54^{th} HIGHWAY GEOLOGY SYMPOSIUM

BURLINGTON, VERMONT, SEPTEMBER 24-26, 2003

PROCEEDINGS

SPONSORED BY:

VERMONT AGENCY OF TRANSPORTATION
NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
VERMONT GEOLOGICAL SURVEY
UNIVERSITY OF VERMONT
NORWICH UNIVERSITY

TABLE OF CONTENTS

Proceedings Papers

The following papers were not presented at the Symposium however they were submitted for publication in the Proceedings:

HIGHWAY GEOLOGY SYMPOSIUM
54th ANNUAL

BURLINGTON, VERMONT
SEPTEMBER 24th – 26th, 2003

The Vermont Agency of Transportation along with the New Hampshire Department of Transportation, Vermont Geological Survey, University of Vermont Departments of Geology and Civil Engineering, and Norwich University Department of Geology welcomes you to the 54th Annual Highway Geology Symposium.

The Local State Steering Committee has strived to put together what we hope is an interesting, educational and enjoyable Symposium. Authors will be presenting some very interesting topics such as geophysical methods, laboratory studies, design considerations and case studies of geo-engineering projects.

The field trip will take us across the breadth of Vermont where we will see some interesting rockfall mitigation applications and fascinating Appalachian geology. We will also visit an active granite quarrying facility and an earthen dam repair project.

Again, welcome, and enjoy the Symposium amidst the beautiful fall foliage we hope to experience this fall in New England.

Thomas D. Eliassen, P.G.
Vermont Agency of Transportation
Host State Committee Chairman
54th Highway Geology Symposium

HIGHWAY GEOLOGY SYMPOSIUM

HISOTRY, ORGANIZATION AND FUNCTION

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

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

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

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

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

List of Highway Geology Symposium Meetings

No.	Year	HGS Location	No.	Year	HGS Location
1st	1950	Richmond, VA	2nd	1951	Richmond, VA
3rd	1952	Lexington, VA	4th	1953	Charleston, W VA
5th	1954	Columbus, OH	6th	1955	Baltimore, MD
7th	1956	Raleigh, NC	8th	1957	State College, PA
9th	1958	Charlottesville, VA	10th	1959	Atlanta, GA
11th	1960	Tallahassee, FL	12th	1961	Knoxville, TN
13th	1962	Phoenix, AZ	14th	1963	College Station, TX
15th	1964	Rolla, MO	16th	1965	Lexington, KY
17th	1966	Ames, IA	18th	1967	Lafayette, IN
19th	1968	Morgantown, WV	20th	1969	Urbana, IL
21st	1970	Lawrence, KS	22nd	1971	Norman, OK

23rd	**1972**	**Old Point Comfort, VA**	**24th**	**1973**	**Sheridan, WY**
25th	**1974**	**Raleigh, NC**	**26th**	**1975**	**Coeur d'Alene, ID**
27th	**1976**	**Orlando, FL**	**28th**	**1977**	**Rapid City, SD**
29th	**1978**	**Annapolis, MD**	**30th**	**1979**	**Portland, OR**
31st	**1980**	**Austin, TX**	**32nd**	**1981**	**Gatlinburg, TN**
33rd	**1982**	**Vail, CO**	**34th**	**1983**	**Stone Mountain, GA**
35th	**1984**	**San Jose, CA**	**36th**	**1985**	**Clarksville, IN**
37th	**1986**	**Helena, MT**	**38th**	**1987**	**Pittsburgh, PA**
39th	**1988**	**Park City, UT**	**40th**	**1989**	**Birmingham, AL**
41st	**1990**	**Albuquerque, NM**	**42nd**	**1991**	**Albany, NY**
43rd	**1992**	**Fayetteville, AR**	**44th**	**1993**	**Tampa, FL**
45th	**1994**	**Portland, OR**	**46th**	**1995**	**Charleston, WV**
47th	**1996**	**Cody, WY**	**48th**	**1997**	**Knoxville, TN**
49th	**1998**	**Prescott, AZ**	**50th**	**1999**	**Roanoke, VA**
51st	**2000**	**Seattle, WA**	**52nd**	**2001**	**Cumberland, MD**
53rd	**2002**	**San Luis Obispo, CA**	**54th**	**2003**	**Burlington, VT**

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

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

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

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

The field trip is the focus of the meeting. In most cases, the trips cover approximately from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours.

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

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

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

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

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

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

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

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

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

HIGHWAY GEOLOGY SYMPOSIUM
54th ANNUAL

BURLINGTON, VERMONT
SEPTEMBER 24th – 26th, 2003

LOCAL ORGANIZING COMMITTEE

Thomas D. Eliassen – Vermont Agency of Transportation
Richard Cross – Golder Associates Inc.
Marc Fish – New Hampshire Department of Transportation
David Westerman – Norwich University
Chris Benda - Vermont Agency of Transportation
Larry Becker – Vermont Geological Survey
Richard Lane - New Hampshire Department of Transportation
Stephen Wright – University of Vermont
Bob Atchinson - Vermont Agency of Transportation
Peter Ingraham - Golder Associates Inc.
James Olson - University of Vermont

Special Thanks To

Meg Boera, Cindy Delaney and Jessica Barber; Delaney Meeting and Event Management , LLC

SPONSORED BY:

Vermont Agency of Transportation
New Hampshire Department of Transportation
Vermont Geological Survey
University of Vermont
Norwich University

HIGHWAY GEOLOGY SYMPOSIUM STEERING COMMITTEE OFFICERS

Mr. Bob Henthorne, Chairman
Chief Geologist
Materials and Research Center
2300 Van Buren
Topeka, KS 66611-1195
PH: 785-291-3860
FAX: 785-296-2526
Email: roberth@ksdot.org

Mr. G. Michael Hager, Vice-Chairman 2004
Wyoming Department of Transportation
P.O. Box 1708
Cheyenne, WY 82009-1708
PH: 307-777-4205
Email: Mike.Hager@dot.state.wy.us

Mr. Richard Cross, Secretary 2003
Golder Associates
RD 1 Box 183A
Solansville, NY 12160
PH: 518-471-4277
Email: dick_cross@juno.com

Mr. Russel Glass, Treasurer
North Carolina DOT (Retired)
100 Wolfe Cove Road
Asheville, NC 28804
Email: frgeol@aol.com

NATIONAL STEERING COMMITTEE 2003
HIGHWAY GEOLOGY SYMPOSIUM

NAME/ADDRESS	**PHONE/FAX/E-MAIL**
Ken Ashton West Virginia Geological Survey P.O. Box 879 Morgantown, WV 26507-0879	PH 304-594-2331 FX 304-594-2575 E-MAIL ashton@geosrv.wvnet.edu
John Baldwin West Virginia Div. Of Highways 190 Dry Branch Rd. Charleston, WV 25036	PH 304-558-3084 FX 304-558-0253 E-MAIL jbaldwin@mail.dot.state.wv.us
Vernon Bump South Dakota DOT (Retired) Geotech. Engr. Activity 700 E. Broadway Ave. Pierre, SD 575010-2586	PH 605-224-7008 FX E-MAIL vernglobump@pie.midco.net
Jeff Dean Oklahoma DOT 200 NE 21st St. Oklahoma City, OK 73015	PH 405-522-0988 FX 405-522-4519 E-MAIL jdean@odot.org
John Duffy California State Dept. of Transportation 50 Higuera Street San Luis Obispo, CA 93401	PH 805-549-3663 FX 805-549-4693 E-MAIL John_Duffy@dot.ca.gov
Russel Glass North Carolina DOT (Retired) 100 Wolfe Cove Rd. Asheville, NC 28804	PH 828-252-2260 FX 828-299-1273 E-MAIL frgeol@aol.com
W.A. Wisner Martin Marietta Aggregates P.O. Box 30013 Raleigh, NC 27622	PH 919-783-4649 FX 919-783-4552 E-MAIL

NATIONAL STEERING COMMITTEE 2003
HIGHWAY GEOLOGY SYMPOSIUM

NAME/ADDRESS	**PHONE/FAX/E-MAIL**
Robert Goddard National Magnetic Field Lab Florida State University 1800 E. Paul Dirac Dr. Tallahassee, FL 32306-4005	PH 850-644-4304 FX 850-644-0687 E-MAIL goddard@magnet.fus.edu
G. Michael Hager Wyoming DOT 5300 Bishop Blvd. Cheyenne, WY 82009-3340	PH 307-777-4205 FX 307-777-3994 E-MAIL mike.hager@dot.state.wy.us
Robert Thommen Rotec Enterprises Inc. P.O. Box 31536 Sante Fe, NM 87594-1536	PH 505-753-6586 FX 505-753-6590 E-MAIL thommen@rotecinternational-usa.com
Richard Humphries Golder Associates 3730 Chamblee Tucker Rd. Atlanta, GA 30341	PH 770-496-1893 FX 770-934-9476 E-MAIL rhumphries@Golder.com
A. David Martin Maryland State Highway Administration Office of Materials & Technology 2323 W. Joppa Road Brooklandville, MD 21022	PH 410-321-3107 FX 410-321-3099 E-MAIL dmartin@sha.state.md.us
Henry Mathis, PE H.C. Nutting Co. 561 Marblerock Wy. Lexington, KY 40503	PH 859-296-5664 859-223-8632 Home FX 859-296-5664 E-MAIL hmathis@iglou.com
Harry Moore Tennessee DOT P.O. Box 58 Knoxville, TN 37901	PH 865-594-2701 FX 865-594-2495 E-MAIL harry.moore@state.tn.us

NATIONAL STEERING COMMITTEE 2003
HIGHWAY GEOLOGY SYMPOSIUM

NAME/ADDRESS	PHONE/FAX/E-MAIL
Nick Priznar Arizona DOT 1221 N. 21st Ave. Phoenix, AZ 85009-3740	PH 602-712-8089 FX 602-712-8415 E-MAIL NPRIZNAR@dot.state.az.us
Christopher A. Ruppen Michael Baker Jr. Inc. 4301 Dutch Ridge Rd. Beaver, PA 15009-9600	PH 724-495-4079 FX 724-495-4017 E-MAIL cruppen@mbakercorp.com
Stephen Senior Ministry of Transportation Rm 220, Central Bldg. 1201 Wilson Ave. Downsview, ON M3M 1J6, Canada	PH 416-235-3743 FX 416-235-4101 E-MAIL stephen.senior@mto.gov.on.ca
Willard L. Sitz Alabama DOT 1409 Coliseum Blvd. Montgomery, AL 36110-2060	PH 334-206-2279 FX 334-264-6263 E-MAIL sitzw@dot.state.al.us
Jim Stroud Vulcan Materials Co. 4401 N. Patterson Ave. P.O. Box 4239 Winston-Salem, NC 27115	PH 336-767-4600 FX 336-744-2019 E-MAIL stroudj@vmcmail.com
John Szturo HNTB Corporation 1201 Walnut, Suite 700 Kansas City, MO 64106	PH 816-527-2275 FX 816-472-5013 E-MAIL jszturo@hntb.com
Michael P. Vierling New York State Thruway Authority 200 Southern Blvd. Albany, NY 12209	PH 518-471-4378 FX 518-436-3060 E-MAIL michael_vierling@thruway.state.ny.us

NATIONAL STEERING COMMITTEE 2003
HIGHWAY GEOLOGY SYMPOSIUM

NAME/ADDRESS	**PHONE/FAX/E-MAIL**
Eric Rorem Brugg Cable Products, Inc. 11807 NE 99th St. Suite 1160 Vancouver, WA 98682	PH 360-253-3438 FX 360-254-2522 E-MAIL erik.rorem@us.geobrugg.com
Chester F. "Skip" Watts Radford University Radford, VA 24142	PH 540-831-5652 FX 540-831-5732 E-MAIL cwatts@runet.edu
Terry West Earth and Atmospheric Science Dept. Purdue University West Lafayette, IN 47907-1297	PH 765-494-3296 FX 765-496-1210 E-MAIL trwest@cas.purdue.edu

HIGHWAY GEOLOGY SYMPOSIUM

EMERITUS MEMBERS OF THE STEERING COMMITTEE

Emeritus Status is granted by the Steering Committee

R.F. Baker*
David Bingham
Virgil E. Burgat*
Robert G. Charboneau*
Hugh Chase*
A.C. Dodson*
Walter F. Fredericksen
Brandy Gilmore
Joseph Gutierrez
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
Mitchell Smith
Sam Thornton
Berke Thompson*
Burrell Whitlow*
Earl Wright
Ed J. Zeigler
Steve Sweeney

*Deceased

HIGHWAY GEOLOGY SYMPOSIUM

MEDALLION AWARD WINNERS

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

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

*Deceased

54th HIGHWAY GEOLOGY SYMPOSIUM SPONSORS

The following companies have graciously contributed toward sponsorship of the Symposium. The HGS relies on sponsor contributions for events such as refreshment breaks, field trip lunches and other activities and want these sponsors to know that their contributions are very much appreciated.

Janod Inc.
555 VT, Suite 122,
Route 78
Swanton, VT, 05488
Phone (802) 868-5058 / Fax (450) 424-2614
www.janod.biz/
info@janod.biz

Janod has specialized in rock stabilization and rock remediation since 1968. Janod was founded in 1968 by Douglas Journeaux, and at that time, soft earth tunneling was the principle part of our operations. In 1970 Janod was introduced to rock slope stabilization when called in by Quebec Cartier Mining to perform some emergency work along the railway. Janod has since become a specialist in rock stabilization, and employs a combination of innovative mechanized equipment and highly trained rock remediation technicians who have an intimate knowledge of geology and influence of climatic conditions on exposed rock structures.

Geobrugg North America, LLC.
Geobrugg Protection Systems
551 W. Cordova Road, PMB 730
Santa Fe, New Mexico 87505
Phone (505) 438 6161 / Fax (505) 438 6166
www.us.geobrugg.com
info@us.geobrugg.com

Geobrugg helps protect people and infrastructures from the forces of nature. The technologically mature protection systems of steel wire nets developed and produced by us are now used all over the world. Our dynamic and static barrier systems offer proven protection against rock falls, avalanches, mud flows and slope failures.

Golder Associates
540 North Commercial Street, Suite 250
Manchester, New Hampshire 03101-1146
Phone (603) 668 0880 / Fax (603) 668 1199
www.golder.com/
amacdonald@golder.com

Golder Associates is an international group of science and engineering companies. The employee-owned group of companies provides comprehensive consulting services in support of environmental, industrial, natural resources and civil engineering projects. Founded in 1960, Golder now has nearly 3,000 employees in over 80 offices worldwide and has completed projects in more than 140 countries.

Boulderscape, Inc.
34249 Camino Capistrano
Suite 215
Capistrano Beach, CA 92624
Phone (949) 661-5087
Fax (949) 661-3397
www.boulderscape.com/
steve@boulderscapeinc.com

Boulderscape is a dynamic rockscape company that has risen to the top of its trade by focusing on extreme detail in it's rock formation projects. Boulderscape is well known as the leader in sculpted positive rock formation projects largely because of their depth of veteran sculptors, who have done projects all over the world. Their uncanny ability to recreate nature with sculpture deletes the need for GFRC rock castings which many times are too costly for large projects. Boulderscape has recently completed sculpting work on a major rail project in New Jersey.

Vertec Contractors Inc.
555 VT, Route 78, Suite 252
Swanton, VT, 05488
Phone (450) 455-9690 / Fax (450) 424-2614
www.verteccontractors.biz/
info@verteccontractors.biz

Vertec specializes in complex and technically challenging projects in areas of steep terrain and difficult access. Using techniques and equipment developed in the European alpine construction industry, our highly skilled technicians can execute difficult projects safely and efficiently. Vertec provides innovative solutions and cost effective technology with less impact to the environment than conventional methods. From installing 45 foot long rock bolts 200 feet up a verti-cal cliff to hoisting and placing 10,000 lb joints of pipe on steep slopes. From inspecting guy wires on a 300 foot tall refinery tower to excavating on a 1:1 slope with a spider excavator. Vertec has the experience and the equipment to successfully complete your most challenging projects.

Haywood Baker Inc.
1130 Annapolis Road, Suite 202
Odenton, Maryland 21113
Phone (410) 551-8200 / Fax (410) 551-1900
www.keller-ge.co.uk
jksidhu@kellerfoundations.com

America's leading Ground Modification[sm] contractor specializing in site improvement, stabilization, underpinning, excavation support, settlement control, foundation rehabilitation, and groundwater control.

PACIFIC BLASTING & DEMOLITION LTD.
3183 Norland Avenue,
Burnaby, BC, CANADA V5B 3A9
Phone (604) 291-1255 / Fax (604) 291-2813
www.pacificblasting.com/
info@pacificblasting.com

For five decades Pacific Blasting has specialized in completing jobs which are beyond-the-ability of most companies. With experts in each field, Pacific Blasting specializes in blasting, drilling, demolition, shotcrete shoring and transport & relocation of heavy industrial machinery and equipment. With, dedication to customer satisfaction and safety as our focus, Pacific Blasting & Demolition tackles difficult jobs, worldwide.

Geo**Design, Inc.**
54 Main Street
Windsor VT 05089
(802) 674-2033 / Fax (802) 674-5943
www.geodesign.net/
j.lens@geodesign.net

Geo**Design** is a New England based Geotechnical Engineering firm specializing in Foundation Engineering & Design, Subsurface Explorations, Earth & Rock Fill Dam Design & Evaluation, Slope Stability Studies, Computer Modeling, Design of Temporary Structures, Geo-Structural Design, Permanent Reinforcement, Coffer Dams and Forensic Engineering. GeoDesign has in-house Laboratory Soil & Rock Testing capabilities. Geo**Design** has offices in Windsor, Vermont and Middlebury, Connecticut.

54th HIGHWAY GEOLOGY SYMPOSIUM **EXHIBITORS**

The host committee for the 54th Annual Highway Geology Symposium would like to express it's appreciation to the following exhibitors and sponsors. You are invited to visit their displays and please be sure to express your appreciation.

Janod Inc.
555 VT, Suite 122,
Route 78
Swanton, VT, 05488
Phone (802) 868-5058
Fax (450) 424-2614
www.janod.biz/
info@janod.biz

Geobrugg North America, LLC.
Geobrugg Protection Systems
551 W. Cordova Road
PMB 730
Santa Fe, New Mexico 87505
Phone (505) 438 6161
Fax (505) 438 6166
www.us.geobrugg.com
info@us.geobrugg.com

Hi-Tech Rockfall Construction, Inc.
P.O. Box 674
Forest Grove, OR 97116
Phone (503) 357-6508
Fax (503) 357-7323
www.hi-techrockfall.com
HTRockfall@aol.com

Applied Research Associates
New England Division
415 Waterman Rd.
South Royalton, VT 05068
Phone (802) 763-8348
Fax (802) 763-8283
www.ara.com/new_england_division.htm
NEDinfo@ara.com

Central Mine Equipment Company
4215 Rider Trail North
St. Louis, Missouri, 63045
Phone (314) 291-7700, (800) 325-8827
Fax (314) 291-4880
www.cmeco.com/
info@cmeco.com

Geokon, Incorporated
48 Spencer Street
Lebanon, New Hampshire 03766
Phone (603) 448-1562
Fax (603) 448 3216
www.geokon.com/
info@geokon.com

Golder Associates
540 North Commercial Street, Suite 250
Manchester, New Hampshire 03101-1146
Phone (603) 668 0880
Fax (603) 668 1199
www.golder.com/
amacdonald@golder.com

Hager-Richter Geoscience, Inc.
8 Industrial Way D-10
Salem, NH 03079
Phone: (603) 893-9944
Fax: (603) 893-8313
www.hager-richter.com/
dorothy@hager-richter.com

American Mountain Management Inc.
Financial Plaza Building
1135 Terminal Way, Suite 106
Reno, Nevada, 89502-2145
Phone (866) 466-7223 (toll free US & Canada)
Fax (450) 455-8762
andre@mountainmanagement.biz

Haywood Baker Inc.
1130 Annapolis Road, Suite 202
Odenton, Maryland 21113
Phone (410) 551-8200
Fax (410) 551-1900
www.keller-ge.co.uk
jksidhu@kellerfoundations.com

H.C. Nutting Company
470-B Conway Court, Suite B-8
Lexington, KY 40511
Phone (859) 455-8530
Fax (859) 455-8630
www.hcnutting.com/
lexington@hcnutting.com

Maccaferri, Inc.
10303 Governor Lane Blvd.
Williamsport, MD 21795
Phone (301) 223-6910
Fax (301) 223-6134
www.maccaferri-usa.com/
njoffice@maccaferri-usa.com

NSA Geotechnical Services, Inc.
15000 West 6th Avenue, Suite 100
Golden, CO 80401
Phone (303) 277-9920
Fax (303) 277-9921
www.nsaengineering.com
info@nsageotech.com

P.E. LaMoreaux & Associates, Inc. (PELA)
106 Administration Road, Suite 4
Oak Ridge, Tennessee 37830
Phone (865) 483-7483
Fax (865) 483-7639
www.pela-tenn.com
info@pela-tenn.com

Pacific Blasting & Demolition LTD.
3183 Norland Avenue,
Burnaby, BC, CANADA V5B 3A9
Phone (604) 291-1255
Fax (604) 291-2813
www.pacificblasting.com/
info@pacificblasting.com

Prometheus Construction
4990 Anita Lane
Santa Barbara, CA 93111
Phone (805) 964-3939
Fax (805) 964-0273
www.prometheusconstruction.com/
info@prometheusconstruction.com

RST Instruments Ltd.
200-2050 Hartley Ave.
Coquitlam, BC
V3K 6W5, Canada
Phone (604) 540-1100, (800) 665-5599
Fax (604) 540-1005
www.rstinstruments.com/
info@rstinstruments.com

Schnabel Foundation Company
Northeast Regional Office
200 Turnpike Road
Southborough, MA 01772
Phone (508) 303-3642
Fax (508) 303-
www.schnabel.com/
Hank@schnabel.com

Technos, Inc.
10430 NW 31st Terrace
Miami, FL 33172
Phone (305) 718-9594
Fax (305) 718-9621
www.technos-inc.com/
info@technos-inc.com

Vertec Contractors Inc.
555 VT, Route 78, Suite 252
Swanton, VT, 05488
Tel (450) 455-9690
Fax (450) 424-2614
www.verteccontractors.biz/
info@verteccontractors.biz

Rotec International, LLC
Post Office Box 31536
Santa Fe, NM 87594-1536
Phone (505) 753-6586
Fax (505) 753-6590
www.rotecinternational-usa.com
thommen@rotecinternational-usa.com

Michael Baker Corporation
4301 Dutch Ridge Road
Beaver, PA 15009-0280
Phone (724) 495-4254
Fax (724) 495-4017
www.mbakercorp.com
cruppen@mbakercorp.com

SIMCO Drilling Equipment, Inc
802 S. Furnas Drive
Osceola, LA 50213
Phone (800) 338-9925
Fax (641) 342-6764
www.simcodrill.com
info@simcodrill.com

SAS Stressteel Inc.
5 Gardner Road
Fairfield, NJ 07004
Phone (973) 244-5995
Fax (973) 244-0544
joe@stressteel.com

AIS Construction Company
797 Rincon Hill Rd.
Carpinteria, CA 93013
Phone (805) 684-4344
Fax (805) 566-0109
www.aisconstruction.com
andy@aisconstruction.com

Monotube Pile Corporation
P.O. Box 7339
Canton, OH 44705
Phone (330) 454-6111
Fax (330) 454-1572
www.monotube.com
monotube@racx.com

Boulderscape, Inc.
34249 Camino Capistrano
Suite 215
Capistrano Beach, CA 92624
Phone (949) 661-5087
Fax (949) 661-3397
www.boulderscape.com/
steve@boulderscapeinc.com

Highway Geology Symposium

Future Symposia Schedule and Contact List

Year	State	Host Coordinator	Telephone Number	Email
2004	Missouri	Bob Henthorne John Szturo	(785) 291-3860 (816) 527-2275	roberth@ksdot.org jszturo@hntb.com
2005	North Carolina	Russell Glass	(828) 298-3874	frgeol@aol.com
2006	Colorado	Jon White	(303) 866-3551	jonathan.white@state.co.us
2007	Pennsylvania	Chris Ruppen	(724) 495-4079	cruppen@mbakercorp.com

54th Highway Geology Symposium September 24 - 26, 2003 Sheraton Burlington Hotel & Conference Center, Burlington, Vermont

GENERAL INFORMATION

The Vermont Agency of Transportation in cooperation with the New Hampshire Department of Transportation, Vermont Geological Survey, University of Vermont Departments of Geology and Civil Engineering, and Norwich University Department of Geology is hosting the 54th Annual Highway Geology Symposium (HGS) scheduled for September 24-26, 2003, to be held at the Sheraton Burlington Hotel & Conference Center in Burlington, Vermont.

The 54th Annual HGS, beginning on Wednesday, September 24, consists of a full day of technical presentations, a full day field trip, and concludes with a final half day of technical presentations on Friday, September 26.

TRB ROCK BOLTING WORKSHOP:

The Symposium will be preceded on Tuesday, September 23rd by a full day Transportation Research Board (TRB) workshop. The workshop will consist of a morning technical session and an afternoon rock bolting demonstration (performed at a nearby highway rock cut). The registration fee for the TRB workshop is $50.00 (Please see registration form).

SPECIAL EVENT:
Lake Champlain Scenic Cruise – Wednesday, September 24th

"Catch the Spirit" aboard Lake Champlain's largest cruise ship. The Spirit of Ethan Allen III is Burlington's newest luxury cruise ship with temperature-controlled decks, topped by an open air deck. While on board, keep your eyes open for the legendary "Champ," the lake's native mythical serpent. The spectacular scenery of the Burlington shoreline and the Adirondack Mountains will provide a beautiful backdrop while you dine on a scrumptious dinner buffet. This special event is not included in the Symposium Registration Fee and is an additional fee of $40 (See registration form).

Guest Tour:

A full-day tour to Stowe has been arranged for Thursday, September 25th for spouses, friends, and other conference guests. The Stowe Day Tour will include time for shopping, strolling in the village, and incredible vistas during the scenic Gondola ride up Mt. Mansfield. We'll also visit the Cold Hollow Cider Mill and the Trapp Family Lodge- still owned and operated by the Trapp family, the inspiration for the classic musical and movie, "The Sound of Music." The cost for this tour is $50. Please see Registration Form.

AGENDA

TUESDAY, SEPTEMBER 23

8:00 AM - 5:00 PM
TRB and HGS 54 Registration

8:30 AM - 12:00 PM
TRB Workshop Session 1:
1. Dr. Ken Fishman of McMahon and Mann Consulting Engineers, P.C - ***Condition Assessment of Rock Reinforcement***
2. **Round Table Discussion on Rock Bolting**

12:00 PM - 1:00 PM
Lunch Break (box lunch provided)

1:00 PM - 4:30 PM
TRB Workshop Session 2:
Rock Bolting Demonstration – Circumferential Highway VT 289, Colchester, VT (Transportation provided)

5:00 PM - 6:00 PM
Welcome Reception & Visit with Exhibitors

WEDNESDAY, SEPTEMBER 24

6:30 AM - 7:30 AM
Continental Breakfast

7:00 AM - 4:30 PM
HGS 54 Registration

7:30 AM - 12:00 PM
Morning Poster Session

General Session
7:30 AM - 8:00 AM
Welcoming Remarks
Pat McDonald, Secretary, Vermont Agency of Transportation
Larry Becker, State Geologist, Vermont Geological Survey

8:00 AM – 9:40 AM
Technical Session I:
1. John F. Szturo, ***Geotechnical Explorations – Great River Bridge***
2. Paul Fisk, Kitty Breskin PE, ***Economic Benefits of Seismic Refraction Investigations for Road-cut Design Studies***
3. Marc Fish, PG, ***Rock bolting, Perspectives from a State DOT***
4. G. Michael Hager, ***Seven S's Of Geotechnical Doom***
5. Harry L. Moore, ***Recent Sinkhole Occurrences Along Highways in East Tennessee, A Historical Perspective***

9:40 AM - 10:00 AM (Break)

10:00 AM - 12:00 PM
Technical Session II:
6. John D. Duffy, Aileen Loe, Morgan Gaudioso, ***Living with Landslides on the Big Sur Coast: The Challenges of Maintaining Highway 1***
7. Jeffrey R. Keaton, ***Earthquake Ground Motion for Design of the Hoover Dam Bypass Bridge (US Highway 93)***
8. Scott L. Murray, PE, ***The Industrial Parkway – So You Want to Build a Road in Mine Spoils***
9. Ed Kase, Tim Ross, Jozef Descour, and Don Green, ***Leveraging Existing Infrastructure: Using What Is Already in the Ground***
10. Daniel Journeaux, Pierre Rousseau, Mark McNeilly, Jay Smerekanicz and Peter Ingraham, ***Rock Slope Scaling and Aesthetic Stabilization of two Historically Sensitive Palisades Sill Slopes in Weehawken, New Jersey***
11. Angela L. Adams, Wanfang Zhou, Jie Wang and Barry F. Beck, ***Using GPR Reflection Patterns And NP Measurements To Predict Sinkhole Risk In Central Florida***

12:00 PM - 1:00 PM
Luncheon with Exhibitors (Provided)

12:00 PM - 4:30 PM
Afternoon Poster Session

1:00 PM - 2:20 PM
Technical Session III:
12. Nick M. Priznar, Kenneth M. Euge, R.G., ***GPR and FWD subsurface investigation techniques used to detect voids in variable subgrade soils on I-40 Near Topock Arizona***
13. Scott Anderson, David E. Peterson, Robert D. Turton, ***Geologic Characterization for Bridge Foundations, Colorado River Bridge, Hoover Dam Bypass Project***
14. Norbert H. Maerz, Ahmed Youssef, ***A Risk-Consequence Hazard Rating System for Missouri Highway Rock Cuts***
15. Robert J. Watters and Kurt Katzenstein, ***Calibration and Accuracy of Rock Fall Simulation Programs***

2:20 PM - 2:40 PM (Break)

2:40 PM - 4:00 PM
Technical Session IV:
16. Joseph A. Fischer and James G. McWhorter, ***Rock Slope Failures: A Legal Case History***
17. Dick Lane, ***Selected Case Histories of Rock Slope Stabilization in New Hampshire***
18. Ted von Rosenvenge, ***Rock Slope Failure Case History Yanacachi, Bolivia, S.A.***
19. Randy L. Kath, Deana Sneyd, and Katie Tyrrell, ***Digital Mappping Assistant And Logger: Two Palm Applications For Digital Collection Of Geologic Data Using a PDA And a GPS Receiver And a Geotechnical Borehole Logging Application***

4:00 PM - 4:30 PM
Questions & Answers and Field Trip Briefing

4:30 PM
Adjourn for the day

5:00 PM
Lake Champlain Scenic Dinner Cruise (Optional)
Buses depart Sheraton Conference Center Entrance at 5:00 PM.

THURSDAY, SEPTEMBER 25

7:00 AM- 5:00 PM
Geology Field Trip

8:30 AM- 5:00 PM
Guest Field Trip
Stowe Tour

6:00 PM- 7:00 PM
Social Hour and Exhibits

7:00 PM- 10:00 PM
Banquet Dinner
Keynote Speakers: Brian Fowler and David Wunsch**, *New Hampshire's "Old Man of the Mountain"***.

FRIDAY, SEPTEMBER 26

7:00 AM- 8:00 AM
Continental Breakfast
Steering Committee Meeting

8:00 AM - 12:00 PM
Morning Poster Session

8:00 AM - 9:40 AM
Technical Session V:
20. Tom Badger, ***A Trim Blast Success: Lake Entiat Vicinity, Washington***
21. Danny J. Van Roosendaal, Nicholas H. Strater and Andrew F. McKown, ***Design of Passive Dowel Systems and Controlled Blasting Measures for Stabilization of Excavated Rock Slopes***
22. Robert L. Dodson, James M. Sheahan, ***Evaluation of Adverse Bedding Orientation on the Clifford Hollow Bridge Foundations***
23. Jeff Dean, ***Dispersive Clay Embankment Erosion***
24. Thomas J. Douglas, Brad Worley, and Colin Mellor, ***A Case Study of Methods used to Study a Sinkhole on Interstate 40, Pender County, NC***

9:40 AM - 10:00 AM (Break)

10:00 AM - 12:00 PM
Technical Session VI:
25. Vincent G. Reidenbach, Jim Nevels, Curt Hayes, ***Statistical Analysis of Unconfined Compressive Strength of Rock Types Found in Oklahoma***
26. Larry R. Bolt, James L. Stuby, Peter H. Li, David Martin ***Maryland's Experience with Large Scale Grouting for Roadway Stabilization in Karst Terrain of I-70***
27. Nancy C. Dessenberger, Robert A. Meyers, Francis E. Harrison, ***Slope Design for Improvements to New Mexico State Highway 48, near Ruidoso, New Mexico***
28. Harry W. Schnabel, ***Case Histories of Tieback And Soil Nail Walls For Roadways***
29. Nancy C. Dessenberger, Francis E. Harrison, ***Effective Interpretation of Borehole Inclinometer Profiles: What is Really Slope Movement and What is Probably Something Else***

11:40 AM - 12:00 PM Concluding Remarks

12:00 PM Symposium Adjourns

Geotechnical Explorations
Great River Bridge
Desha County, Arkansas – Bolivar County, Mississippi

John F. Szturo R.G.
HNTB Corporation
715 Kirk Drive
Kansas City, Missouri 64105

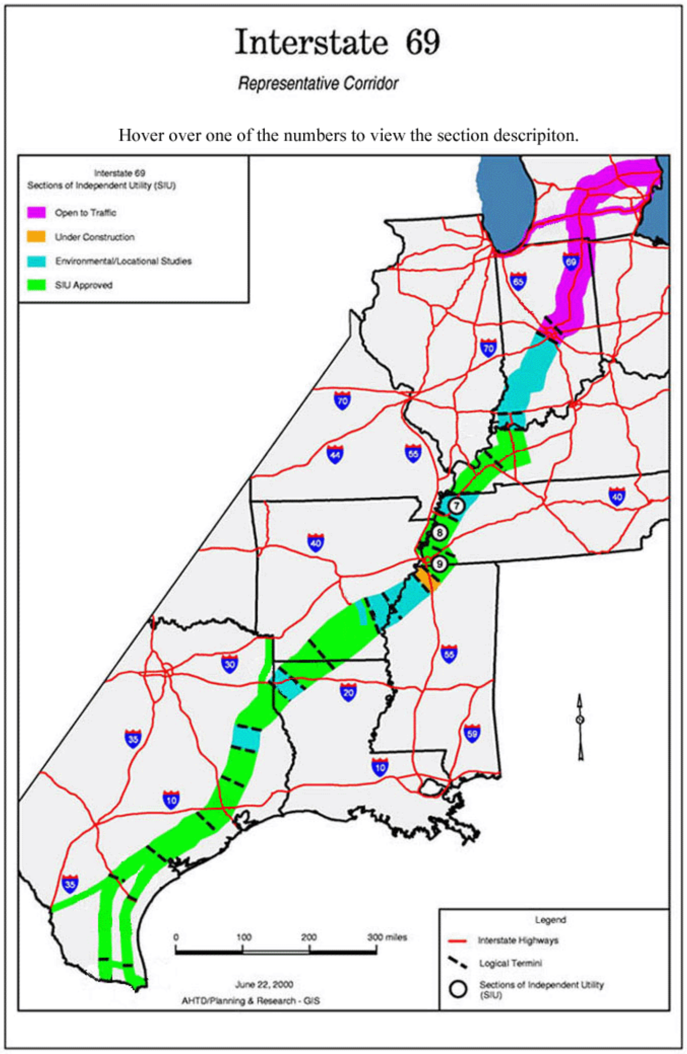
54th Annual Highway Geology Symposium

September, 2003

Burlington, Vermont

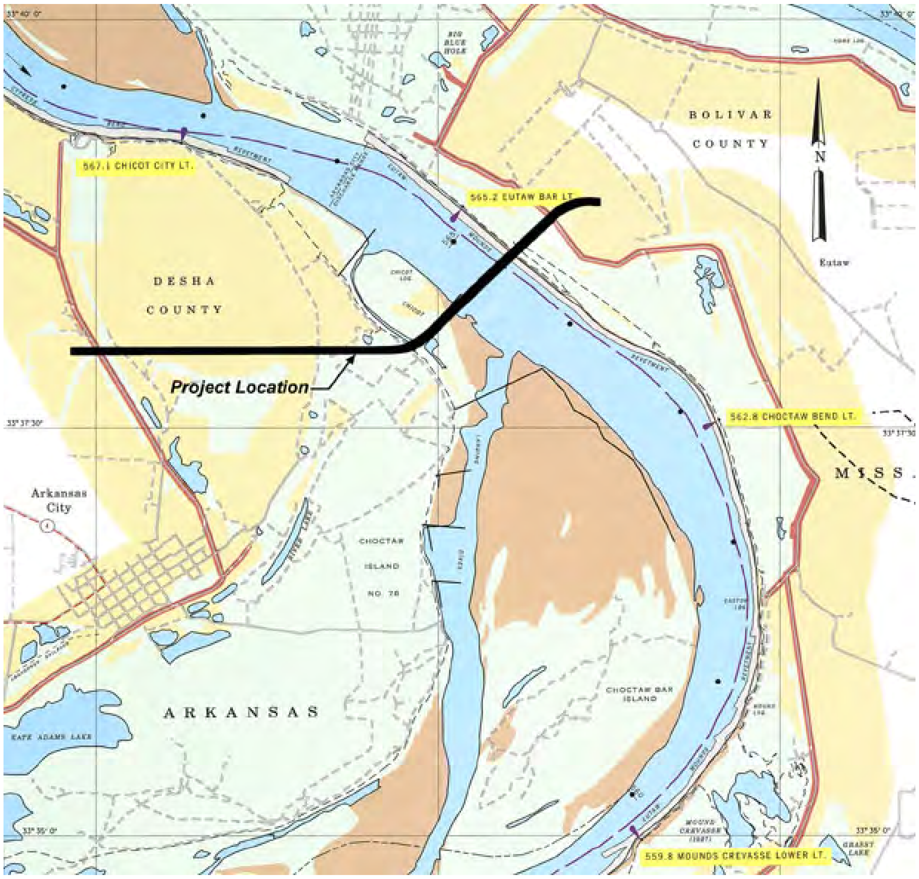
Introduction

The Great River Bridge is the Mississippi River crossing of the high priority "NAFTA" corridor, a planned interstate route from Detroit, Michigan to Brownsville, Texas. Formally known as I-69, the roadway already exists from Detroit to Indianapolis. Additional segments are either in the planning/eis stage, in design, or under construction. The Great River Bridge Project, administered by the Arkansas Highway and Transportation Department with cooperation from the Mississippi Department of Transportation, is funded to proceed to final design.



I-69 Corridor

The Great River Bridge project consists of a four-lane divided roadway and bridge across the Mississippi River just north of Greenville, Mississippi. The project alignment crosses the Mississippi River approximately two miles north of Arkansas City, Arkansas, near the old Chicot Landing site at Mississippi River Mile 564.6 above the Gulf of Mexico. The project begins approximately 2,600 feet west of the Arkansas City levee in Arkansas and ends approximately 2,800 feet east of the existing levee in Mississippi. The total length of the project is approximately 22,800 feet, with approximately 22,570 feet (4.27 miles) of continuous bridge.



Project Location

The low level approach bridge on the Arkansas side is approximately 13,232 feet (2.5 miles) in length. The approach bridge on the Mississippi side is approximately 3,075 feet (0.6 miles) in length.

The length and arrangement of the main river spans were set by engineering, economic and navigation considerations evaluated as part of the conceptual studies. The river span arrangement consists of a main span cable stayed bridge opening of 1520 feet between the two towers, twin anchor spans of 685 feet, and additional transition spans totaling 3,372 on the west bank for a total of 6,262 feet (1.2 miles).

GEOLOGIC SETTING

The project is located in the Mississippi Alluvial Valley, an extensive lowland that extends from Southeast Missouri, southward 600 miles to the Gulf of Mexico. The region is a vast floodplain with valley width in the project area of approximately 80 miles. Ground surface elevations generally range from 110 to 140 feet above sea level. The constructed levee system rises approximately 30 feet above the local ground surface.

The deposits of the valley are of irregular thickness and are made of two units, a lower layer of sands and gravels (substratum), and an upper layer of soft clayey and silty beds (top stratum). The lower layer makes up most of the alluvial mass and occurs closer to the surface at the margins of the valley. The upper fine grained layer, is more unpredictable in material and thickness, having been reworked and replaced by the dynamics of the river during recent geologic time.

The formation of the alluvial valley is directly related to the accumulation and melting of glaciers, starting some 60,000 years ago (Pleistocene). Ice accumulations were formed on the continent from water withdrawn from the ocean, thus, causing a gradual lowering of sea level. With the lowering of the sea level came a steepening of the river gradient and the river was allowed to cut a deep trench-like valley into the underlying marine bedrock layers. This depth of entrenchment can be up to 250 feet below the present ground surface.

As the glaciers melted and sea levels rose to their present static elevation about 5000 years ago (Holocene), the alluvial sediments were deposited in the valley. The general alluvial sequence with its upward decrease in particle size, results from the progressive decrease in slope brought about by the rising sea level and consequent filling of the valley. First the trenches were filled with clean gravelly sands, then clean sand, and finally the filling of silts and clays by flood waters.

The variability of the upper deposits of the valley is directly related to the meandering of the river. The meandering of the river has left well-defined traces in the erosional and depositional features on the surface of the alluvial plain. The river has moved back and forth over the same location at different geometry and dynamics of either erosion or deposition. This system has formed the patchwork of various upper level deposits.

These variable upper level deposits are identified as features such as oxbow lakes, point bars, natural levees, bar accretions, channel fillings, and backwater deposits. Many of these features are readily recognized on topographic maps and aerial photographs as traces of the former channel.

Natural levees are sediments deposited on either side of a stream by the natural deposition of coarse-grained materials contained in overflow waters. There is a decrease in grain size away from the crest of the levee and in a downstream direction. The materials are predominantly sandy and silty clays.

Point bars are formed on the inside bendways whenever the stream migrates. Although the deposits extend to a depth equal to the deepest portion of the thalweg of the parent stream, only the uppermost, fine-grained portion is included in the topstratum. These types of deposits characteristically form an alternating series of deposits that conform to the curvature of the migrating channel and indicate the direction of meandering. Ridges are developed during high stream flows and swales are laid down during falling stream stages.

Backswamp deposits consist of fine-grained sediments laid down in broad shallow basins during periods of stream flooding. Backswamp areas typically have very low relief and a distinctive, complicated drainage pattern in which the channels alternately serve as tributaries and distributaries at various flood periods. Sediments that make up the deposits are clays and silty clays with occasional lenses of silt and sands.

Abandoned channel (clay plugs) deposits are sediments laid down in a meander loop after a stream shortens its course by the converging of arms of a loop when a stream occupies a swale or chute and abandons the outer position of the loop. Generally, coarser grained materials are deposited in the cutoff areas or arms while clays are laid down in the loop area.

Man-made flood control efforts and navigation projects have cut numerous loops from the river, shortening its course by several hundred miles from its former natural state. Shortening the river decreases the total distance, raised the gradient, increases the velocity, and promotes constant scour, keeping the navigation channel open with a minimum amount of dredging. These projects have created many of the abandoned channels, oxbow lakes and filled clay plugs.

Abandoned courses are lengthy segments of a river abandoned by a stream when it is making a gradient adjustment to a change in valley slope. The abandoned course may vary from a few miles to tens of miles in length and is filled with sediments. Data are insufficient to describe the sediments filling abandoned Mississippi River courses. However, indications are that the old course fills with a wedge of sand, thickest where the new course diverges from the old, and gradually thinning downstream. Abandoned courses of smaller streams are filled with clays and silty clays that are similar in composition and thickness to those in clay plugs (abandoned channels). Often an underfit stream occupies the older course and often meanders within the old mender belt destroying segments of the course or it may outline the course when there are no other indications of its presence.

Channel bar deposits are those sediments laid down within the banks of the present courses of the Mississippi River. These deposits consist of sediments deposited behind and between man-made dikes extending out into the channel form its banks; the “islands” developed within the channel of the river; and the sediments reworked by the stream action during the thalweg changes related to high and low water regimes.

TERTIARY DEPOSITS

The alluvial plain is located in a great structural downwarping called the Gulf Coast Geosyncline. Downwarping is a result of the accumulation of the marine sediments forming the Gulf Coastal Plain.

Indurated sedimentary deposits of Tertiary Age (Jackson Formation) form the floor of the entrenched valley under the alluvium at the project location. These deposits are characterized as deltaic marine clays with scattered beds of sand or gravel. Cementation by calcification may also rarely be found.

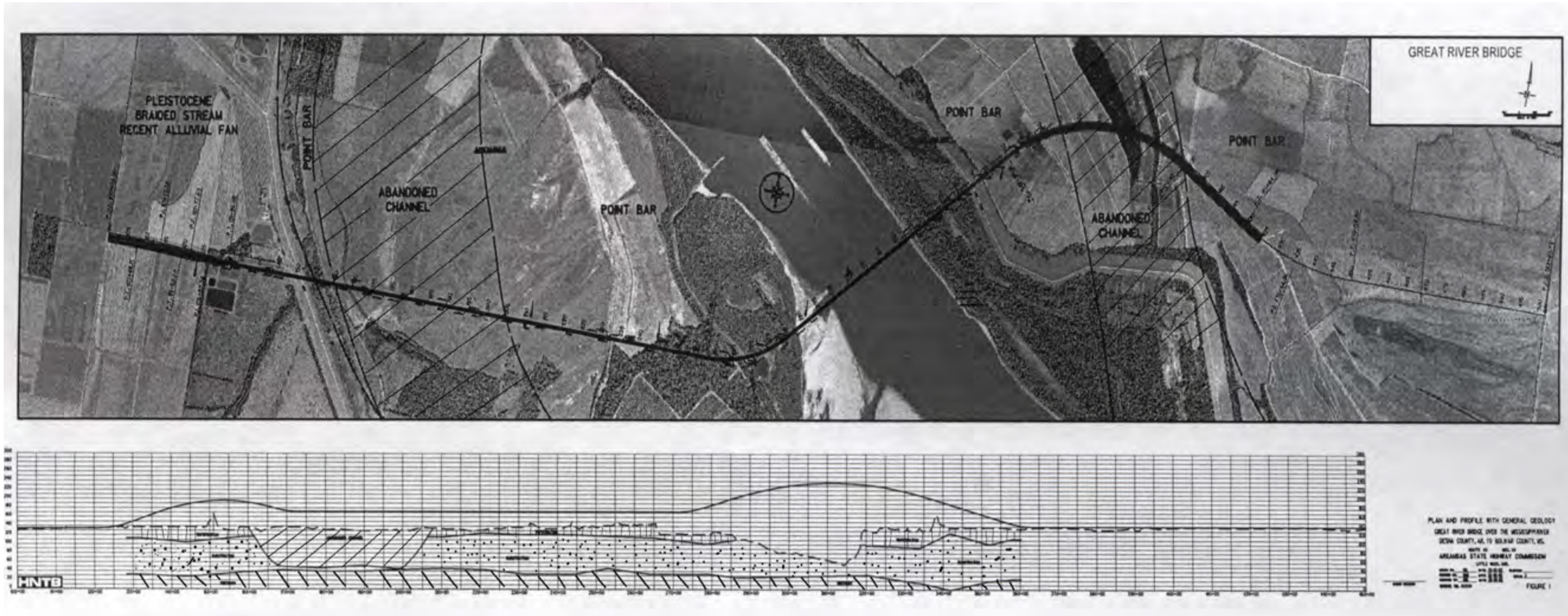
Geologic units underlying the alluvial deposits within the project area belong to the Jackson Group that is made up of Eocene Age Yazoo and underlying Moody's Branch Formation. The unweathered sediments of the Yazoo Formation are mostly homogenous fat clays with widely scattered thin zones of bentonitic silty clays. The deposits are generally dark gray to blue in color and contain fossil shell fragments. The sediments exhibit a very low permeability and are considered to be very strong with a low compressibility.

Typical Undisturbed Sample of Yazoo Clay

The underlying Moody's Branch Formation may be divided into upper hard slightly cemented silty clays with lignite nodules, fossil fragments, glauconitic sands, and a lower layer of very dense silty sands with fossil fragments. The silty sands are slightly clayey with glauconite, trace lignite fragments and occasional clay seams. The upper layer silty clays exhibit relatively low permeabilities and are considered to be very strong with a low compressibility. The Moody's Branch formation overlies the Eocene Age Cockfield Formation of the Claiborne Group.

Seismic Setting

The site is located approximately 200 miles south of New Madrid, Missouri. The New Madrid region has been the most seismically active region of central and eastern North America. The events of the winter of 1811-1812 are well documented, and have been the subject of a significant volume of research over the years. Nuttli's study of the damage and felt effects of these events indicates that the surface wave magnitudes were on the order of M_s 8.5. Other studies have reached similar conclusions regarding the magnitude of that series of events.



General Plan and Profile

Between 1813 and 1990 over 23 earthquakes having magnitudes of 4.5 or greater have been documented in the New Madrid area. Considering that M_s 8 or larger events are anticipated every 550 to 1,200 years, the design earthquake is essentially a repeat of the 1811 and 1812 events.

The seismic evaluation tasks to date include evaluation of existing seismic data. field work, evaluation of fault rupture potential, level ground liquefaction, the timing of liquefaction occurrence, earthquake-induced settlements, lateral spreading, and seismic slope stability and flow failure.

Seismic Field Work

The field work consisted of performing downhole shear wave velocity measurements in four boreholes, cone penetration tests with porewater pressure measurements (CPTU) soundings at 36 locations in preliminary phase, 18 locations final phase, and seismic shear wave velocities measured during five soundings.

Liquefaction – Lateral Spreading

Level ground liquefaction analyses was conducted using the SPT and CPT-based procedures suggested by NCEER (1997). The analyses indicated sporadic to extensive risk of liquefaction ranging from approximately 30 to 75 feet below the ground surface. The Arkansas riverbank is subject to lateral spreading as a result of the free-face condition and the presence of recently deposited relatively loose sands.

Geotechnical Investigation

Preliminary and final exploration programs were undertaken for the project. The preliminary program was intended to generally characterize the subsurface, identify the abandoned channels filled with soft clay, profile the top of tertiary bedrock, as well as establish seismic and scour parameters.

Preliminary Investigation

The preliminary investigation would establish seismic parameters for liquefaction potential, lateral spreading and embankment stability. Embankment stability and height had to be established to locate the bridge ends. The investigation also established the scour parameters. In this environment scour typically controls the foundation design. Within the present channel, the 500 year flood event scours all the alluvium and 50 feet into the Tertiary bedrock. The river foundations would have to be well below elev -20, a depth of 150 below the normal river level.

Dredged caissons are the typical foundations for the towers and approach piers in this type of subsurface environment. The preliminary investigation would evaluate practical constructability, and develop bearing capacities for foundation costs which would be included in the economic span length studies. Conceptual bridge type studies were undertaken in conjunction with the preliminary geotechnical program and would result in final span type, sizes and locations.

Preliminary investigations consisting of widely spaced borings over the 4 mile long project. Cone penetration and seismic shear velocity testing were also included in the initial investigations. A series of 20 borings were drilled at approximately 1000 foot spacing along the project alignment. No borings were drilled from the river. All preliminary borings penetrated the alluvium which ranged from 80 to 140 feet thick and up to an additional 140 feet into the Tertiary materials. Total boring depths ranged from 50 to 220 feet.

The borings were drilled with Failing 1500 drill rigs utilizing mud rotary methods. Sampling generally consisted of 3-inch thin wall tubes in the cohesive top stratum, SPT's every 5 feet in the granular alluvium, and 3-inch thick wall tubes in the cohesive Tertiary sediments. In order to obtain additional data for earthquake engineering, four-inch PVC casing was grouted to 200 feet deep at 4 boring locations to allow for subsequent geophysical seismic velocity testing.

The geophysical testing involved conventional down-hole geophysical tests to evaluate the dynamic properties of the soils for site response analysis. Along with the seismic velocity testing, a series of 36 Cone Penetration Test Soundings (CPT's) with porewater pressure measurements were also performed.

A groundwater study was also undertaken for the project. Seven open observation wells were installed at key points along the 4 mile long project. Readings were taken at regular intervals over the course of approximately 2 years. The groundwater study will assist in the need and development of foundation requirements and the use of seal courses.

Source and recharge of the alluvial groundwater is almost entirely from the Mississippi River. The groundwater table fluctuates directly with river levels as there is a direct interchange between the river and the alluvial groundwater. There is a delayed response of only a few days between higher river levels and higher groundwater levels. Recharge is more by way of the river than by percolation of rain and surface water.

Typical Drill Set Up

Final Investigations
The conceptual design report combined the results of the studies of all the disciplines involved with selecting the most economic, constructible bridge. A cable stayed bridge was selected for the main river crossing. Trestle and approach spans would either be steel girder or precast concrete girders.

The main span and west four approach piers will be supported by dredged caisson foundations. The approach, low level trestle type bridge will be supported by either driven piles, or drilled shafts with seismic factors heavily influencing foundation type, size and depth. The final design geotechnical investigation would determine dredged caisson bearing capacities, and foundation type of other piers based on subsurface conditions, earthquake and scour parameters.

The final design bridge type consists of 122 substructure locations. It is anticipated the 5 foundations in or near the river would consist of dredged caissons with estimated construction cost of $20 to $30 million each. Multiple borings were planned at these locations. These foundations were to be excavated to over 200 feet below the ground and water elevation. Excavatability, possibility of obstructions, stability, and bearing would be investigated. Otherwise at the remaining 107 locations, single borings would be taken to investigate and provide the foundation and earthquake design parameters.

Using Pitcher Barrel Sampler

At one boring location on the Mississippi side bank the Yazoo clay material was continuously sampled to provide the future contractor information regarding means and difficulty of excavation. The boring was accomplished with a 4-inch triple tube core barrel and saw tooth carbide bit. Recovery was generally high providing a valuable representation of the of the subtleties of the subsurface.

Preserving Continuous Core

Water Borings

A total of 12 borings were planned to be taken from the water, 4 at the tower pier and 2 each at the other 4 pier locations in the water. Borings were planned to be drilled from a "jack-up" barge, a self contained barge which normally works the offshore oil rigs in the Gulf of Mexico. The boat was capable of supporting 24 persons 24 hours per day 7 days per week. The work crew consisted of two captains/pilots, deckhand, cook, 2 – 5 man drilling crews, 2 geologists, and two supervisor personnel. A support tug and barge provided drilling mud storage and aid in maneuvering against the swift current. Including the support tug and survey crew the water borings enlisted a total of 23 personnel. The drill boat had a working deck and drill hole where a Failing Model 1500 was placed.

"Jack-Up" Barge

Borings were located using GPS methods and drilled utilizing methods typical of the offshore petroleum industry, advanced with 4-inch I.D. heavy wall API drill pipe and approximately 4 tons of drill collar to stabilize the drill string in the strong current and advance the hole. No casing was used. Drilling mud was mixed and expended throughout the drilling process.

Samples were taken with an underwater wireline split spoon sampling system. The underwater hammer system consisted of a 175 pound weight sliding along a 5 foot long rod. Standard Penetration Tests using conventional rods with 140 lb. hammer and automatic hammer above deck were taken to correlate the underwater wireline system. Three inch undisturbed samples were taken using conventional push techniques. Two inch undisturbed samples were also taken using the weight of the drill string or the underwater hammer system.

Drill Rig and mud pit

Drill bit, rods and underwater hammer

Running the Underwater Wireline Penetration Test

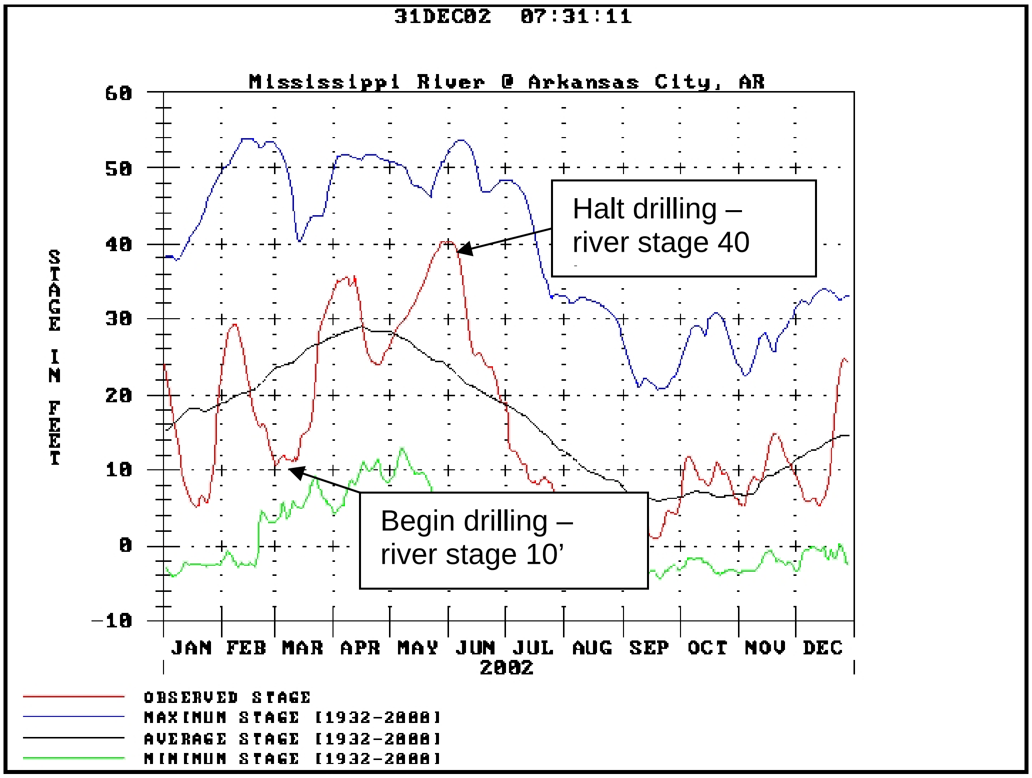
The water based drilling was not without problems. Instances occurred where it was difficult to maintain an open boring with the "uncased" process. The self propelled jack-up barge also experienced difficulty maneuvering against the swift river currents when the legs were lowered.

A rapidly rising river, swift currents, and high water also caused havoc with the drilling operations. At the time the borings began during mid March 2002, the local river stage was at 10 feet, some 12 feet below the average stage for the date. Heavy spring rains in the Missouri, Mississippi and Ohio river basins caused the river to rise 16 feet over the course of the next two weeks. The drill barge was unable to move on to new boring locations, failing to overcome the strong currents, even with the assistance of the service tug.

Swift currents also hindered the ability to set the rods to the bottom and maintain plumbness in the now 70 feet deep water even with the 4 tons of drill collar at the lead. At this point operations ceased and crews demobilized, the drill barge was sent to safe harbor in Greenville, MS. While occurring steep daily overhead, the drilling operation waited for the river to ebb.

Over the course of the next two weeks the river did fall back about 10 feet at which point drilling operations resumed. However, soon after the operations resumed, rains had the river on the rise once again. The remaining boring locations were all toward the middle of the channel in the swift, 70 feet deep water. Over the course of the last two weeks of

April 2002, the river rose to a stage of 40 feet, 30 above the stage at the onset of operations in mid March, 16 feet above the mean average for the date, and 3 feet above flood stage. When the jack-up barge attempted to set on location, severe scour under the legs caused an unstable, shifting, platform. The river was now against the levees and 4 miles wide. The drill barge personnel declared the conditions unsafe and a decision was made to demobilize the operation without completing the borings for the mid channel tower of the bridge.



Mississippi River Hydrograph – 2003 - USACE

Water borings resumed in September of 2002 with a river stage near 0, some 40 feet lower than when operations ceased in April. This time the borings were undertaken with more conventional methods. A “spud” barge with spuds holding the barge on location was utilized. Since the overburden had been characterized in the previous operations, a 16-inch casing was lowered to the mud line, a vibratory hammer then set the casing to practical refusal on the hard Tertiary clay materials. A large pump and jet pipe was then used to clean the granular materials from the casing. An additional 6-inch casing was now set, and the drilling proceeded using conventional rotary drilling methods with fishtail bits. The sampling consisted of Standard Penetration Tests and 3 inch undisturbed thick wall tubes extruded in the field and preserved in cartons sealed with wax.

East Bank – River Stage 10’

East Bank – River Stage 30’

Vibrating 16 inch casing


Water Jet Used to Flush Casing


Conventional Drilling – Second Water Attempt

Summary
The Great River Project involved many aspects of geotechnical investigation including literature and existing data research, conventional drilling, sampling and testing using a variety of techniques and samplers, ground water study, seismic studies involving shear wave velocity, and cone penetration testing.
It also involved drilling over water in deep swift current.

In all, the program cost approximately $4.5 million and included 7 firms and various subcontractors and took 2 years to complete. Many different methods of drilling and sampling were attempted with various degree of success.

Acknowledgements

Owners
Arkansas Highway and Transportation Department
Mississippi Department of Transportation

Geotechnical Subconsultants
Burns Cooley and Dennis – Jackson, Mississippi
Geotechnical Services Inc. – Omaha, Nebraska
Geotest Inc. – Houston, Texas
Haley and Aldrich – Boston, Massachusetts
McClelland Engineers – Fayetteville, Arkansas
URS Corp – St. Louis, MO

Others
Mr. Graham Davis
Mr. Wayne Duryee
Mr. David Knudsen
Mr. Kevin Kuhl
Superior Energy Services – Venice, Louisiana
Miller Construction – Vicksburg, Mississippi

Economic Benefits of Seismic Refraction Investigations for Road-cut Design Studies

Kitty Breskin, P.E., Maine DOT
Paul Fisk, NDT Corporation

Abstract:

Subsurface investigations for highway design projects must provide data of varying type and quality at different stages of the design process. A program of drilling and test pits is the most common method of collecting subsurface information but it may not be the most efficient way to gather this data. Seismic refraction coupled with a limited drilling program and a small number of test pits can provide an accurate, cost-effective way to gather subsurface data. The availability of this information in early stages of a highway design project is critical, so that informed decisions can be made between different possible alignments.

A combination of test pits and/or drilled subsurface investigations with geophysical seismic refraction surveys have yielded accurate subsurface information at different stages of design for a number of Highway projects designed by MDOT. Soil stratification, depth to ledge and degree of fracturing were provided by seismic refraction at closely spaced intervals and confirmed by a limited program of drilling or test pits in easily accessible locations. The information was more reliable than when only drilled data were used, and obtained at a fraction of the cost of a conventional program of closely spaced borings. Case histories will be presented documenting the successful application of seismic refraction techniques to provide top of bedrock information for road cut design studies. Emphasis will be on the accuracy of the results and the monetary savings.

The role of the subsurface investigation in the highway design process

In many rural highway projects there is some question at the preliminary design stage of whether and how the alignment should be changed for safety improvements. Unlike in private sector development, the State will acquire Right-of-Way to build a road, and so the principal constraints of alignment choice are cost and impacts to wetlands or historic properties. There are generally many possible alternate alignments, and the most cost-effective choice isn't clear until some subsurface information is available. At this stage of design, rough subsurface data is needed so that preliminary decisions can be made. This data will be supplemented in the final design, and it need not be precise.

In Maine the data generally used in the preliminary design phase of a highway project are resource maps and as-built plans if available, with discussions with Maintenance personnel and local residents. Raw deflections from the FWD test may be used to get a rough idea of the weak areas of the pavement structure or parts of the roadway which are likely to have shallow ledge. In past practice, the final alignment has been chosen with no other site-specific data for large highway projects, due to the high cost of off-road investigations and the awkwardness of drilling on properties that may not be acquired for the construction. In the past several years, Maine

DOT has moved toward adding seismic refraction studies before the final alignment is chosen in areas where significant highway realignment or deep side-hill cuts are proposed.
Seismic refraction analysis allows the engineering design team to determine a very rough estimate of rock excavation quantity and cost as compared to the cost of, for example, adding retaining walls or relocating a property owner on an alternate alignment. These decisions can be made based on actual site-specific data to ensure that tax money is spent in the most efficient manner. Areas of organic or saturated soils can also be located and a decision made on whether to design for them or choose an alternate alignment.

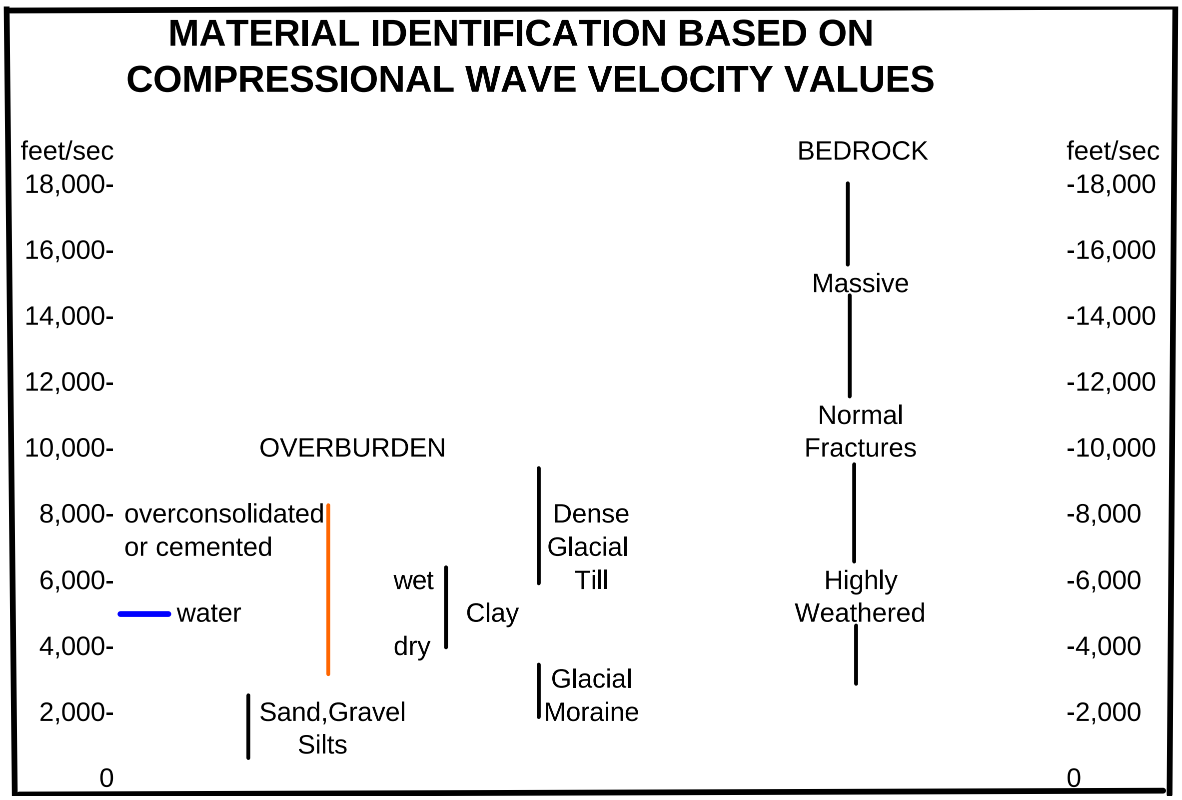
In final design of a highway project, the data needed is more specific, and borings or test pits are required to determine soils characteristics for the design of pavement structures, retaining walls, large culvert foundations, and slope stabilization measures. The seismic refraction data can be used to limit the number of borings or test pits required to identify specific soils while obtaining more continuous general subsurface information.

The refraction data will indicate the lateral extent and depth of a soil type where there has been a change. In Maine we have large areas of glacially deposited clay-silt soils existing as layers and lenses within granular layers. A layer of saturated soil that exits into a slope can reasonably be interpreted as a clay-silt, and the refraction study will show the lateral extent and thickness of this layer for design purposes. Interpretation of seismic refraction data by an engineering geologist can indicate strata of saturated marine clay-silt or heavily fractured ledge so that conservative slopes can be designed in these areas and adequate ROW acquired during the design phase of a project. Drill refusals may indicate either a large boulder or shallow ledge, and in some areas of Maine the diameter of many boulders exceeds 2 meters. A number of borings and test pits will still be required for any highway realignment project, but they are used to determine specific soil characteristics, where the refraction study is used to show the lateral extent of each general soil type.

Raw deflections from an FWD test can indicate areas of shallow ledge under an existing paved roadway, but other conditions may cause an extremely flat reading. Refraction will show the extent and approximate depth to ledge both for cost estimation and design purposes. It will also show the degree of fracturing so the engineer will be able to estimate ledge to be ripped as compared to ledge blasting for cost estimation. If heavily fractured ledge is indicated in a side slope, borings will be needed to determine the orientation of the joints with respect to the cut line to determine the clear zone required for rock falls.

Uncertainty in Subsurface Investigations

The seismic refraction survey method has several inherent limitations that affect the accuracy of the results. In general, depth calculations should be expected to be within 10% of the actual depth except in very shallow (less than 15 feet) bedrock conditions. In shallow bedrock conditions errors can be greater due to rapid changes in bedrock topography between sensors or localized overburden velocity variations. Other limitations include: as depth of investigation and geophone spacing increase, resolution decreases; thin layers may not be detected; velocity inversion add uncertainties to depth calculations; weathered bedrock and consolidated or cemented materials have similar velocity values. (The attached table relates measured compressional wave velocity values with general material identifications.) Knowledge of the local geology improves the accuracy of data interpretations.



New England geology is highly variable with changes in bedrock and overburden types and properties in relatively short distances. Bedrock topography can vary from outcropping to hundreds of feet deep in pre-glacial valleys, and can be highly competent with little weathering or variable, highly weathered and fractured. Overburden materials are as variable, ranging from loose sands and peats to very dense, overconsolidated glacial tills. Overburden materials can be saturated or can have perched water tables. Overburden can have areas of nested boulders.

In past practice, Maine DOT tried to design and build highway projects within a single budget cycle. This didn't work well, due to the difficulty in determining scope and budget for a project in the absence of public contact and subsurface data. Scopes and budgets tended to creep well beyond the programmed value of the project. The current practice is to design a project within one budget cycle and build it the next so that engineering design is not constrained by an inadequate budget. The subsurface investigation must therefore be accomplished years before the project will be built. Where data must be obtained under sideslopes or in people's drives or lawns, it is preferable to limit the intrusion and disturbance required. Getting a drill rig up a steep slope or doing borings or test pits in an abutting property owner's front yard is a serious intrusion. Particularly during an alignment study when the property in question may never be acquired, a subsurface investigation program should be designed to minimize the impacts to abutting property owners.

It is difficult and expensive to get large equipment to some areas. There are limits to the slopes that a drill rig or excavator can climb, and in wooded areas it is time consuming to remove trees to create a path. In this case, seismic refraction is far superior to any drilling program both in impacts and in cost.

Seismic refraction surveys offer many advantages in acquiring subsurface information away from an existing roadway. Seismic refraction equipment is very portable and can be easily hand carried into densely wooded, topographically extreme areas without roads, without the need for tree removal. Seismic data can also be acquired across rivers, swamps and lakes by using hydrophone sensors. Seismic refraction data provides a continuous top of bedrock and soil stratification profile which allows identification of bedrock valleys, shallow bedrock or shear zones that might be easily missed by a convention drilling program.

Maine's Experience

Extensive use of seismic refraction at the preliminary alignment stage is a new process for Maine DOT. Geotechnical engineers in Maine have embraced the technique as a way to avoid the difficulties that come in construction when final design is done using inadequate subsurface information.

Maine DOT has used this technique in the past to collect off-site data during final design. This has been very useful, particularly in areas where large boulders in the soil matrix are common, to determine which refusals are rock and which are ledge.

Realignment, widening for paved shoulders, and ditching was needed on a project on Route 4 in Avon. Boulders with a diameter of two to four meters were common within the soil matrix and on the surface. An extensive boring program through the roadway and up steep slopes on the south side of the road showed many locations with drill refusals at shallow depth, but Maine DOT engineers could not assume that refusal indicated top of bedrock for design purposes. A seismic refraction analysis demonstrated which of the many refusals were boulders and which were ledge refusal. This knowledge enabled MDOT to design stable side slopes, and acquire the Right-of-Way necessary to build the project. A project on Route 11 in Auburn where the highway alignment was constrained by a river to the right, with outcrops and large boulders visible in a slope to the left, required that the roadway be widened to include a paved shoulder and drainage. A reasonably accurate profile of the top of rock led designers to use of a shallow ditch and a shallow tire-shred French drain as underdrain to avoid a cut that would have left a thin, unstable layer of soil on a steeply sloping bedrock surface, and extensive ledge cut for traditional underdrain.

In some recent projects, unanticipated subsurface conditions encountered during construction caused serious erosion, permitting and Right of Way acquisition problems. On a project where a limited off-site boring and soundings exploration program was done due to the difficulty of reaching the area over privately owned land, two borings indicated relatively competent igneous rock, and hand soundings every 10 meters profiled the bedrock surface. A ¼h:1v rock slope was designed for this area. As the construction blasting progressed, the construction Resident discovered that both borings were in isolated pockets of igneous intrusion in an area of heavily fractured mica-schist, with joints dipping into the cut. MDOT is in the process of acquiring additional ROW to lay this slope back for stability and to build a new septic system for an abutting property owner whose system was affected by this cut. This type of experience has overcome any lingering doubts that some MDOT designers had about the need for early, continuous off-site data.

These problems during construction can be avoided when appropriate information is available during the preliminary alignment phase of a project. Maine has several projects in early design now, where seismic refraction data is being used to set the alignment. For projects in environmentally sensitive areas, or where public acceptance is a significant factor in the choice of a new alignment, it is critical that a complete range of data be available to the designer in the early stages of a project to prevent problems during construction.

A project in Madrid requires relocation of Maine Route 4 at two curves: where the highway crosses the Appalachian Trail, and at a horseshoe corner bounded by a class-A stream and a high bedrock knob. Realignment is needed both for traffic safety and to move the highway away from a failing slope. The roadway must be relocated through terrain that is densely wooded and extremely steep. The local scenic byways committee does not support the change with enthusiasm, and the project is politically challenging. On this project, a preliminary alignment was roughed in using the 20 foot contours on the USGS topographic quadrangle for this area. MDOT survey collected topographic data for 30 meters on each side of this line. The survey crew also staked out seismic lines nine meters to the right and to the left of this line using GPS units, and seismic refraction analysis of the proposed alignment was done concurrently with the survey. When the survey data came into the office, profiles through the seismic lines was sent to NDT and the top of ledge and all available soils information was plotted on the profile. An input file was created from this top-of-rock data for direct use as a below-grade surface in the MDOT highway alignment software program. This allows the design team to make the best possible alignment decisions based on reasonably accurate off-site data, obtained at a minimal cost, through terrain that precluded the use of conventional drilling equipment. This data will also aid in explaining the design constraints and costs to the public.

A project on Route 27 in Coburn Gore required relocation of the highway to flatten dangerous curves where the existing roadway was bounded by a Class-A water body on the left and by steep ledge-and-boulder cliffs to the right. A seismic survey was done as the alignment was being chosen, to ensure that the most cost effective option was chosen. Test pits were dug at the ends of the seismic lines to confirm the soil types and demonstrate the average boulder size. It would have been either difficult or impossible to get borings in these areas

A project on Route 2 in Dixfield requires some realignment of several short sections of highway to flatten curves through hilly terrain, where the roadway is already bounded by steep slopes, and FWD indicated that ledge or boulders are shallow under the existing roadway. It would have been possible to check for subsurface ledge in the preliminary alignment using a drill rig, but widely spaced borings convey only limited information in glacial topography, as the borings can easily miss changes in subsurface conditions. We chose to do a seismic survey of the areas where deep cuts were proposed to obtain continuous data, so that the final alignment choice was based on good information. Some drilling will be required in the final design to determine rock quality and fracture orientation in the ledge cuts, but these borings can be targeted to specific locations, and done when the abutting property owner knows that the State will need to acquire the land.

Costs

Seismic refraction is an efficient and cost effective technique to profile top of rock for highway alignment studies. Drilling is an effective way to collect data at specific points under roadways, but it is risky to interpolate between widely spaced points in glacially deposited terrain. Many borings or probes are needed to collect drilled off-site data for preliminary design that can be more easily obtained using geophysical methods.

The Maine DOT drill crew costs approximately $1000/day including drill rig, driller, inspector, driller assistant and flaggers. The number of borings that can be completed in a day depends on the depth of the borings, and the difficulty of getting there. A shallow probe under the roadway or shoulder will take at least 20 minutes from start to finish. If soils information is needed, the shallow boring is likely to take 45 minutes to an hour. If the boring must be deeper or is more difficult to get the rig to it, it is easy to see how the cost of drilled information can become quite high.

On two recent projects where substantial lengths of entirely new roadway were constructed, drilled subsurface investigations have cost approximately $30,000 per mile for drilling only. These costs do not include the drill inspector or interpretation of the soils data. The borings for these projects were in widely spaced clusters, and the investigations left the designers with very little information between the borings. Construction problems have included unanticipated areas of deep peat and serious erosion from unstable cut slopes in clay-silts of the Presumpscot Formation. For the project in Madrid mentioned above, the preliminary subsurface investigation included 0.9 miles of seismic refraction analysis. This investigation gave the designer a continuous bedrock surface 0.89 miles long and 60 feet wide for the entire length of the offsite project work. The average slope of the existing ground is approximately 15% in this area although many areas are much steeper, and the land is densely wooded. Deep cuts are anticipated. It would have been difficult or impossible to get a drill rig to the site. The seismic analysis provided a continuous bedrock surface profile in addition to preliminary information on rock quality and overburden type. This investigation cost $28,800 for the seismic survey and the geophysical analysis. The data report included a spreadsheet that was converted directly to an input file for use in the highway design program used at Maine DOT.

Geotechnical engineers at Maine DOT are moving toward using seismic refraction analysis in preliminary highway alignment studies, to be supplemented by a limited program of borings and test pits during final design. This improves the quality of data provided by our subsurface investigations while reducing the total cost.

Rock bolting
Perspectives from a State DOT

Marc Fish, PG
New Hampshire Department of Transportation
Bureau of Materials & Research
Stickney Avenue
Concord, NH 03302-0483
603-271-3151
mfish@dot.state.nh.us

ABSTRACT

Within a construction project, other construction activities often dominate over the remediation of a rock cut. This occurs because the rock cut remediation portion of the project is usually one of the lesser expenses on the job. For this reason a general contractor is awarded the project and then one or two subcontractors are hired by the general contractor to remediate the rock cut. As a contingency item, rock bolts are included in the scope of the job with a condition that the Department will make a final determination as to weather rock bolts are needed after the face of the rock cut has been excavated. Once the decision to install rock bolts has been made, things start to happen rather quickly. Winter is often just around the corner and the general contractor and the DOT become eager to get the job completed prior to the first snow. Before rock bolting is started, the subcontractor usually submits an installation plan, describing how they intend to meet the job's engineering design criteria. Unfortunately, it is not possible to cover every conceivable scenario within the plan. In addition, the DOT often has ideas contrary to that of the subcontractor on how the job should be done. A balance must be reached whereby the state DOT does not have to take ownership of the subcontractors plan and the subcontractor has enough freedom to get the job done their way. Several items must be decided prior to the start of work, which include: Is the subcontractor qualified to install the rock bolts? Have they installed rock bolts in the past? Will their experienced personnel be on the job site? Can the subcontractor install the bolts according to the engineering design criteria? Additionally, items must be addressed in the field, as they appear, which include: What is the condition of the rock on the face of the cut? Will concrete pads be needed to place the bearing plates upon? Can the rock bolt holes be completely filled with grout? Within the rock bolt holes, will artesian water conditions prevail? And will the rock bolts be loaded properly?

INTRODUCTION

The installation of pre-stressed rock bolts is a procedure, which requires a substantial amount of skill and inspection. After the bolts have been installed it is not possible for an inspector to determine if they were installed correctly. This is due to the fact, that only the ends of the bolts are exposed on the face of the cut. The depth of the bolt, the grout coverage, the lengths of the bonded and un-bonded zones, and the construction of the bolt itself cannot be observed. The angle at which the bolt was installed is the only thing that can be checked with any confidence. If the end of a bolt were exposed, it would be possible to check the load that was placed on the bolt by conducting a lift-off test. To conduct a lift-off test, traffic control measures must be initiated to gain access to the cut. A man lift or a crane and a hydraulic jack must be mobilized to lift an inspector to the end of the bolt. The test will turn into a major undertaking if it is done after the completion of the project. For these reasons, it is critical that an inspection occurs while the bolts are being installed.

NEW HAMPSHIRE'S REGIONAL GEOLOGY

New Hampshire is known as the granite state, not so much because of the amount of granite within the state but because there is a substantial amount of hard shallow bedrock. Sedimentary rocks do not exist anywhere within the state. The bedrock geology in the state is exclusively plutonic and metamorphic in nature with a little bit of volcanics (figure 1). Bedding planes are practically non-existent and folding is extremely common. The Connecticut River valley has some of the most highly folded rocks within the state where phyllites and schists are the dominant rock types.

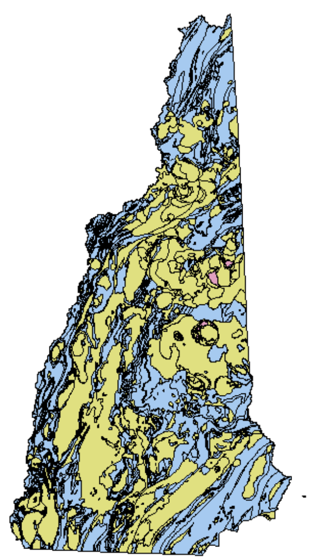


***Fig. 1**: New Hampshire's simplified bedrock geology map. Olive is Plutonic, Blue is Metamorphic and Red is Volcanic.*

NEW HAMPSHIRE'S ROCK BOLTING HISTORY

The bolting of rock cuts began in New Hampshire in the early 1970's with the placement of more than 100 rock bolts and 70 rock tendons in the Barron Mountain rock cut after a major slope failure. Since this time, several different contractors have installed approximately 150 pre-stressed rock bolts throughout the state. These bolts are found within roadway rock cuts in the towns of Woodstock, Manchester, Albany, Pinkham's Grant, Hinsdale and Harts Location.

PROJECT INITIATION

In New Hampshire, rock cuts are typically remediated in one of three ways: as part of a resurfacing project, an emergency slope stabilization project, or part of a 4R safety improvement project. Because these projects are multi-facetted (i.e. roadway resurfacing, realignments and improvements as well as guardrail, ditch and drainage work) they are awarded to a general contractor. The actual rockwork, including the rock bolting, is subcontracted out to one or several subcontractors. When it comes to the overall cost of the project, the cost for rock bolting is actually quite small. Rock bolts are included within the scope of the construction contract only as a contingency item. The project geologist decides whether rock bolting will be required once the cutting back or scaling of the rock slope has been completed. Because rock bolting is one of the last items to be completed on the project, time becomes a concern. Either the onset of winter is just a few weeks away or the general contractor becomes anxious to finish the project on time.

ROCK BOLT INSTALLATION PLAN

The initial step in the rock bolting process is the submittal of a rock bolt installation plan. The construction contract requires the subcontractor to submit his plan to the general contractor who then submits the plan to the project administer who then forwards the plan onto the project geologist. A typical installation plan has many pages and is broken up into several parts.

1. Specifications
2. Drawings
3. Certificates
4. Testing equipment and procedures
5. Project resume

Usually, only a few pages contain specific information about the installation procedures along with the subcontractors project resume. The remaining pages are just photocopies of equipment specifications, shop drawings and equipment operation procedures. Once the project geologist receives the plan it is reviewed for completeness and whether the bolts can be installed according to the installation plan and the engineering design criteria. The installation plan must contain enough information to allow the project geologist to determine if the subcontractor is qualified and capable of installing the rock bolts. He must be able to determine if the subcontractor has installed rock bolts of this type in the past and whether their experienced personnel will be on the job site. After a completed review, a memo is written to the project engineer describing the plan's deficiencies. The project engineer delivers the memo to the general contractor who forwards it on to the subcontractor. At this point, a meeting or a phone call is made between the subcontractor and the project geologist to discuss the plan's deficiencies. Adjustments are made and the submittal procedure is then repeated. This is a time consuming process with a number of steps taking a certain amount of time to accomplish. It is important to reach a balance, whereby the DOT does not have to take ownership of the contractor's plan, by telling the subcontractor how to do the job, and the subcontractor has enough freedom to get the job done their way. In reaching this balance, the DOT is limiting its liability to any future construction claims.

<u>THE INSTALLATION OF THE ROCK BOLTS</u>

Unfortunately, it is not an easy task for the subcontractor to follow the installation plan. During construction, unforeseen or different rock conditions might be encountered requiring changes to be made to the installation plan. It is not until work actually begins that the project geologist gets his first up-close look at the slope from a crane or man-lift (Figures 2-4).

Fig. 2: *Inspecting rock cut from a platform suspended by a crane*

Fig. 3: *Inspecting rock cut with a man lift*

It is possible to encounter poor rock conditions at the locations chosen for the rock bolts to be installed. This could be due to severe weathering and fracturing that was not observed earlier from either the top or the bottom of the cut. Because of the drill rig's or the drilling platform's limitations, the angles between the rock on the face of the cut and the bearing plate might be high enough to make the drilling and bolting difficult or impossible. If this is the case, either new locations for the bolts must be found, concrete pads must be built or the rock on the face of the cut must be chipped away to reduce these angles.

The angles at which the driller is to drill the holes is not an easy task for either the project

Fig. 4*: Checking the quality of the rock with rock hammer*

Fig. 5*: Marking location for the rock bolt*

geologist or the driller to measure. The drilling platform and the drill rig are metallic, which adversely affect the needle on a compass. For this reason, compass measurements need to be taken from a distance (Figures 6 & 7).

Fig. 6*: Measuring the dip of the proposed rock bolt hole*

Fig. 7*: Measuring the strike of the proposed rock bolt hole*

Sometimes a bolt hole will intersect a water-bearing seam and artesian water conditions might prevail. If this occurs, the hole must be abandoned and drilled elsewhere, or grouted and re-drilled until the artesian water conditions no longer exist. Unfortunately, the artesian water conditions might not be observed until after the bolt has been grouted in the hole (figure 8).

If the angles at which the bolt holes were drilled are not perpendicular to the face of the cut, concrete pads might have to be built to support the bearing plates (figure 9). Sometimes these pads are relatively thin and help to smooth the surface of the rock face. Other times these pads are thick and are made with forms and reinforcing steel. Weather conditions can play an integral part with the making of these pads. Both cold and hot weather inhibits the proper curing of the concrete. There is also a concern as to the long-term durability of these concrete pads. If the pads deteriorate, the bolt will lose its load and become ineffective.

Fig. 8*: Ice developing from artesian water conditions*

Fig. 9*: Concrete pad built for bearing plate*

As an alternative to building the concrete pads, a subcontractor might choose to chip out the rock surrounding the bolt. This will allow the bearing plate to fit perpendicular to the bolt and be secure against the rock face (figure 10). If the rock is hard, this is a difficult and time consuming process. If the rock is not chipped out or concrete pads are not built, wedges or beveled washers can be used if the angles between the rock bolt and the face of the cut are small (figure 11). If the angles between the rock bolt and the face of the cut are large, the use of wedges or beveled washers could actually cause the bolt to shear-off or they could make it difficult to impart the proper load upon the bolt. Because

Fig. 10*: Chipping rock away from around rock bolt*

Fig. 11*: Wedges & washers making up differences in angles & rust on the bolt*

these wedges or beveled washers are made of hardened iron they can sometimes cause the ends of the bolts to rust (figure 11). To inhibit rusting, the bolt ends need to be completely encapsulated with epoxy resin or painted.

Grout coverage is another item that is difficult to determine. The engineering design criteria calls for complete grout coverage around the entire length of the bolt, but this is extremely difficult to measure. The grouting operation is usually terminated when grout flows from the top of the hole. Even though adjacent bolt holes may have been drilled to the same depths, because of seams and fractures within the rock itself, some of the holes may require only a few bags of cement while others may require more.

When it comes to drilling the bolt holes and the placing and grouting of the bolts, some subcontractors will use unique approaches. Cranes can be used to suspend large drill rigs over the face of the cut, while a man lift can be used to raise the driller up to the drill rig (figures 12-14). After the holes have been drilled, the crane can lift the rock bolts into the air while the subcontractor guides the bolts into each hole from a man lift (figure 15).

Fig. 12*: Large drill rig to be lifted by crane*

Fig. 13*: Large drill rig is suspended by crane to drill bolt holes*

Fig. 14*: The drillers operate the drill rig from a man lift positioned next to the drill rig*

Fig. 15*: Rock bolts are inserted using a crane & man lift*

As an alternative to lifting the drill rig with a crane, a drilling platform could be used to place the drill rig and the drillers upon. A crane could then suspend the platform over the face of the cut allowing the drill rig to drill the bolt holes. The platform could also be used to place the bolts into the holes (figures 15-18). The bolts can be grouted after they have been placed into their respective holes through a grout tube. This can be accomplished by using a grout pump stationed on the ground below each bolt. To attach the end of the grout tube to the grout pump a man lift or the drilling platform can be used to access the end of the bolt (figure 19).

***Fig. 16**: Large drill rig placed on drilling platform*

***Fig. 17**: Drill rig on drilling platform suspended by crane*

***Fig. 18**: Inserting rock bolts from suspended platform*

***Fig. 19**: Grouting bolt after insertion into hole*

Instead of using a large drill rig, another approach might be to use a small drill rig to drill the holes (figure 20). The drill rig is powered by a separate compressor, which can be located at some distance away (figure 21). An advantage with this set up is that the boom of the drill rig can be removed from the remainder of the rig and placed onto the drilling platform (figure 22). This provides for greater room on the drilling platform as well as less weight to suspend from the crane. Once the holes have been drilled, the platform can be used to insert the bolts into the holes (figure 23). If the bolts are inclined downward, they can also be grouted in a different sequence. Instead of inserting the bolts into their holes prior to grouting, grout can be initially placed into the holes followed by the rock bolts (figures 24).

Fig. 20*: Small drill rig*

Fig. 21*: The compressor is separate from the drill rig*

Fig. 22*: The boom can be removed from the drill rig & placed on a platform & suspended by a crane*

Fig. 23*: Rock bolts inserted from a platform*

Fig. 24*: The holes can be grouted prior to inserting the bolts*

Once the bolts have been placed and grouted in the holes, cement pads have been made, or the rock has been chipped out around the bolt ends, loads can be placed on the bolts.

Obtaining the desired load on the rock bolts is extremely important. It is not possible for the inspector to look at an installed bolt and to determine if it has been loaded properly. For this reason, it is critical that the inspector observes the loading of each bolt and conducts an elongation and lift off test at that time (figures 25). When wedges are used, care must be taken to not pinch the bolt. When a bolt has been pinched it could possess a lesser load than that which was desired. When conducting the elongation test, an independent reference point must be used that remains stationary during the entire length of the test. A separate metal bar and a plate can be used that is positioned directly beneath the rock bolt (figure 26).

Fig. 25*: Stressing bolt with a hydraulic jack from a man lift*

Fig. 26*: Measuring bolt elongation using an independent reference point*

CONCLUDING REMARKS

The engineering design criteria as well as the installation plan needs to be somewhat flexible. The condition of the rock on the face of the cut might be different from that which was expected. This could require moving the bolts to different locations, requiring or eliminating wedges or concrete pads or changing the angles and depths to which the bolts are to be installed.

The subcontractor might change his personnel on the job site during the course of the project. The job foreman could be pulled off the job for a few days leaving instructions with the crew as to what is to be accomplished in his absence. If issues develop with regards to changes in the installation procedure or malfunctioning equipment no one with authority is available to make decisions on the subcontractor’s behalf. The general contractor and the DOT are hesitant to make decisions on the subcontractor’s behalf because they do not want to be held financially responsible for those decisions. This also holds true for telling the subcontractor how to get the job done. If the DOT or the general contractor tells the subcontractor how to accomplish the job, they are now responsible for paying the subcontractor for the work that they have instructed, regardless if the procedure works or not.

Inspecting the installation of the rock bolt is critical. If the installation is not done according to the engineering design criteria the risk of a slope failure is significantly increased. Without careful inspection, it is not possible to determine if the subcontractor has met the engineering design criteria. The DOT’s construction project administer usually does not have the time nor the experience to inspect the rock bolt installation. He is relying on the project geologist to make sure the subcontractor meets the contract specifications. He is also under pressure to get the job done, on time and within budget. Rock bolting is usually one of the final items to be finished on the project. For this reason the general contractor can place pressure on the subcontractor, the project geologist and the project administrator to sign off on the rock bolting phase of the project, so they can receive their final payment.

There is also a desire to use local subcontractors. Money is placed back into the local economy making the residents within the community happy. General contractors want to use a subcontractor that they are comfortable with as well as one that costs the least amount of money. Local subcontractors have an advantage when it comes to relationships with the general contractor as well as being able to provide the service for the lowest cost. Having a state licensed business as well as state licensed engineers also gives the local subcontractor an advantage. Politically, it is not easy to disqualify a local subcontractor from doing the job. Through their submitted installation plan and persistence, they can usually demonstrate that they can get the job done and meet the requirements of the contract specifications.

REFERENCES

1. NHDOT Standard Specifications for Road and Bridge Construction and Special Provisions, 2002.
2. Post-Tensioning Institute, Recommendations for Prestressed Rock and Soil Anchors
3. Lyons, J., Bothner, W., Moench, R., Thompson, J., Bedrock Geological Map of New Hampshire, US Geological Survey, Denver, CO, 1997.

SEVEN S'S OF GEOTECHNICAL DOOM

G. Michael Hager
Chief Engineering Geologist
WYDOT-GEOLOGY

ABSTRACT: I have discovered in my 26+ years in the field of Geotechnical engineering that there are guiding principles controlling our field of interest. I call them the Seven S's of Geotechnical Doom, since they influence everything we do. They are: Sleet\Snow, Slides, Settlement, Swelling Soils, Seismic, Silt & Clay, and last but not least, Scour, and don't forget those Twins; Shear Stress, Slope Stability and Soil Slopes. I will discuss each separately as to how and why they are the scourge of Geotechnical engineering.

1. **Sleet\Snow**: These are frozen members of water, which are the biggest curse in Geotechnical engineering. They show up everywhere creating Geotechnical problems. However, the controlling these Geotechnical problems has made some geo-product companies very inventive and rich. WYDOT alone has used or installed over 18 million square yards of these geo-products.

2. **Slides**: These are the bread and butter of all state highway geotech departments. They are also considered the most exciting types of investigations we work on. To be a **<u>Slidebuster</u>** is to be in a place of distinction. WYDOT is currently tracking 200 slides that have affected the Wyoming highway system.

3. **Settlement**: Highway Departments strive to build and maintain smooth pavement surfaces. The settlement of fills, pipe crossings, and bridge approaches are a constant headache in geotech departments. Settlement keeps these departments busy and under the gun to prevent problems before they happen and fix existing ones. WYDOT Geology has participated in a research project to investigate pipe settlements using clay soils for backfill.

4. **Swelling soils**: Wyoming has its share of bentonite rich clay shales which wreak havoc on the highways. WYDOT Geology has come up with a preventive solution which effectively encapsulates the foundation shales in an Impermeable Membrane, which prevents swelling.

5. **Seismic**: A.K.A. earthquakes, another substitute S. Earthquakes influence Geotechnical engineers in many ways. They cause landslides and force us into including seismic loads into our foundation and slide mitigation designs.

6. **Silt & Clay**: Every Geotechnical problem that can't be blamed on water surely has silt and clay involved. Check the list: poor compaction, settlement, frost heaves, collapsing soils, swelling soils. Mother Nature has cursed us with the final weathering products of the erosional cycle. The State of Wyoming has its share of these "bad boy" soils. One of the state's major economic mineral contributors is the sale of bentonite. There are three major bentonite mills in the state that should have been a clue to some geotechnical highway engineers. What would we do without Atterberg limits?

7. **Scour**: When Mother Nature thinks we are bored she sends a slug of water our way to see if our bridge foundations are deep enough. This happened to WYDOT on August 27, 2002, when a 500 year flood event hit one of our Interstate bridges, causing severe damage to the foundation of the N.B.L. of I-25, closing this lane for four months, until heroic efforts by all WYDOT parties involved got the bridge repaired and the road open. Lately, scour has been on the forefront of all DOT's menus in both the Bridge and Geotechnical programs/departments. In the last 13 years WYDOT Geology has investigated over 60 structures for scour potential. It's a national mantra.

SEVEN S'S OF GEOTECHNICAL DOOM

G. Michael Hager
Chief Engineering Geologist
WYDOT-GEOLOGY

I have discovered throughout my 26+ years in the field of geotechnical engineering that there are some guiding principles or terms that control our field of interest. I call them the 7 S's of Geotechnical Doom, since they seem to influence everything we do. They are: Sleet\Snow, Slides, Settlement, Swelling Soils, Seismic, Silt & Clay, and last but not least, Scour. I will discuss each separately as to how and why they are the scourge of geotechnical engineering.

In life we have the seven deadly sins (Lust, Gluttony, Greed, Sloth, Anger, Envy, and Pride). Pride is universally acknowledged as the worst of the Seven Deadly Sins since all the others stem from it. The same goes for water and the seven S's of Geotechnical Doom. For those of us employed by state DOT'S, we are really just asset managers. There are three main types of assets: 1) fixed - e.g. buildings, bridges, signs, or anything attached to the ground; 2) rolling - e.g. trucks, cars, drill rigs, etc.; 3) linear - e.g. highways, guardrails, bike paths, etc. As managers, we are charged with maintaining, upgrading and keeping these assets in a safe and usable condition. My Seven S's of Geotechnical Doom seem to hinder or control our work in many ways. The photos at the head of each Doom below are current members of the Geotechnical Engineering Hall of Fame (*G.E.H.F., 2003*). We also have the Seven Wonders of the Ancient World. Only the Cheops pyramid is still standing. They must have had a good foundation!

ACKNOWLEDGMENTS:
The author would like to thank, Greg Miller, Mark Falk, Holly Till, and Connie Fournier for their work in editing, reviewing and producing the graphics for this paper.


Karl von Terzahgi

1. Sleet\Snow: These, of course, are the frozen version of water, which is where I'm headed. Water is the ultimate problem child in our line of work. We are, unfortunately, inhabitants of the Big Blue Marble, Third Rock from the Sun, the water-covered Planet, Earth. Look at almost every problem area on a highway system and water will be the prime suspect. I try to emphasize to my geologists when they do a geotechnical investigation to detect the water. Where is it? What is the soil moisture? How much is there? Where is it coming from and going to? Solve these questions and then you will know the problem and how to fix it. Be a Water Detective is what I preach. The entrance to the University Of Wyoming Engineering Building has this Quote: "Strive on - the control of nature is won, not given." I say "The control of *water* is won, not given." However, getting rid of the water is half the battle. Geotextile manufacturers have made millions of dollars helping us control water. Since 1980, WYDOT alone has installed over 18 million square yards of geotextiles of various kinds. When they decided to go to the moon, NASA engineers were concerned about landing problems on the lunar soils. Karl von Terzahgi, the father of modern geotechnical engineering, was contacted about a million dollar plus research project to study the problem. Terzahgi told NASA he could save them the money if they knew if there was water on the moon. He basically told them "No Water! No Problems! Save your money." Extensive studies were conducted by the FHWA in 1973 to analyze the effects of water on and within road structures. The second of four conclusions states that "when excess water is retained within the pavement structure, rates of damage to the section increase." The most common landslide triggering mechanisms as are identified as intense rainfall, rapid snowmelt, changes in water level, volcanic eruption, and earthquake shaking (*Wieczorek, 1996).* It is worth noting that three of the five mechanisms are directly related to water.

Nilmar Jambu

2. Slides: These, of course, are the bread and butter of Highway Departments. They are also some of the most exciting types of investigations we work on. To be a "Slidebuster" is a title of great distinction. As I look back on my career in geotechnical engineering, 95% of what I've done or worked on are landslides that have affected the transportation system in one way or another. Even as I was recovering from 9½ hours of brain surgery after I suffered a stroke, my first phone call was from one of my staff to say a slide just occurred on one of our Interstates. My response was "Good Luck! Not much I can do from here." In 1993, on the day my predecessor, Gary Riedl ,retired, I answered a call from a District Engineer who wanted someone to come look at some potential slide cracks in the road. The next day I was flying to Sheridan, Wyoming, to look at these cracks. Silly me--12 days later the road was gone and we were called on to fix another in what was to become known as "Slide Alley" - 30 miles of Interstate 25 between Sheridan and Buffalo, Wyoming, built on and out of some of the worst soils in Wyoming. At least one major slide a year hit us until the drought took over in 1998. WYDOT alone is currently tracking 200 slides that have affected the Wyoming highway system.

This leads me into S #3 - the "BAD-BOY" soils, silt and clay. I t has been my observation that in all my years of fixing(100+) slides, at least in Wyoming the best and most economical fixes have involved Toe Berms in some fashion. The key to toe berms of course is having good site data both surficial and drill hole data. Knowing the geometry of the slide is critical in locating the toe berm, so that it counterbalances the driving forces. Getting rid of the water is also critical, but tends to be hit or miss and can be time consuming. We have tried many exotic slide remediation over the years including reticulated minipiles, soil nails, tie-back anchors and drilled shaft drainage curtains.

Arthur Casagrande

3. Silt & Clay: Every geotechnical problem that can't be blamed on water probably has silt or clay involved. Check the list: poor compaction, settlement, frost heaves, collapsing soils, and swelling soils. Mother Nature has cursed us with the final weathering products of the erosional cycle. The State of Wyoming has its share of these "bad boy" soils. One of the state's major economic mineral contributors is the sale of bentonite. Bentonite is a natural clay of the smectite family. Its platey structure makes it a versatile mineral used in civil engineering, the foundry industry, paper mills, drilling mud, and detergents. It is found in two forms. The first is a calcium form which doesn't swell or gel in water. The second is a sodium form which swells and forms thixotropic gels in water (*Colin Stewart Minchem web site, 2003).* There are 14 bentonite mills in the state (*Wyoming Geo-Notes #74).* Wyoming would be much the poorer without the volcanic ash and sea water that once covered our state. The western volcanoes and Cretaceous seas provided a perfect environment for the formation of bentonite layers in Wyoming, some of which are up to 5 feet thick. These layers should have been a clue to some geotechnical highway engineers. The table below is a chemical analysis of a typical "Wyoming" bentonite *(Colin Stewart Minchem\Web site, 2003).*

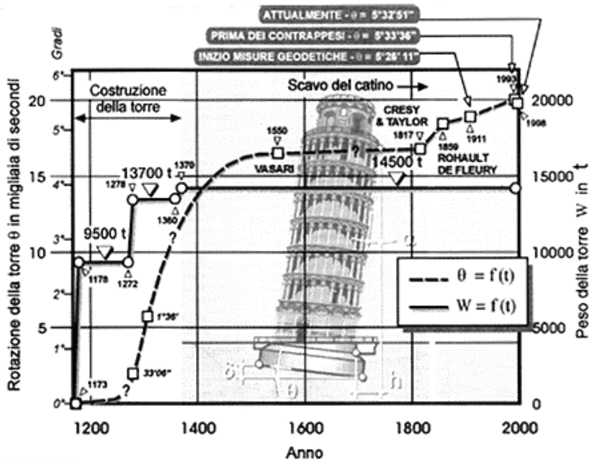
Conquest Wyoming Bentonite Specifications
% plus 75 microns Natural Sodium Wyoming Bentonite 8.0-15.0

Chemical(Oxides)	**Percent**
SiO2	**52.0-58.0%**
Al2O3	**16.0-20.0%**
Fe2O3	**3.0-4.5%**
CaO	**0.50-2.0%**
Na2O	**1.8-3.0%**
MgO	**1.8-2.6%**
K2O	**0.35-0.65%**
Bound Water	**4.5-6.0"%**
Moisture @105^{o}C	**10.0-14.0%**
Free Quartz	**5.0"%max.**
Free Cristobalite	**1.0%max.**

What would we do without the Atterberg limits? The A-line comes from Cassagrande's first name. The physical and chemical properties of silts and clays are dependent on water and its percentage within the soil. Plasticity Index, Plastic, and Liquid Limits were invented in order to classify and define these soil types**.**

Charles Augustin de Coulome

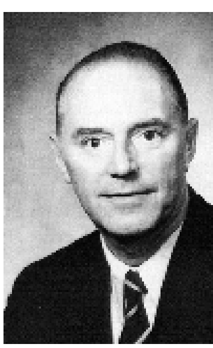
4. Settlement: See #3 as to why fills and pipes settle. Who cares about void ratios, compression index, coefficient of consolidation, Terzaghi time factors? We do of course! If Terzaghi is the father of geotechnical engineering, then Coulomb must be the grandfather. Nice uniform**.**



The Leaning Tower of Pisa is probably the best known case of settlement in the world. For years they have been trying to stabilize the soils beneath the tower. Settlement, as we know, is just a case of water being squeezed out of the soils. Our challenge as geotechnical engineers is to try and predict how much settlement is going to happen and how fast it will occur. The key is good samples, good logs and a master soils laboratory technician. All the formulas have been worked out years ago. It's just a matter of plug and go. Settlement is also one of the two main failure mechanisms for soils. Shear failure, of course, is the other.

Ralph Brazelton Peck

5. Swelling Soils: These are the "bad boy" cousins of the clay family. We bake them, rip them, lime treat them, encapsulate them, and still, they rise up to cause trouble. (See table for Chemical Analysis of Wyoming Bentonite, the main cause of swelling soils in Wyoming.) Add water to a swelling soil and the troubles begin.



1**Harry Bolton Seed**

6. Seismic: aka earthquakes, another substitute S. Earthquakes influence geotechnical engineers in many ways. They cause landslides and force us into including seismic loading into our foundation and slide mitigation designs. Also, liquefaction is a result of seismic events. California's well-documented problems with liquifiable soils have caused all states to review liquefaction potential at bridge foundation sites across the nation**.** Many landslides are triggered every year by seismic events. These are so far unpredictable and limited mostly to the Western U.S. Poor soils and water saturated soils are a disaster waiting to happen.

The most common landslide triggering mechanisms as are: intense rainfall, rapid snowmelt, changes in water level, volcanic eruption, and earthquake shaking (*Wieczorek 1996.)* It is worth noting that three of the five mechanisms are directly related to water.

George F. Sowers

7. Scour: When Mother Nature thinks we are bored she sends a slug of water our way to see if our bridge foundations are deep enough. This happened to WYDOT this past August 27,2002, when a 500 year flood hit one of our Interstate bridges, causing severe damage to the foundation of the NBL of I-25. The bridge was closed for four months, until heroic efforts by all WYDOT parties involved got the bridge repaired and the road open. Lately, scour has been on the forefront of all DOT's menus in both the Bridge and Geotechnical Programs or Departments. In the last 13 years WYDOT Geology has investigated over 60 structures for scour potential. It's now a national concern. Scour protection is one of the main focuses of many Bridge Departments and Hydraulic Sections. As geologists we are charged with finding scour resistant materials or quarries to supply large size aggregate. The force of water astounds many a motorist caught in a flood or crossing a flooded highway. Many lives are lost every year due to motorists' cars being washed away in floods.

Coors Light Twins
Diane & Elaine Klimaszewski

8. Twins: These are the double S terms, like Shear Stress, Slope Stability and Soil Slopes. These may not be doom terms, but are geotechnical in nature just the same.

Shear Stress: A concept to be understood and feared by all soil Engineers. The direct shear tests are a basic soils test in order to evaluate the strength of a soil states (*Hough, 1957*). In stress-controlled tests the shearing force is increased in such a manner that the development of shearing stress follows a predetermined pattern. Usually in these tests the objective is to increase the shearing stress at a constant rate, although in certain cases, tests are or have been conducted so that the shearing stress is increased in increments.

Slope Stability and Soil Slopes: These terms very much intertwined, and are a major concern for all DOT geotechnical engineers, because if you design a slope correctly, and it is built correctly, then it should never give you problems in the future. There are always exceptions. Water will always get into a slope and affect its stability. Maybe not this year, but there is always next year. Stable slopes are safe slopes.

References

Cedergren, H. (1974).*Drainage of Highway and Airfield Pavements*, John Wiley & Sons, Inc.,New York, NY.

Colin Stewart Minchem(2003),http://www.csminchem.co.uk/bentonite_cwb.htm
Hough(1957.*Basic Soils Engineering, The Ronald Press Co.,New York, NY.*513p.
S.W,A.W.(2003):http://ce.eng.usf.edu/pharos/wonders/other.html

Turner, J.P.,(2003).*Evaluation of Tire Chips for Highway Drainage Applications,FHWA-WY-03/02F, Department of Civil and Architectural Engineering, University of Wyoming, Laramie, WY, (August,2003),68p.*

Wieczorek, G. (1996). "Landslide Triggering Mechanisms",Chapter 4 in Landslides: Investigation and Mitigation, Special Report No. 247, National Academy Press, Washington D.C., pp76-90.
G.E.H.F. (2003), http://www.ejge.com/People/hallfame.htm *Geotechnical Engineering Hall of fame(2003)*

http://www.ejge.com/ *W3G World Wide Web of Geotechnical Engineers(2003)*

Wyoming Geo-Notes #74

I would be remiss if I did not list the 7 wonders of the Ancient World:

1. Great Pyramid of Khufu (Cheops).
2. Hanging Gardens of Babylon.
3. Statue of Zues.
4. Temple of Artemis of Ephesus.
5. Masoleum of Halicarnassus.
6. Colosus of Rhodes.
7. Alexandria Lighthouse.

or

The 17 Wonders of the Modern World:

1. The Channel Tunnel.
2. The Clock Tower (Big Ben) in London, England.
3. The CN Tower in Toronto, Canada.
4. Eiffel Tower in Paris, France.
5. The Empire State Building in New York City, USA.
6. The Gateway Arch in St. Louis, USA.
7. The Golden Gate Bridge in San Francisco, USA.
8. The High Dam in Aswan, Egypt.
9. Hoover Dam in Arizona/Nevada, USA.
10. Itaipú Dam in Brazil/Paraguay.
11. Mount Rushmore National Memorial in South Dakota, USA.
12. The Panama Canal.
13. The Petronas Towers in Kuala Lumpur, Malaysia.
14. The Statue of Cristo Redentor in Rio de Janeiro, Brazil.
15. The Statue of Liberty in New York City, USA.
16. The Suez Canal in Egypt.
17. The Sydney Opera House in Australia.

Then there are the 17 Forgotten Wonders:

1.Abu Simbel Temple in Egypt.
2. Angkor Wat in Cambodia.
3. The Aztec Temple in Tenochtitlan (Mexico City), Mexico.
4. The Banaue Rice Terraces in the Philippines.
5. Borobudur Temple in Indonesia.
6. The Colosseum in Rome, Italy.
7. The Great Wall of China.
8. The Inca city of Machu Picchu, Peru.
9. The Leaning Tower of Pisa, Italy.
10. The Mayan Temples of Tikal in Northern Guatemala.
11. The Moai Statues in Rapa Nui (Easter Island), Chile.

12. Mont-Saint-Michel in Normandy, France.
13. The Throne Hall of Persepolis in Iran.
14. The Parthenon in Athens, Greece.
15. Petra, the rock-carved city in Jordan.
16. The Shwedagon Pagoda in Myanmar.
17. Stonehenge in England.

Don't forget the 11 Natural Wonders:

1. The Bay of Fundy in Nova Scotia, Canada.
2. The Grand Canyon in Arizona, USA.
3. The Great Barrier Reef in Australia.
4. Iguaçú Falls in Brazil/Argentina.
5. Krakatoa Island in Indonesia.
6. Mount Everest in Nepal.
7. Mount Fuji in Japan.
8. Mount Kilimanjaro in Tanzania.
9. Niagara Falls in Ontario (Canada) and New York State (USA).
10. Paricutin Volcano in Mexico.
11. Victoria Falls in Zambia/Zimbabwe.

*****As compiled by the web site of the Seven Wonders of the Ancient World SWAW (2003).***

The World Wide Web or Internet has opened up access to volumes of Geotechnical data. One of the best sites that has Links to many Geotechnical sites can be found here; http://www.ejge.com/ W3G World Wide Web of Geotechnical Engineers(2003).

For presentation at the 54th Annual Highway Geology Symposium, Burlington, Vermont, September 24-26, 2003

Recent Sinkhole Occurrences along Highways in East Tennessee, A Historical Perspective

By
Harry L. Moore
Tennessee Department of Transportation
Geotechnical Engineering Section
Knoxville, Tennessee
37914

Abstract

During the last five years an increase in the number of sinkhole collapse incidents along highways in East Tennessee has been recorded. Most of the sinkhole occurrences have been located in unpaved ditch lines along the roadways. The sinkhole collapses occur in residual soils of Cambrian and Ordovician carbonates within the Valley and Ridge Physiographic Province. The soil-carbonate interface is usually pinnacle type and very irregular, with some soil slots extending as much as 70 to 90 feet below the surface. The sinkhole occurrences tend to be related to precipitation events; however, annual precipitation totals do not fully support this observation.

A comparison of the last five years of sinkhole incidents was made with incidents over the past 33 years, showing several spikes in occurrence over the period. Precipitation data for the middle region of East Tennessee was compared to the number of sinkhole incidents that occurred annually over the 33-year period. With the average annual precipitation approximately 48 inches, there have been six periods of above average precipitation (1972-1974, 1979 and 1982 as one period, 1989-1991, 1994, 1996-1999, and 2002) and one major period of below average precipitation (1985-1988).

There have been four major episodes of sinkhole collapse during the years from 1969 through 2002: 1980, 1984, 1987, and 1998. Twenty-five cases of sinkhole collapse alone occurred in 1998 and most required partial road closure during repair. Historically, from 1977 to 1987 74 % of sinkhole incidents occurred in the ditch line of roadways. Overall, from 1969 through 2002, 86.5% of the sinkhole collapse incidents were in ditch line locations, with 93% of those ditch line collapses occurring in unlined ditches.

Introduction

Continued highway construction and development in the Valley and Ridge Province of middle East Tennessee and adjoining areas has resulted in the occurrence of numerous induced sinkhole collapses over the years. The location of these new roads and bridges as well as developments over belts of active karst increases the incident of induced sinkhole collapse.

Karst landforms characterize the landscape in the East Tennessee area including the Valley and Ridge as well as small portions of the Blue Ridge and Cumberland Plateau provinces. These karst landforms consist of a variety of sinkholes, karren (solution furrows etched on exposed limestone), and numerous cave systems, many of which include subsurface streams.

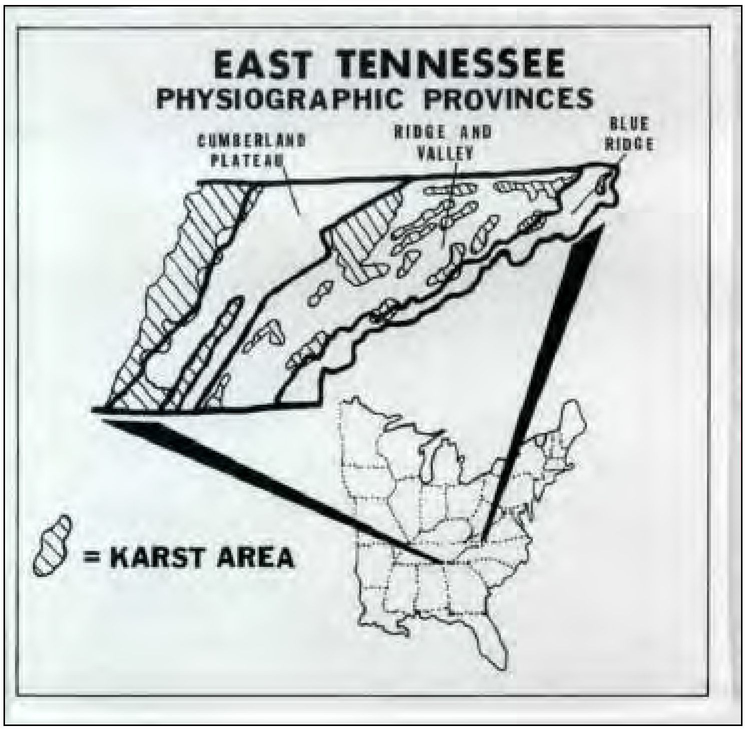
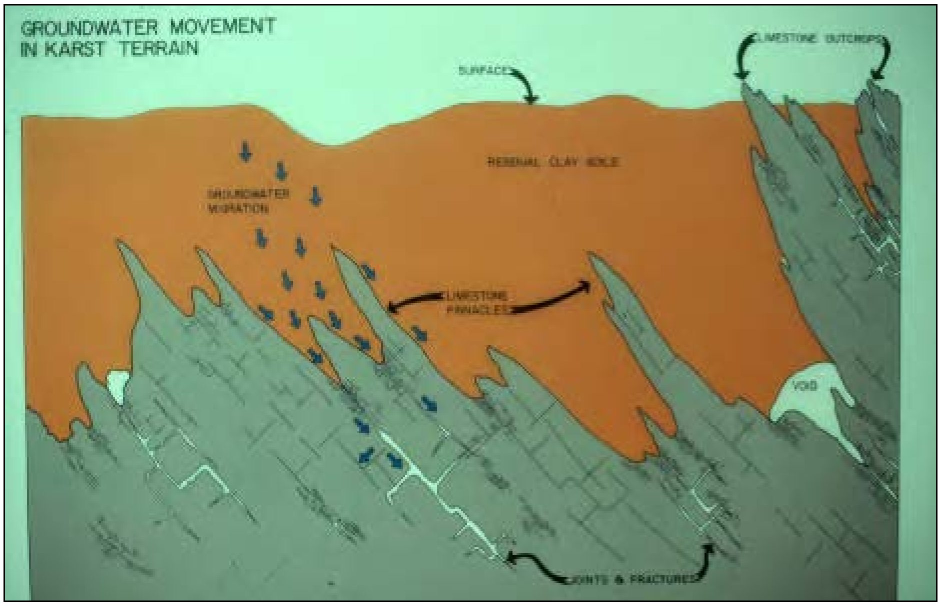


Figure 1. This map shows the general areas of karst in East Tennessee relative to the Physiographic Provinces.

Cave development in the East Tennessee region follows the more soluble zones or lines of weakness within the host bedrock. These weak areas are generally parallel to zones of fractures (joints and faults) or bedding planes of the rock. Solution enlargement of these weak areas of fractured rock generally coalesce into interconnecting passages that form intricate caverns, some of which are profusely decorated with cave formations (speleothems or dripstone).

Surface expression of the subsurface conditions in karst areas is usually manifested as depressions and sinkholes. The sinkholes vary in size from several feet to several hundred feet across and up to tens of feet in depth. Some of the sinkholes may have active swallets or openings into the cave environments, while others may be simply silted in and covered with vegetation. Additionally, numerous rock outcrops are present, as well as sinking streams, cave entrances and springs. All of these conditions are present along highways in the East Tennessee region.

Sinkhole development along the highways usually results from the collapse of the residual clay soil into cavities developed in the subsurface soil due to erosion of the residual clay. As these “soil cavities” enlarge and approach the surface, the remaining soil-bridge over the cavity looses strength and collapses forming a typical sinkhole collapse. The eroded soil is flushed down into the solution cavities in the bedrock. This type of induced sinkhole failure has been previously described by Donaldson (1963), Jennings, et al (1965), Moore (1981, 1987), and Newton (1981, 1984).



This drawing illustrates the pinnacle/soil interface between the carbonate bedrock and residual soil in the East Tennessee region.

The majority of sinkhole collapse incidents experienced along Tennessee highways are usually the soil collapse type (Moore, 1987). Extremely rare are occurrences of bedrock collapse into large open caves.

Recent Sinkhole Collapse

Within the past five years a number of sinkhole collapse incidents have been experienced along the state highways in East Tennessee. Twenty-five cases of sinkhole collapse alone occurred in 1998 and most required partial road closures during repair. Fifty-four cases of collapse have occurred in the past five years.

There have been four major episodes of sinkhole collapse based on TDOT office records during the years from 1969 through 2002: 1980, 1984, 1987, and 1998. Historically, from 1977 to 1987 a total of 74% of highway related sinkhole collapses occurred in roadway ditch lines (Moore, 1987). The 1987 study revealed that of 74% of the ditch line collapses, 93% occurred in unlined (sod or clay) ditches. Overall, from 1969 through 2002, 86.5% of the sinkhole collapse incidents were in ditch line locations, again with 93% of those ditch line collapses occurring in unlined ditches.

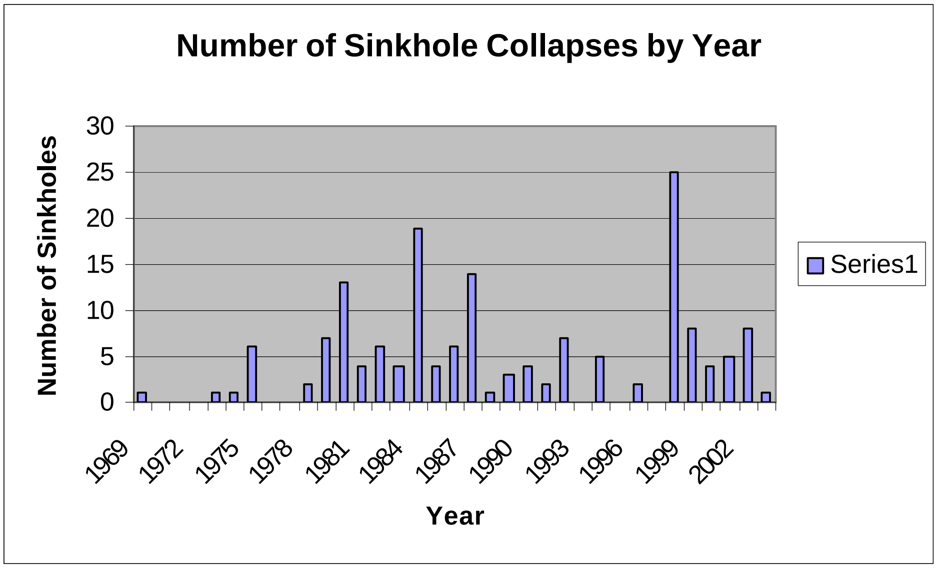


Table 1. This table shows the annual number of sinkhole collapse incidents from 1969 through 2002. (163 sinkhole collapse cases studied)

All major highways including Interstate routes experienced sinkhole collapses within the past five years. One ditch line collapse on I-75 in Anderson County was measured to be over 30 feet in diameter. Most of the sinkhole collapses were on the order of 10 to 12 feet in diameter and up to 8 to 10 feet in depth. No particular geologic formation was found to be more dominant than another in areas where the sinkholes had formed. However, it was observed that those areas where excavation had approached the pinnacle type soil/rock interface (within 8 to 10 feet) experienced more collapses than other areas where there was no excavation or very deep soils that existed below the road grade.

This sinkhole collapse occurred in the summer of 2002 along I-640 in Knoxville, Tennessee

This collapse was located in the median of I-75 in Anderson County, approximately 20 miles north of Knoxville. Note the marked area in the grass outlining the sinkhole boundary which was measured to be approximately 30 feet in diameter.

A number of ditch-line type sinkhole collapse problems occurred along highways in East Tennessee in the winter of 2002-2003. This is along U.S. 321 in Blount County.

Some of the recent sinkhole collapse problems were just open pit shafts, as deep as 10 to 12 feet.

TDOT maintenance forces were used to correct many of the collapse problems, such as this one on an I-140 off ramp near Alcoa, Tennessee.

This large collapse occurred in the ditch line of an off ramp along I-140 in Blount County, near the Knoxville airport in March of 2002.

Ditch line collapse problems have been common along some roads in East Tennessee. (I-140, Blount Co.)

Precipitation Data

Annual precipitation data for the middle areas of East Tennessee was obtained from the Tennessee Valley Authority and the National Weather Service in Morristown, Tennessee to see if there was a correlation between precipitation events and increased sinkhole collapse incidents along the highways. It was hypothesized that during high rates of precipitation there would be a corresponding increase in sinkhole collapses.

With the general average annual precipitation rate for the middle areas of East Tennessee being approximately 48 inches, there have been six periods of above average precipitation during the years from 1969 to 2002. These include the following: 1972-1974, 1979and 1982 (as one period), 1989-1991, 1994, 1996-1999, and 2002. The average annual precipitation of these high years was 59.9 inches. There was also one major period of below average precipitation (1985-1988) where the four-year average was 34.53 inches.

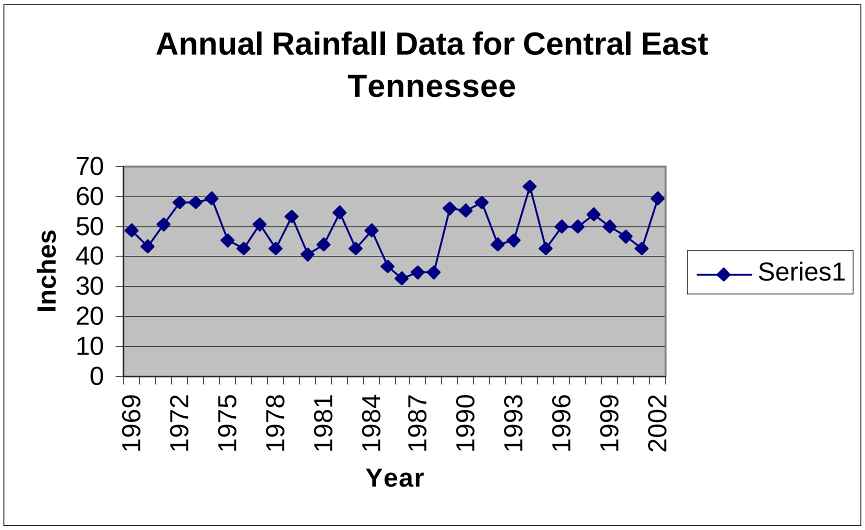


Table 2. This chart illustrates the annual rainfall data (in inches) for the central region of East Tennessee from 1969 to 2002.

Conclusions

As was first surmised, there appears to be some link to precipitation. However, it is not what was first believed. There has been a general perception that the sinkholes tend to occur during very wet periods. And generally the data tend to support this. However, our data also shows a sinkhole occurrence spike during low periods of precipitation.

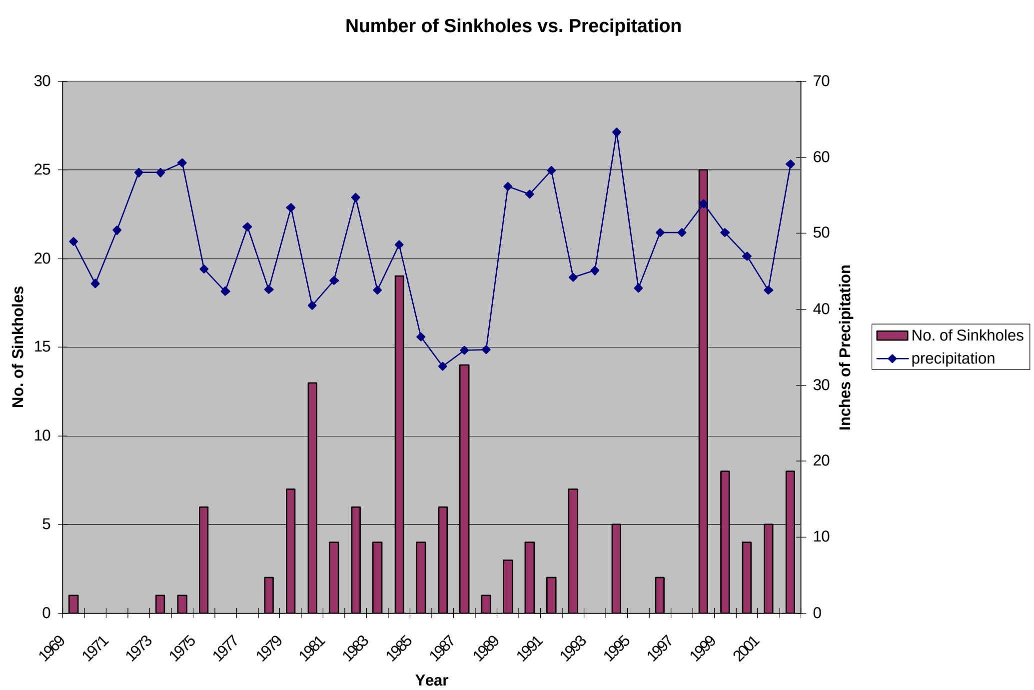
Obviously, the occurrence of sinkholes during high levels of precipitation is usually predicted due to the increase in surface and subsurface erosion. Not clearly understood is the occurrence of sinkholes during drought conditions. A correlation of the drought conditions may be made to mining or quarry operations where those operations de-water or lower the local groundwater table causing induced sinkhole collapses to occur. There are a number of case histories where quarrying adjacent to highways in karst areas resulted in induced sinkhole incidents (Benson et al, 1998; Newton 1973, 1981).

It is believed that the lowering of the water table results in a loss of support to the roofs of cavities in bedrock that were filled with water. This also applies to residual clay soils that overlie openings in the cavernous bedrock that were filled with water before the decline of the water table (Newton, 1971). As drought conditions lower the water table, the same mechanism causes the migration of the residual soils from near the surface to lower areas within the cavernous bedrock causing subsidence and collapse to occur. This may explain the increase in sinkhole collapse incidents during dry periods.

This study revealed that from 1969 to 2002 where 163 cases of highway sinkhole collapse were recorded in East Tennessee, approximately 86.5% of the sinkhole occurrences were located in highway ditch lines. This study also supported a previous study by the author (1987) that of all the ditch line collapse incidents, 93% involved unlined ditches.

It is readily evident that channeling surface water into unlined ditches or even retention basins or "holding ponds" will increase the incidence of sinkhole collapse. Recognizing the impact of development (road building, subdivisions, shopping malls, etc.) on karst environments in advance of construction can result in minimizing or even avoidance of the sinkhole problem altogether. Providing some element of impervious lining (geomembrane, pavements) for all drainage ditches in karst areas will greatly limit the incidence of induced sinkhole formation.

Typical ditch line sinkhole collapse in East Tennessee. (SR 72, Loudon Co.)



This table illustrates the relationship between annual precipitation rates versus annual sinkhole occurrences in East Tennessee.

References

Benson, R. C., R.D. Kaufmann, L.B. Yuhr, and David Martin, 1998. “Assessment, prediction and remediation of karst conditions on I-70, Frederick, Maryland”, Proceedings of the 49th Annual Highway Geology Symposium, Prescott, Arizona, September 10-14,pp. 307-312.

Donaldson, C.W., 1963. “Sinkholes and subsidence caused by subsurface erosion”, Proceedings of the 3rd Regional Conference for Africa on soil mechanics and foundation engineering, pp. 123-125.

Jennings, J.E., Brink, A.B.A., Louw, A. and Gowan, G.D., 1965. “Sinkholes and subsidence in the Transvaal Dolomites of South Africa”, Proceedings of 6th International Conference of Soil Mechanics, pp.51-54.

Moore, Harry L., 1981. “Karst problems along Tennessee highways: An overview”, Proceedings of the 31st Annual Highway Geology Symposium, Austin, Texas, August, pp.1-28.

Moore, Harry L., 1987. “Sinkhole development along “untreated” highway ditchlines in East Tennessee”, in Proceedings of 2nd Multidisciplinary Conference on Sinkholes and the Environmental Impacts of Karst (ed. Barry Beck), Orlando, Fla., February 9-11, pp.115-119.

Newton, J.G., 1971. “Sinkhole problems in and near Roberts Industrial Subdivision, Birmingham, Alabama”, USGS Circular #68, 42p.

Newton, J.G., 1973. “Sinkhole problem along proposed route of Interstate 459 near Greenwood, Alabama”, USGS Circular # 83, 63p.

Newton, J.G., 1981. “Induced sinkholes: An engineering problem”, Journal of the Irrigation and Drainage Division, ASCE, Vol. 107, No. IR2, June, 1981, pp. 175-185.

Newton, J. G., 1984. “ Review of induced sinkhole development”, in Sinkholes: Their Geology, Engineering, and Environmental Impact (Ed. By Barry Beck), in Proceedings of the 1st Multidisciplinary Conference on Sinkholes, Orlando, Fla., October, pp. 3-9.

Living with Landslides on the Big Sur Coast: The Challenges of Maintaining Highway 1

John D. Duffy [1]
Aileen Loe[2]
Morgan Gaudioso[3]

Abstract

Coastal erosion along the 75-mile long Big Sur Coast comprises a broad spectrum of processes. This is an emergent coast with the young Coast Ranges steadily rising from the Pacific Ocean. Landsliding, bluff erosion, and discharge from inland watersheds are the primary sources of sediment discharge to the nearshore system. The most prominent man-made feature along the coast is California State Route 1. The appeal of the wild coast and traveling along the edge of land and sea has made Highway 1 a popular destination for traveler's worldwide. Keeping the highway open and safe in this dynamic natural setting is the challenge of the California Department of Transportation. Landsliding is one of the most noticeable erosional features along the coast and has a large impact on the highway. Given the travel demands along the route and a focus on reducing overall environmental impacts associated with highway repairs, engineering solutions have changed emphasis over time. From grand civil engineering approaches attempting to stabilize large landslides, the shift now is toward less ambitious approaches to achieve adequate stability with some compromise for local instabilities. This approach means highway repairs with fewer direct environmental impacts and quicker response to reopening the road after a landslide-related closure; the consequences may require more intensive maintenance and associated traffic delays. This change in engineering approach is illustrated over two recent El Nino storm periods. After the storms of 1983, highway repair from one large landslide resulted in the removal of 3.1 million cubic meters of earth and a one-year road closure. After the storms of 1998, highway repairs from four large landslides resulted in the removal of only 700,000 cubic meters of earth and a three-month road closure. Even with this evolution in repair techniques, excess landslide material is still generated, creating a challenge for its proper handling and disposal on this coast. The current management strategy for handling residual material from any source or phenomenon (e.g. whether by slope

[1] Senior Engineering Geologist, California Department of Transportation, 50 Higuera Street, San Luis Obispo, California, 93401, USA; phone 805-549-3663, fax 805-542-4693; john_d_duffy@dot.ca.gov
[2] Senior Environmental Planner, California Department of Transportation, 50 Higuera Street, San Luis Obispo, California, 93401, USA; phone 805-549-3103, fax 805-542-4693; aileen_loe@dot.ca.gov
[3] Student Assistant, California Department of Transportation, 50 Higuera Street, San Luis Obispo, California, 93401, USA; phone 805-549-3135, fax 805-542-4693; morgan_gaudioso@dot.ca.gov

erosion or watershed discharge) is exporting to landfills. These sites are located at some distance from the source and result in direct impacts to coastal upland habitats and indirect impacts from truck-hauling operations. Other options include reducing material, re-using material, recycling material. Also considered is replenishing the marine sediment load. Efforts to develop sound solutions and support regulatory decisions are underway and include several levels of technical research to more fully characterize the natural processes at work.

Introduction

The ongoing natural processes that shape the unforgettable landscape in Big Sur also create the greatest challenges for maintaining a reliable highway. The backdrop is the relatively young Santa Lucia Mountains that fall steeply to the Pacific Ocean. These facts combined with the unique aspects of the climate and the role of the Pacific Ocean contribute to both the area's beauty and its geological instability. Water in the form of surface runoff from the mountains above and wave action at the toe of the slopes below is the primary agent in the erosion processes that shape the landscape. The force of water is most evident during periods of heavy storms, which produce both high volumes of runoff and pounding surf.

Landslides and washouts of variable severity result in frequent road closures. Depending on their complexity, repairs to restore the highway can cause delays that extend over long periods of time. With detour opportunities very limited, Highway 1 is the lifeline to several well-established communities, state parks, national forest and is highly popular for recreational travel (Figure 1) (California Department of Transportation, 2000).

Figure 1: Highway 1 along the Big Sur Coast in Monterey County, CA

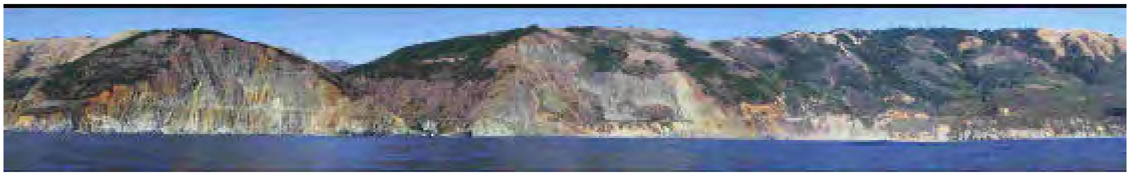


Figure 2: The Santa Lucia Mountain Range

Geology & Landslides on the Big Sur Coast

The Big Sur coast is located within the long and geologically complex part of the Coast Ranges geomorphic province, which extends for about seven hundred miles within California from Santa Barbara County to the Oregon border. The Big Sur coast is noted for its dramatically high, steep slopes, which rise from sea level to over 3000 feet within less than three miles. Uplift of the Santa Lucia Mountains and continuing wave erosion at their base has formed precipitous slopes in many types of bedrock and overlying surficial deposits (Figure 2). The richly varied geologic composition here has led to an abundance of landslides (California Division of Mines & Geology, 2001).

Rock types of the Coast Ranges belong to all three major rock classes: igneous, metamorphic and sedimentary. The most widespread geologic unit is the Franciscan Complex, composed of variably metamorphosed fine to medium grained graywacke sandstone and highly sheared shale. Other minor components of the Franciscan Complex include serpentinite, greenstone and chert. Overlying the Franciscan complex, all of the rock types tend to be weak, intensely sheared and slightly metamorphosed sedimentary rocks or unconsolidated surficial deposits.

Within the northern portion of the coast a block of distinctive rocks are bounded by the San Andreas fault on the east and the Sur-Nacimiento faults on the west. This rock mass is known as the Salinian block. The geology of the Salinian block is quite different from the rest of the Coast Ranges. The Salinian block bedrock consists of granitics and sedimentary rocks which in general are more resistant to landsliding than typical Franciscan bedrock.

Slope Stability Along Highway 1

Landslide mapping performed by the California Geological Survey (CGS)[4] throughout the corridor has identified over 1500 landslides within about a three-mile-wide section along the 75-mile-long study area. A recent Caltrans evaluation indicates that 88 locations along the highway currently exhibit stress or influence related to underlying movements (California Department of Transportation, 2001). The implications for Highway 1 are obvious: maintaining a reliable linear feature, such as a highway, in this unsteady landscape is challenging at best.

[4] Formerly the California Division of Mines & Geology.

Many natural conditions and processes influence slope stability along this stretch of Highway 1. Looking at the geology the following factors all contribute to the global instabilities encountered here: uplift of the Coast Ranges, inclination of the slopes and the underlying rock types and geologic structures. Seasonal conditions and exposure to rough seas also play a primary role; specific factors include fire, rainfall and ocean wave action. With rainfall averaging up to 60 inches per year and storm waves up to 30 feet, the influence of winter storms is significant. In addition to the natural processes, construction of the original highway has also contributed to localized instabilities.

The more than 1500 landslides mapped by CGS in the Highway 1 corridor along the Big Sur Coast tend to be the larger, deep-seated slides that affect large areas. The predominant types of landslides described in the corridor are:

- ***Rock Slide*****:** A slide involving bedrock in which much of the original structure is preserved.
- ***Rock Fall:*** A landslide in which a fragment or fragments breaks off of an outcrop of rock and falls, tumbles or rolls downslope.
- ***Earth Flow:*** A landslide composed of mixture of fine-grained soil, consisting of surficial deposits and deeply weathered, disrupted bedrock.
- ***Debris Slide:*** A slide of coarse-grained soil, commonly consisting of a loose combination of surficial deposits, rock fragments, and vegetation.
- ***Debris Flow:*** A landslide in which a mass of coarse-grained soil flows downslope as a slurry.

Highway Repair Strategies

Strategies for maintaining a functioning highway in the dynamic Big Sur environment include three basic techniques or approaches: Relocation or Separation, Stabilization, Management and Protection (Transportation Research Board, 1996).

These techniques are not mutually exclusive, nor listed in any particular order. A repair strategy for an individual location may include one or more of these techniques, depending on the specific site conditions.

Relocation or Separation. This strategy involves moving the roadway alignment away from the problem area, thereby separating the highway from further influence of the natural land movements. This action protects the public investment in the facility and allows the natural processes associated with landsliding to continue without interference. This technique includes minor realignments as well as construction of bridges, viaducts, and tunnels. In many cases, separation projects are considered “long-lead” projects that require substantial planning and detailed investigations; these relatively high cost projects must compete for funding statewide and can take considerable time to implement.

Stabilization. Stabilization refers to techniques applied to a slope to prevent or minimize movements from either above or below the highway. Examples of stabilization techniques include buttresses, retaining walls, crib walls, shoreline armor, anchor bolts and reinforced earth embankments. Completely removing an unstable mass (landslide) is also a legitimate stabilization technique. Aspects of stabilization approaches may also include modifications to control surface or subsurface water to avoid retention or concentration of water that could lead to severe erosion or saturation and ultimately slope failure.

Management and Protection. Minimizing damage to the highway and disruption to those who depend on it requires a three-pronged management approach: prevention, anticipation and response. *Prevention* includes actions taken in advance that would avert a failure from affecting the highway or minimize the potential for damage. Preventive actions include maintenance activities as well as more intensive projects designed to avert a potential failure. *Anticipation* refers to actions taken in preparation for breaks or disruptions in service that cannot be avoided and putting mechanisms in place to facilitate future responses. Anticipation actions are not considered maintenance activities, as they generally involve projects. *Response* activities are conducted when there is a break in service or an imminent threat to traveler safety or integrity of the facility.

The primary objective of management and protection is to reduce the potential for damage, while not eliminating the underlying conditions of movement or failure. Management and protection techniques apply a wide variety of measures to protect the traveler from harm and the highway from damage; they also involve techniques that best "live with" on-site movements by balancing the forces of the instabilities. Techniques include reducing the driving forces of a landslide (for example, removing a mass of material from the head of a landslide) or increasing the resisting forces (such as buttressing a slope). Balancing forces may prevent or reduce the likelihood of a larger scale event (or movement), but slopes may continue to move more gradually. While localized (or smaller) instabilities may continue to be apparent, global stability may be improved.

Managing Instabilities

Landslide remediation efforts have evolved as new technologies, constraints, and perspectives have emerged over time. Advancements in understanding geological processes, improved investigative tools and new design methods have influenced and improved landslide investigations and mitigation.

Perceptions of landslides on the coast have also evolved as people become more aware of the nature of landslides and how they contribute to the place known as Big Sur. Now, with the wealth of information available, landslides are being understood as the primary natural process that shapes the landform and creates the essential character of the coast.

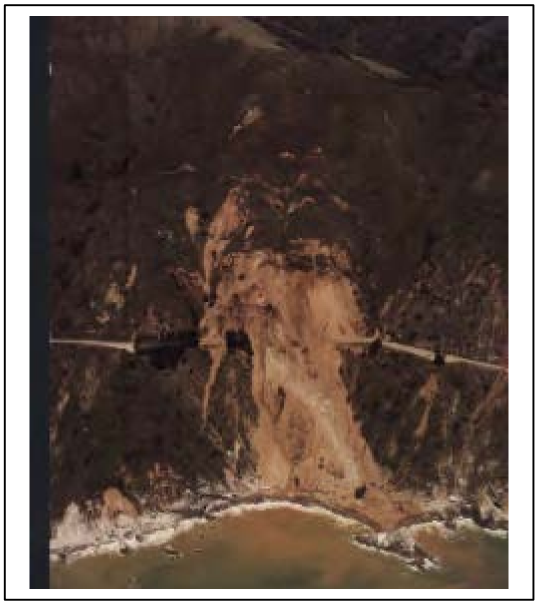


Figure 3: The repair of Highway 1 after the 1983 JP Burns landslide caused a one-year road closure.

With this informed perspective, efforts to “stop the slopes from sliding” have evolved into efforts to manage instabilities while respecting landslides as part of the natural landscape. The philosophy can be said to have evolved from “moving the mountains” to “living with landslides.” From grand civil engineering projects attempting to stabilize large landslides, the shift now is toward less ambitious approaches to achieve adequate stability with some allowance for local instabilities. This means highway repairs with fewer direct environmental impacts and a quicker reopening of the road after a landslide-related closure. Such approaches may require more intensive maintenance and associated traffic delays.

The evolution in engineering approach described above is illustrated by comparing Caltrans responses to two recent El Nino storm periods. After the storms of 1983, highway repair from one large landslide, the JP Burns landslide (Figure 3), impacted approximately 300 meters of roadway. This slide was approximately 180 meters wide and 300 meters in slope length comprising approximately 23 acres. An estimated 2.3 million cubic meters of material was displaced. The repair required the removal of 3.1 million cubic meters of material and a one-year road closure.

Highway repairs during the 1997/1998 winter season were extensive. Over 60 locations required repairs of varying degrees. At three locations large paleo-landslides had moved destroying a total of almost 600 meters of roadway (Figures 4, 5 & 6). By employing new strategies the repair of the slides resulted in the removal of only 700,000 cubic meters of earth and a three-month road closure. The dramatic decrease in earthwork also results in fewer overall environmental impacts (Figure 7) (California Department of Transportation, 2002).

Figure 4: The Duck Ponds Landslide-1998

- **Roadway affected:** ~120 meters
- **Landslide dimensions:** ~900m x 900m x 60m
- **Landslide area:** ~275 acres
- **Displaced portion:**
  - ~9 acres
  - ~1.5 million cubic meters
- **Repair Strategy:**
  - Management
    - Increase resisting forces
    - Reduce driving forces
    - De-watering: > 1.8 million liters/day
- **Earthwork:** ~340,000 cubic meters removed, 1/3 of the total displaced

Figure 5: The Big Slide Landslide-1998

- **Roadway affected:** ~90 meters
- **Landslide dimensions:** ~600m x 200m x 40m
- **Landslide area:** ~100 acres
- **Displaced portion:**
  - ~32 acres
  - ~7.5 million cubic meters
- **Repair strategy**
  - Management
    - Reduce driving forces
    - De-watering: ~190,000 liters/day
- **Earthwork:** ~45,000 cubic meters removed, ~5% of total displaced

Figure 6: Grandpas Elbow Landslide - 1998

- **Roadway affected:** ~360 meters
- **Landslide dimensions:** ~900m x 360m x 50m
- **Landslide area:**
  - ~82 acres
  - ~20 million cubic meters
- **Repair strategy:**
  - Management
    - Remove driving forces
    - De-watering: ~380,000 liters/day
- **Earthwork:** ~26,000 cubic meters removed, 1% of total displaced

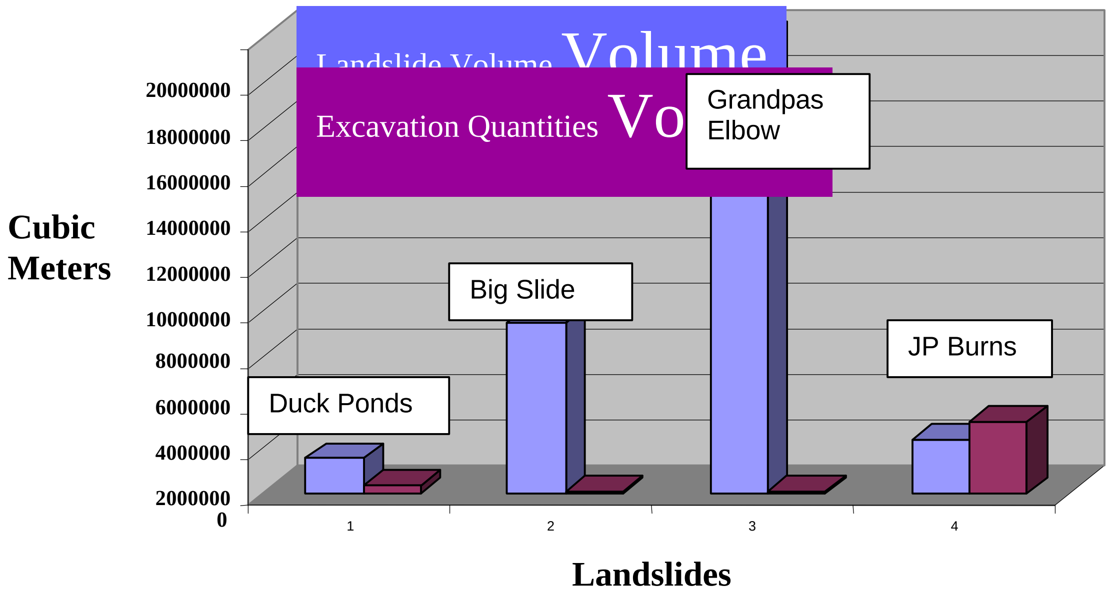


Figure 7: Comparison of Landslide Volume vs Excavation Quantities Associated with Highway Repair

Methods of Handling Excess Material

Despite this major advance in repair techniques, excess material remains the one common denominator. Soil, rock and debris generated by landslides and their subsequent repairs need to be moved in order to restore the highway to service. Among the outstanding challenges is finding the best solutions for the proper handling and transfer of this material on this coast.

Assuming the most vigilant actions minimize the quantity of material that would require any transfer (i.e. opportunities to first reduce, reuse and recycle are maximized), residual material frequently still requires handling under many circumstances. Choosing the appropriate method or combination of methods to deal with excess material is at least in part related to its source. Discussion continues about how material sources (and specifically the mechanisms by which material is generated) factor into this decision-making process. For example, where excess material is seen as being generated by mechanical excavation (as part of the highway repair response to a natural landslide event), decisions have been made to exclude replenishment-type actions. Where the source can be accepted as largely natural, and the repair activity is not seen as increasing the overall volumes, methods for replenishment have been considered. The potential for environmental impacts is a driving factor in the decision-making process. Adaptation and tolerance of the receiver site or habitat must be evaluated to determine the severity and duration of potential effects (Monterey County Planning Department, 1986). This kind of analysis in addition to developing a (quantitative) understanding of the ambient sediment budget on this coast is desired by resource managers toward advancing aspects of this decision-process.[5]

Deposit of material where a beneficial use has not been determined (i.e. received regulatory approval) is generally hauled to its destination by truck. Trucking is the conventional method for transporting material on the coast. However, options for transport by sea to another terrestrial destination or an approved ocean disposal site, might include barges to haul material.

Best Earthwork and Excess Materials Handling Practices

In responding to a debris flow or other landslide activity, the issue that most influences the time required to re-open the highway is earthwork. The quantity of material involved, the proximity of sensitive environmental resources, the availability of temporary or permanent disposal sites, and the distances to such sites are critical factors in determining appropriate options for dealing with excess material. Identifying and refining best practices for materials handling is a major focus. The disadvantages of some practices argue for their use only as a last resort or after other measures have been applied to reduce the quantities of material involved. Caltrans will continually seek to reduce the disadvantages of certain practices. This current best earthwork and materials handling practices and identifies strategies for future exploration and development are described below.

Materials handling along Highway 1 will be conducted in terms of three basic principles: reduce, reuse, and recycle. Although these approaches are not mutually exclusive, they are to be considered and implemented in the following order of priority:

Reduce – Reducing the quantity of material produced or to be dealt with is the first-order priority. This involves identifying activities with the least environmental impacts, a limited "footprint" or area of disturbance, and minimizes remedial earthwork relative to the volume of material displaced by the event. An example of this would be in-situ stabilization techniques (Figure 8).

Strategies for reducing the amount of material generated by a landslide-related repair include a commitment to using the best techniques within each of the three basic strategies for approaching a repair. Separation techniques allow natural processes to continue without impacting the highway (viaducts, bridges, or tunnels). Stabilization techniques enable steeper slopes and limit the highway footprint (retaining walls or engineered slopes); in-situ stabilization (rock bolts or soil nails) minimize excavation. Management techniques balance the forces (reduce driving forces, increase resisting forces) rather than completely excavate the instability.

---

[5] Initial components for developing a sediment budget along the Big Sur Coast may be initiated by the US Army Corps of Engineers through a Coast of California Study.

Figure 8: Use of new techniques and specialty equipment minimizes excavation associated with highway repair.

Reuse – Reuse material that is viable for other highway maintenance or reconstruction projects. Rock and soil suitable for other highway repairs would be re-used locally or in other parts of the corridor, as needed. For structural use, the material cannot be high in organic matter; for revegetation efforts, organic material such as duff or topsoil is an important component. Material may require double handling as it may be stockpiled and processed for later use.

Strategies for re-use of excess materials focus on the vicinity of the slide or repair activity or other location along the Highway 1 corridor to avoid the potential adverse impacts of hauling the material to another site. Re-use would include such techniques as highway shoulder backing, shoulder widening, berming at locations to delimit boundaries of pullouts, remediating eroded embankments, and reprofiling the highway. These strategies would be considered viable where they are consistent with other preservation values. Other re-use or fill strategies could be identified in developing particular highway repair projects, and would be funded as part of the larger project undertaking. Temporary stockpiling of material is often required between excavation and re-use activities.

Recycle – Recycling refers to the re-use of slide material for non-highway purposes and includes the return of the material into the natural system. Slide material may be transferred for use in a public or private development project. Such recycling helps balance system inputs and outputs and reduces the amount of excess material that must be disposed of permanently.

The following practices may be used exclusively or in combination, depending upon site conditions, quantities of material, and other factors as described for each. Even where reduction and reuse are successful, quantities of material requiring recycling or disposal will likely remain.

Where material cannot be used for highway repair or maintenance, surplus is made available for other purposes. Suitable fill for land use development and large rock are

Figure 9: Reuse and recycle strategies involve temporary stockpiling, often along the roadside.

often commercially viable. Material would require transport to a receiver site or suitable processing facility, which may require temporary stockpiling (Figure 9).

Replenish – Replenishment refers to restoring sediment supplies to natural systems by removing or bypassing man-made barriers. This would consider the circumstances under which the highway (or highway management practices) may inhibit natural flow of sediment.

Determining appropriate replenishment strategies relies on information that a natural pathway for sediment is interrupted in some way. Most basically, this would apply to circumstances where the highway acts as a bench and detains material on its downward trend, or where the highway crosses a natural drainage via embankment fill and culvert that may also retain sediment. In this context, the reference to a pathway considers the role of the ocean as a natural sediment sink. Transferring material directly across the barrier could include either controlled or uncontrolled placement; bypassing options to deposit material in a manner that can avoid impacts to the nearshore environment are also included under this heading. Options under this heading include the following techniques: localized sidecasting, on-site detention and nearshore bypass.

It should be noted that these options are being considered on an experimental basis and are not widely available under current environmental regulations and restrictions.

> ***Localized Sidecasting-*** This option refers to relocating material from the roadway and directly placing it along the edge or pushing it downslope. The localized nature of the activity keeps the material within its place of origin from a geological standpoint.

On-Site Detention- This practice involves a more controlled approach to sidecasting. A berm or mechanism for catchment may be used to prevent material from being directly deposited onto the beach or into the ocean. This practice may involve importing material from other areas along the coast (assuming criteria for compatibility are developed); in some situations, this may include loading large old landslides and allowing the material to gradual move down toward the ocean. With this method, material would be expected to enter the marine system over time through erosional processes that may be gradual or episodic in nature (Figure 10).

Nearshore Bypass- This technique represents a direct introduction of material to the ocean while avoiding direct placement or settlement of soft sediments into nearshore and intertidal habitats. Methods could include pumping or barging material for release beyond the intertidal but still relatively close to shore; these options are more commonly associated with dredging activities for harbor management.

Dispose – Any remaining excess material that cannot be put to any other beneficial use in a timely fashion is considered disposal. Excess material is ultimately disposed of at approved receiver sites located at some distance away from the source of material. Most commonly this results in landfill on private or public lands. It may also apply to appropriately permitted ocean sites.

Landfill sites- Landfills are terrestrial sites approved for receiving material. They include commercial facilities (such as a quarry), approved developments requiring fill or opportunistic filling of pockets of land near the highway (Figure 11). The method of transport is by truck.

Figure 11: This opportunistic landfill site within the Big Creek Reserve also demonstrates good practices for successful revegetation.

Ocean sites- This option takes material a considerable distance offshore for deposit at sea. Under this approach, excess slide material could be transferred to barges and shipped or pumped to a designated offshore disposal site.

Current Management Practices and the Regulatory Environment

Implementation of these "best practices" assumes the options are made available through applicable regulatory programs. Currently, the only options available for excess material are re-use, recycling and disposal where the receiver identified is a commercial landfill (or a temporary stockpile). Four terrestrial sites, located on private and public lands along the corridor, are also under consideration for receiving material. Replenishment (Figure 12) options are being explored on a trial basis; one such site is at a location known as Pitkins Curve, where an "on-site detention" strategy is being evaluated. For the past 15 years, the only accepted strategy has been to haul material by truck to an upland site.

While impacts to terrestrial environments are more easily evaluated and monitored, impacts to the marine environment are more difficult to estimate and understand. There is little argument about the role of the ocean as a natural sediment sink for inputs from rivers and bluff erosion. Landsliding is another natural process that introduces sediment to the littoral system, but which occurs more episodically (Hapke, 2001). Landsliding on the Big Sur Coast is common, though largely unpredictable on a localized basis.

The Department of Transportation has been involved in a number of landslide repair projects over the years that have included disposal of landslide debris along the shoreline. Various marine studies have documented the results of these actions. A few notable coastal landslide events have precipitated a regulatory response that effectively prohibits the deposit of material into the marine environment from landslide related highway repair. Meanwhile, in spite of the best efforts to reduce

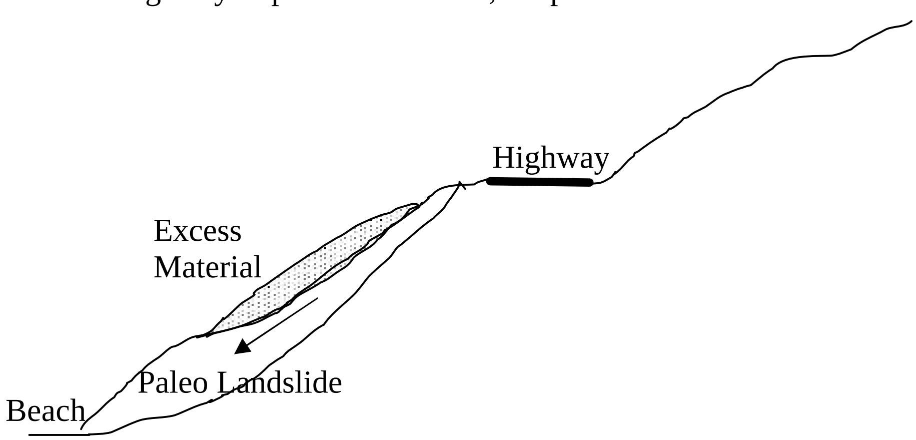


Figure 12: Replenishment, incrementally loading paleo landslides with excess material.

overall impacts and minimize the volumes of excess material generated, there remains an unavoidable need to handle excess material. However, no solid direction has emerged for any particular strategy on the disposition of material.

As part of a corridor-wide planning effort, the Department has engaged stakeholders from regulatory agencies, non-governmental organizations and the community to develop collaborative solutions to the problem.

Conclusion

The Department of Transportation is engaged in continuing dialogue and research to help identify better solutions to the problem of handling excess material associated with landslide-related highway repairs. The discussion also centers on a commitment by the Department to minimize the quantities of excess material generated by landslide-related highway repairs.

Throughout California, there is a growing awareness of the long-term trends of loss of natural sources of sediment inputs associated with dams in coastal watersheds inhibiting fluvial sources and hard shoreline structures inhibiting bluff erosion. While sediment contributions from landslides may be a relatively smaller source of input, the present management approach (hauling material by truck to upland sites) further contributes to the problem by inhibiting a natural source of sediment from entering the littoral system.

Guiding policy on this issue from a broader perspective may be preferred to the existing situation where individual entities struggle to achieve their mission, such as the Department of Transportation under its duty to manage Highway 1, a widely valued infrastructure component that also provides for primary coastal access.

References

California Department of Transportation, District 5. (2000). *Mitigated Negative Declaration: Disposal Sites for Storm Damage Repair, Highway 1 Near Big Sur*.

California Division of Mines & Geology (now, California Geological Survey). (2001). *Landslides in the Highway 1 Corridor: Geology and Slope Stability Along the Big Sur Coast*.

California Department of Transportation. (2001). *Potential Slope Instabilities in the Highway 1 Corridor: Road Condition and Hazard Potential at Sites between San Carpoforo Creek and Carmel Highlands.*

California Department of Transportation, Public Affairs Management, District 5. (2002). *Summaries of Storm Damage and Maintenance Practices Working Group Meetings for the Coast Highway Management Plan*.

Hapke, C. (2001). *Pilot Study for the Long-Term Volumetric Sediment Contribution from Landslides: Big Sur Coastline, California.*

Monterey County Planning Department. (1986). *Big Sur Coast Land Use Plan, Local Coastal Program, Monterey County, California*.

Transportation Research Board. (1996). *Landslides: Investigation and Mitigation*. Special Report 247.

Earthquake Ground Motion for Design of Hoover Dam Bypass Bridge (US Highway 93)

Jeffrey R. Keaton[1]

ABSTRACT

The Hoover Dam Bypass Project is a 3.5-mile corridor on U.S. Highway 93 in Clark County, Nevada, and Mohave County, Arizona, crossing the Colorado River approximately 1,500 feet downstream of Hoover Dam. The 1,896-foot-long bridge will include a 1,090-foot-long main arch span.

The arch bridge was designed on the basis of a nonlinear dynamic analysis using three-component seismograms at each abutment. A 1-s spectral acceleration of 0.139 g was selected as the target ground motion on which to anchor design earthquakes and response spectra. The target acceleration would be produced by a magnitude 6.2 earthquake on the Mead Slope fault at 16 km, or by a magnitude 7.0 earthquake on the California Wash fault at 36 km. The recommended design response spectrum for the river bridge was the maximum motion from the two earthquakes.

A Composite Source Model was used to produce synthetic, three-component seismograms at each abutment for nonlinear dynamic analysis of the river bridge. Input parameters for the Composite Source Model included specific geographic fault location, parameters pertaining to the physics of fault rupture (length, width, average displacement, rake and rupture velocity), and seismological parameters of the source (seismic moment and stress drop) and site area (Green's functions). The acceleration time history records generated with the Composite Source Model were adjusted to bring their acceleration response spectra into close agreement with the design response spectrum.

INTRODUCTION

United States Highway 93 (U.S. 93), a North American Free Trade Agreement (NAFTA) route, has experienced congestion caused by switchbacks on the two-lane road leading to the Hoover Dam site and the restrictions at the concrete arch-dam crossing. The Hoover Dam Bypass Project is a 3.5-mile corridor beginning at approximately milepost 2.2 on U.S. 93 in Clark County, Nevada and crossing the Colorado River approximately 1,500 feet downstream of Hoover Dam, and terminating in Mohave County, Arizona near milepost 1.7 (Figure 1).

The major project stakeholders consist of the Federal Highway Administration, the States of Arizona and Nevada, the Bureau of Reclamation, the Western Area Power Authority, and the National Park Service. Central Federal Lands Highway Division is in the lead management role for all elements of project procurement, design and construction. HDR Engineering, Inc. is providing design and construction support services for the Hoover Dam Bypass Project. An integrated group of professionals from HDR Engineering, T.Y. Lin International, Sverdrup Civil, Inc. and several supporting subconsultants, including AMEC Earth & Environmental, makes up

[1] AMEC Earth & Environmental, 1290 North Hancock Street, Suite 102, Anaheim, California 92807;
Phone: 714-779-2591; e-mail: jeff.keaton@amec.com

the consultant team, collectively know as Hoover Support Team. Design work began in the summer of 2001 and construction is scheduled to be completed in the summer of 2007. The river bridge will be approximately 1,896 feet long, and will include a 1,090-foot-long main arch span.

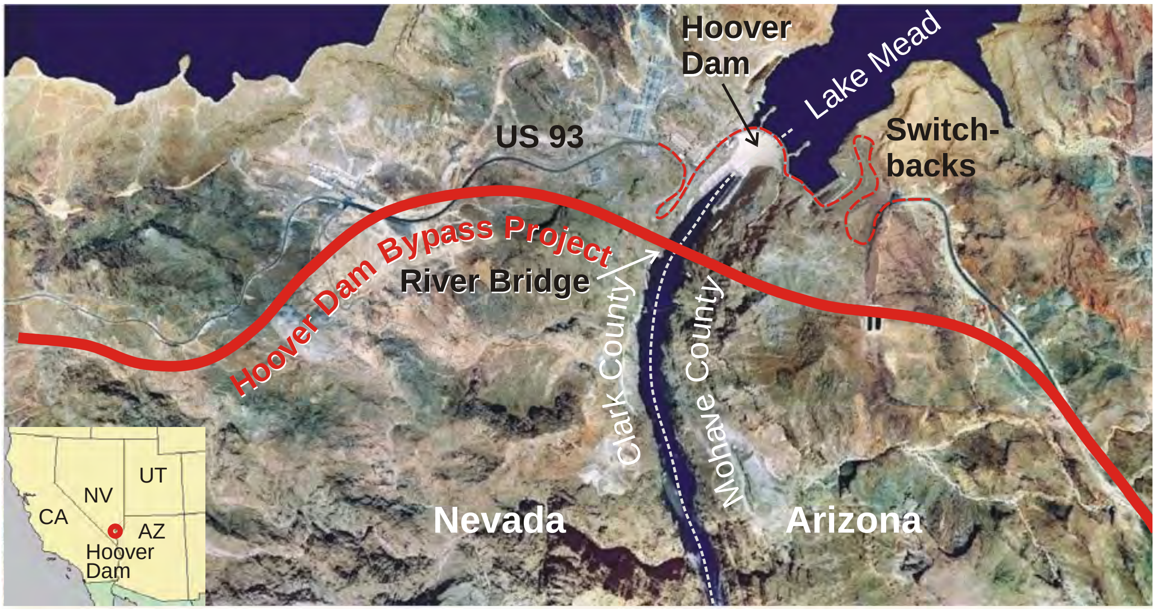


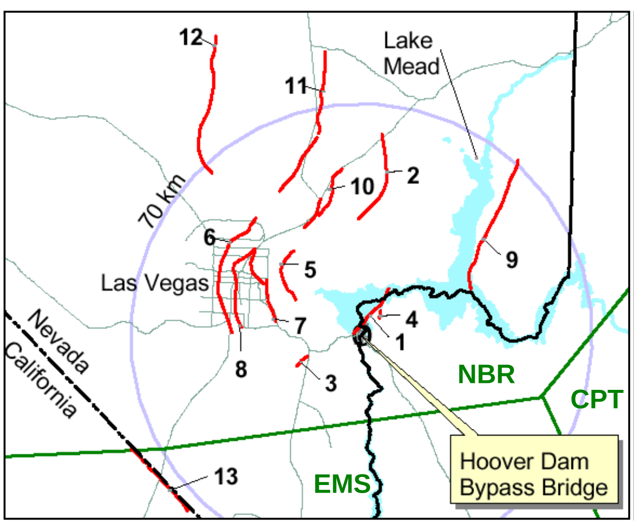
Figure 1. Location of the Hoover Dam Bypass Project.

SEISMOTECTONIC SETTING

Hoover Dam and the proposed Bypass Project bridge and approach structures lie within the physiographic and neotectonic regime of the Northern Basin & Range. The seismic source zone defined as the Eastern Mojave Subprovince of the Southern Basin & Range lies immediately south of the site, whereas the Colorado Plateau–Basin & Range Transition seismic source zone lies to the east. Nearby seismic source zones include the Western Mojave Subprovince of the Southern Basin & Range, the California Basin & Range, the Southwestern Plateau Margin Zone of the Colorado Plateau, and the northwestern part of the Arizona Mountain Zone. The Northern Basin & Range seismic source zone of Wong and others (1992) is equivalent to the Southern Nevada Basin & Range Zone of Euge and others (1992) and the Lake Mead source zone of Anderson and O'Connell (1993). The Northern Basin & Range seismic source zone displays distinctive tectonic habit and related physiography and seismicity. Landforms and dominant geologic structure of the Northern Basin & Range seismic source zone were produced by tectonic processes that began in the mid-Tertiary with large-scale deformation, sedimentation and magmatism (Menges and Pearthree, 1989). This mid-Tertiary tectonism was driven by southwest-oriented crustal extension and was dominated by strike-slip faulting (Bohannon, 1984). The Lake Mead and Las Vegas Shear Zones are large-scale, late-Cenozoic features located within the Northern Basin & Range produced by the mid-Tertiary extensional deformation. Subsequent west-trending Basin & Range extensional deformation dominated by moderately to steeply dipping normal and oblique faults probably began in the late Miocene and was superimposed on older structures. The result of this deformation is expressed in the present landforms, with northwest-trending uplifted mountain ranges separated by broad alluvial basins containing thick accumulations of continental sediments. Most Quaternary faults identified in the region appear to be high-angle normal faults with evidence of multiple faulting events (Anderson and O'Connell, 1993).

The Northern Basin & Range source zone appears to be tectonically active, with a moderate level of seismicity and a number of neotectonic faults that would be considered active or potentially active sources of earthquakes (Anderson and O'Connell, 1993; Euge and others, 1992). A large amount of the historical seismic activity in the region has been artificially induced from nuclear explosions at the Nevada Test Site and reservoir-induced seismicity related to the impoundment of Lake Mead (Rogers, 1977; Rogers and Lee, 1976). However, much of the historical seismicity of the Northern Basin & Range source zone does appear to be tectonic in nature. Earthquakes in this source zone are common and generally of small magnitude. The largest historical earthquake within this source zone was the magnitude 6.1 event that occurred in the Clover Mountains near the Utah-Nevada border in 1966, about 160 km (100 mi) north-northeast of the project site (Euge and others, 1992). Euge and others (1992) estimate the maximum credible event for faults within the Northern Basin & Range source zone to be Mw 7.75, whereas Anderson and O'Connell (1993) place it at Ms 7.25.

Active faults were defined as those having evidence of movement within the past 11 ky (thousand years). Known and suspected active faults within 160 km (100 mi) of the bridge site were considered as sources of earthquakes for design. Geomorphologic or soil stratigraphic evidence indicates that 17 faults have experienced movement within the Holocene or latest Pleistocene (within the last 35 ky). Of the 17 faults, 13 are within the Northern Basin & Range source zone (Figure 2) and 2 faults are within the Colorado Plateau–Basin & Range Transition Zone, with the remaining 2 faults being the Death Valley fault zone and the Garlock fault in California. Two faults relatively close to the bridge site govern seismic design. These faults are the Mead Slope fault and the California Wash fault. The USGS seismic hazard mapping project excluded the Echo Bay-Overton Arm fault because slip rate data has not been developed.



No	Fault Name
1	Mead Slope
2	California Wash
3	Black Hills
4	West Fortification Ridge
5	Frenchman Mountain
6	Eglington-Decatur
7	Cashman-Whitney Mesa
8	Las Vegas Valley
9	Echo Bay-Overton Arm
10	West Dry Lake Range
11	West Arrow Canyon Range
12	West Sheep Range
13	Pahrump Valley

ID	Seismic Zone
NBR	Northern Basin & Range
EMS	Eastern Mojave Subprovince of Southern Basin & Range
CPT	Colorado Plateau-Basin & Range Transition

Figure 2. Quaternary faults within about 80 km of Hoover Dam Bypass Project.

The Mead Slope fault is located in Mohave County, Arizona within the Northern Basin & Range seismic source zone on the piedmont slope below Fortification Hill on the eastern side of Lake Mead and evidently continues some distance beneath Lake Mead (Pearthree, 1998). Anderson and O'Connell (1993) studied the fault in some detail and conclude that it is a steeply-dipping, oblique-slip fault with significant reverse and possible left-lateral strike-slip movement. Evidence suggests that the last movement on the fault was late Quaternary and possibly early Holocene. Anderson and O'Connell (1993) conclude that the minimum fault length is 11 km and the maximum length is 17 to 18 km with a northwesterly strike and a southeasterly dip. Pearthree (1998) reports a 7-km length, which is the length of the fault not concealed by Lake Mead. No slip rate or recurrence interval has been determined for the Mead Slope fault. Pearthree (1998) estimates it to be <0.2 mm per year. Anderson and O'Connell (1993) assign a maximum credible earthquake of Mw 6.75 to the Mead Slope fault based on conservative estimates of fault length and field evidence. The Wells and Coppersmith (1994) relationship produces a maximum magnitude of Mw 6.5 for an assumed length of 18 km.

The California Wash fault is located in Clark County, Nevada within the Northern Basin & Range seismic source zone on the western side of the Muddy Mountains (Wyman and others, 1993). The California Wash fault was named by Bohannon (1983), but identified as the Muddy Mountains fault and West Muddy Mountains fault by Menges and Pearthree (1983) and Wyman and others (1993), respectively. Wyman and others (1993) identify the California Wash fault as a down-to-the-west normal fault with a south-southwesterly strike and a length of 32 km. Wyman and others (1993) and Anderson and O'Connell (1993) conclude the last movement to be late Quaternary to early Holocene. Anderson and O'Connell (1993) assign a late Quaternary length of 30 km and the U.S. Geological Survey (USGS) Seismic Hazards website (2001) assigns a length of 40 km to the fault. Wyman and others (1993) report the slip-rate at 0.01 to 0.2 mm per year with a preferred slip rate of 0.05 mm/year. The USGS Seismic Hazards website (2001) reports a slip-rate of 0.1 mm/year. Anderson and O'Connell (1993) report evidence for up to 2.1 m of offset of the last event. The recurrence interval for the California Wash Fault is reported 10 to 100 ky (Anderson and O'Connell, 1993; Wyman and others, 1993). Anderson and O'Connell (1993) assign a maximum credible earthquake of Mw 7.25 based on the length of the fault and the evidence for 2.1 m of offset, whereas Wyman and others (1993) assign a maximum earthquake of Mw 6.5 to 7.25 with a preferred Mw 7.0. The Wells and Coppersmith (1994) relationship produces Mw 6.9 for an assumed length of 40 km.

DESIGN RESPONSE SPECTRUM

Both deterministic and probabilistic analytical techniques can be applied to the task of quantifying seismic hazard. The preceding section of this paper that addresses earthquake potential of faults within 160 km of the bridge site are part of the deterministic approach, even though the definition of an active fault is inherently probabilistic. A deterministic approach is required for design of some high-consequence facilities, such as dams, where failure could have catastrophic off-site effects. A probabilistic approach is the basis for building codes and for the current standard AASHTO (1996) specifications for highway bridges. The probabilistic approach recognizes that a structure might be exposed to forces that exceed those used in design, but also recognizes that such an event has an acceptably low probability of occurring. The AASHTO specifications use an acceptable exceedance probability of 10 percent in 50 years for bridges with spans less than 500 feet and constructed using conventional steel and concrete girders. The design team for the Hoover Dam Bypass Bridge believed that its length and importance warrant consideration of a 100-year design life with an acceptable exceedance probability of 10 percent for the earthquake ground motion. A probability of 10 percent in 100 years corresponds to an average recurrence interval of 950 years.

Earthquake ground motion for the Hoover Dam Bypass Bridge was approached with a method that is simultaneously consistent with the probabilistic nature of building codes, but also based on deterministic geoseismic information to develop reasonable design parameters. Regional probabilistic seismic hazard mapping was conducted by the USGS in 1996 and published on their website (http://geohazards.cr.usgs.gov/eq/). The USGS hazard maps are the basis for ground motion used in the International Building Code, and depict peak and spectral accelerations on rock sites corresponding to probabilities of 10, 5, and 2 percent in 50 years. A probability of 5 percent in 50 years corresponds to a recurrence of 975 years, a value that is very close to the 950 years corresponding to a probability of 10 percent in 100 years. Design ground motion for the International Building Code is taken as 2/3 of the acceleration corresponding to 2 percent probability in 50 years. Although the International Building Code is not the design basis for the Hoover Dam Bypass Project, the ground motion that would apply also was determined.

Peak horizontal acceleration commonly is used as an index parameter for seismic hazards. The bridge that will span the Colorado River will have a low fundamental frequency. Therefore, the low frequency ground motion was considered to be more important in its design than high frequency ground motion. However, higher frequency ground motion was more important in the design of the approach bridges. Horizontal acceleration values for the Hoover Dam Bypass Bridge abutments from the USGS website are shown in Table 1.

Table 1. Summary of peak and spectral accelerations in g

Nevada Abutment

	50-Year Exceedance Probability			2/3 of
	10%	5%	2%	2% in 50 yr
PGA	0.140	0.205	0.331	0.220
Sa 5 Hz	0.324	0.477	0.799	0.533
Sa 3.3 Hz	0.276	0.411	0.637	0.425
Sa 1 Hz	0.086	0.129	0.208	0.139

Arizona Abutment

	50-Year Exceedance Probability			2/3 of
	10%	5%	2%	2% in 50 yr
PGA	0.140	0.205	0.330	0.220
Sa 5 Hz	0.323	0.476	0.797	0.531
Sa 3.3 Hz	0.275	0.411	0.636	0.424
Sa 1 Hz	0.086	0.129	0.208	0.139

Note: PGA designates peak horizontal ground acceleration on rock sites; Sa designates spectral acceleration on rock sites for frequencies of 5, 3.3, and 1 cycle per second.

The 1-Hz spectral acceleration of 0.139 g was taken to be the anchor point for developing a design response spectrum for the project because the bridge has a relatively low fundamental frequency and the value was consistent with the International Building Code without exceeding by much the 5-percent 50-year spectral acceleration for 1 Hz. Linear interpolation between the 1-Hz spectral acceleration values for 5 and 2 percent exceedance probability results in a average recurrence of 1057 years for the 0.139 g motion.

The attenuation relationship developed by Abrahamson and Silva (1997) was used to evaluate the distance at which earthquake magnitudes would be expected to produce the a 1-Hz spectral acceleration of 0.139 g, consistent with the design philosophy described above. An earthquake of Mw 6.2 at a hypocentral distance of 16 km would produce the 0.139-g spectral acceleration with Abrahamson and Silva's hanging-wall effect. Similarly, an Mw 7.0 earthquake at a hypocentral distance of 36 km would produce the 0.139-g spectral acceleration.

Mw 6.2 for an earthquake at 16 km matches well with the deaggregation for the 5-percent 50-year 1-Hz spectral acceleration shown on the USGS website. The distance would be consistent with a hypocenter on the Mead Slope fault, and the magnitude is less than the maximum credible magnitude described in the Seismotectonic Setting section of this paper. The deaggregations for the 2-percent 50-year 1-Hz spectral acceleration shown on the USGS website would be consistent with an Mw 7.0 earthquake at 36 km on a southward extension of the California Wash fault. Mw 7.0 is the maximum earthquake used by the USGS in their probabilistic seismic hazard model, which is less than the maximum credible magnitude described above. Therefore, design earthquakes for the bridge site would have Mw 6.2 and 7.0 and occur on faults that exist at distances of 16 and 36 km.

Acceleration response spectra (Figure 3) for these two design earthquakes were calculated using the relationship developed by Abrahamson and Silva (1997). The spectral ordinates based on International Building Code procedures are also shown on Figure 3. The design spectrum for the project was taken as the higher spectral acceleration for each period, meaning that the Mw 6.2 earthquake governs at periods below 1 s (at frequencies higher than 1 Hz).

COMPOSITE SOURCE MODEL

Design of the bridge that will span the Colorado River will be based on a nonlinear dynamic analysis, whereas the approach bridges will be designed on the basis of a response spectrum analysis. Structural analysis of the main span requires three-component seismograms at each bridge abutment, whereas structural analysis of the approach bridges requires only an appropriate acceleration response spectrum for each structure. Two methods are available for developing three-component seismograms for use in design: 1) modify an actual recording of an historical earthquake to match the shape of the design response spectrum, 2) develop synthetic seismograms from an appropriate model of earthquake sources in the site region.

The earthquake sources in the vicinity of the bridge site are mostly normal faults. Very few recordings of historical normal faulting earthquakes are available to use in developing seismograms for design of the bridge. Furthermore, additional problems exist with modifying records of historical earthquakes for use in design, such as appropriate tectonic style of fault rupture, earthquake magnitudes, source-to-site geometry, and site conditions of the recording instrument compared to the design location.

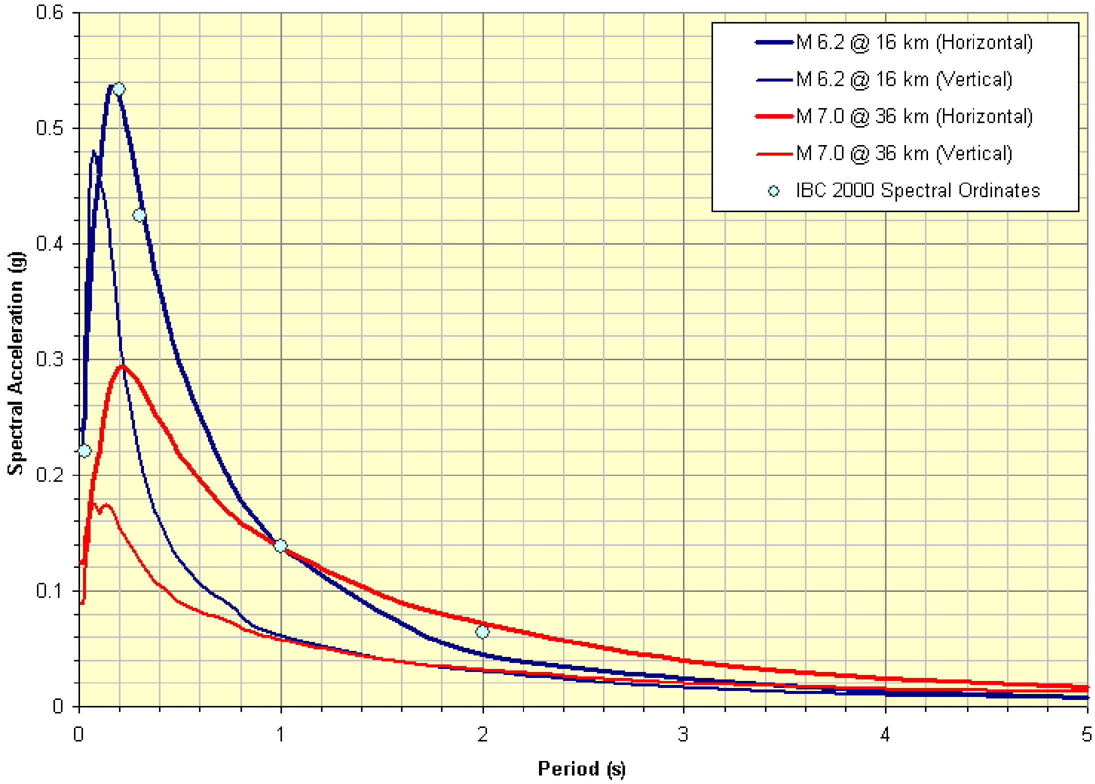


Figure 3. Acceleration response spectra for the Hoover Dam Bypass Project

Synthetic seismograms can be generated two ways: 1) a band-limited white noise (random motion) model can be used, or 2) a mathematical model based on the physics of fault rupture and seismic-wave propagation can be used. The first method can produce seismograms that have reasonable frequency content, but they do not correspond to geology or tectonic style. The second method incorporates physical and seismological parameters of earthquake sources and site geology. The Composite Source Model of Zeng and others (1994) was used to develop synthetic seismograms for use at the Hoover Dam Bypass Project. Until recently, the Composite Source Model was used to model ground motions generated by well-documented earthquakes (Su and others, 1994a, b; Zeng and Anderson, 1996; Anderson and Yu, 1996; Keaton et al., 2000a). It has been used for engineering design on two building projects in Utah, one dam project in California, and the Hoover Dam Bypass Bridge Project in Arizona and Nevada.

Key elements of the Composite Source Model are shown on Figure 4. The model consists of a fault with a specific location and finite dimensions. The dimensions of the fault and the average displacement must be consistent with seismic moment, hence the design earthquake magnitude. Asperities on the fault plane are modeled as circular features, the number and radius of which are constrained by seismic moment. These randomly distributed asperities produce the variability in the frequency content of the seismograms. The hypocenter of the earthquake can be assigned at any point on the fault, and the orientations of the horizontal components are specified.

The sense of slip on the fault is modeled by the rake of subevents which generate a pulse of energy as the constant-velocity rupture front passes the center of each asperity. Subevent rake is

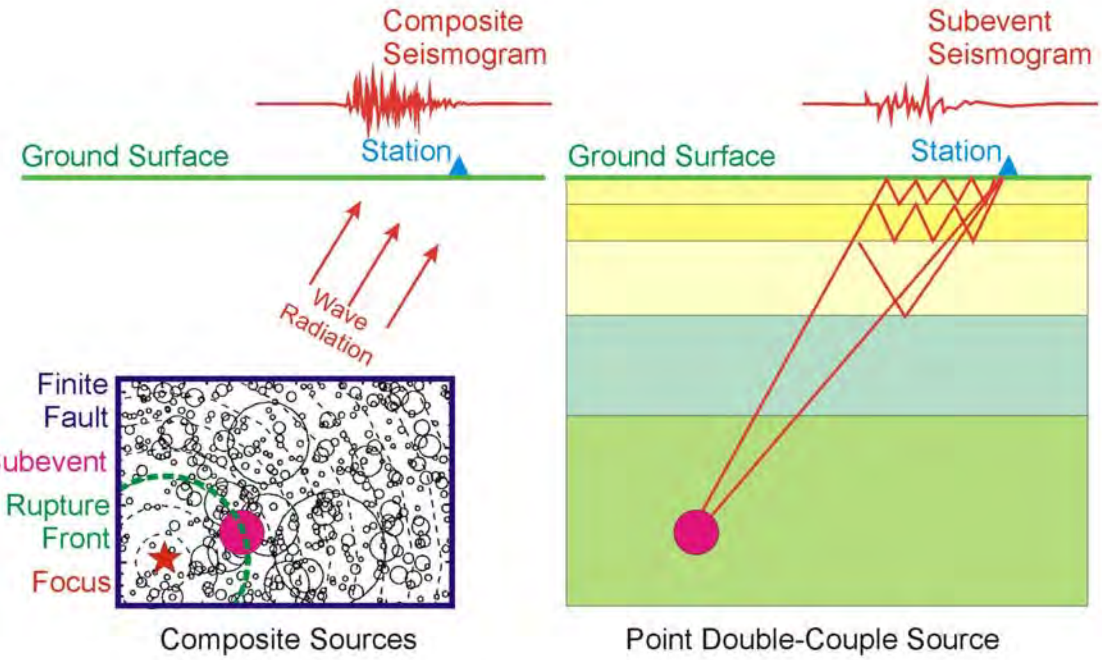


Figure 4. Key elements of the Composite Source Model

-90º for purely normal-slip faults, whereas rake is 0° for purely left-lateral faults, 180° for purely right-lateral strike-slip faults, and 90° for purely reverse-slip faults. The duration of each subevent is controlled by asperity radius and stress drop. The time function of each subevent is transferred to a geographically located site using synthetic Green's functions for a flat layered medium. A schematic representation of the ray paths contributing to the seismogram at the site generated by a single subevent is shown on the right-hand side of Figure 4. The seismograms from all of the subevents are superimposed to produce the synthetic seismogram for the earthquake (left-hand side of Figure 4). The model also includes a ray-scattering effect.

Synthetic Green's functions require that the upper 1,000 km of the earth's crust be represented in terms of compression- and shear-wave velocity and density. The compression-wave velocity in uppermost part of the volcanic bedrock along the Colorado River canyon was measured by a refraction seismic survey as part of the geotechnical investigation. A compression-wave velocity of 6000 feet per second was obtained. A sample of the rock material was collected and the density was determined to be approximately 145 pounds per cubic foot using Archimedes' principle on a representative sample. Shear-wave velocity was assumed to be approximately 60 percent of the compression-wave velocity. Pertinent values for deeper layers were taken from Su and others (1996) for a site in Las Vegas. The velocity structure is summarized in Table 2.

Basin & Range earthquakes commonly have focal depths of 12 to 15 km. A hypocentral distance of 16 km corresponds to a horizontal (epicentral) distance of 10.6 km for an earthquake with a focal depth of 12 km. A horizontal distance of 10.6 km from the bridge site is near the northeast end of the Mead Slope fault, and this fault was used in the Composite Source Model as the source of the Mw 6.2 earthquake. An earthquake with a moment magnitude of 6.2 has a seismic moment of 1.995E+25 dyne-cm. The Mead Slope fault dips to the southeast at 70°.

Table 2. Velocity structure for Composite Source Model

Depth (km)	Compression Wave Velocity (km/s)	Compression Wave Attenuation (Qp)	Shear Wave Velocity (km/s)	Shear Wave Attenuation (Qs)	Density (gm/cc)
0.2	1.8	150	1.1	70	2.35
0.7	3.2	150	1.9	70	2.4
0.6	3.6	300	2.1	150	2.4
1.5	5.0	400	2.9	200	2.5
2.2	5.8	800	3.4	400	2.7
10.7	6.2	800	3.5	400	2.75
16	6.5	800	3.8	400	2.9
1000	7.8	800	4.6	400	3.3

Therefore, a maximum depth of 15 km for the fault plane corresponds to a fault width of 15.96 km. A fault length of 14 km and a shear modulus of 3.5E+11 dynes/cm^2 correspond to a seismic moment of 1.995E+25 dyne-cm with an average displacement of 25.5 cm. A mean average displacement of 29 cm is predicted for normal faulting earthquakes of moment magnitude 6.2, according to Wells and Coppersmith (1994). Therefore, these fault dimensions and displacement were judged to be reasonable for the Composite Source Model. Stress drops of 20 and 50 bars and attenuation factors of 0.030 were used in the model.

The USGS assigned a maximum moment magnitude of 7.0 to the California Wash fault. No other faults capable of generating moment magnitude 7 earthquakes are known to exist within 40 km of the Hoover Dam Bypass Bridge site. The south end of the California Wash fault is located approximately 36 km north of the bridge abutments, and was selected to be the source of the moment magnitude 7.0 earthquake. It was necessary to extend the California Wash fault southward so that a hypocentral distance of 36 km would coincide with a focal depth of 12 km on the westward-dipping fault plane. An epicentral distance of 33.9 km corresponds to the 36-km hypocentral distance and 12-km focal depth. An earthquake of Mw 7.0 has a seismic moment of 3.162E+26 dyne-cm. The California Wash fault dips to the west at 60°. Therefore, a maximum depth of 15 km for the fault plane corresponds to a fault width of 17.32 km. A fault length of 42.524 km and a shear modulus of 3.5E+11 dynes/cm^2 correspond to a seismic moment of 3.162E+26 dyne-cm with an average displacement of 122.7 cm. A mean average displacement of 91 cm is predicted for normal faulting earthquakes of moment magnitude 7.0, according to Wells and Coppersmith (1994), and 124 cm is predicted to be the displacement that is one standard deviation above the mean. Therefore, these fault dimensions and displacement were judged to be reasonable for the Composite Source Model. Stress drops of 100 and 150 bars and attenuation factors of 0.025 were used in the model.

The geometries of the Mead Slope and California Wash faults with respect to the Hoover Dam Bypass Bridge abutments are shown on Figure 5. It can be seen that the bridge is vertically above the southeast-dipping Mead Slope fault plane. A southwest connection from the Mead Slope fault to the Black Hills fault is shown on Figure 5, but the area of the fault over which rupture occurred in the Composite Source Model is shown by the broad hachures.

The Mead Slope fault seismograms for the Mw 6.2 earthquake are shown on Figure 6 for the Nevada abutment and on Figure 7 for the Arizona abutment. Acceleration response spectra for the horizontal and vertical components of the seismograms are shown on Figure 8.

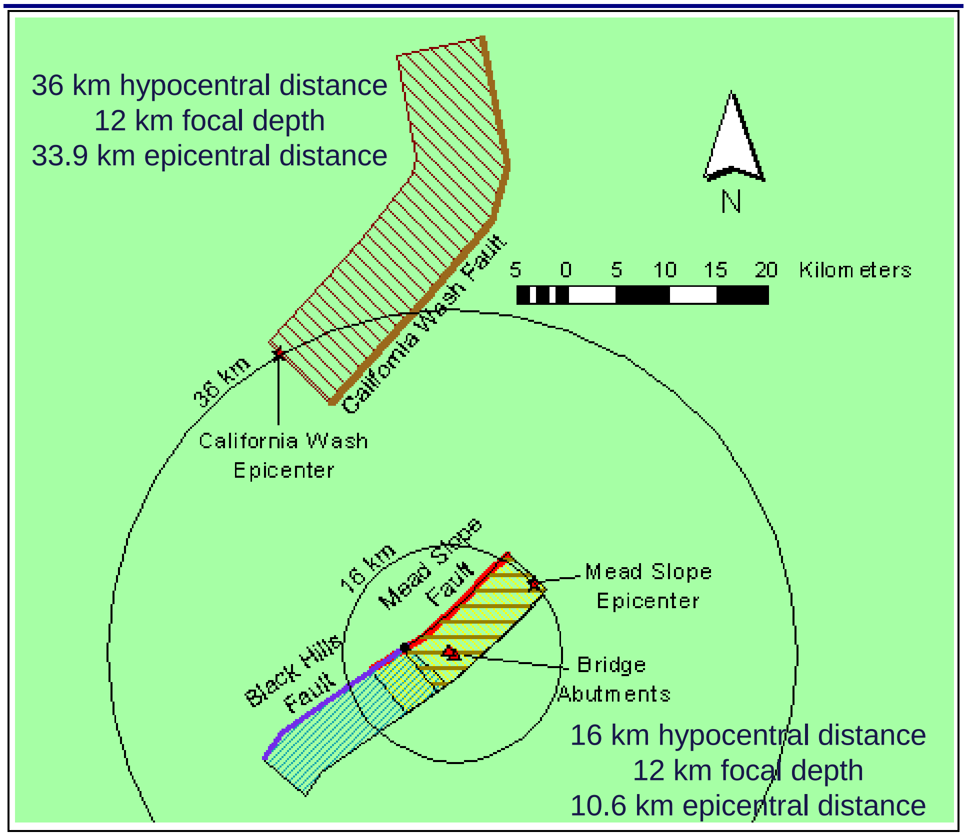


Figure 5. Source-to-site geometry of Mead Slope fault and California Wash fault

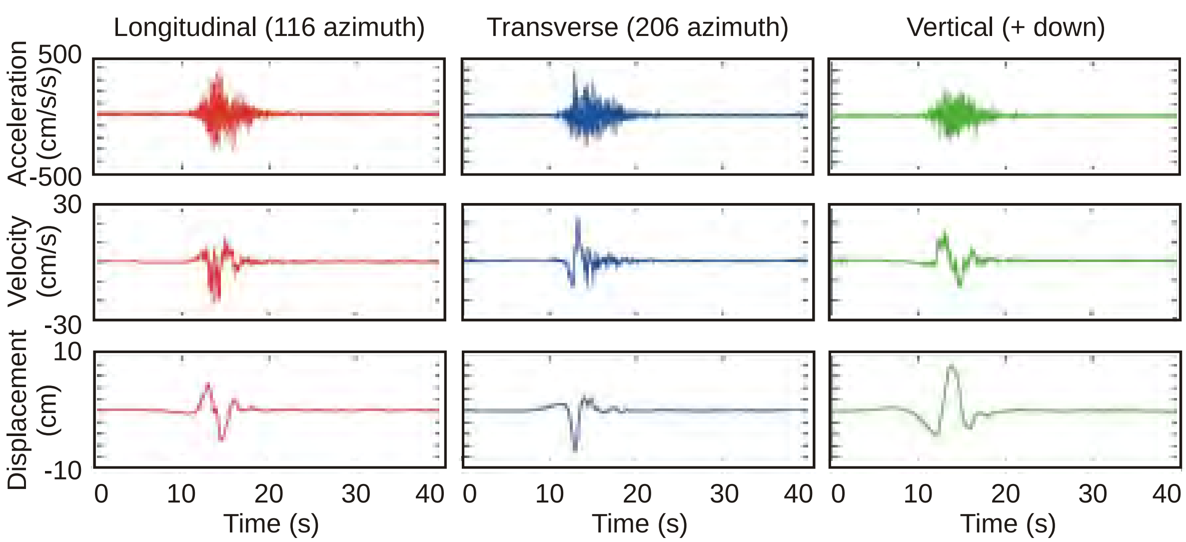


Figure 6. Mead Slope fault seismograms for the Nevada abutment

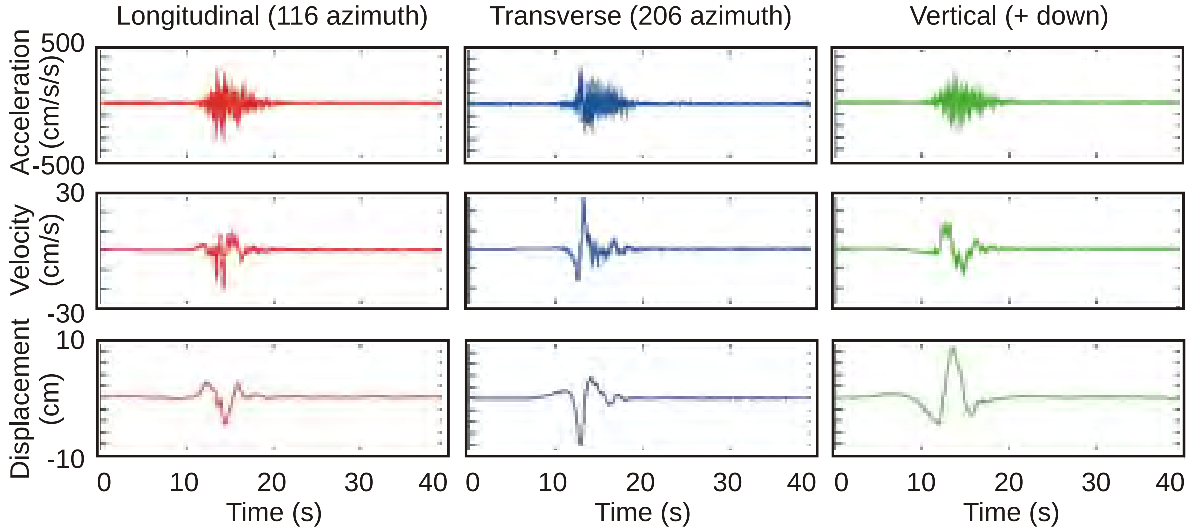


Figure 7. Mead Slope fault seismograms for the Arizona abutment

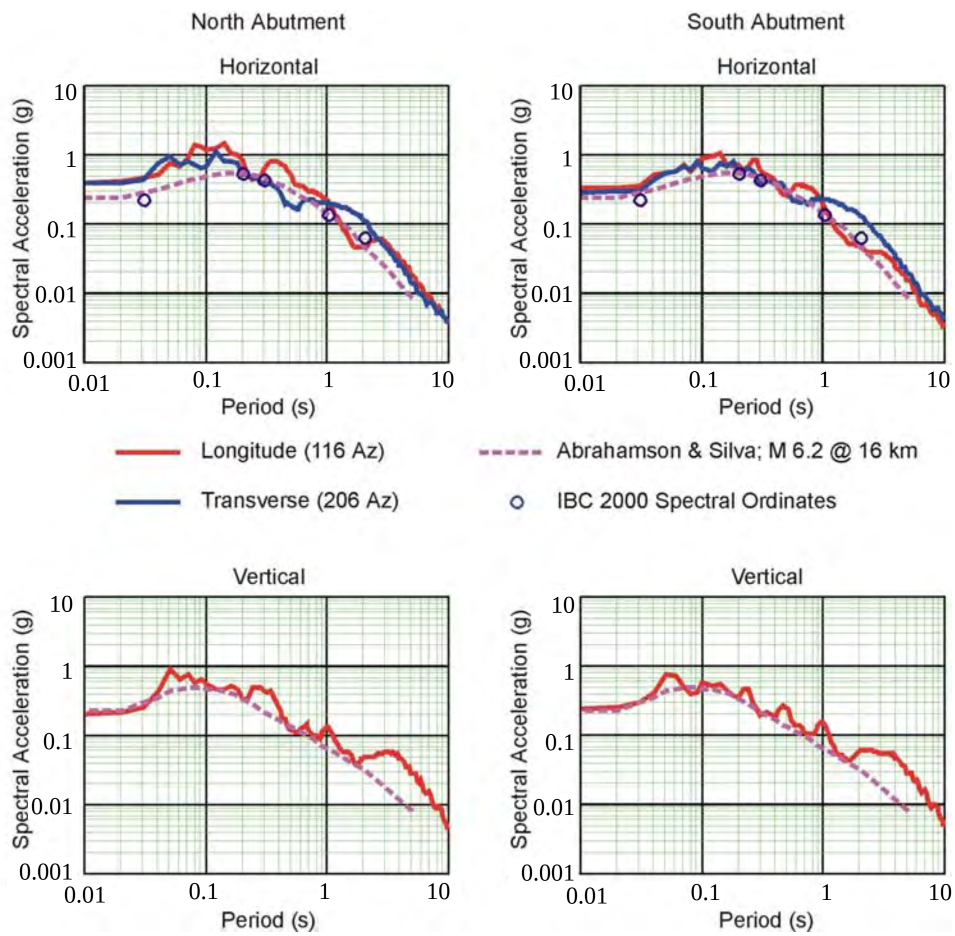


Figure 8. Mead Slope fault acceleration response spectra

The California Wash fault seismograms for the Mw 7.0 earthquake are shown on Figure 9 for the Nevada abutment and on Figure 10 for the Arizona abutment. Acceleration response spectra for the horizontal and vertical components of the seismograms are shown on Figure 11.

The seismograms are realistic in general appearance, including maximum amplitude of motion and duration of strong shaking. The velocity records for the Mead Slope fault earthquake show a single, prominent spike of maximum velocity. This spike in the velocity record has been called the 'fault fling' effect and has been recorded at locations that are close to faults producing large earthquakes. The fault fling effect is not present in the California Wash record because the abutments are too far from the fault.

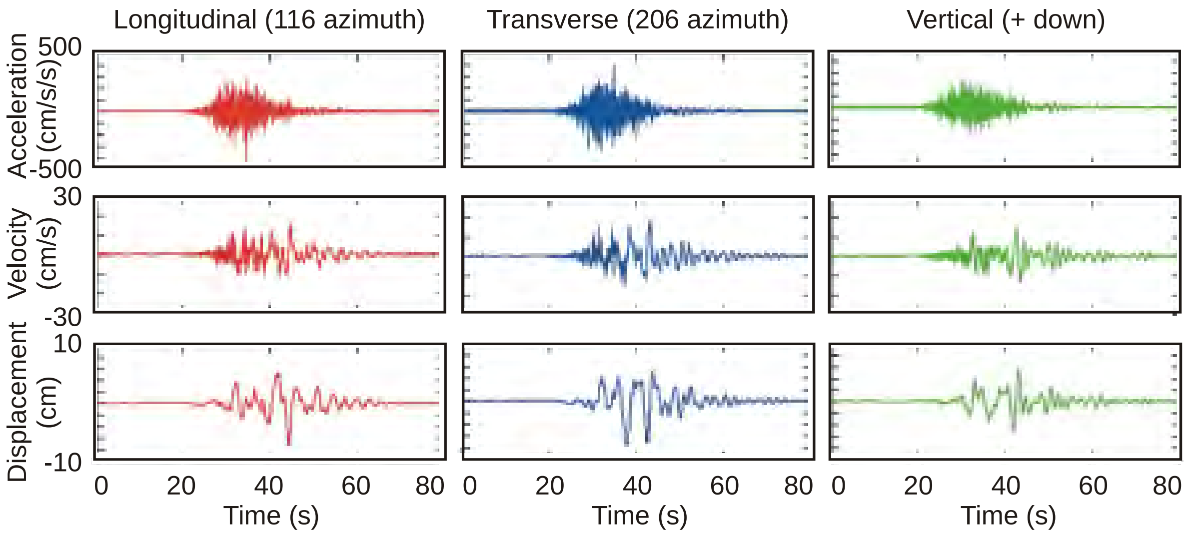


Figure 9. California Wash fault seismograms for the Nevada abutment

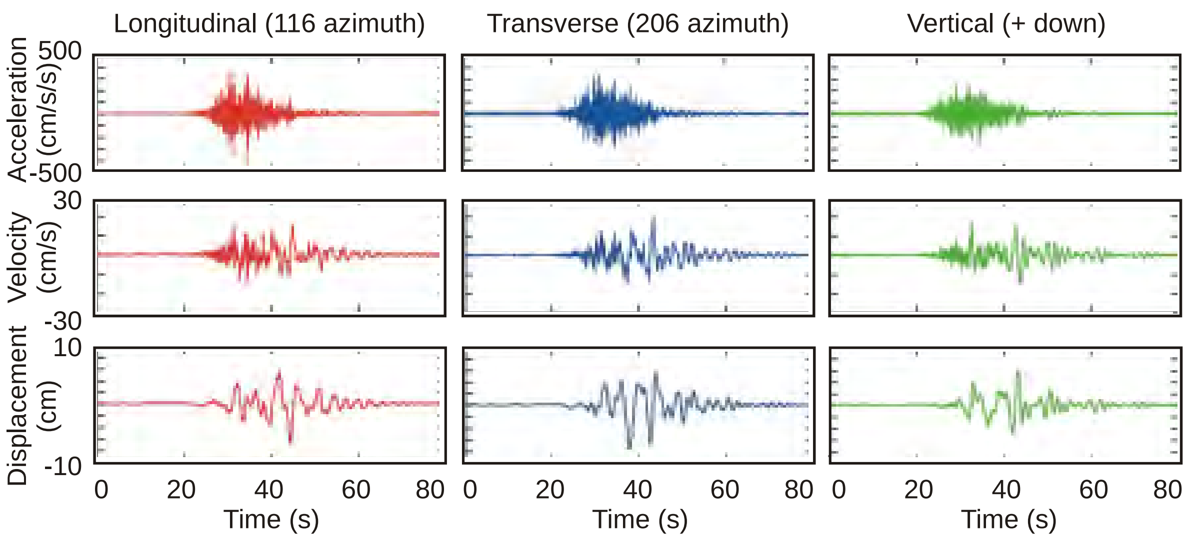


Figure 10. California Wash fault seismograms for the Arizona abutment

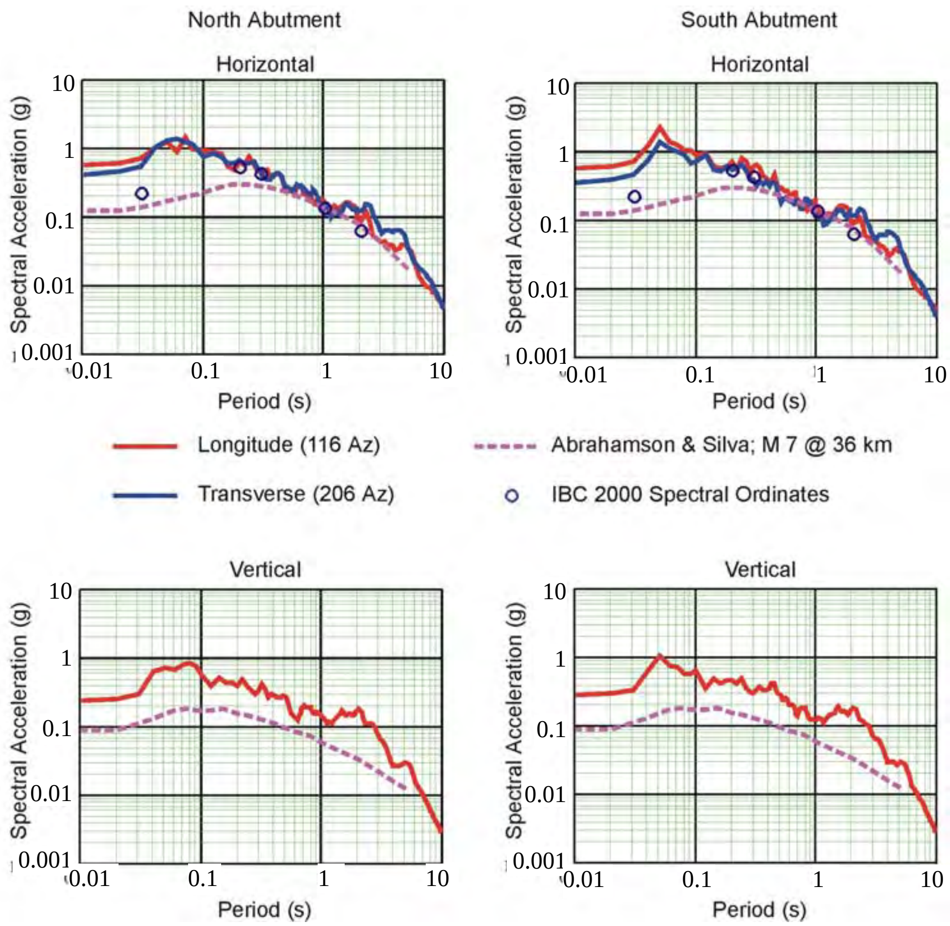


Figure 11. California Wash fault acceleration response spectra

The response spectra for the synthetic seismograms match reasonably well the target response spectra calculated with the attenuation relationship of Abrahamson and Silva (1997). The Mead Slope records are very close to the target spectra. The California Wash records are close to the target spectra in the period range above 0.4 s for the horizontal components. At periods less than 0.4 s, the horizontal components exceed the target spectrum by a factor of two or three. The vertical components of the California Wash records exceed the target spectrum at all periods. The reason that the California Wash records do not more closely match the target spectrum may be related directivity effects caused by the pronounced bend in the California Wash fault and nonlinear site response effects that are not included in the Composite Source Model.

SPECTRAL RESPONSE ADJUSTMENT

The acceleration response spectra calculated for the Composite Source Model seismograms were judged to differ too much from the design response spectrum for the seismograms to be used directly in the nonlinear dynamic analysis of the bridge structure. Adjustments were made to acceleration-time history records by computing the response spectra and determining the amount of adjustment needed in different period ranges to nearly match the design spectrum using the procedure described by Keaton and others (2000b). Fourier spectra are computed for the acceleration-time histories, and the Fourier amplitude spectra are adjusted while the Fourier phase spectra are preserved. The inverse Fourier transform is used to recompute acceleration-time histories. The procedure is iterated to achieve reasonably optimal spectral matching. Velocity- and displacement-time history records and response spectra are computed from the adjusted acceleration-time histories in the usual way. Low-frequency harmonic motion introduced by the scaling procedure was removed by tapering the displacement-time history records to zero at reasonable times based on the shapes of the original acceleration-time histories. Corresponding velocity- and acceleration-time histories are computed by numerical differentiation and final acceleration-response spectra are recomputed. The results of this scaling for the California Wash earthquake motion at the Nevada abutment are shown on Figure 12.

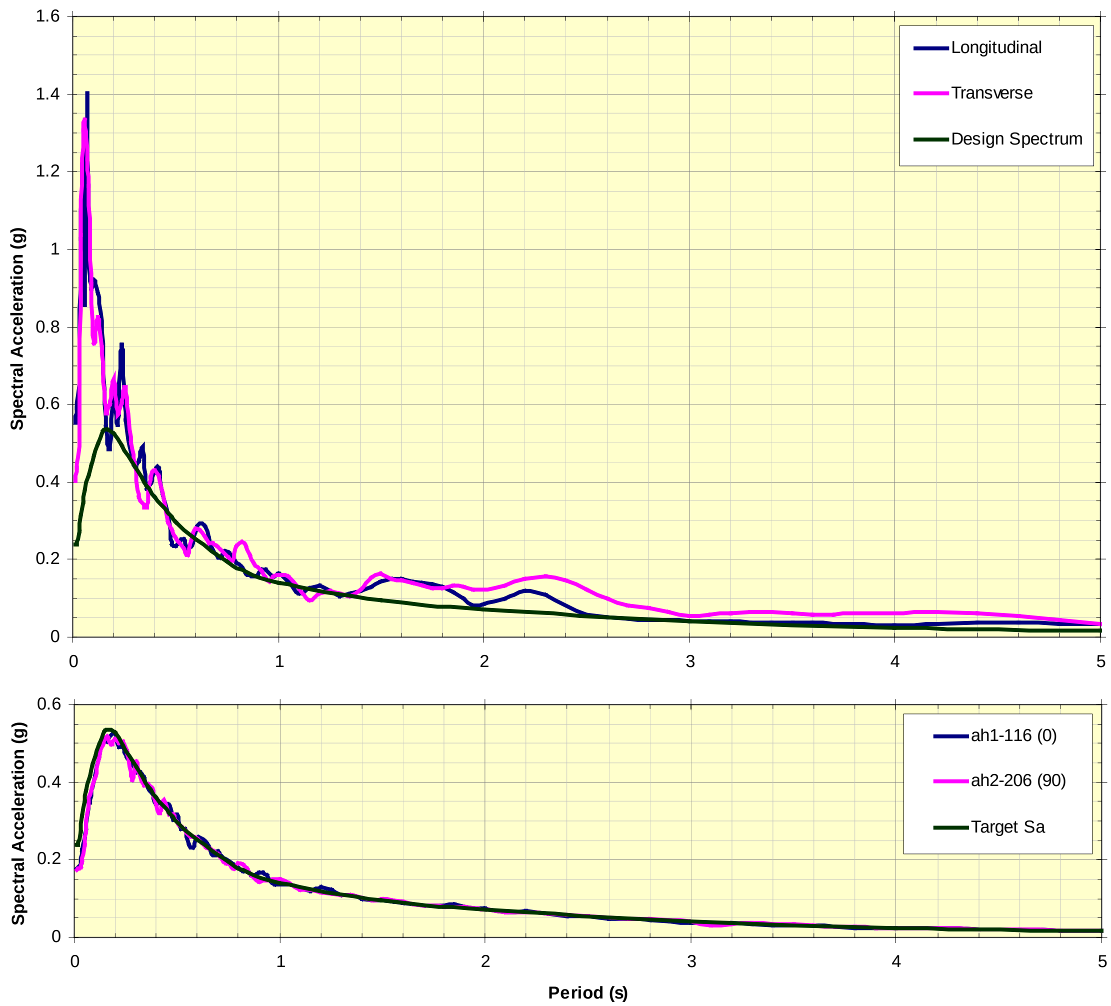


Figure 12. California Wash earthquake horizontal response spectra directly from Composite Source Model (top) and adjusted to match design spectrum (bottom)

RECOMMENDED GROUND MOTION

Three-component seismograms oriented longitudinal to the bridge, transverse to the bridge, and vertical were produced for each abutment of the bridge for the Mw 6.2 Mead Slope earthquake and for the Mw 7.0 California Wash earthquake. T.Y. Lin International used the motions in their dynamic analysis of the bridge structure. Horizontal displacement-time histories from the California Wash earthquake are shown on Figure 13 and a horizontal abutment-displacement map is shown on Figure 14. The abutment displacement map is shown with an arbitrary offset in the longitudinal direction of -3 cm for the Nevada abutment and +3 cm for the Arizona abutment. The vertical displacement-time histories are not shown in this paper, but were included in the dynamic analysis of the bridge structure.

ACKNOWLEDGEMENTS

The seismotectonic analysis of earthquake sources was conducted under the direction of Ralph E. Weeks (AMEC Earth & Environmental, Phoenix, Arizona) with assistance from David Burton Slemmons (Professor Emeritus, University of Nevada-Reno). John G. Anderson and Yuehua Zeng (Professors, Seismological Laboratory, University of Nevada-Reno) assisted with spectral matching adjustments of the Composite Source Model seismograms. Michael L. Rucker (AMEC Earth & Environmental, Phoenix, Arizona) conducted the refraction-seismic survey.

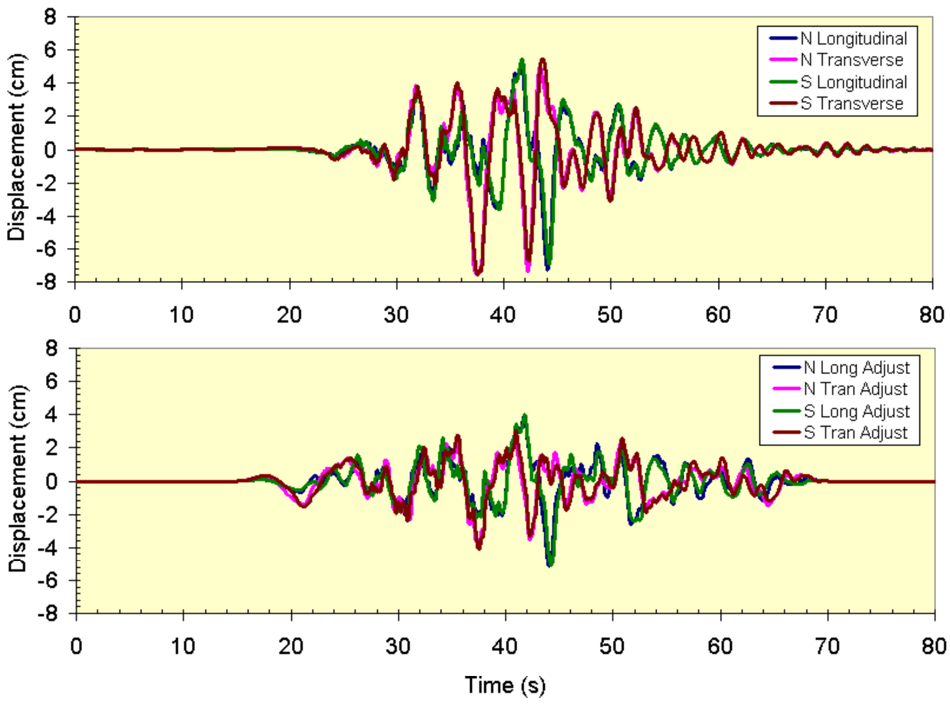


Figure 13. California Wash earthquake displacement-time histories from the Composite Source Model (top) and adjusted to match the design acceleration response spectrum (bottom)

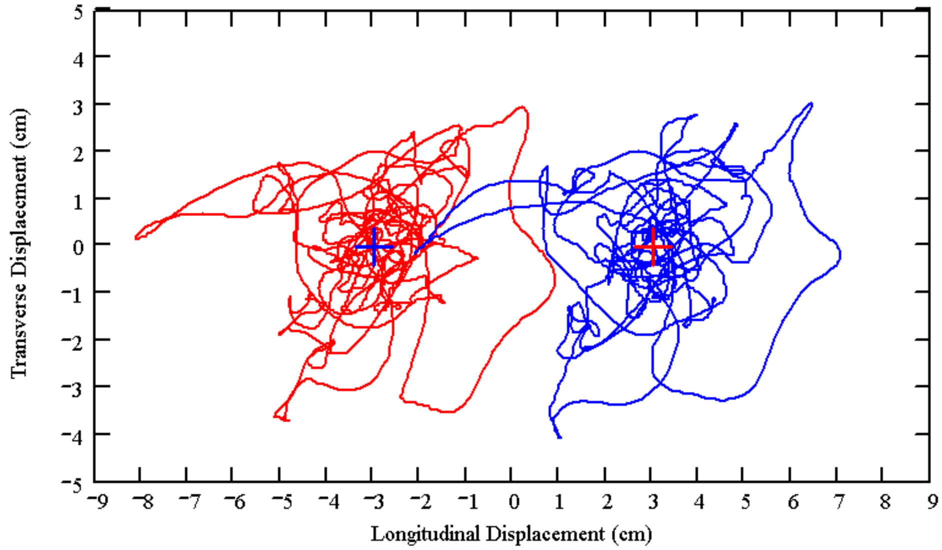


Figure 14. Horizontal displacement map for the adjusted California Wash earthquake for the Nevada (left red) and Arizona (right blue) abutments of the Hoover Dam Bypass Bridge

REFERENCES

Abrahamson, N.A. and Silva, W.J., 1997, Empirical Response Spectral Attenuation Relation For Shallow Crustal Earthquakes, Seismological Research Letters, Vol. 68, No.1, p. 94-127.

American Association of State Highway and Transportation Officials (AASHTO), 1996, Standard Specifications for Highway Bridges, 16th Edition, Washington, D.C.

Anderson, J.G., and Yu, G., 1996, Predictability of Strong Motions from the Northridge, California, Earthquake: Bulletin of the Seismological Society of America, v. 86, no. 1B, p. S100-S114.

Anderson, L. W. and O'Connell, D.R., 1993, Seismotectonic Study of the Northern Portion of the Lower Colorado River, Arizona, California, and Nevada for Hoover, Davis, and Parker Dams, Bureau of Reclamation, Geotechnical and Engineering and Geology Division, Geology Branch, Seismotectonics and Geophysics Section, Seismotectonic Report 93-4.

Bohannon, R.G. 1983, Mesozoic and Cenozoic tectonic development of the Muddy, North Muddy, and northern Black Mountains, Clark County, Nevada, in D.M. Miller, V.R. Todd, and K.A. Howard (eds.), Tectonic and Stratigraphic studies in the eastern Great Basin: GSA, Memoir 157, p. 125-148.

Bohannon, R.G. 1984, Nonmarine sedimentary rocks of Tertiary age in the Lake Mead region, southeastern Nevada and northwestern Arizona: USGS Professional Paper 1259, 72 p.

Euge, K.M., Schell, B.A., and Po Lam, I., 1992, Development of Seismic Acceleration Contour Maps for Arizona, Phoenix: Arizona Department of Transportation. Report No. AZ92-394.

Keaton, J.R., Anderson, J.A., and Zeng, Y., 2000a, Synthetic normal-faulting-earthquake seismograms: Proceedings of the Sixth International Conference on Seismic Zonation, Earthquake Engineering Research Institute, Oakland, CA, CD-ROM Paper 00204.pdf, 6 p.

Keaton, J.R., Zeng, Y., and Anderson, J.A., 2000b, Procedure for scaling earthquake records to match acceleration response spectra for engineering design: Proceedings of the Sixth

International Conference on Seismic Zonation, Earthquake Engineering Research Institute, Oakland, CA, CD-ROM Paper 00130.pdf, 6 p.
Menges, C.M. and Pearthree, P.A., 1983, Map of neotectonic (latest Pliocene-Quaternary) deformation in Arizona, Arizona Bureau of Geology and Mineral Technology, Open File Report 83-22. 40 p.
Menges, C.M. and Pearthree, P.A., 1989, Late Cenozoic Tectonism in Arizona and its Impact on Regional Landscape Evolution in Geologic Evolution of Arizona, Eds., J.P. Jenney and S.J. Reynolds, Tucson: Arizona Geological Society Digest 17.
Pearthree, P.A., 1998, Quaternary Fault Data and Map of Arizona, Tucson: Arizona Geological Survey, Open-File Report 98-24.
Rogers, A.M., 1977, A Preliminary Assessment of the Seismic Hazard of the Nevada Test Site Region; Bulletin of the Seismological Society of America, V. 67, p. 1587-1606.
Rogers, A.M. and Lee, W.H.K., 1976, Seismic Study of Earthquakes in the Lake Mead, Nevada-Arizona Region, Bulletin of the Seismological Society of America, V. 66, No. 5, p. 1657-1681.
Su, F., Anderson, J.G., Ni, S.D., and Zeng, Y., 1996, Site amplification in Las Vegas estimated from the Little Skull Mountain Earthquake, in dePolo, C.M., ed., Seismic Hazards in the Las Vegas Region: Nevada Bureau of Mines & Geology Open-File Report 98-6, p. 137-156.
Su, F., Zeng, Y., and Anderson, J.G., 1994a, Simulation of Landers earthquake strong motion using a composite source model (abstract): Seismological Research Letters, v. 65, p. 52.
Su, F., Zeng, Y., and Anderson, J.G., 1994b, Simulation of the Loma Prieta earthquake strong ground motion using a composite source model (abstract): 1994 Fall Meeting, American Geophysical Union, p. 448.
Wells, D.L. and Coppersmith, K.J., 1994, New Empirical Relationships Among Magnitude, Rupture Length, Rupture Width, Rupture Area, And Surface Displacement, Bulletin of the Seismological Society of America, Vol. 84, No. 4, p. 974-1002.
Wong, I.G., Unruh, J.R., Wright, D.H. and Lettis, W.R., 1992, Seismicity Evaluation for Parker Dam, Arizona-California, unpublished report prepared by William Lettis & Associates for U.S. Bureau of Reclamation.
Wyman, R.V., Karakouzian, M., Bax-Valentine, V., Slemmons, D.B., Peterson, L., and Palmer, S., 1993, Geology of Las Vegas, Nevada, United States of America, Bulletin of the Association of Engineering Geologists, v. 30 p. 37-70.
Zeng, Y., and Anderson, J.G., 1996, A composite source model of the 1994 Northridge earthquake using genetic algorithms: Bulletin of the Seismological Society of America, v. 86, p. S71-S83.
Zeng, Y., Anderson, J.G., and Yu, G., 1994, A composite source model for computing realistic synthetic strong ground motions: Geophysical Research Letters, v. 21, no. 8, p. 725-728.

The Industrial Parkway –
So You Want to Build a Road in Mine Spoils

By

Scott L. Murray, PE, Vice President
Mark A. Litkenhus, PE, Associate
Fuller, Mossbarger, Scott and May Engineers, Inc.
Lexington, Kentucky

Abstract

The Kentucky Transportation Cabinet is constructing a $90 million roadway in Boyd, Carter and Greenup Counties. The Industrial Parkway begins with an intersection at I-64 about six miles east of Grayson, extends about 15 miles northward and ends at-grade with US 23 near Wurtland. The corridor has been extensively surfaced-mined to recover coal reserves. Bedrock belongs to the Conemaugh and Breathitt formations, representing the Pennsylvanian geologic age. Rock along the alignment consists primarily of non-durable shales that degrade to soil-like consistencies when exposed to weathering elements.

Past mining operations required mass excavation of bedrock to uncover coal reserves. Non-durable shales and other materials were used to refill mine benches and backfill highwall cuts, or were wasted in large hollow fills. These spoils are present in thicknesses ranging from a few feet to over 70 feet. The materials were not placed in an engineered manner, but rather were bladed, end-dumped and pushed into place. Such operations resulted in thick deposits of soil-like materials that continue to degrade and settle over many years, even under their own weight. Roadway elements constructed on such materials can experience substantial settlements. Buried highwalls, acid drainage producing materials, sediment ponds and existing landslides were also encountered at bridge and roadway locations.

This paper discusses how the site was characterized by performing preliminary geotechnical overviews as well as more comprehensive geotechnical explorations. It also focuses on applications of dynamic compaction, drilled-shaft foundations, geo-grids, acid drainage remediation, chemical modification of subgrades, problems encountered during construction, and the successes achieved.

Project Location and Background Information

The Industrial Parkway is a roadway about 15 miles long in Northern Kentucky that links Interstate 64 in Carter County with US 23 in Greenup County, near the community of Wurtland. The Parkway connects these major transportation arteries with the expectations of promoting the development of local industry—for which it is named. Site grading for the roadway, which initially supports two lanes of traffic, has been completed for an ultimate four-lane configuration. The design and construction of the roadway were split into four sections. Sections 1 through 3 are currently open to traffic and Section 4, located at the US 23 end of the project, is scheduled for completion this year (2003).

In 1995, the Kentucky Transportation Cabinet (KYTC) selected the team of Presnell Associates, Inc. (now Qk4), Palmer Engineering Company, American Consulting Engineers, Inc., and Fuller, Mossbarger, Scott and May Engineers, Inc. (FMSM) to design and oversee construction of the Industrial Parkway. FMSM is the geotechnical engineering consultant responsible for field exploration efforts; laboratory testing of recovered samples; engineering analyses to evaluate potential roadway settlements, embankment and cut stability, subgrade conditions for pavements, and structure foundations; and geotechnical engineering support during construction activities. This paper focuses on the geotechnical conditions encountered at the site, and the design and construction features implemented to accommodate the unique geotechnical aspects of the roadway corridor. Figure 1 is a regional map showing the location of the Industrial Parkway.

Figure 1. Regional Location Map

Physiographic and Geologic Setting

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, formed from sediments deposited during the Middle Pennsylvanian geologic period. Coal seams present in the area are all part of the Breathitt formation and include the Princess Nos. 3, 4, 5, 6, 7 and 8 seams. 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.

Coal Mining Operations

Ridges and side slopes above the lower valley areas were extensively surface mined to recover coal reserves at various locations along the corridor. Recovery methods included mountain top removal, cross-ridge cut, contour cut and auger mining. Some underground mining operations also occurred within the area, and some pre-law mining areas exist at a few locations along the roadway. The conditions left behind as results of such past activities presented some of the most significant challenges in designing and building the Industrial Parkway. Figures 2 through 4 show some of the conditions present at the site at the beginning of the geotechnical field exploration efforts.

Figure 2. Back-Stacked Highwall Area

Spoil materials generated from mining operations were placed in hollow fills, used to back-fill or cover rock highwalls created during contour cut mining, and spread in thick deposits on mine benches. The back-filled areas resulting from mountain top removal and cross-ridge cut operations were often graded to gently rolling terrain. Back-fill materials used to cover highwalls were typically placed on slopes between 15 and 30 degrees. Many valleys and hollows below mined coal seams were used to waste large quantities of mine spoils, resulting in the presence of large hollow fills and side-hill fill areas. Wet and marshy zones formed on flat-lying crests of hollow fills and on flatter portions of mined benches that had no positive drainage features. Numerous silt ponds were constructed to collect surface run-off and sediment from previously disturbed and reclaimed mine areas.

Figure 3. Mine Bench and Hollow Fill Areas at Station 13+160

Figure 4. Silt Pond and Mine Bench Area Encountered at Project Site

Potential Acid Drainage-Producing Bedrock

Acid drainage in the Pennsylvanian rock of Eastern Kentucky is produced by the oxidation and hydration of iron sulfide minerals (pyrite and marcasite) that are present in some of the strata. This reaction produces hydrogen ions (acidity), sulfate ions and soluble ferrous iron. The subsequent oxidation of the ferrous iron to insoluble ferric iron produces additional acidity. In undisturbed rock strata, this reaction proceeds at a very slow rate because of the low amount of oxygen present. When the pyrite/marcasite-bearing strata are exposed to air and moisture, the reaction proceeds at a relatively rapid rate. Because of the presence of significant quantities of carbonate minerals (principally calcite and siderite) in many of the rock units, the acidity is generally quickly neutralized and the iron is precipitated out as ferric hydroxide. Despite the relatively rapid neutralization, acid drainage is undesirable—ferric hydroxide in the stream channels can be injurious to some aquatic life, and elevated sulfate levels can adversely affect drinking water quality. Figure 5 provides an example of iron leaching from acid-producing materials exposed during previous mining operations at the site.


Figure 5. Acid Runoff

Pennsylvanian-age rocks of Eastern Kentucky were typically deposited in a fresh to brackish water, fluvial-deltaic environment. As a result, most of the iron sulfide mineralization in these bedrock units was produced by the reduction of sulfate in sea water during periodic rises in sea level and consequent back-flooding of the delta. Because organic material is a reducing agent, coals and carbonaceous shales often have significant pyrite/marcasite concentrations. The iron sulfide minerals are also present in some inorganic strata as a result of other reducing reactions. The Industrial Parkway corridor lies within the Princess Coal Zone. Because intrusions of seawater were less frequent in the upper part of the delta, there are fewer acid-producing strata in this zone than in strata deposited in the Lower Delta Plain.

However, as part of the geotechnical exploration for this project, FMSM reviewed the potential for acid-producing bedrock strata to be encountered at the site, and evaluated possible effects such materials might have on design and construction. Applicants for coal mining permits are required to perform potential acidity (PA) and neutralization potential (NP) testing on recovered samples of rock strata that will be disturbed (overburden) during coal recovery efforts. The results of these tests are used to develop acid-base accounts to determine the acid runoff potential the overburden materials will exhibit when placed in spoil fills. FMSM reviewed permit applications and records of four former surface mines that had been located within or near Section 1 of the Industrial Parkway alignment.

The results of this research indicated that potentially acid-producing rock strata (coal seams, carbonaceous shale, and siltstones and sandstones containing coal stringers and partings) would be encountered during roadway construction. The reviewed information further suggested that acid-producing strata at the site would be relatively thin and discontinuous as compared to the overall volume of bedrock excavated during roadway construction. Acid-base accounting presented in the mine permit applications, showed that the non-acid producing rock strata would be capable of neutralizing the amounts of acid producing materials identified when both were placed together in a well-mixed fill (roadway embankment).

The greatest concern developed from the research was how to address acid-producing materials that would be exposed in open rock cut faces. Design and construction features to address potential acid-producing materials included the following:

1. Line cut ditches with limestone to help reduce the erosion potential and to help neutralize any acidic leachate from exposed cut faces.

2. Install limestone-lined retention basins at the ends of ditches to help collect and neutralize acid run-off.

Figure 6. Limestone Lined Ditches

3. Identify and separate acid-producing materials from other embankment materials and treat with agricultural lime, encapsulated a minimum of five feet on non-acid producing embankment material.

Figure 6 shows some of the limestone-lined ditches utilized for this project.

Silt Ponds and Hollow Fills

Previous mining operations utilized sediment ponds to help control surface runoff and collect silt from the mine sites. Prior to constructing any roadway elements such as embankments, cuts or structures, these ponds commonly had to be breached, and the significant quantities of soft, wet soils they contained were removed and/or stabilized. Large hollow fills used to waste excess overburden and spoil materials generated during the mining efforts were also present at the site. Because of the random nature of the material present in the fills, and the lack of compaction effort used to place such materials, any roadway elements constructed on such hollow fills are subject to severe settlements and slope stability problems. Figures 7 and 8 show a typical silt pond and hollow fill, respectively. The hollow fills at the site often had spoil materials present in thicknesses greater than 100 feet.

Figure 7. Silt Pond

Figure 8. Hollow Fill Encountered at Site

Generally, the ponds were breached and the wet, soft materials were removed prior to beginning roadway construction. The approach in dealing with hollow fills was to avoid them if possible. If the alignment could not be adjusted away from a hollow fill, then it was recommended that the alignment coincide with the primary axis of the hollow fill, a necessary precaution to avoid positioning the new roadway section at an awkward and potentially unstable angle to the axis of the fill.

Deep Mine Spoil Materials and Buried Highwalls

Surface mining activities within the roadway corridor generated tremendous quantities of mine spoil materials. These materials were not only wasted within hollow fills as previously described, but were also placed on large, flat mine benches following extraction of coal reserves, resulting in hundreds of acres of relatively flat topography. The depth of the spoil materials in such areas ranged from about 30 feet to over 80 feet. In many instances, buried rock highwalls were also encountered. Roadway embankments built on these spoils are subject to significant total settlements and severe differential settlements. Roadway cuts constructed in these areas are also subject to stability problems because of the heterogeneous nature of the materials and the presence of seeps, perched water, and soft, wet, organic zones. Figure 9 shows one of these large, flat spoil areas within the Industrial Parkway corridor.

Figure 9. Flat Spoil Area at Project Site

Other Geotechnical/Geological Considerations

In addition to all the conditions associated with past mining operations, the project site had its share of “normal” geotechnical challenges with which to contend. The shale within the Conemaugh formation is extremely non-durable and quickly degrades to soil-like conditions when exposed to water. Using these materials for embankment construction requires special treatment to develop adequate placement and compaction. Because most of the rock encountered in the cuts was of this nature, non-durable shale was likely to be the subgrade material used for pavement support.

Deep alluvial soils were also present at some valley locations where large roadway embankments were planned. Maintaining embankment stability and controlling settlements were primary issues in these areas. To help address these concerns, flatter embankment slopes, rating embankment construction to help maintain stability, monitoring settlements using settlement platforms, and utilizing stability berms at the toes of some embankments were some of the design methods employed.

The new roadway includes several bridges and culverts. Because of the unusual conditions encountered, use of deep foundation systems such as driven piles and drilled shafts were necessary to provide adequate support for structural loads.

Drilling and Laboratory Testing

Field drilling and sampling efforts consisted of commonly used procedures and techniques for conducting geotechnical explorations for transportation facilities. Rock core borings and rock soundings were performed within critical cut intervals, while sample borings, rock core borings and soundings were drilled in proposed embankment areas and at structure locations. Disturbed bag samples of predominant soil types and mine spoil materials were also collected along the alignment. The frequency of drilling and sampling was, of course, increased when unusual features such as buried highwalls or deep mine spoils were encountered.

Rock Testing

Selected rock samples were subjected to slake durability index (SDI) and jar slake testing, and unconfined compressive strength testing. The SDI and jar slake tests provide indications of the effects weathering elements will have on the bedrock when exposed in open cuts or used as fill materials for embankments. Both of these tests are performed on rock comprised primarily of shale. The Kentucky Transportation Cabinet separates shale into four categories for design purposes depending upon SDI and jar slake test values, as follows:

Table 1. KYTC Shale Classifications

Classification	SDI (%)	Typical Jar Slake Category
Durable	95 to 100	6
Non-Durable, Class I	80 to 94	4 or 5
Non-Durable, Class II	50 to 79	3 or 4
Non-Durable, Cass III	0 to 49	1 or 2

Ninety-seven percent of the tested samples for this project yielded SDI values less than 95 percent. Almost forty percent of the shale samples classified as Non-Durable, Class III.

Soil Testing

Soil classification, standard Proctor, California bearing ratio (CBR), unconfined compressive strength, one-dimensional consolidation, consolidated undrained triaxial shear strength and chemical stabilization tests were performed on selected soil samples recovered as part of this project.

In general, soils classified as SM, SC or CL according to the Unified Soils Classification System (USCS), and as A-4 or A-6 according to the AASHTO system of soil classification. The predominant material type encountered across the project was mine spoil. The spoil typically yielded classifications of GC and CL, depending upon the percentages of rock fragments present within the samples, and consisted primarily of non-durable shale that had slaked and weathered to a soil-like consistency.

CBR testing yielded very low values ranging from a low of 1.2 to a high of 2.7, indicating the soils would provide relatively poor ability to support pavement structures. KYTC suggests either chemically or mechanically modifying subgrade soils that exhibit CBR values less than six. To evaluate what type of chemical modification would be most appropriate for the mine spoil materials encountered at the site, FMSM performed chemical stabilization tests on several remolded samples of spoil materials. Both lime-modified and cement-modified samples were evaluated, and both showed significant improvements in unconfined compressive strengths as compared to untreated samples. However, the cement-treated samples yielded substantially higher compressive strengths than did lime-modified specimens.

As a result of such testing, cement-modified subgrades were recommended. In lieu of using chemical modification, the Contractor was given an option to use durable rock from offsite to construct a two-foot, rock roadbed for the pavement subgrade.

Table 2 shows the results of CU triaxial tests. These values were used to model embankment and cut stability sections during subsequent engineering analyses.

Table 2. Results of CU Triaxial Tests

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

Cut and Embankment Slope Stability

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 3 summarizes slope geometries for cuts in mine spoil materials.

Table 3. Slope Geometries for Soil/Mine Spoil Cuts

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

In general the same recommendations were used for embankments as presented in Table 3 for cut slope geometries. However, specific foundation conditions and embankment geometries were evaluated to develop final slope recommendations for fill sections.

Potential Embankment Settlements

Accurate predictions of settlements that roadway elements may experience when constructed on mine spoil materials are not possible. Conventional methods of estimating consolidation settlement for mine spoil materials are not considered to be reliable due to the heterogeneous nature of the fill and the relatively uncontrolled manner under which it was placed. Because the majority of mine spoils encountered at this site consisted of non-durable shales, magnitudes of settlements were estimated using procedures developed by Tommy C. Hopkins and Tony L. Beckham of the Kentucky Transportation Center, and presented in Embankment Construction Using Shale. These procedures are based on significant research of shale embankment construction in Kentucky and basically suggest using a percent of the shale thickness or embankment height to estimate settlement. Estimated embankment settlements for this project ranged from a few inches for embankments less than twenty feet tall, to almost three feet for taller embankments constructed on deep spoil materials.

Because of the magnitudes of potential total settlements, and because site conditions indicated that in some situations severe differential settlements could occur, special provisions were incorporated in the design and construction of selected embankments to help reduce potential settlements. These measures included the use of geogrids within the embankments, dynamic compaction of foundation materials, or a combination of both. Figure 10 shows dynamic compaction operations at one of the bridge approach embankment locations. Dynamic compaction was typically used in areas of deep mine spoil materials to help create more uniform conditions within the upper 10 to 20 feet of foundation materials, and to provide a better transition zone from thinner to thicker spoil deposits.

Figure 10. Dynamic Compaction of Mine Spoil

Summary

Natural geologic conditions and the results of extensive surface mining along the Industrial Parkway corridor created unique geotechnical challenges to designing and building a quality transportation facility in Greenup, Carter and Boyd Counties, Kentucky. Deep deposits of mine spoil materials, non-durable shale bedrock, buried highwalls, wet, soft surface areas, poor subgrade materials, acid-producing rock strata, and deep alluvial soils were all encountered and successfully addressed to accomplish the Kentucky Transportation Cabinet's goal of building a connecting roadway between Interstate 64 and US 23. The completion of the project has initiated anticipated industrial development in the region and should provide such a catalyst for years to come. The task was accomplished through the cooperative efforts of ordinary citizens, local and regional interest groups, industry, the Kentucky Transportation Cabinet, other governmental agencies and the Industrial Parkway Design Team. The project's successful completion is a tribute to all those involved.

Industrial Parkway Exit Ramp (East) over I-64

For more information about the Industrial Parkway project you may contact:

Scott Murray (smurray@fmsmengineers.com) or Mark Litkenhus (mlitkenhus@fmsmengineers.com)
Fuller, Mossbarger, Scott and May Engineers, Inc.
1409 North Forbes Road
Lexington, Kentucky 40511-2050
859-422-3000
visit us on the web at fmsmengineers.com

LEVERAGING EXISTING INFRASTRUCTURE: USING WHAT IS ALREADY IN THE GROUND

Ed Kase, Tim Ross, and Josef Descour
NSA Geotechnical Services, Inc.
15000 West Sixth Avenue, Suite 100
Golden, CO 80401

Don Green
AWK Consulting Engineers, Inc.
1611 Monroeville Avenue
Turtle Creek, PA 15145

ABSTRACT

Expanding capacity for highways seems to be a never-ending task. Much of the highway infrastructure in the United States was built in the 1960's and 1970's. Even when designs and blueprints are available, whether structures were built to specifications is not always known. As bridges are replaced or upgraded, it becomes critical to understand whether the existing foundation can be used in the new design. By using multiple methods, NSA Geotechnical Services is able to determine if existing bridge piers are founded on competent rock, allowing them to be reused in the new design.

Encountering unforeseen ground conditions or existing structures are major contributors to cost overruns in large construction projects. Having accurate data can greatly reduce the cost of highway projects. Established technologies like electrical resistivity surveys and ground penetrating radar surveys, augmented by newer technologies like seismic imaging, can provide detailed characterizations of ground conditions and existing structures. In this case, using RockVision3D resulted in a more thorough understanding of problems to be accounted for in engineering designs.

For this case study, the images of the bridge foundation produced by RockVision3D, coupled with data from the GPR survey and electrical resistivity survey, show definitively that the existing structures are founded on competent rock. Rather than having to specify totally new foundations for the bridge, accurate information about the existing structures allow them to be incorporated into the new design. This greatly reduces the cost of the completed project. Having better information enables better decision-making.

INTRODUCTION

Expanding capacity for roads and highways seems to be a never-ending task. Much of the highway infrastructure in the United States was built in the 1960's and 1970's. Although designs and blueprints are available, whether structures were actually built to specifications is not always known. Understanding whether bridge piers were constructed as designed allows contractors to use existing infrastructure in expanding highway capacity, saving time and money on large construction projects.

The Pittsburgh area is in the midst of multiple construction projects with a goal to alleviate congestion on current roadways by expanding the carrying capacity of these existing roads. One possible “pinch point” in most road projects is the capacity of bridges. As old bridges are replaced or upgraded, it becomes critical to understand whether the existing foundation can be used in the new design. By using seismic tomography, corroborated by electrical resistivity and ground-penetrating radar (GPR), NSA Geotechnical Services is able to determine if existing bridge piers are, in fact, founded on competent rock, allowing them to be reused in the new highway design.

BACKGROUND: SEISMIC IMAGING

Seismic tomography is based on the principle that acoustic waves have different propagation velocities through different types of ground. That is, seismic waves travel faster in strong, competent material and slower in weaker materials (e.g., voids, broken or weathered rock, soil) (Nur, 1987; Shea-Albin, et al., 1991; Yu, 1991). Velocity tomographic images represent the ground velocity as measured between seismic sources and receivers. The accuracy and resolution of a tomographic image is a function of the source and receiver geometry.

To determine the seismic velocities of a survey area, the time required for seismic energy to travel from known source and receiver locations is measured. The velocity is then computed by dividing the distance traveled from source to receiver by this travel time. In ground with a homogenous velocity distribution, this distance is simply a straight-line distance, or straight ray path, from the source to the receiver. However, in ground with velocity variations, this distance may significantly increase due to curvature of the ray path through higher velocity ground between the source and receiver. With appropriate source and receiver geometry, it is possible to iteratively construct an accurate velocity model of the ground surveyed. Distortions in the velocity model may appear in varying degrees as a consequence of the ground characteristics and the source and receiver geometry. Figure 1 demonstrates this distortion for a marginal, two-dimensional, source and receiver geometry.

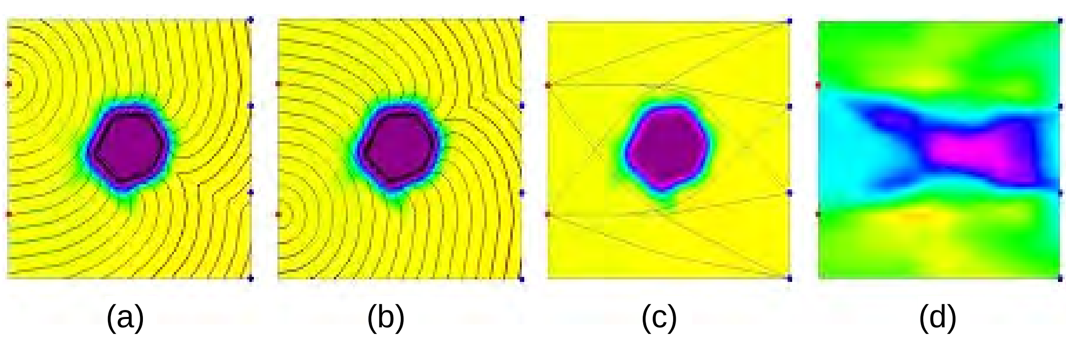


Figure 1. Example of tomographic reconstruction.

There are numerous factors that may cause variations in velocity. Different ground types usually have different material/seismic properties, but variations within the same ground type are also commonly encountered. Variations in stress, fracture extent, water saturation, soil compaction, etc., all may have a significant effect on velocity. In areas where geological features such as fracture zones, faults, subsidence zones, or cavities exist, the seismic waves may travel at a lower velocity, or may travel across an increased distance to pass around the anomaly and suffer increased attenuation. The same type of behavior may be noted in rocks of varying lithology as harder, more competent materials propagate seismic waves at higher velocity and lower attenuation than softer, less competent or less consolidated rocks.

Seismic tomography, in its current state, is best employed as a tool for detailed investigations. Other methods such as ground-penetrating radar (GPR) and electrical resistivity (ER) are methods for doing reconnaissance.

ETNA INTERCHANGE

AWK Consulting Engineers, Inc. approached NSA Geotechnical Services (NSA) to conduct a geophysical investigation to ascertain whether the existing bridge foundations at the Etna interchange were founded on competent rock. The specific objective of the survey was to determine the depth of the piles and render an opinion about whether or not the existing piles are point-bearing on rock or friction piles. NSA proposed using its seismic velocity cross-borehole tomography system, RockVision3D™, as the primary investigative technique. To corroborate the tomography survey results, NSA conducted a resistivity survey for one of the piles at each survey location. NSA also conducted a cross-borehole, GPR survey as an additional corroborating investigation method.

Procedure

The location to be surveyed was Wall U-Ramp U of the US Route 28 Etna Interchange. Three cased boreholes were provided for the pier locations. These three holes were configured so that two of the panels between boreholes cross through the pile cluster (Figure 2). The third panel crossed through ground that does not include piles to provide control for the survey. Calculating the location of instruments within the boreholes was critical to ensure the accuracy of the source and receiver locations during the survey; therefore, a verticality survey was conducted for each borehole. Both the seismic and GPR survey used the same boreholes.

Seismic Cross-hole Tomography Survey

NSA's seismic velocity cross-borehole tomography, RockVision3D™, was used to collect field data and generate two-dimensional and three-dimensional tomographic images of the ground defined by the boreholes around each pier. RockVision3D™ is a seismic tomography system that provides two-dimensional and three-dimensional velocity images of the ground between boreholes and/or boreholes and the surface. RockVision3D™ provides information on ground conditions or anomalies with differential seismic velocities or attenuation characteristics.

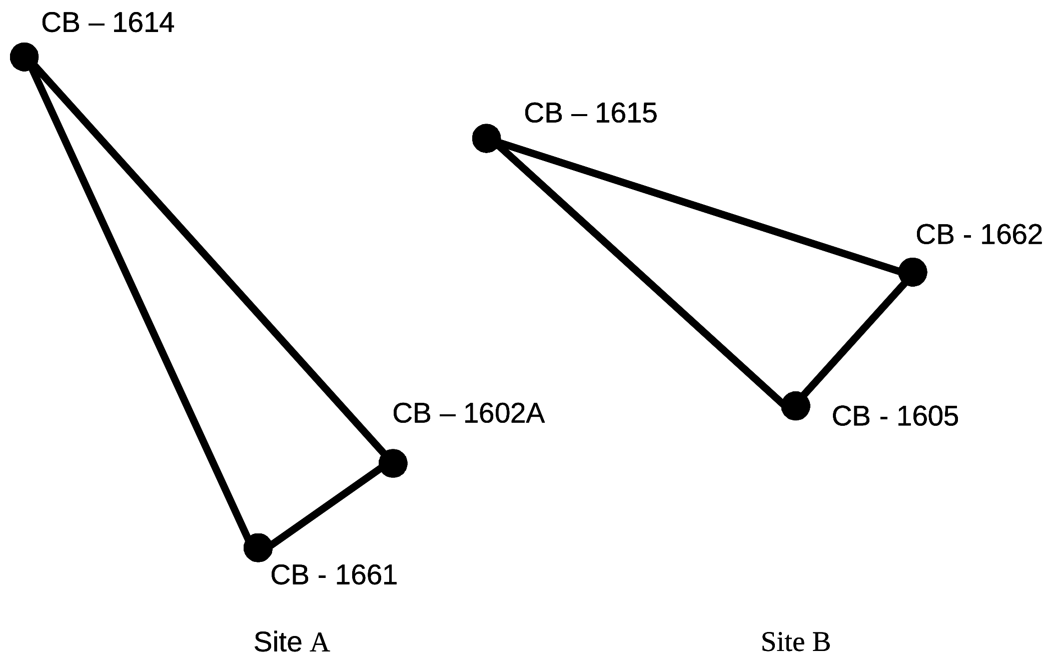


Figure 2 Borehole layout for sites A and B.

NSA conducted this cross-hole velocity tomography survey by pairing a seismic source and a string of receivers in adjacent boreholes to propagate and capture seismic signals transmitted between source and receiver boreholes. Each source-receiver array did result in data that formed a vertical panel containing information on the velocity structure between the boreholes. The collected seismic data were analyzed, filtered, and processed creating a series of two-dimensional tomograms and three-dimensional contour velocity images and section views of the ground between borehole pairs. For this survey, a hydrophone string with 18 hydrophones served as down-borehole receivers; the hydrophones were spaced at 3.28-ft (1-m) intervals, resulting in each string having receivers along 56 ft (17 m) of its length. An Etrema magnetostrictive, swept-frequency source was used to generate seismic energy in boreholes adjacent to the hydrophones. Each seismic signal initiation triggered the recording of seismic waves by the seismograph. Where steel or Raymond/monotube concrete piles exist within the surveyed area, they would be indicated in the tomograms as relatively higher seismic velocity zones than the surrounding ground.

Resistivity Survey

To conduct the resistivity survey, an electrical node was connected to a conductor previously connected to one of the piles at each of the survey sites. A second electrical node was attached to a grounding electrode in line with and beyond one of the boreholes used for the seismic and GPR surveys; this borehole was perforated to measure conductivity using the groundwater. A borehole probe, with terminals spaced 1 ft (0.305 m) apart, measures this potential difference. Because the probe can only measure resistivity in water, the resistivity data start in each drill hole at the top of the water table. The potential difference measurements were taken at 3.28-ft

(1-m) intervals from the top of the water table to the bottom of each borehole. The potential difference in the electrical field would be minimal from the water table to the bottom of the steel piles, and variations in resistivity are a consequence of differential ground conductivity. At the bottom of the steel piles, the electrical field created by the energized pile and the remote node would decrease, and the potential difference measured by the probes in the borehole would increase significantly. Immediately below the depth of the steel portions of the piles, the electrical field would approach zero, and the potential difference measured by the borehole probe would return to zero.

Ground Penetrating Radar Survey

NSA also conducted a cross-hole GPR survey to corroborate the seismic and resistivity surveys. The same procedures were followed as in the seismic survey except a radar transmitter was used as a down-hole source which transmitted a signal to a radar receiver positioned in adjacent boreholes. It should be noted that one major difference between the seismic and GPR data collection methodologies is that the GPR transmitter and receiver were always positioned at the same elevation in the source and receiver holes, whereas for the seismic survey, many different elevations were used. Where there are steel or Raymond/monotube piles in the ground between any of the boreholes, the velocity and the magnitude of the radar signals would be affected. The average velocities of the signals passing through piles would be lower as the radar signal velocity of the piles would be less than the surrounding ground. Attenuation of the radar signal would be greater for signals passing through piles as a significant portion of the signal would be reflected and dispersed by the piles.

Data Analysis and Interpretation

Seismic Cross-Hole Tomography

The data quality for both sites A and B was acceptable. The images indicated the measured seismic velocity of a volume of ground defined by the boreholes. For site A (Figure 3), the individual piles appeared to be detectable as relatively higher seismic velocity anomalies within the fill and soil material above the bedrock to a depth of approximately 690 ft (210 m).

For site B (Figure 4), only the pile cluster was indicated above the bedrock as a zone of relatively higher velocity ground, extending to approximately 680 ft (207 m). For both sites A and B, there was also a very clear indication of low-velocity anomaly pockets in the top of the bedrock where the piles were driven into the bedrock.

The seismic tomography surveys for both site A and site B indicated that the piles were point-bearing on rock, and, in fact, the piles were driven into the bedrock surface when installed.

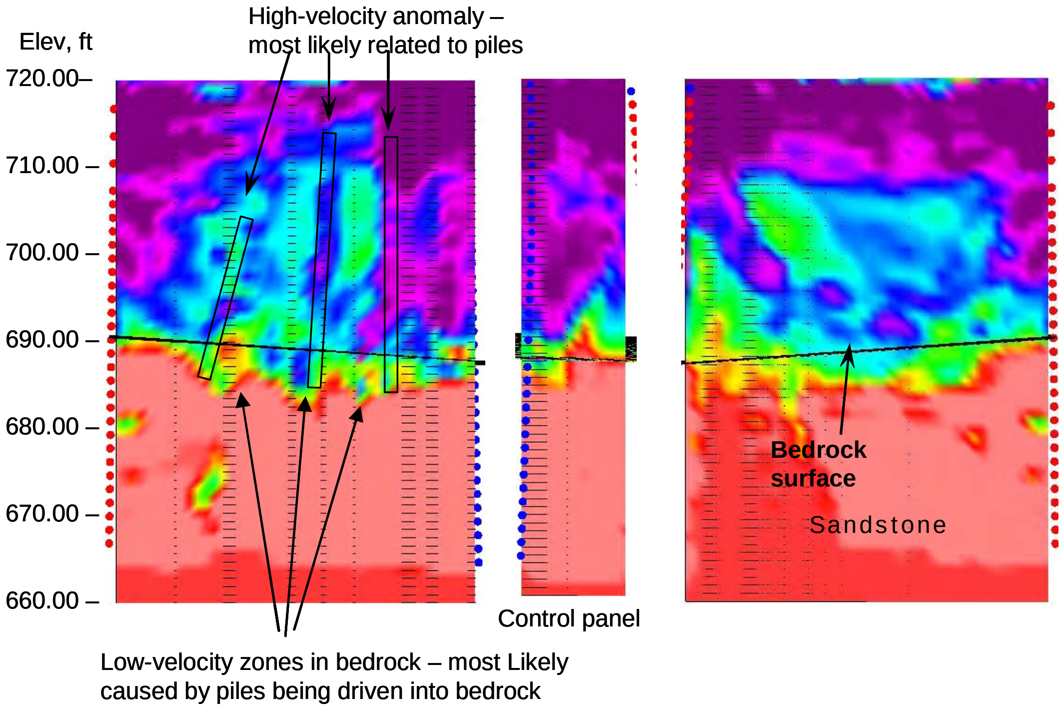


Figure 3 Ramp U, Site A. Side view of vertical tomograms between survey boreholes.

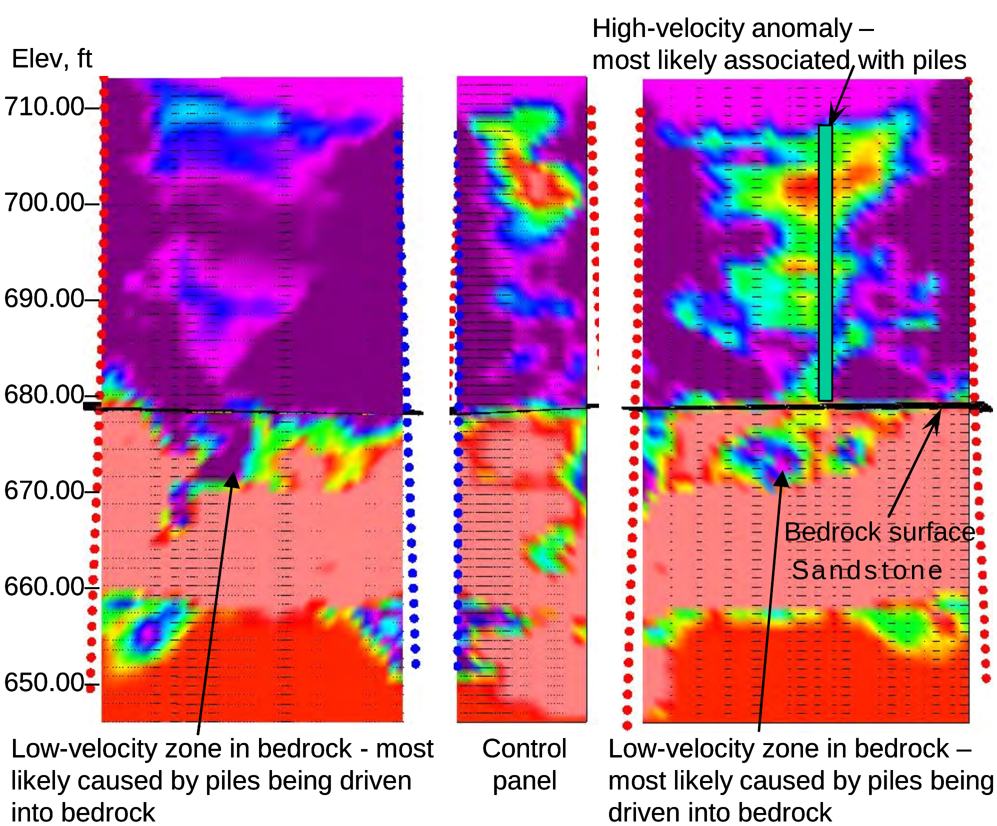


Figure 4 Ramp U, Site B – Side view of vertical tomograms between boreholes.

Resistivity

The data quality for both sites was acceptable. These figures showed the depth in the borehole from the surface versus potential difference in the electrical field created by the energized piles and the remote node. The graph for site A (Figure 5) clearly indicated that there was a continuous electrical conductor (the pile) from the node connected to the top of the piles to an elevation of approximately 689 ft (210 m). As a consequence of the probe borehole being several feet away from the pile and the triangular shape of the electric field created by the energized pile and remote surface probe, it was concluded that the steel portion of the pile is slightly deeper than this elevation. The drill-hole information indicated the top of sandstone bedrock at 688.9 ft (210 m) elevation

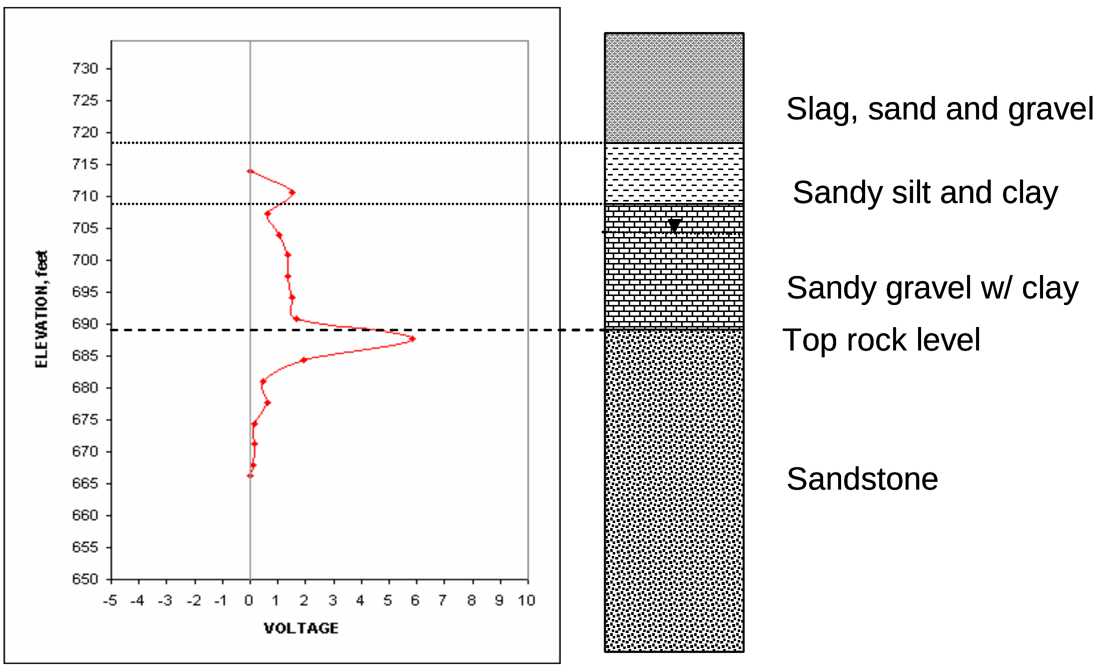


***Figure 5. Ramp U, Site A – Resistivity survey profile and associated geology*.**

The graph for site B (Figure 6) clearly indicated that there was a continuous electrical conductor (the pile) from the node connected to the top of the piles to an elevation of approximately 679 ft (207 m). The drill-hole information indicated the top of sandstone bedrock at 678.3 ft (206.9 m) elevation. Again, because of the geometry of the electrical field and the probe, the metal portion of the pile most likely extended slightly deeper than 679 ft elevation (207 m).

The resistivity surveys for sites A and B indicated that the piles were point-bearing on rock.

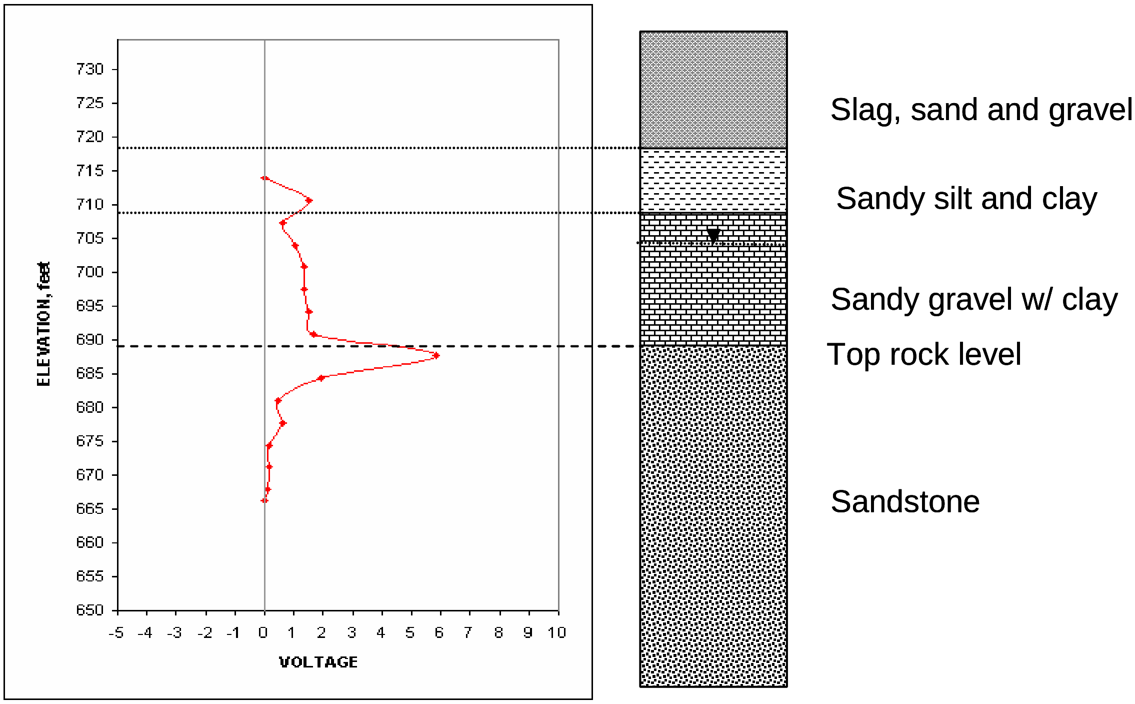


Figure 5. Ramp U, Site A – Resistivity survey profile and associated geology.

GPR Survey

The data for site B were acceptable and provided clear indications of the pile cluster tip elevations. The data for site A were poor, most likely due to high radar conductivity of the ground around this site. As a consequence, the GPR survey for site A was inconclusive.

For site B, the first image (Figure 7) showed the control panel between drill holes CB-1605 and CB-1662; the second image showed the panel through the pile cluster between drill holes CB-1615 and CB-1662; the third image showed the panel through the pile cluster between drill holes CB-1615 and CB-1605. The control panel showed the GPR signal passing through the ground above the top of the sandstone bedrock. At an elevation of approximately 678 ft (206.8 m), which corresponds to the top of bedrock, the signal became stronger and had a higher velocity.

The second and third panels, through the pile clusters, indicated a total attenuation of the GPR signal above 678 ft (206.8 m) elevation. This was expected and was most likely a consequence of the GPR signals being reflected and attenuated by the steel piles. Once the GPR transmitter and receiver were below the elevation of the piles, the signal was detectable.

The GPR survey for site B indicated that the piles are point-bearing on rock.

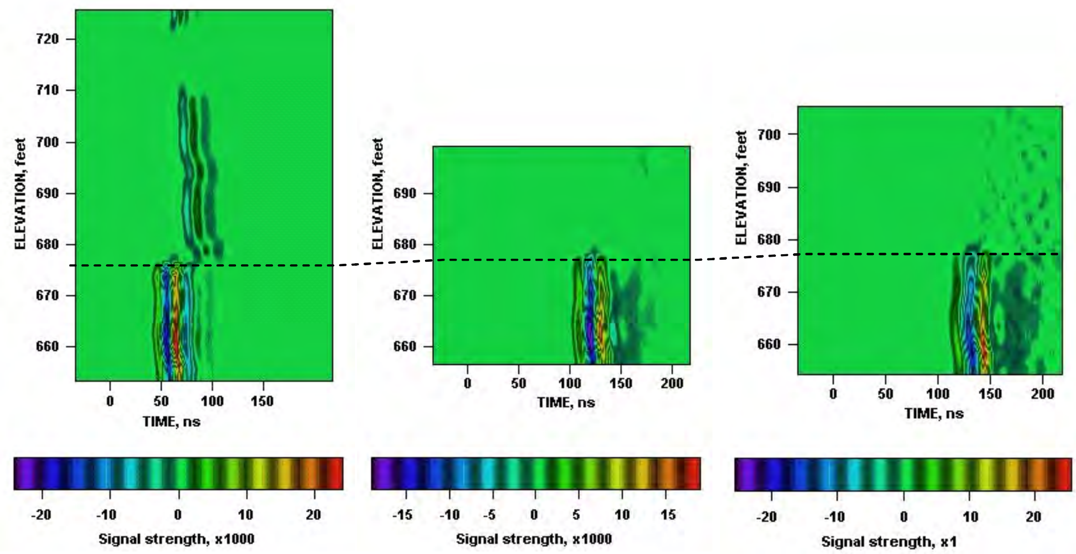


Figure 7. Ramp U, Site B – Radar panels.

Summary Results

The seismic and resistivity surveys conducted for site A ***strongly indicate that the piles were point-bearing on bedrock,*** and there were no indications in either of these surveys that the pile tips do not extend to bedrock. The GPR survey for this site was inconclusive and provided no information on the pile tip locations. The seismic, resistivity, and GPR surveys conducted for site B ***strongly indicate that the piles were point-bearing on bedrock,*** and there were no indications in any of the surveys that the pile tips do not extend to bedrock.

ECONOMICS

The cost for the geophysical investigation conducted by NSA was on the order of $25,000. In the absence of a definitive analysis regarding the condition of these bridge foundations, the only option would have been to assume none of the existing infrastructure could be used and an entirely new bridge foundation system designed and installed. That would raise the costs on this project by $2.5M or 100 times the cost of geophysical investigation.

Had the piles been friction piles, piles not driven to bedrock, they still could have been incorporated into the new design for this interchange. However, the new design would have been engineered differently for friction piles. In general, there is great benefit in knowing the ground conditions around an existing bridge foundation in the redesign of a highway interchange.

NEW METHODOLOGIES

In August of 2001, NSA was awarded an Advanced Technology Program grant from the National Institute of Standards and Technology for a program that combines three-dimensional ground imaging with numerical modeling. This project will integrate seismic tomography and holography with state-of-the-art numerical modeling technologies. Holographic/tomographic seismic imaging will be used to develop “seismically calibrated” models - engineering computational models based on the actual anisotropic, non-homogeneous constituents of the in situ three-dimensional rock and soil mass.

In addition to using the results of seismic investigation to create these “seismically calibrated” models, NSA is making advances in two areas related to seismic imaging. The state-of-the-art for current seismic investigations depends on picking first arrivals of seismic energy and using those data to compute seismic velocities. NSA’s new analysis methods will use the full waveform of seismic energy received, improving the accuracy and detail of seismic models.

NSA is also developing computational modeling technology that will process data from surface-mounted sensors. By relying more on surface-mounted instrumentation, the costs for performing seismic investigations will decrease because the cost of drilling boreholes is eliminated. NSA is also developing self-locating, wireless sensors as part of this effort. Self-locating sensors will eliminate the need for all sensor locations to be surveyed. By using wireless communication technology for these devices, the effort of positioning and removing the wires currently required to transmit data from seismic sensors to a seismograph is eliminated. These innovations will result in reduced costs and effort related to performing the data collection portion of seismic investigations.

CONCLUSIONS

The images of the bridge piers produced by RockVision3D™, coupled with data from the GPR survey and resistivity testing, show definitively that the existing structures are founded on competent rock. Rather than having to specify totally new structures to support the new bridge, accurate information about the current structures allow them to be incorporated into the new design. This greatly reduces the cost and schedule for the completed project. Having better information enables better decision-making. By removing the guesswork, projects can be executed safely and economically in a timely manner.

REFERENCES

Nur, A., 1987. Seismic Rock Properties for Reservoir Descriptions and Monitoring. In *Seismic Tomography with Applications in Global Seismology and Exploration Geophysics*, G. Nolet, ed., D. Reidel Publishing Co., Dordrecht, The Netherlands, pp. 203-237.

Shea-Albin, V. R., D. R. Hanson, and R. E. Gerlick, 1991. Elastic Wave Velocity and Attenuation as Used to Define Phases of Loading and Failure in Coal.” USBM Report of Investigation 9355, U.S. Bureau of Mines, Department of the Interior, 43 pp.

Yu, G., 1991. Elastic Properties of Coals. Ph.D. Thesis, MacQuarie Univ., Sydney, Australia, 133 pp.

Rock Slope Scaling and Aesthetic Stabilization of Two Historically Sensitive Palisades Sill Slopes in Weehawken, New Jersey

Daniel Journeaux[1], Patrick Harrison[2], Pierre Rousseau[1], Mark McNeilly[3], Jay Smerekanicz[4] and Peter Ingraham[4]

ABSTRACT

A section of the Hudson-Bergen Light Rail project in northern New Jersey follows the base of a 180-foot high bluff formed by the Palisades sill through a densely populated area along the Hudson River. The project involves converting an old railroad line to a mass transit light rail line. Minor rockfalls are generally acceptable to railroad companies but not mass transit systems. Rock stabilization was required at two locations, Kings Bluff and the Weehawken Tunnel East Portal. Several options for stabilizing these slopes were studied to satisfy the conflicting requirements of stakeholders, which included public concerns for preservation of the historic Palisades bluffs, so stabilization measures were limited to those that do not create visual impacts. Hence, draped wire rope nets and mesh were not allowed.

The Palisades sill comprises diabase columnals up to 30 feet in diameter, some of which have been eroded to an angle shallower than the columnar jointing. Stabilization measures include scaling, installation of tensioned and untensioned rock bolts, dental shotcrete and shotcrete buttresses. To reduce visual impacts, scaling was limited to rocks that could be readily removed with scaling bars, rock bolt ends were camouflaged with colored shotcrete, and structural shotcrete and shotcrete buttresses were sculpted and colorized to blend into the natural slope. To address site access concerns and reduce impacts on rail construction, innovative Tyrolean hoist systems were erected at each bluff to stage materials for rock stabilization crews working on the slope face. The stabilization methods used have allowed construction of the light rail project to continue unimpeded, and have addressed stakeholder concerns for preservation of the historic Palisades bluffs.

Rock stabilization at both sites was performed under a design-build contract, requiring close cooperation between all parties. The design-build contract enabled the work schedule to be considerably compressed as the design and construction proceeded concurrently.

INTRODUCTION

The New Jersey Palisades are a well known landmark which consists of a 600-foot high diabase sill that forms an escarpment along the Hudson River. New Jersey Transit/21st Century Rail is constructing the Hudson-Bergen Light Rail project at the base of the escarpment along an old Conrail railway alignment, and Washington Group International is providing design, build, operation and maintenance services. The rock slopes adjacent to the new railway alignment in Weehawken, New Jersey were scaled and/or cut at the beginning of the last century, at which time rock buttresses were constructed near the Weehawken Tunnel East Portal and the toe of the Palisades escarpment was cut. The remaining slopes are typically 80

[1] Janod Contractors, 555 VT, Suite 122, Route 78, Swanton, VT 05488 (Daniel@janod.biz, Pierre@janod.biz)
[2] Hudson-Bergen Light Rail Transit System Project, 150 Warren Street, Suite 201, Jersey City, NJ 07302 (Patrick.harrison@wgint.com)
[3] Golder Associates Inc., The Federal Trust Building, 24 Commerce St., Suite 430, 4th Floor, Newark, NJ 07102 (mmcneilly@golder.com)
[4] Golder Associates Inc., 540 N. Commercial St., Suite 250, Manchester, NH 03101 (jsmerekanicz@golder.com, pingraham@golder.com)

degrees to vertical. The two slopes that are the focus of this paper are Kings Bluff, which is roughly 190 feet high (Figure 1), and the slopes above and adjacent to the Weehawken Tunnel East Portal, which are roughly 170 feet high (Figure 2).

Figure 1. King's Bluff rock slope.

Figure 2. Weehawken Tunnel East Portal rock slope.

In 2002, a rockfall occurred at Kings Bluff during construction of a concrete railway bridge abutment, causing cosmetic damage to the new abutment. In response to concerns for worker safety, work on the abutment and railway bridge structures was halted until slope stabilization measures could be implemented. Conventional slope stabilization and rock fall mitigation methods consisting of draped wire rope nets and rock bolting were designed for both sites as early as 1998 and were scheduled for construction following completion of the railway bridge. The rockfall event and concern for worker safety increased the priority of rock slope stabilization to allow the contractor to resume bridge construction. Upon discussing the planned rock slope stabilization with local stakeholders, New Jersey Transit recognized the unique historic and geologic character of the Palisades bluffs. Working with the local authorities and Palisades preservation groups, New Jersey Transit elected to complete the slope stabilization in a manner that preserved the natural character of the slopes while providing rockfall protection for the light rail alignment.

To expedite rock slope stabilization, New Jersey Transit elected to complete the work as a design-build effort. Because of their past experience at national and state parks with similar aesthetic concerns, Washington Group International retained Janod Contractors and Golder Associates to complete the work. Based on discussions with the Mayor and other stakeholders, the selected stabilization methods included rock bolts, rock dowels, rock buttresses and dental shotcrete. All of the stabilization methods were to be aesthetically treated to blend into the natural slopes. Scaling was limited to blocks that could be removed using a standard scaling bar. As no trim blasting was allowed on the project, larger blocks were to be stabilized in place.

GEOLOGIC SETTING

The Palisades were formed by a Lower Jurassic age diabase sill intruded between Upper Triassic sediments of the northeast corner of the Newark Basin (Figure 3). As noted above, the Palisades Sill forms a prominent ridge on the west bank of the Hudson River, attaining a height up to 600 feet, and a thickness up to 1,700 feet. Four lithologies are present at both sites (in upward stratigraphic order):

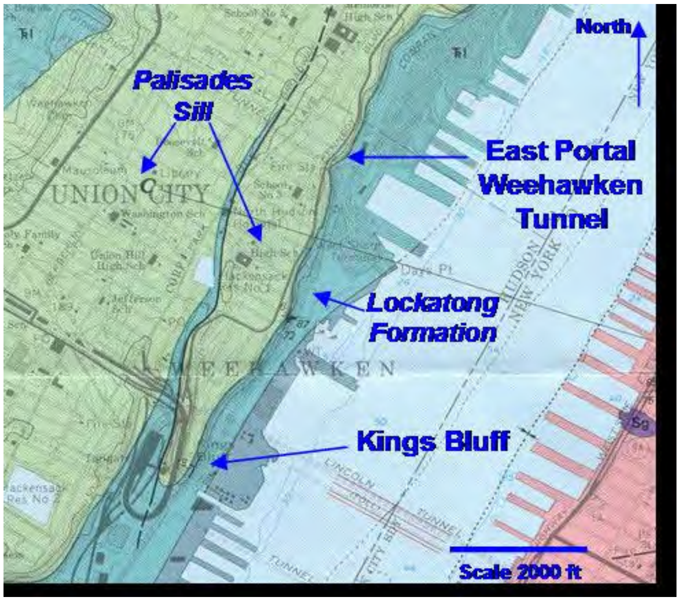


Figure 3. Site location map (Baskerville, 1994).

- Triassic Lockatong Formation – laminated to thick bedded gray and black siltstone and sandstone, occurring as contact-metamorphosed hornfels where intruded by diabase (Baskerville, 1994);

- Chill margin basalt – fine grained, nearly aphanitic, very hard basalt in the lower 20 to 40 feet of the sill (Baskerville, 1994);

- Olivine Zone – medium to coarse grained, olivine-rich basalt, soft, highly weathered zone, 1 to 5 feet thick, chiefly occurring as a zone roughly parallel to the lower sill contact, and in vertical dikes originating from the horizontal zone;

- Diabase – medium to light gray, medium to coarse grained, very hard diabase, consisting of calcic plagioclase and augite (Baskerville, 1994).

Figure 4. Vertical OZ injection dike.

The chill margin, olivine zone and diabase comprise the Palisades Sill. The bases of both slopes contain thin- to medium-bedded Lockatong Siltstones thermally metamorphosed by the diabase. The chill margin basalt and diabase are very hard, having laboratory measured compressive strengths of 18,000 to 30,000 pounds per square inch.

The geologically famous olivine zone (OZ) occurs mainly as a subhorizontal zone between the finer-grained chill margin and coarser diabase, and is sub-parallel to the intrusion contact with the lower

Lockatong (Puffer et al., 1992). The OZ has previously been thought to be formed by fractionation of the parent magma body of the sill (Walker 1969). At both sites, the OZ also occurs in near-vertical dikes with lateral gradational contacts with the diabase. These dikes originate at the horizontal OZ, and contain very closely spaced near-vertical, planar joints. Recent geochemical studies suggest the diabase in the Palisades is not the parent lithology of the OZ, thereby contradicting the fractionation theory (Puffer et al., 1992; Puffer and Husch, 1996). Recent theories propose the OZ was the fractionation product of a separate magma body from the Palisades sill, and that the parent fluid of the OZ was injected into the cooling, partially crystallized "mush" of the Palisades diabase. Field work conducted for the slope stabilization design supports this theory, with several olivine-rich basalt zones found in the rock slopes as near-vertical dikes originating at the horizontal OZ, with gradational contacts with the diabase (Figure 4).

The OZ lithology is much less resistant to chemical weathering than the underlying chill margin basalt and overlying diabase, and differential weathering has undercut the overlying diabase and has formed a shallow sloping bench on the bluffs. This bench can form a "launching ramp" for rockfalls originating higher up the slope (Figure 5). The weathering of the OZ leads to undermining of upper diabase columns, which may be the prevalent mode of failure. Weathering of the vertical OZ dikes also contributes to the formation of rock blocks in the upper portions of the diabase zone.

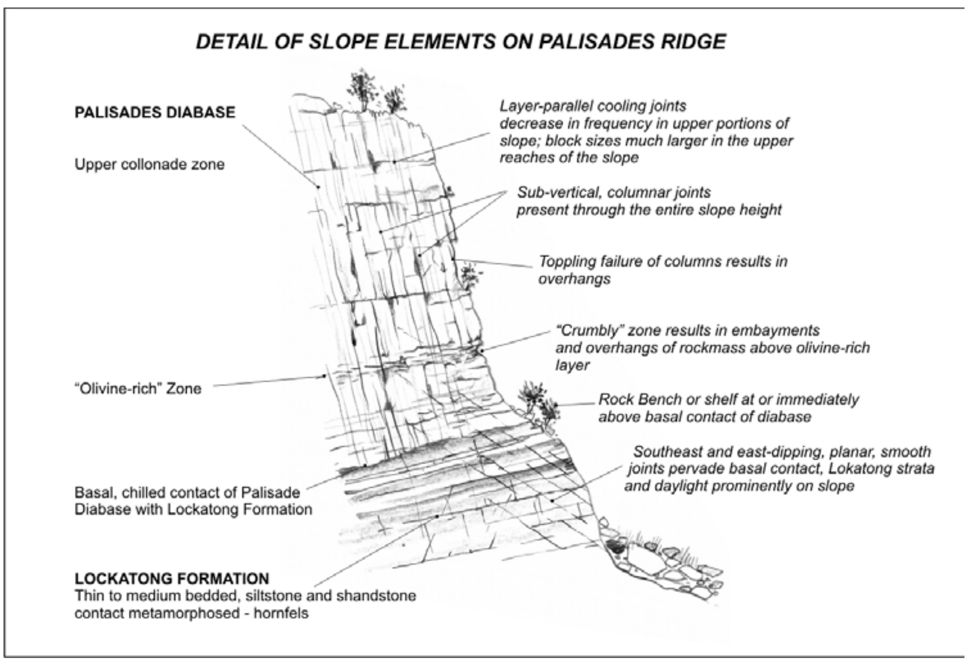


Figure 5. King's Bluff profile.

The chill margin basalt and diabase are characterized by six well developed joint sets (Figure 6). Most joints were formed by the cooling of the parent magma body. Discontinuities within the Lockatong Formation consist of bedding planes dipping gently to the northwest and vertical joints. Discontinuities within the Palisades Sill consist of steep, primary helical, columnar cooling joints, with spacings of 10 to 30 feet, splaying to near vertical at the crest of the slopes. These types of thermal cooling joints have been well documented in other basalt sills in the Newark Basin (Faust, 1978). Secondary columnar cooling joints are vertical to near vertical, and have spacings of one to five feet. The cooling joints form four discrete near vertical joint sets. A sub-horizontal joint set, parallel to the gentle northwest dipping Lockatong, is also present in the diabase. The helical, columnar and sub-horizontal joint sets form typical

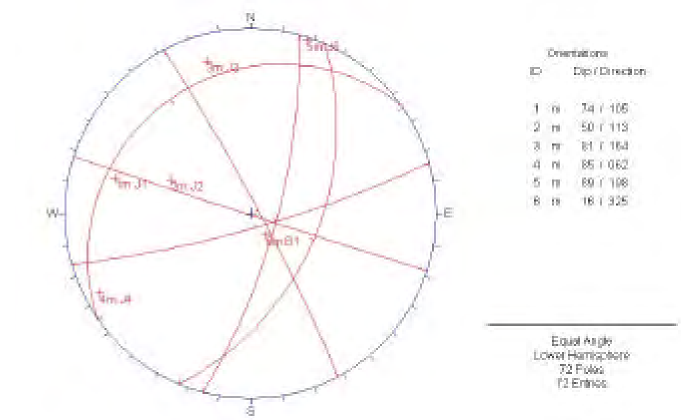


Figure 6. Joint sets at King's Bluff.

rock blocks measuring roughly 2 feet square in the lower chill margin, and up to 5-10 feet square in the upper diabase zone. An additional well developed joint set dips moderately to the southeast at the base and middle portions of the slope. This joint set cuts across all the others indicating formation by post Jurassic-age tectonism. Weathering of intersecting helical joints has caused the formation of precarious overhanging features at both sites, and was the subject of detailed stabilization design.

ROCK SLOPE STABILIZATION

Rock scaling was performed during January 2003 at Kings Bluff and during February 2003 at the East Portal Weehawken Tunnel projects. This work was performed during an unusually cold winter season. This work was done using scaling bars by workers on rappel. To minimize the visual impact on the appearance of the slopes, no air bags, jacks or explosives were allowed (Figure 7). At least 200 cubic yards of scaled rock were removed from each site prior to slope stabilization. Rubber-tire blasting mats were used to protect new concrete structures at the base of both slopes and earth berms were put in place to help contain scaled rocks. Traffic control was used when larger boulders or rock masses were scaled from the slope.

Figure 7. Scaling operations at King's Bluff.

After scaling, both slopes were inspected by geologists and engineers working side-by-side with the rock scaling personnel to identify areas requiring stabilization. Working together on rappel, rock bolt locations were identified and marked on the slope. Because of the restrictions imposed on scaling methods, many rock blocks that would normally be removed during scaling had to be stabilized in place. This restriction, together with the prohibition of installing permanent wire rope net drapes, increased the number of rock dowels and bolts required to stabilize the slopes. Because some rock blocks with some open joints were to remain in place, passive rock dowels or lightly tensioned rock bolts were favored to limit the potential for shifting or breaking rock blocks during bolt tensioning. Where overhangs had developed, buttresses consisting of fiber reinforced shotcrete were keyed into the slope with rock dowels. Dental shotcrete was used to stabilize open joints and reduce erosion of small rock fragments in joints between columnals. Drains were installed in the dental shotcrete to reduce hydrostatic pressure behind the shotcrete.

Design of the rock stabilization used a limit equilibrium approach (Hoek and Bray, 1981), and considered groundwater and seismic (pseudo-static) conditions (Humphries et al., 2001). Calculations used a basic friction angle (f) of 35° (Hoek and Bray, 1981), and field-measured asperity angle (i) of 3.4° (total of 38.4°), a laboratory-determined diabase density of 187.7 pounds per cubic foot, and typical free-body diagram geometries to determine the rock bolt force needed to retain a typical rock block. Designed factors of safety were: 1.5 for dry, static conditions; 1.3 for wet conditions; and 1.1 for pseudo-static (earthquake acceleration) conditions. A combination of grouted rock dowels (grade 75, 1.0-inch diameter, 20-30 feet long), grouted rock anchors (double corrosion protected, grade 150, 1.25-inch diameter, 30 feet long), and grouted rock pins (grade 75, 1.0 inch diameter, 8 feet long) were used to stabilize the slope.

CONSTRUCTION CONSIDERATIONS

Access for installation of rock stabilization was limited at the crest of the slopes by a private residence at King's Bluff and an active roadway at the East Portal Weehawken Tunnel. Ongoing utility and rail construction reduced available access and staging space at both locations. To overcome access issues and facilitate staging and getting rock bolts and drills to rock stabilization crews, an innovative Tyrolean hoist system was constructed at both slopes. Using a three-drum hoist, the Tyrolean system was used to deliver rock bolts, drills and drill steel to any location on the slopes (Figure 8).

Figure 8. Tyrolean hoist system at East Portal Weehawken Tunnel.

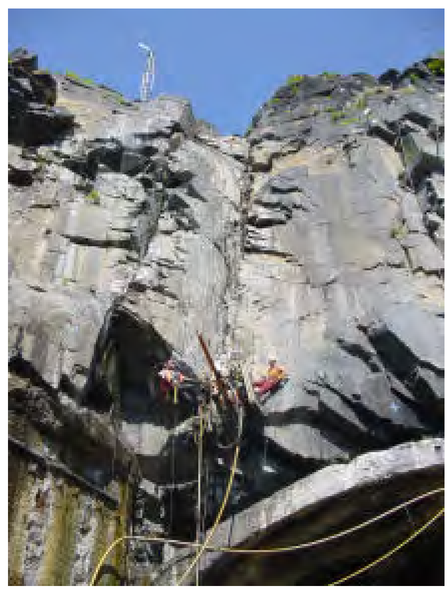


Figure 9. Wagon drill at East Portal Weehawken Tunnel.

Initial drilling methods used percussion (bencher) drills on "wagon drill" frames, as well as hand operated plugger drills. The wagon drills employ a steel box frame suspended on a cable and raised or lowered with a winch. The wagon drill frame doubles as compressed air conduit and is easily positioned and secured at the rock bolt and dowel locations (Figure 9). Rock stabilization personnel worked on rope rappel, and were secured to anchorages separate from those used for the wagon drills.

Initial drilling for installation of slope drains and the first rock dowels experienced very low production rates with the percussion drills due to the high compressive strength of the diabase. Further, dust produced by the drills was also considered a problem by the project health and safety officer as well as the residents of the home at the top of King's Bluff. To increase drill production rates, the percussion drills were replaced with down-hole hammer drills. Dust suppression was addressed with addition of a high pressure water injection system to the drill air supply. The combination of down-hole hammer drills and water injection had the added benefit of reducing site noise levels considerably. Up to four wagon drill rigs were used per slope; light air track rigs were used on one portion of the East Portal slope.

Calculations showed that rock dowels and bolts would need a minimum grouted anchor length of 4 feet beyond major fractures to adequately pin rock blocks to the slope. Drilling indicated that open discontinuities (up to several feet) existed as much as 20 feet from the face. Open or soil-filled joints were spanned by the bolts using a fabric "sock" that allowed bonding of the bolt and grout to rock and prevented excessive grout takes in jointed ground. Where accessible, open joints were backfilled with shotcrete to fully encase the bolts and fix the rock blocks in place.

Dental shotcrete and shotcrete buttresses were placed using fiber reinforced shotcrete and conventional dry-mix methods (nozzleman and man-lift) following most of the rock dowel installation. Where the olivine zone was exposed and potential for differential weathering existed, the area was protected with shotcrete, reinforced where necessary with short dowels and rebar. Overhanging areas, such as a large wedge at King's Bluff formed from the upper corner of a columnal (Figure 10), and a feature called the "Gorilla's Head" at the Weehawken Tunnel East Portal (Figure 11), received extensive rock mass reinforcement, including shotcrete buttresses below the structures to reduce overhangs, and tensioned rock bolts. Permanent drains were installed through the shotcrete to relieve hydrostatic pressure behind the shotcrete.

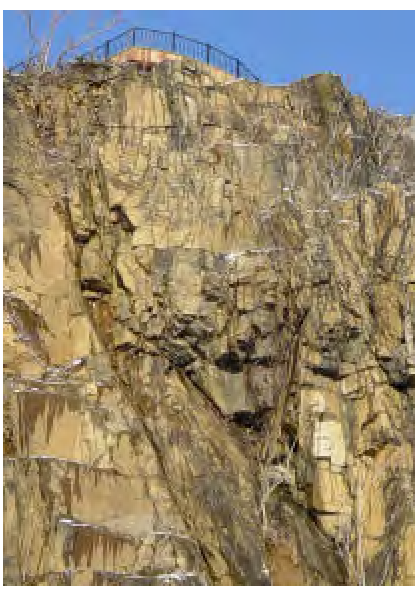


Figure 10. Large wedge overhang at King's Bluff.

Figure 11. "Gorilla's Head" formation above East Portal Weehawken Tunnel.

All structural shotcrete was covered with a layer of unreinforced, sculpted and colorized shotcrete. The surface finishing of the shotcrete layer was performed by Boulderscape, Inc. of Capistrano Beach, California. Following training for work on rappel, the Boulderscape workers began texturing coats of colored shotcrete over the fiber reinforced shotcrete (Figure 12). Particular care was taken to replicate the fracture patterns in the natural rock and to blend the treated areas into the untreated natural rock. Stains were later applied to the cured and textured shotcrete to color the rocks to the same hues as the natural outcrops, including iron and manganese staining, and buff to dark gray-green weathered and unweathered rock colors (Figure 13).

Figure 12. Structural shotcrete at King's Bluff.

Figure 13. Textured and colored shotcrete at King's Bluff.

CONCLUSIONS

Construction of aesthetically acceptable structures and rock slopes for transportation projects in urban and environmentally sensitive areas such as national parks is becoming common. Stakeholder concerns commonly drive the need for aesthetic rock slope stabilization and new construction. Measures to minimize the visual impacts of rock stabilization, such as recessed rock bolt heads, have been used at state and national parks. Construction techniques such as the use of Tyrolean hoists, down-hole hammers, water injection dust suppression and use of textured and colored shotcrete to blend repairs into natural rock slopes will become more commonplace where stakeholder concerns require construction that preserves the natural environment.

The use of a design-build contract enabled the rock slope stabilization to be completed in a compressed schedule. Solutions to specific rock stabilization problems were completed on a "fast-tracked" basis with minimal impact to the project's overall schedule by having the contractor and field geologists/engineers working together on the slope. Use of innovative construction techniques drew on the international experience of the contractor and allowed rapid adaptation to site access constraints and enabled the project to address abutter concerns.

Partnering with stakeholders to identify acceptable preservation methods was a key element of the project's success. Close cooperation between New Jersey Transit, 21st Century Rail, Washington Group and the Janod Team enabled the project to satisfy stakeholder interests regarding preservation of the Palisades Bluffs and produced improved rock slope safety for the Hudson-Bergen Light Rail Line.

ACKNOWLEDGEMENTS
The authors would like to acknowledge the contributions to this challenging project by their colleagues at the New Jersey Transit Authority, Washington Group International, Twenty-First Century Rail, and Janod Contractors. Special appreciation is extended to Charles "Ty" Dickerson, P.E. of New Jersey Transit, and Mayor Richard Turner of Weehawken. Special appreciation is also extended to our colleagues at Golder Associates for their assistance in design and construction of the projects, and for assistance in preparing and review of this paper.

REFERENCES

Baskerville, C.A., 1994. Bedrock and Engineering Geologic Maps of New York County and Parts of Kings and Queens Counties, New York, and Parts of Bergen and Hudson Counties, New Jersey. U.S. Geological Survey Miscellaneous Investigation I-2306, map scale 1:24,000.

Faust, G.T., 1978. Joint Systems in the Watchung Basalt Flows, New Jersey. U. S. Geological Survey Professional Paper 864-B, 45 p.

Hoek, E. and Bray, J.W., 1981. Rock Slope Engineering. Revised 3rd Edition, Institute of Mining and Metallurgy, E&FN Spon, London, 358 p.

Humphries, R., Warbuton, W. and Daly, J., 2001. An Approach to Earthquake Analysis of Rock Slopes. 52nd Annual Highway Geology Symposium Proceedings, May 16-18, 2001, Rocky Gap State Park, Cumberland, MD, p. 129-138.

Puffer, J.H., Husch, J.M. and Benimoff, A.I., 1992. The Palisades Sill and Watchung Basalt Flows, Northern New Jersey and Southeastern New York: A Geological Summary and Field Guide. New Jersey Geological Survey Open-File Report OFR 92-1, 27 p.

Puffer, J.H. and Husch, J.M., 1996. Early Jurassic Diabase Sheets and Basalt Flows, Newark Basin, New Jersey: An Updated Geological Summary and Field Guide. Field Trip Guide for the 68th Annual Meeting of the New York State Geological Association, The College of Staten Island/CUNY, Staten Island, NY, October 18-20, 1996, p. 1-52.

Walker, K.R., 1969. The Palisades sill, New Jersey: A re-investigation. Geological Society of America Special Paper 111, 178 p.

USING GPR REFLECTION PATTERNS AND NP MEASUREMENTS TO PREDICT SINKHOLE RISK IN CENTRAL FLORIDA

Angela L. Adams[1], Wanfang Zhou[2], Jie Wang[3] and Barry F. Beck[4]

ABSTRACT

Much of central Florida is a mantled karst terrane, where sinkhole collapse and subsidence pose a significant threat to property and the environment. Florida's well-known sinkhole collapses occur where a surficial aquifer in sand is separated from the karstic Floridan limestone aquifer by a clayey aquitard. Where there is a breach in the confining/perching strata, water in the surficial aquifer percolates downward into the Floridan aquifer causing subsurface erosion of the surface sediment and sinkhole development. Therefore, sinkhole risk assessment in central Florida relies on: (1) detecting areas of down warped deformation in the mantling sediment and; (2) determining whether the groundwater is leaking downward, undermining the stability of the site. Sinkhole collapse/subsidence hazards were evaluated by two complimentary geophysical techniques—ground penetrating radar (GPR) and natural potential (NP). GPR was used to characterize the size and location of shallow subsurface features (buried sinkholes and depressions in the water table) based on the interpretation of reflection patterns derived from stratigraphic layers. NP measurements were used to corroborate the existence of downward groundwater leakage at locations of buried sinkholes. Three risk-levels were identified: (1) Buried sinkholes with active leakage; (2) Inactive (plugged; no leakage) buried sinkholes that are in metastable equilibrium; (3) possible sinkhole features (radar interpretation indistinct) with no leakage. By combining NP and GPR data it is possible to evaluate the vulnerability of buried karst features, and thus to assess which features are more hazardous.

INTRODUCTION

A dynamic assemblage of mantled karst terrane exists in central Florida where sinkhole collapse and subsidence pose a significant threat to property and the environment. The study site, as shown in Figure 1, is located in central Florida, which is underlain by two limestone units, the Ocala and Suwannee Limestones. Overlying the carbonate sequence is the Hawthorn Group, comprising interbedded carbonates and siliciclastics. Above the Hawthorn Group is an unconsolidated and undifferentiated unit comprising quartz sand, clay, phosphate, organics (peat) and shell deposits that blanket nearly all of Florida in varying thickness and composition.

---

[1] Geophysicist, P.E. LaMoreaux & Associates, Inc., 106 Administration Road, Ste. 4, Oak Ridge, TN 37830. aadams@pela-tenn.com (865) 483-7483 x. 107
[2] Senior Hydrogeologist, P.E. LaMoreaux & Associates, Inc., 106 Administration Road, Ste. 4, Oak Ridge, TN 37830. wzhou@pela-tenn.com (865) 483-7483 x. 104
[3] Geologist, P.E. LaMoreaux & Associates, Inc., 106 Administration Road, Ste. 4, Oak Ridge, TN 37830. jwang@pela-tenn.com (865) 483-7483 x. 102
[4] Vice-President for Karst and Chief-of-Operations, P.E. LaMoreaux & Associates, Inc., 106 Administration Road, Ste. 4, Oak Ridge, TN 37830. bbeck@pela-tenn.com (865) 483-7483 x. 101

THE MANTLED KARST OF CENTRAL FLORIDA

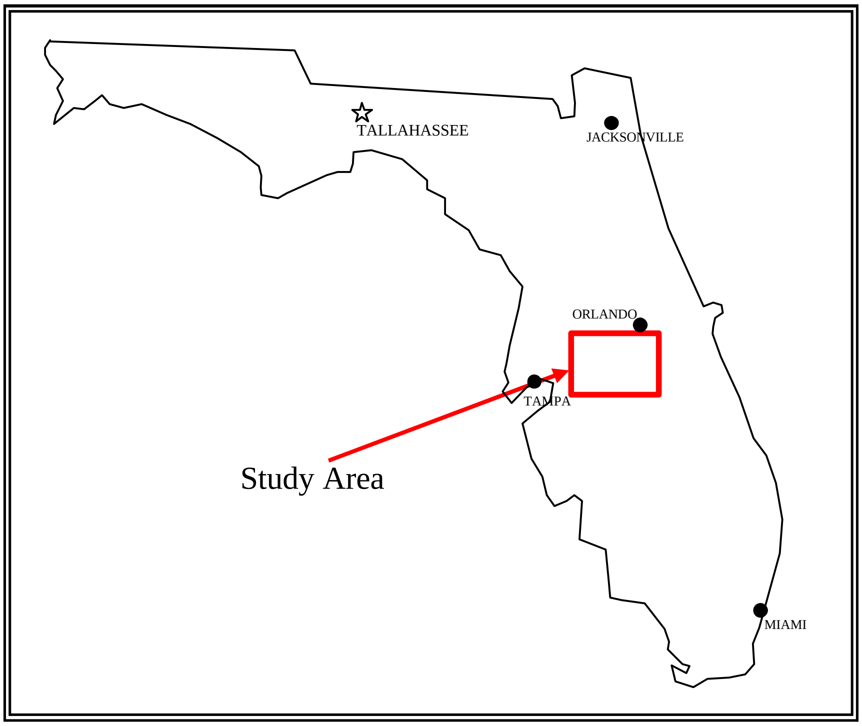


Figure 1. Site Location. Not to scale.

Karst is a distinctive topography resulting from geological weathering and erosional processes of soluble carbonate rocks that are overlain by unconsolidated sediments (Beck and Sayed, 1991). In mantled karst regions, the carbonate units are not exposed at land surface, but their presence may be indicated by sinkholes and the hummocky topography that results as covering deposits settle into the irregular surface and voids within the highly soluble carbonate rocks beneath them (Benson and Yuhr, 1987, Chen and Beck, 1989). The evolution of karst terrane and sinkholes is a long-term, sporadic process. Karst features in central Florida include sinkholes, springs, sinking streams, subsurface rather than surface drainage networks and highly transmissive but heterogeneous aquifers. Sinkhole collapse constitutes the significant geologic hazard in karst because of its inherent suddenness.

A large area of Florida is prone to karst related problems. Karst related environmental impacts as well as significant financial loss are well documented. Therefore, calls for an effective predictive method have become increasingly a necessity to proper site characterization. A legitimate karst risk assessment in central Florida includes: (1) detecting areas of down warped deformation in the mantling sediment and; (2) determining whether the groundwater is percolating downward to further undermine the stability of the site.

HYDROGEOLOGIC FRAMEWORK

Florida is underlain by an extensive system of aquifers. The principal aquifer and one of the most productive in the world is the Floridan aquifer system, a sequence of thick carbonate deposits underlying most of Florida. The high productivity of this aquifer is due to the development of secondary porosity caused by dissolution or karst processes. Hydrogeologic and stratigraphic units of central Florida are shown in Figure 2.

SYSTEM	SERIES	LITHOSTRATIGRAPHIC UNIT	HYDROGEOLOGIC UNIT
QUATERNARY	HOLOCENE	UNDIFFERENTIATED PLEISTOCENE-HOLOCENE SEDIMENTS	SURFICIAL AQUIFER SYSTEM
	PLEISTOCENE		
TERTIARY	PLIOCENE	TAMIAMI FORMATION	INTERMEDIATE AQUIFER SYSTEM OR CONFINING UNIT
	MIOCENE	HAWTHORN GROUP PEACE RIVER FORMATION BONE VALLEY MEMBER ARCADIA FORMATION TAMPA – NOCATEE MEMBERS	
	OLIGOCENE	SUWANNEE LIMESTONE	FLORIDAN AQUIFER SYSTEM
	EOCENE	OCALA LIMESTONE AVON PARK FORMATION OLDSMAR FORMATION	
	PALEOCENE	CEDAR KEYS FORMATION	
CRETACEOUS AND OLDER		UNDIFFERENTIATED	SUB-FLORIDAN CONFINING UNIT

Figure 2. Stratigraphic and hydrogeologic classification for south and central Florida (after Scott, 1992)

The surficial aquifer system is composed primarily of Pliocene-Holocene unconsolidated siliciclastics (quartz sand, clay, organics and shell), and generally correlates with undifferentiated sand and clay deposits that blanket central Florida. These deposits may exceed 100 ft in thickness where they infill karst features or are remnants of ancient dune deposits. Employing complementary geophysical surveying techniques, such as natural (electrical) potential (NP) and ground penetrating radar (GPR) holds considerable promise to map these buried karst features.

GEOPHYSICAL INVESTIGATION

Ground Penetrating Radar

Fundamentally, GPR is a noninvasive, environmentally safe method of locating and mapping shallow subsurface features in situ. The method uses ultra-wideband radio frequencies to “echo-locate” features of interest. A successful GPR survey yields a cross-sectional image of the subsurface, with signal quality and depth of penetration dependent on local soil and rock electrical properties. Using different frequency antennas, a survey can target depths from a few inches to tens of feet. In general terms, the depth of penetration is controlled by electrical conductivity and EM wave speed in the subsurface. Reflection amplitudes are a function of contrast in dielectric constants across a reflector interface. The dielectric constant is the measure of inductive capacity of a material that results from an applied electrical field (Sheriff, 1984).

Method of Study

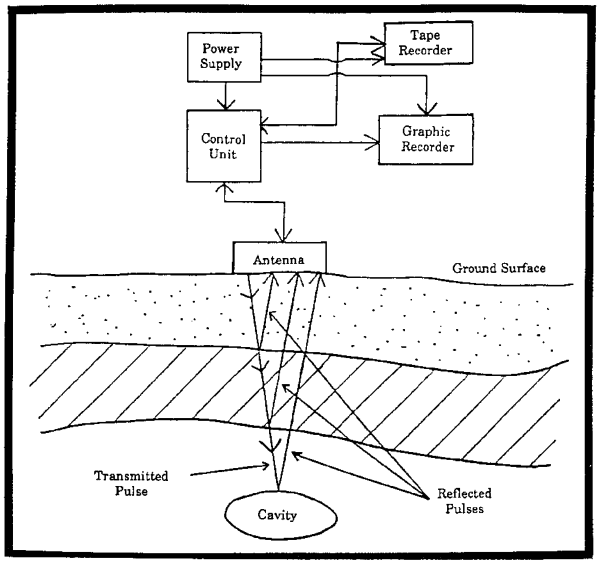


Figure 3. Conceptual diagram showing the assembly and operation of ground penetrating radar. The model used in this study did not have a tape recorder for making a reproducible record of the data. Therefore, the signal was sent directly from the control unit to the graphic recorder for printout of a hard copy.

GPR data were used primarily to rapidly identify the shallow subsurface features found beneath selected areas of central Florida by collecting data along traverses across the land surface. GPR data were collected using a SIR System III with a 100-MHz antenna manufactured by Geophysical Survey Systems, Inc. Radar pulses are generated by a surface antenna and then propagated into the ground. When they encounter a reflective interface, a portion of the radar energy is reflected back to the surface and recorded at the antenna. The wave nature of the reflected energy produces a series of positive and negative peaks (peaks and troughs) which are sensed by the receiver if they exceed a certain threshold. The antenna alternately radiates signal pulses, and then receives the reflected signals (Fig. 3). Normally, each reflection produces three detectable surges of alternate polarity which are plotted as three parallel bands on a strip chart (Fig. 4). Land traverses of GPR equipment require a 2-person field crew one to drive the vehicle and one to steer the antenna. Optimum traverse speed used for the study ranged between 1-2 mile per hour.

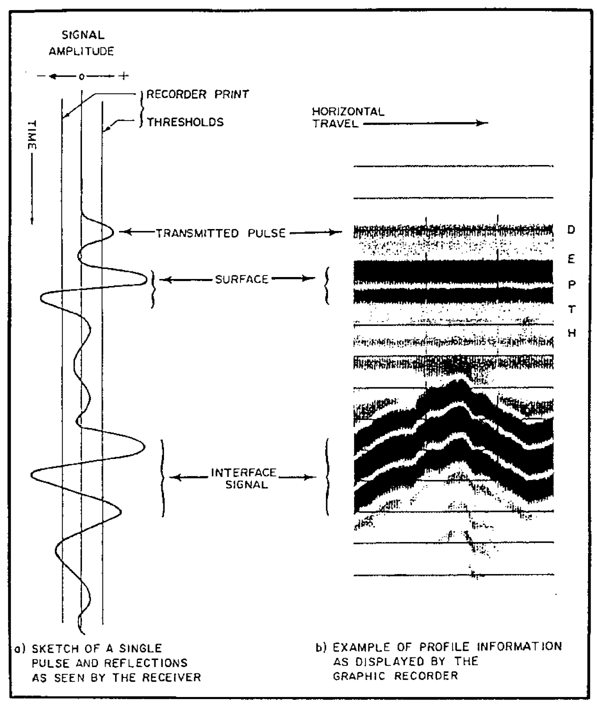


Figure 4. Diagrammatic explanation showing how the reflected radar wave is plotted on the strip chart. (Modified from GSSI, 1986)

Natural Potential

The natural potential (NP) method - also known as self-potential, spontaneous potential and streaming potential (SP) - has been used to locate areas of groundwater flow in karst terrane. The NP method is a passive electrical technique that involves measurement of naturally occurring ground potentials. The two main sources of NP signals important in environmental and engineering studies are streaming potentials, due to movement of water through porous subsurface materials, and diffusion potentials resulting from differing concentrations of electrolytes within the groundwater. The background potentials developed by electrolytes flowing through a porous media, streaming potentials, are used for the study of seepage. As water flows through a capillary system, it collects and transports positive ions from the surrounding materials. The positive ions accumulate at the exit point of the capillary, leaving a net positive charge. The untransported negative ions accumulate at the entry point of the capillary, thus leaving a net negative charge. If the streaming potentials developed by this process are of sufficient magnitude to measure, the entry point and the exit point of zones of concentrated seepage may be determined due to the negative and positive (respectively) self potential anomalies. The measurement of NP can be used to characterize groundwater flow in karst terrane because electrical potential gradients are generated by the horizontal flow of water along fractures or conduits and the vertical infiltration of water into fractures or shafts. The drift-corrected NP data represent the effect of local streaming potential, which is caused predominantly by groundwater flow (Ernstson, K. and Scherer, H.U., 1986). Infiltration of neutral water (pH of 7) causes a negative anomaly and negative anomalies have been documented in the field in areas with active infiltration (Zhou et al., 1999). A schematic diagram (Fig. 5) shows the application of NP to the Floridian Aquifer.

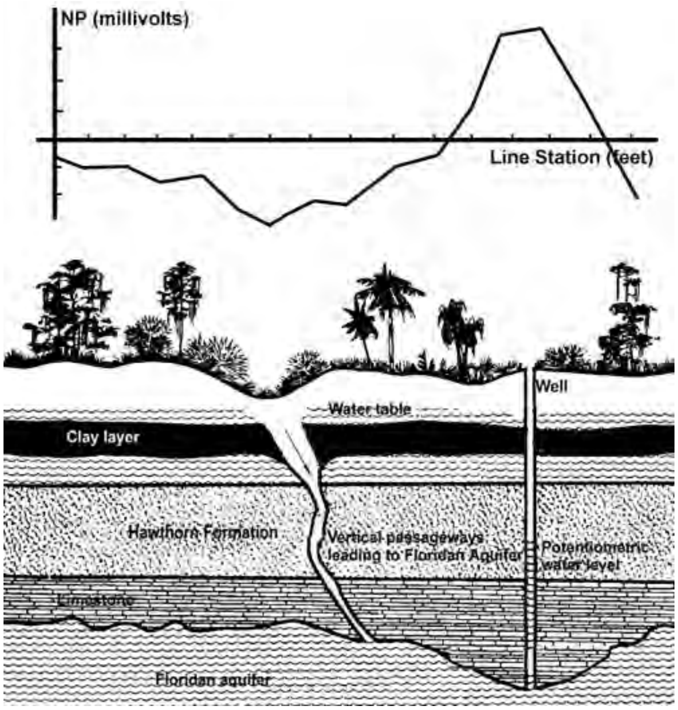


Figure 5. A schematic diagram showing the application of NP to the Floridian Aquifer for evaluation of sinkhole risk.

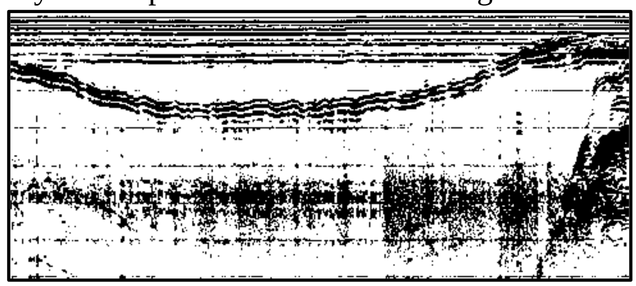
Method of Study

The NP measurement system consists of two non-polarizing copper/copper sulfate porous pot electrodes, a voltmeter and IP wire. Procedurally one electrode was kept at a fixed base station and the second (roving) electrode was moved along the traverse. At the base station, the electrode was buried to provide a stable environment. At each data point, the roving electrode was placed approximately 6 inches deep into the topsoil/sand to ensure a good contact. NP data were collected every 25 feet along each transect. At each station two readings were taken to ensure that the measurement was representative of that location. If the difference between the two readings was less than 2 mV under 100 mV and 5 mV over 100 mV, the readings were recorded and used to represent that data point. If the difference was greater, additional measurements were taken until the difference between two readings was acceptable. The base station was visited at the beginning of the transect and at the end of the transect, for the purpose of drift correction.

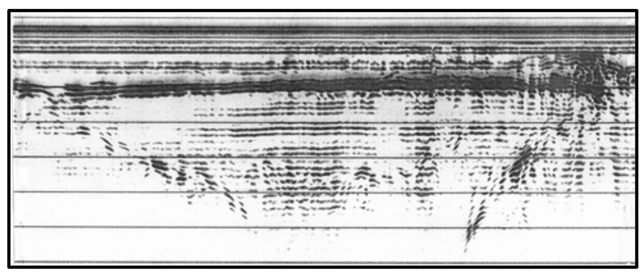
DEVELOPMENT OF THE RISK LEVELS

Building on Benson and Yuhr's 1987 characterization of GPR sinkhole anomalies we maintain that a significant indicator of risk level is whether or not a breach in the clay layer is present allowing for active infiltration of water into the subsurface. Such downward flow may cause erosion and subsequent surface collapse or subsidence. Three types of distinct reflection patterns were identified. Each reflection pattern has a characteristic signature on the radar record that indicated the presence of a subsurface structure possibly associated with sinkhole development. The definition of the reflection pattern was based on the interpretation of irregularities in the ubiquitous sand/clay boundary and depressions of the shallow groundwater level in the GPR images. The characteristic radar signatures of subsurface karstic activity or a buried sinkhole may often consist of one or more of the following:

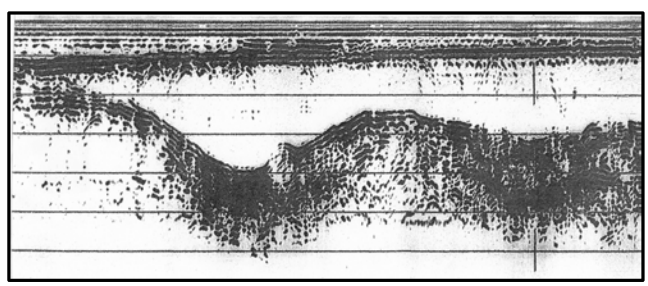
- Downwarping or abrupt displacement of shallow strata, overlain by a depressed water table (Fig. 6 at right).



- A funnel- or wedge-shaped area where either transparent reflectors are present or there is an area of irregular, disturbed reflectors (Fig. 7 at right). Disturbed reflectors may indicate a zone of collapsed sediment.



- Repetitive undulations in the deeper reflectors indicative of closely-spaced areas of subsidence (Fig. 8 at right).



Subsurface diagnostic features are used to define the sedimentation history and to locate possible breaches in the ubiquitous clay layer. In general, low angle, parallel reflections are downwarped to form a depression. These reflections are accompanied by discontinuous or segmented reflections that suggest structural displacement and subsurface subsidence. Horizontal reflections overlying the subsidence indicate subsequent fill. Rapid surficial deposition may cover karst structures associated with sinkhole development completely. The reoccurrence of these features in GPR profiles over more than 20 miles of transects led to the identification of three reflection patterns of commonly found karst features. Based on the interpretation of the GPR and NP profiles the findings can generally be placed into three risk levels (in order of most concern):

1) Indications of buried sinkholes where the shallow water table is depressed, indicating current drainage of water into the deeper part of the subsurface (Fig. 9). These sinkholes are of most concern because the leaking of shallow groundwater downward into the limestone also facilitates the erosion of the shallow sediment downward into dissolved voids, which is the main mechanism forming cover-collapse or cover subsidence sinkholes.

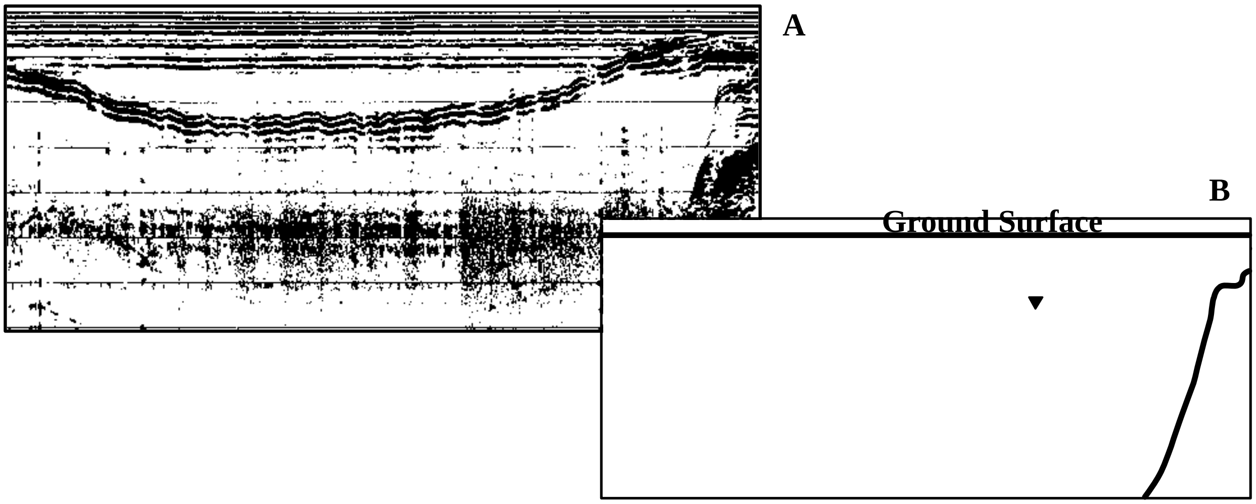


Figure 9. A: Unmigrated GPR radar strip chart with distance along traverse in feet and estimated depth below ground surface in feet. B: Line drawing interpretation of GPR profile. The water table is indicated by the inverted triangle.

2) Inactive, buried sinkholes that are in metastable equilibrium. These sinkholes show no inflection in the shallow water table (Fig. 10). The bottom of this kind of sinkhole may be temporarily blocked by clayey materials. However, these sinkholes may be rejuvenated when the current conditions are disturbed. In their present condition, they may nevertheless be prone to slow subsidence when loaded.

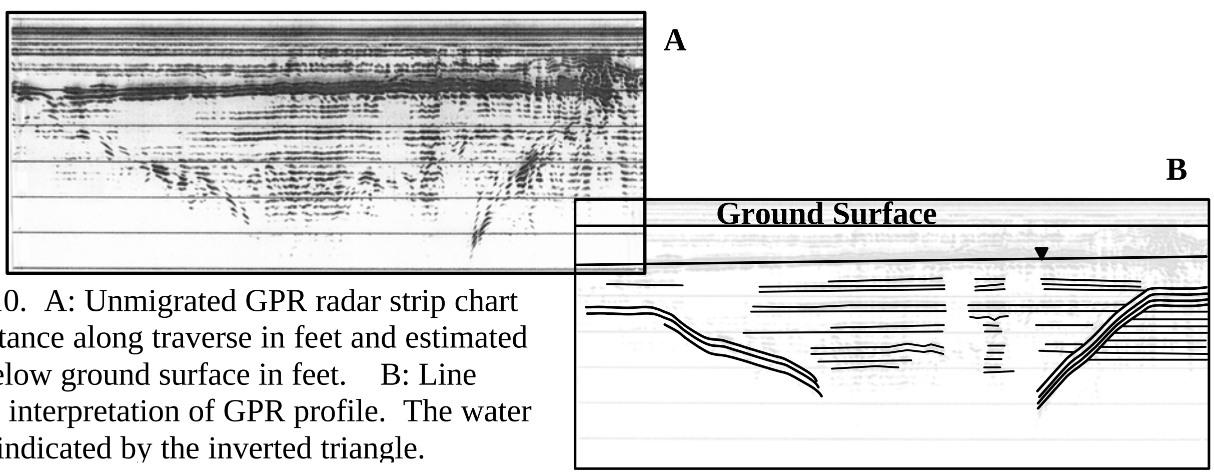


Figure 10. A: Unmigrated GPR radar strip chart with distance along traverse in feet and estimated depth below ground surface in feet. B: Line drawing interpretation of GPR profile. The water table is indicated by the inverted triangle.

3) Possible sinkhole features. These are areas where the interpretation of the radar profile is inconclusive as to whether or not the observed feature is due to karst activity or some other geologic or manmade condition.

Risk levels 1 and 2 represent depressions that have been subsequently filled to the existing ground surface. The fill is represented by horizontal reflections that may fill the depression or completely cover the subsided area. Evidence of a breach in the clay layer differentiates the two risk levels. Those containing a possible breach are risk level 1. These breaches within the depression may provide a significant hydraulic connection between surface waters, shallow aquifers, and the underlying aquifer. Where there is no major contrast in electrical properties between two materials at an interface, low amplitude reflections are present and a "transparent" zone may exist. In Florida this is quite common and infers a stratigraphy of continuous homogenous sand and/or clays.

The depth of a subsurface reflector can be estimated if the propagation velocity of a radar pulse is known. The relation between propagation velocity and depth is expressed by the following equation:

$$V_m=2D/t \tag{1}$$

where

V_m is the propagation velocity; in feet per nanosecond

D is the depth to the reflector in feet; and

t is the two-way travel time, in nannoseconds.

Types of materials in the subsurface cannot be directly identified by the use of GPR; however, reflection patterns or geometry can be useful for identification (Barr, 1993). Interfaces between the materials indicated on the GPR record become complex and may appear chaotic when sediments have been disturbed.

CONCLUSIONS

Karst hazards in central Florida range from minor subsidence to major catastrophic events resulting in property loss and rebuilding. The occurrence of sinkholes damages roads, bridges, buildings and pipelines. It is becoming more imperative that the engineers in charge of construction have an effective tool to predict the potential for significant karst hazards. Since, the ground penetrating radar did not reach the depth of limestone it is impossible to directly associate surface topography with subsurface voids. However, depressions and features in the overlying sediments are indirect evidence of buried solution cavities. Subsurface deformation features were detected using GPR methods in areas where surface depressions were not observed. The GPR method also detected subsurface sediment deformation features indicative of sinkhole development, even in areas where surface depressions were not visible. Hydrogeologic conditions, lithology can be inferred from interpretation of GPR data. Breaches in confining beds identified on a GPR profile may indicate potential for groundwater flow between upper and lower aquifers, which can be further defined by the addition of NP measurements. The addition of a second, complementary geophysical data set provides confirmation of the interpretation and a greater understanding of mechanisms and paths of anomalous seepage flow. As indicated in figures 10 and 11, karst development has affected the shallow, unconsolidated sediments in central Florida.

REFERENCES

Barr, G.L. 1993, Application of ground-penetrating radar methods in determining hydrogeologic conditions in a karst area, west-central Florida: US Geological Survey Water Resources Investigations Report 92-4141, 26p.

Beck, B. F. 1988, Environmental and Engineering Effects of Sinkholes the Processes Behind the Problems: Environmental Geology and Water Science, v. 12, no. 2, p. 71-78.

Beck, B. F. and S. Sayed (1991). The sinkhole hazard in Pinellas County: A geologic summary for planning purposes. Winter Park, FL, Florida Sinkhole Research Institute.

Benson R.C. and Yuhr L.B., 1987 Assessment and long term monitoring of localized subsidence using ground penetrating radar. Proceedings of the Second Multidisciplinary Conference on Sinkholes and the Environmental Impacts of Karst: Orlando, Florida. A.A Balkema Publishers pp.161-169.

Chen J. and B.F. Beck (1989). Qualitative Modelling of the Cover-collapse Process. Proceedings of the Third Multidisciplinary Conference on Sinkholes: St. Petersburg, Florida. A.A Balkema Publishers pp.89-95.

Ernstson, K. and Scherer, H.U., 1986. Self-potential variations with time and their relation to hydrogeologic and meteorological parameters, Geophysics, 51:1967-1977.

Knochenmus, L.A. and Robinson, J.L.,1996, Descriptions of anisotropy and heterogeneity and their effect on ground-water flow and areas of contribution to public supply wells in a karst carbonate aquifer system: U.S. Geological Survey, Water-Supply Paper 2475, 47 p.

Kindinger, J.L., Davis, J.B., and Flocks, J.G.,2001, Subsurface Characterization of Selected Water Bodies in the St. Johns River Water Management District, Northeast Florida U.S. Geological Survey Karst Interest Group Proceedings, U.S. Geological Survey Karst Interest Group Proceedings, St. Petersburg, Florida, February 13-16, 2001: USGS Water-Resources Investigations Report 01-4011

Tihansky, A.B., 1999, Sinkholes, west central Florida—A link between surface water and ground water, *i*: Galloway, Devin, Jones, D.R., and Ingebritsen, S.E., 1999, Land Subsidence in the United States: U.S. Geological Survey, Circular 1182, p. 121-141.

Zhou, W.F., Beck, B.F., Stephenson, B.J., 1999. Investigation of groundwater flow in karst areas using component separation of natural potential measurements, Environmental Geology, 37(1-2), 19-25.

GPR and FWD subsurface investigation techniques used to investigate voids in variable subgrade soils on I-40 Near Topock Arizona

Nick M. Priznar, *Engineering Geologist, Arizona Department of Transportation*
Kenneth M. Euge, R.G., Engineering Geologist, and **Kerry Hennon**, R.Gp., Geophysicist*, Geological Consultants Inc., Phoenix, AZ*
Mark Edwards, *Geophysicist, Ninyo & Moore, Phoenix, AZ*

ABSTRACT

Interstate 40, in Mohave County Arizona is part of one of the principal transportation corridors in the United States. Unpredictable development of small to medium sized voids has occurred over a 4-year period directly beneath the paved sections of this route between Mileposts 4 and 7.9. Voids typically occur near transverse pavement cracks, in the vicinity of embankment cut and fill sections. The underlying natural ground varies between gypsiferous expansive clay, dune sands and terraced alluvial fan deposits.

Three techniques were used to evaluate the condition of the subgrade:

Ground Penetrating Radar surveys were executed in areas of known voids in order to ascertain the feasibility of developing signature profiles of radar anomalies at void locations. Test surveys using a variety of GPR antennas were also conducted to develop the exploration techniques most likely to be locally successful. After completion of this test, several miles of additional surveys were conducted to locate anomalies where voids were anticipated to occur. In total 1,120 radar anomalies were identified. After additional evaluation the number was reduced to 255 priority locations.

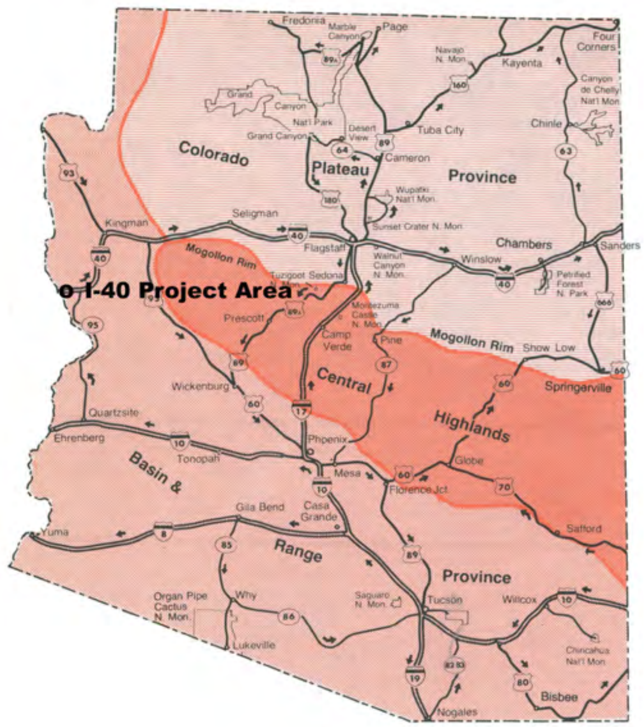


Figure 1: Project area location map (courtesy of Mountain Press Publishing, Missoula, MT.

In order to further discriminate the possibility of detecting a void at these locations a Dynatest falling weight deflectometer was utilized to evaluate the strength of the pavement and subgrade material at 89 of these positions. Test sites with D 1 value greater than 19 were judged more likely to represent an anomalous location where a void might occur. Eight Priority 1 GPR anomalies that identified subgrade voids or disturbed zones were confirmed by FWD tests where D1 values were greater that 19.

Finally the subgrade was investigated at 39 locations with a shallow mechanical earth auger and air probing equipment to record the conditions of the anomaly sites. Some of the borings were located in areas where there were obvious pavement distress and other sites which demonstrated statistically low D1 values. Of these 39 sites 8 small voids were detected and 15 sites demonstrated suspiciously weak subgrade conditions. None of the sites investigated revealed voids of a size that would require immediate maintenance.

LOCATION

The project is located along both the east and westbound Interstate 40 from Milepost 4.2 to 7.9 east of Topock Arizona in Mohave County. This is roughly 130 miles South of Las Vegas Nevada, and 4 miles east of the Colorado River (Figure 1).

BACKGROUND DATA

Figure 2: I-40 view looking westbound.

The typical highway section consists of two 38-ft roadways, one eastbound and one westbound. Each direction consists of two 12-ft travel lanes, a paved 10 –ft outside shoulder and a paved 4-ft inside shoulder (Figure 2). The as-built cross slope is 1.5%. The eastbound and westbound roadways are separated by a varying width graded median. The project elevation ranges from approximately 650 feet to 900 feet.

Average annual precipitation is less than 8 inches, and the average daily temperature ranges from 40° F. in January to 108° F. in July.

I-40 was constructed in the 1960’s along the former alignment of Old Route 66. Parts of the westbound alignment were constructed directly over the former paved surface and embankments. It was at this time that the replacement drainage structures were constructed for the new divided two-lane highway, and older drainage infrastructure was abandoned in place.

Throughout the 70's and 80' the highway provided service to the great migration of interstate commercial and California bound traffic. Significant deterioration of the pavement and embankment failure begin to occur in specific areas underlain by rain saturated Bouse Formation and Holocene deposits during the late 1980's. Plans were made to isolate the expansive material by over-excavating and covering the structural native soils with a geomembrane and non-expansive fill.

Figure 3: Partially patched transverse pavement cracking.

Subsequently, normal service resumed for a period of approximately ten years when subgrade failure again began to manifest itself as large transverse pavement cracks (Figure 3) crossing both east bound and west bound lanes developed into 3' dia. voids precipitating pavement structure to collapse (Figure 4). Although this may be considered a small area of disturbance it could pose a significant hazard to vehicles traveling at a 70mph.

<u>REGIONAL GEOLOGY</u>

The project area is located in the Basin and Range Physiographic Province, on the eastern flank of the Lower Colorado River Corridor (Figure 1). It is bounded on the south by the Mohave Mountains and the Sacramento Wash on the North. The locality can be described as a series of late Tertiary alluvial, lacustrine/marine, and aeolian sediments, which were deposited in a preexisting Miocene trough, composed of deformed and faulted igneous and extrusive volcanic bedrock.

Figure 4: Repair of collapsed pavement section.

Prior to the emplacement of the modern Colorado River the regional drainage system may have been dominantly confined to a closed basin. The major sediments of the

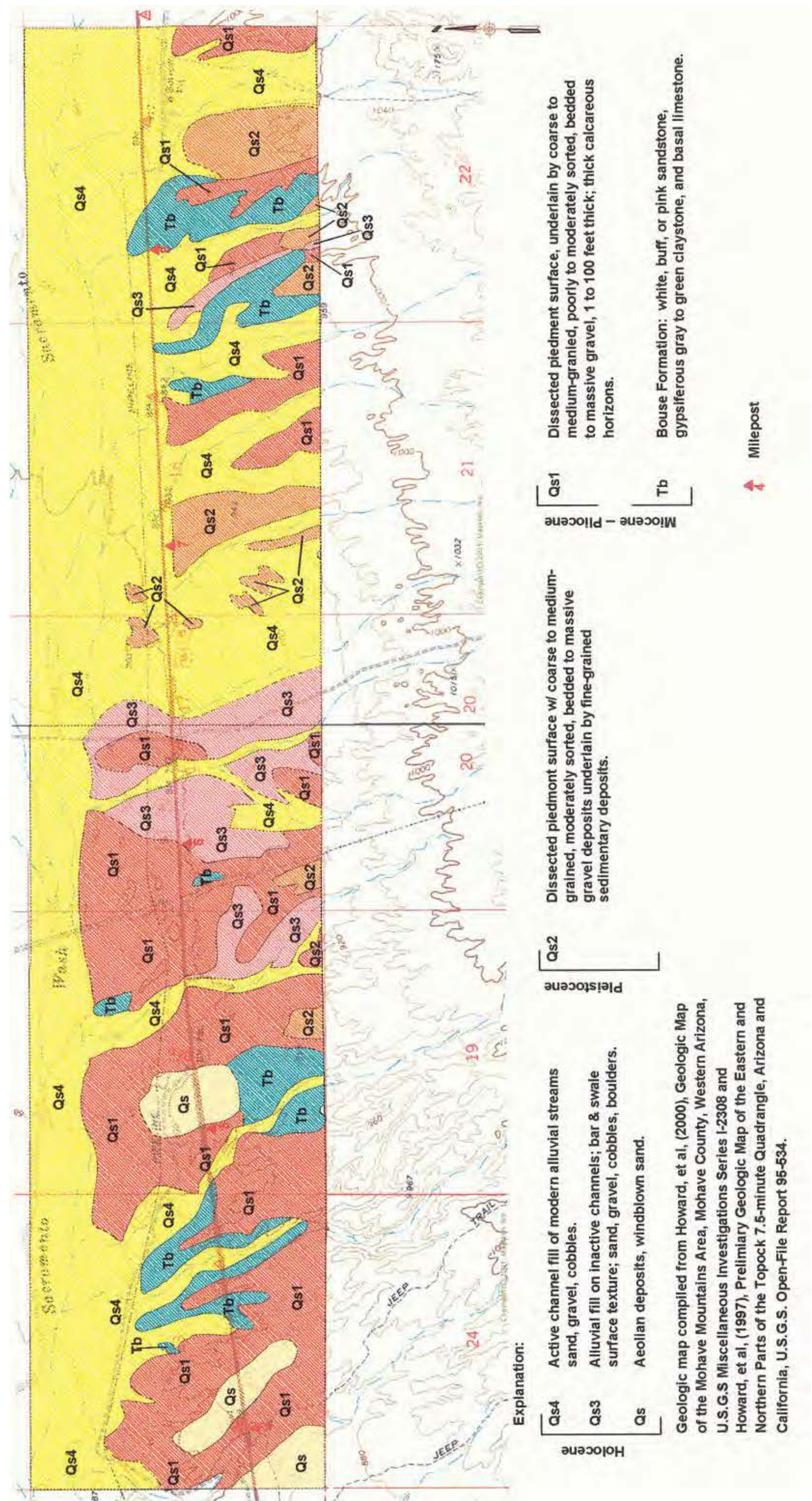


Figure5: Surficial geology of the I-40 project area.

area consists of alluvial fans deposits, and lake/marine deposits unconformably resting on a Miocene and older, basement rock. Upon advancement of the Colorado River through the region, a new base level was established ultimately draining pluvial lakes and simultaneously eroded and buried portions of the former topography. This process created inserted sets of "alluvial veneers on erosion platforms" (Wilshire and Reneau, 1992), in the project location, reflecting the incision of the Colorado River and its tributaries onto the older deposits as it evolved into present position.

GENERAL STRATIGRAPHY

Within the project area the general stratigraphy is composed of the three general lithologic groupings (Figure 5). Miocene Bouse Formation eroded and terraced Alluvial Fan Deposits Pleistocene and Pliocene, and Holocene sediments consisting of the modern stream channel deposits and fine-grained aeolian sands. The profile (Figure 6) and grade of Arizona's eastern section of I-40 intersects this complex set of deposits and has created variable subsurface conditions between Milepost 4.2 – 7.9.

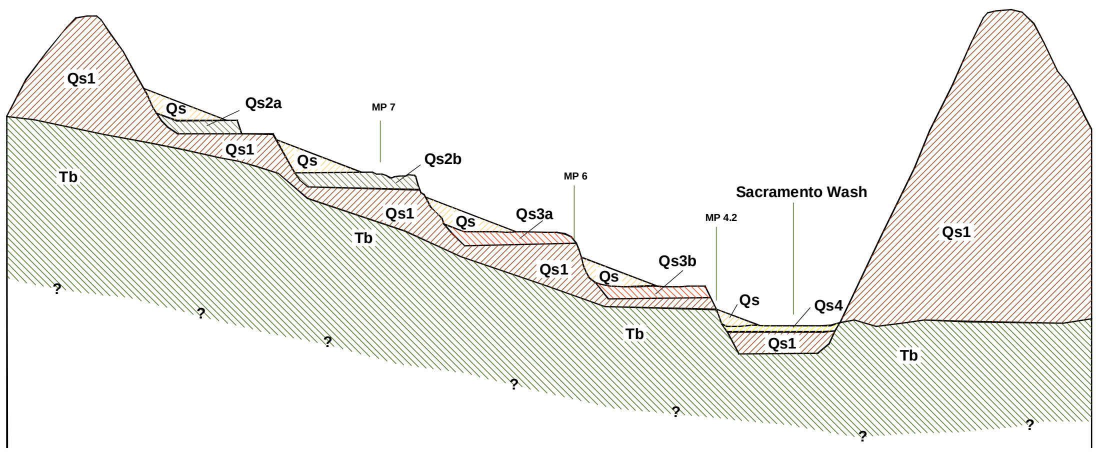


Figure 6: Conceptual geological cross section depicting stratigraphic relationships across I-40 alignment. See Figure 5 for description of geologic units. Adapted from Wilshire & Park, 1992.

<u>BOUSE FORMATION</u>

The Bouse formation is composed of three recognizable lithologies that were described by Metzger in 1973. These units are a Basal Limestone, an Interbedded unit composed of clay, silt and sand, and an upper Tuffa Unit. Of these three only the Interbedded Unit has been recognized in the project area (Figure 7).

Interbedded Unit

Figure 7: Interbedded unit of the Bouse Fm. (Courtesy of Doyl Wilson, Mohave Community College).

Within the project area the Interbedded Unit typically is recognized as a pale olive to yellowish green fissile and expansive claystone that outcrops in many of the cuts and underlies a great deal of the I-40 alignment. The clay stratum displays prominent gypsiferous lenses with a desiccated and fissured texture when exposed in fresh cuts (Figure 8). Amorphous greenish debris often covers the lower slopes and blooms with a “popcorn” texture characteristically associated with expansive sedimentary bedrock. Additionally the Interbedded unit appears to be susceptible to piping and dissolution along small desiccation features and mud cracks. Associated with these clays are interbedded silt and fine-grained sand strata that are very poorly consolidated and weakly cemented. Both the underlying and overlying contacts of the Bouse Formation are locally disconformable and represent periods of weathering and erosion before and after its deposition.

ALLUVIAL FAN DEPOSITS

Figure 8: Typical exposure of expansive clay within Bouse formation exhibiting desiccation and fissure features.

The Alluvial Fan deposits are in disconformable contact with the Bouse Formation filling former paleo-topographic irregularities. It can be generally located as the prominent higher eroded hills, and truncated cliffs that are visible parallel to the highway that rise in grade to the south. These deposits are composed of “unsorted to poorly sorted, coarse, and angular to subangular clastic debris derived from pre-Tertiary and Tertiary rocks” (Howard, 1990). Generally these deposits are exposed as a dissected alluvial fan deposit. Commonly these units display alternating lenses of well bedded to massive sand and gravel deposits.

Additionally dissection, planation and reworking the material inserted a series of younger terraces onto the local drainages. Younger alluvial deposits associated with the westerly flowing Sacramento Wash and wind blown sand have locally obscured this material.

These older deposits often display calcareous horizons up to 18 inches thick. The top of this horizon forms a crust that is more durable and resistant to erosion than the local material above and below it. Local (desiccation?) jointing in these materials appears to facilitate formation of voids just below the base of the cemented zone. When exposed to seasonal rainstorms these joints conduct surface water flow and enlargement of the voids continue until the cemented horizons can no longer support the load of the overlying soils (Figure 9).

Figure 9: Near vertical fracture and associated piping features within the alluvial fan deposits.

<u>HOLOCENE SEDIMENTS</u>

The Holocene sediments can be separated generally into two broad categories. Unconsolidated clay silt, sand and gravel cobbles and boulders associated with the modern stream washes, and aeolian (wind blown sand) deposits.

Both the older fan deposits and the Bouse Formation underlie the modern alluvial deposits of the project area. Generally their depth varies from 5 to more than 20 feet.

The aeolian deposits vary in thickness throughout the area and, in places, mask the near surface exposures of the Bouse Formation and the Alluvial Fan Deposits. Dune sand is present in the area but its surface appears to be generally stabilized. However, dry and loose excavated aeolian subgrade embankment soil flows easily backfilling boreholes.

INVESTIGATION OF PAVEMENT DEFORMATION:

Because of the severe pavement cracking and void formation initially identified by ADOT maintenance personnel, ADOT Geotechnical Services requested, through their on-call geotechnical services contract, an investigation be performed to locate areas of suspect voids, soil piping, and other subgrade distress. Geological Consultants Inc. and Ninyo & Moore proposed to use ground penetrating radar (GPR) to locate anomalies related to voids and other distress features along selected sections of the I-40 alignment east of Topock, Arizona.

Ground Penetrating Radar

Ground Penetrating Radar (GPR) was the geophysical technique selected to assist with the investigation. Radar is the optimum geophysical technique because it has the highest spatial resolving capability of any method and data can be quickly collected and interpreted. This method can identify subsurface changes in soils stratigraphy and man-made objects that create dielectric contrasts with the surrounding soil. Generally, geologic material have relative dielectric constants of 20-50 whereas air filled voids have a value of one. A large contrast between suspect voids, or other discontinuities, and the surrounding soil can cause obvious anomalies in the radar data. However, the presence of a change of electromagnetic properties in itself does not constitute a subgrade failure. In the course of normal construction it is very possible to have zones of uneven compaction efforts, different types of soils, differing moisture conditions, differing gradations of soil, all of which can contribute to varying electromagnetic properties.

Figure 10: GPR survey and antenna test at selected site along shoulder of I-40.

A two phased program was proposed: the first was a test GPR program at three sites along the alignment where ADOT maintenance personnel had conducted emergency repairs of pavement collapse associated with void features. During the test, various frequency antennas were evaluated along the same GPR profile lines to determine which one would provide enhanced resolution for the given site conditions (Figure 10).

A total of 5,400 linear feet of GPR were run at the three sites. Figure 11 is a plan of a typical test area depicting the GPR locations and results. Three prominent types of anomalies were resolved during the test program: (1) Point anomalies, identified as discrete features, were identified below the pavement section but were not encountered in adjacent GPR profiles. Point anomalies possibly represent an open crack or fissure of limited extent (Figure 12a). (2) Linear, two-dimensional anomalies where subsurface

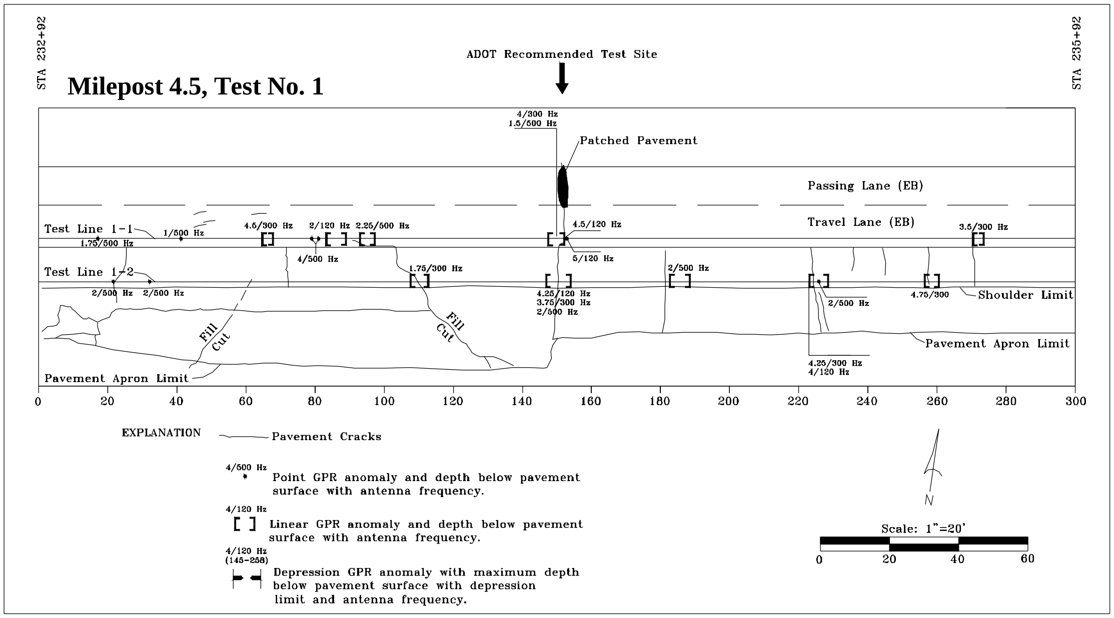


Figure 11: Typical GPR survey anomaly location map generated from data obtained along selected section of I-40 alignment during test program.

features were identified over a discrete interval along the survey line and extended to a unique depth below the pavement section. The features may represent a wide, open crack, fissure or disturbed zone within the base course or subgrade (Figure 12 b). (3) Depression anomalies, where the boundaries of the anomaly appear to be gradual

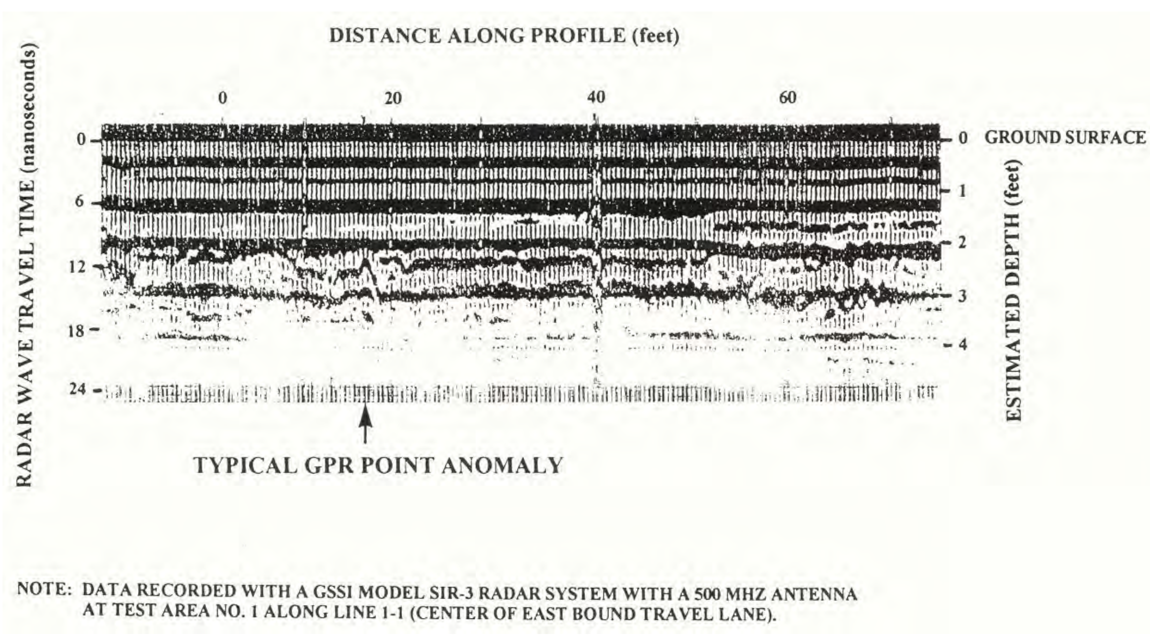


Figure 12a: GPR survey real-time record depicting typical GPR point anomaly.

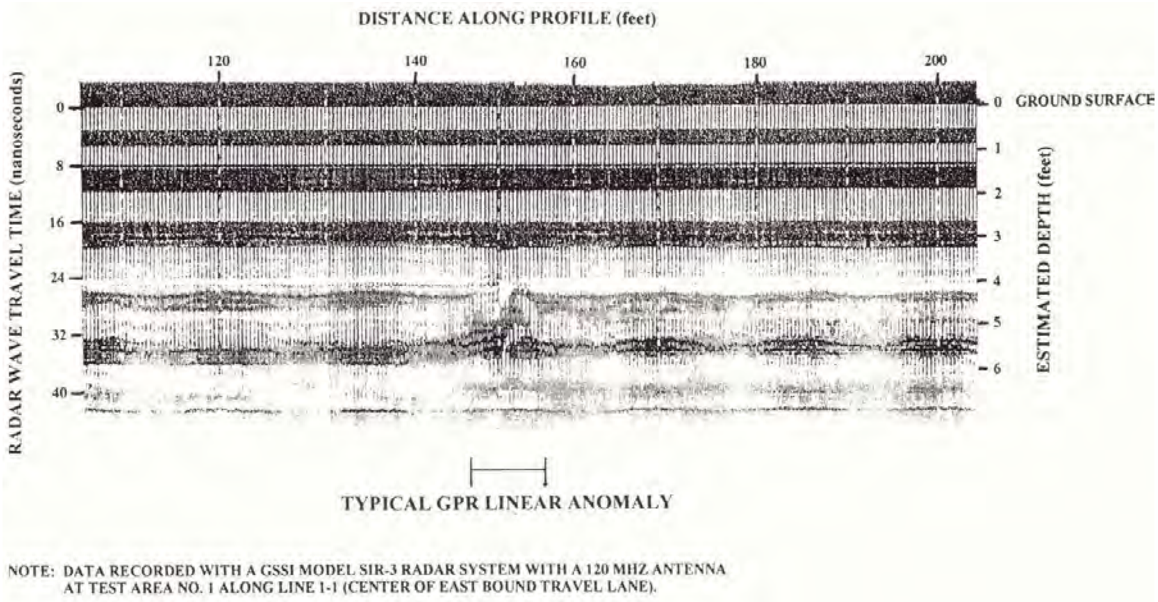


Figure 12b: GPR survey real-time record depicting typical GPR linear anomaly.

(or steep), either extend to unique depths below the pavement section or to a depth that was beyond the GPR penetration. These anomalies possibly represent a cut/fill transition zone, a consolidation/settlement zone within a fill section, or heaved expansive subgrade section (Figure 12c) (Geological Consultants, 2001a).

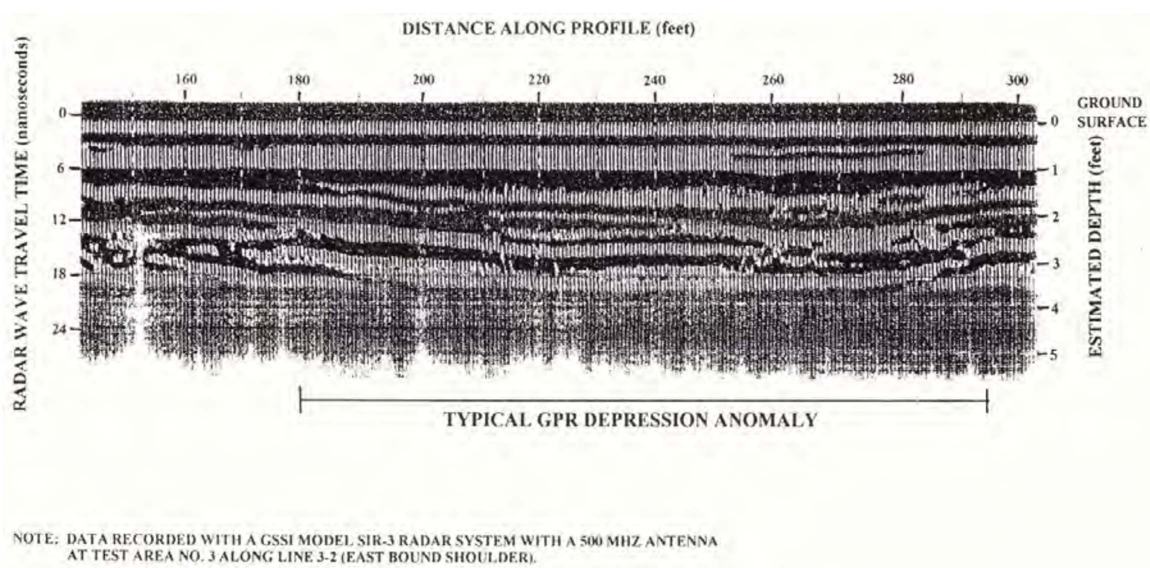


Figure 12c: GPR survey real-time record depicting typical GPR depression anomaly.

Because of the apparent success of the GPR test program, ADOT authorized Geological Consultants Inc. and Ninyo & Moore to proceed with the production survey of the remaining critical areas identified by ADOT. The joint GPR survey team conducted approximately 37,590 linear feet of GPR profile lines. Within the production survey area 1,120 radar anomalies were delineated. Of these, a total of 255 anomalies were interpreted to represent significant deviations from the typical roadway section design. The 255 anomalies were classified as Priority 1 anomalies defined by prominent radar responses that may represent voids or disturbed zones. Because of anomaly characteristics, these anomalies were interpreted to have a significantly higher potential to cause pavement distress and therefore the anomalies were targeted for subsequent direct subsurface evaluation (Geological Consultants, 2001b; Ninyo & Moore, 2001). Following the completion of the production GPR surveys, ADOT Geotechnical Services requested Geological Consultants Inc. and Ninyo & Moore to determine which of these anomalies were more likely to represent the more deleterious subgrade condition sites. At this point the list was reduced to 89 anomalies.

In order to examine the data more closely another test was tried. To that end the 89 GPR anomalies were further tested by Falling Weight Deflectometer (FWD). The FWD measures the mechanical resistance of the pavement and subgrade to applied stress.

FWD Testing

"The Falling weight deflectometer (FWD) (Figure 13) has been frequently used to evaluate structural integrity of pavement. The device applies an impulsive force on the surface of pavement and measure surface deflections at several locations including the place of loading. Although the test is dynamic, the data is regarded as pseudo-static data. According to common practice, using the peak load and the corresponding peak deflections, layer moduli are estimated in a static domain such that the measured peak deflections coincide with the corresponding calculated deflections based on the assumption of the theory of linear elasticity." (*Matsui K. et al 1998)*

Figure 13: ADOT Falling Weight Deflectometer (FWD) system.

"This system develops forces from the acceleration caused by the arrest of a falling weight. The (pavement) surface will bend in response to the applied load much as

it would deflect due to the weight of a passing wheel. The structure will bend downward and exhibit a "deflection basin". The deflection response when related to the applied loading can provide information about the strength and condition of the various elements of a pavement. Analysis can be used for load transfer, void detection and back-calculation." (Law Engineering, 2000)

In practice the FWD drops a calibrated weight, and imparts a stress, through a plate onto the pavement surface. An array of seven sensors, (D1-D7), positioned at 0, 8, 12, 18, 24, 36, & 60 inches from the load plate measure maximum deflections in the pavement surface, (deflection basin), and records the data through several iterations of drops.

According to FHWA-LTTP (2000), *"A pavement section with surface discontinuities such as cracks and or joints, or subsurface discontinuities such as voids, will generally exhibit higher deflection than a pavement section without such discontinuities."* Therefore, a location within the pavement section that exhibits a greater than average response to stress in the center of the deflection bowl, (lower than average resistance to mechanical stress at position D1), which also coincides with a geophysical anomaly (a characteristic electromagnetic signature corresponding to a suspected void in the subgrade), should indicate positions where voids are most likely to be encountered.

For this project, testing was conducted with a Dynatest Falling Weight Deflectometer Systems at eighty-nine positions identified by Geological Consultants Inc./ Ninyo and Moore. These locations appeared to be most likely to indicate voids in the subgrade. FWD testing was also performed at 12-foot offsets to determine the stress environment immediately before and after the indicated position. There were 39 GPR locations in the eastbound lane and 50 GPR locations in the westbound lane. Correspondingly there are 117 FWD testing locations on the eastbound lane and 150 FWD testing locations in the westbound lane.

Statistical Analysis

A statistical analysis was performed on all the FWD data to calculate an arithmetic mean for eastbound and westbound D1 deflections. Typical results of the FWD testing and statistical analysis are presented in Figure 14. These averages were used as a gross indicator of average response to mechanical stress. A standard deviation was established for each set of data. D1 deflections outside one standard deviation, with less resistance to average mechanical

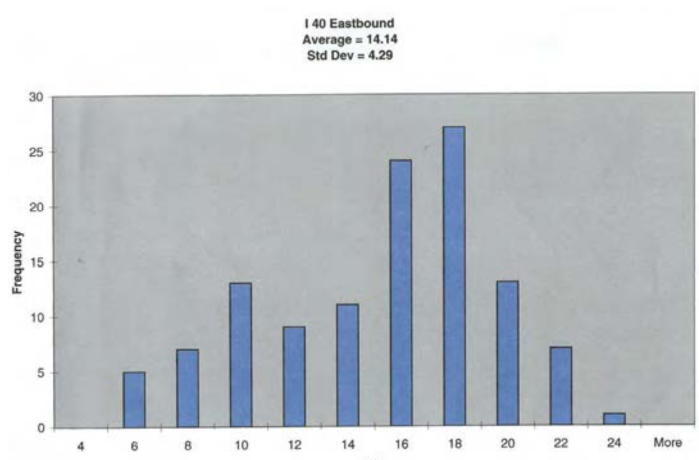


Figure 14: Results of FWD testing and statistical analysis.

stress, (higher deflections) were identified as being most likely to indicate a subgrade condition of probable deleterious void formation.

The average east bound DI value calculated as 14.13 with a standard deviation of 4.30. Therefore D1 values above 18.43 were considered anomalous. The average westbound DI value calculated as 15.94 with a standard deviation of 3.75. Therefore values above 19.69 were considered anomalous. Generally values at or above 19.0 were suspect.

Subgrade Drilling and Air Probe Investigation

The subgrade investigation was conducted in the fall of 2001. Locations of geophysical anomalies and subsequent borehole locations were located in the field by Ninyo & Moore.

ADOT Geotechnical Operations Section advanced a total of thirty eight (38) test borings. Each boring was advanced with two independently operated drilling rigs. A Sprague and Hayward model B324 was utilized to drill 12-inch diameter pavement core to the base of the pavement section (Figure 15). A 763 bobcat loader, equipped with a model 15 auger drive unit and a 9 inch diameter flight auger was utilized to bore an inspection hole from 4 to 6 feet, below the top of pavement surface. Some of the boreholes were completely hand dug beneath the pavement (Figure 16). The sidewalls of the borehole were air blown clean to a depth of approximately 3 feet. A 4-foot long 1" diameter air probe was inserted to the bottom of the boring to determine the relative density of the material immediately below the disturbed soils (Figure 17) and to search out soft zones or voids (Figure 18).

Figure 15: Bobcat loader with 9-inch diameter flight auger for drilling inspection boreholes. Note 12-inch diameter core for pavement section. Excessive thickness due to successive overlay repairs of roadway section,

Figure 16: Hand-dug boreholes were required at some sites to minimize void collapse and fracture filling.

Grab samples were gathered from the auger borings and submitted to the ADOT Materials Laboratory for gradation, Atterburg limits, moisture and calculated "R"Values. A total of 28 samples were tested for Ph, resistivity, and sulfate soundness to determine if corrosive agents were present in the subgrade material.

Figure 17: Air-probe used to evacuate loose soil from test borehole exposing void and fracture features.

<u>RESULTS OF SUBGRADE DRILLING PROGRAM</u>

Subgrade material encountered during the boring investigation generally consisted of Silty Sand (SM) with varying amounts of angular gravel and cobbles, Sand well graded (SW), Sand poorly graded (SP), and occasional lenses of Clay (CL) and sand, cobbles, and boulders (GW). Asphalt base course, where detected, varied inconsistently between 1 to 6 inches of angular gravel and cobbles. Native soils and some fill material consisted of moderately dense calcified zones and cemented horizons of sub-angular to rounded granitic and sedimentary gravel with a sandy to silty matrix. Zones of loose caving material were often encountered below dense cemented layers and could either be an area of low density native soil (Alluvial fan deposits, or eolian sands) that has been bridged over by the cemented layer or an area of low density subgrade. Oversize cobbles and boulders were rarely observed in a few of the boring locations.

Highly plastic, expansive, olive green clay of the Bouse Formation was observed in the cut slopes and in the ditches parallel to the highway but were not encountered in any of the borings in the paved section subgrade. However, piping and dissolution of clayey material was observed in the east bound shoulder near station 416+86 (Figure 19). Deleterious cracking and distorted drainage infrastructure was observed within the investigated area when the improvements were in intimate contact with expansive green clay of the Bouse Formation. Aggressive corrosion was

Figure 18: Cleaned borehole exposing fissure crack in subgrade soil.

also observed in culverts and drains in contact with this material. Laboratory testing of expansive clay in the area indicated Liquid Limits ranging from 72 to 90 and Plasticity indexes ranging from 46 to 56.

OVERALL TESTING RESULTS

Figure 19: Soil erosion pipes in clay section of Bouse formation exposed in eastbound shoulder ditch at Station 416+86.

A total of three boreholes displayed voids at or near co-existing GPR and FWD anomalies. However voids were also detected in random borings drilled in locations with pavement cracks and construction joints. The voids observed were less than two inches in diameter. However, clusters of small voids approached 6 to 12 inches in total cross sectional distance. Fissuring through the pavement and into the subgrade was observed in six borings. Most fissuring when present was detectable to a depth of two feet. The fissures are important because they may become conduits and enlarge into voids. Fissure widths varied from ¼ inch to one inch.

Void development, geophysical anomalies and weak subgrade were generally concentrated in areas constructed on changing bedrock units, near cut and fill boundaries. Higher FWD D1 data did indicate areas of pavement distress with associated subgrade deterioration. Eight locations demonstrated good correlation between all test methods and weak subgrade material where FWD D1 values were greater than 19. However, only 50% of the potential locations were investigated with borings.

Voids were also observed in boreholes where the D1 values were compatible to a higher than average resistance to mechanical stress. Therefore voids can exist at GPR locations that do not have a significant deleterious effect on the structural section or the compacted subgrade. The GPR did accurately locate the position of abandoned CMP's, and box culverts which eliminated some of the proposed evaluation borings and were not specifically identified by FWD data.

The GPR anomalies that were bored did indicate loose (very low density) soil materials in the subgrade under cemented zones, gravel lenses and beneath a geo-membrane. The results of the GPR survey program indicate that this method is a reliable indirect investigative technique for locating areas of disturbance surrounding void features and zones of low density subgrade soil. However, as with any indirect, nondestructive testing method, GPR and FWD anomalies do not automatically indicate that voids are present. Due to varying electromagnetic properties of native soils, fill soil, subgrade materials, and other variables such as moisture and clay content, not all subgrade features of interest, including voids, may be detected by the GPR survey method. Size of voids, GPR surveyor experience, and the equipment selected for the GPR surveys also play a role in whether or not voids will be successfully detected. Direct exploration, such as drill holes or test pits should also be performed to verify the anomalies identified.

In this investigation, the observations of the material in the boring suggest that certain locations may become sites of piping, fissuring and void development in the future. At the time of this investigation, the detected anomalies did not appear to present an immediate threat to pavement soundness. It appears that void development can be controlled by aggressive maintenance of all pavement cracks to prevent infiltration of surface water into the subgrade and aggressive maintenance of drainage infrastructure facilities that may be in close proximity to expansive soil or bedrock.

REFERENCES

Bull, W.B., Geomorphic Responses to Climate Change, Oxford University Press, New York, 326 p.

Chronic H., 1983, Roadside Geology of Arizona, Mountain Press Publishing Company, PO Box 2399, Missoula, Mt 59806

Geological Consultants Inc., 2001a, consultants report entitled “Results of Task 1 Test Program, Geophysical Investigation of Subsurface Voids, I-40, Milepost 2.0 to 8.0, Mohave County, Arizona”, Geological Consultants Project No. 2001-105, March 14, 2001, 5 p.

Geological Consultants Inc., 2002b, consultants report entitled “Geophysical Investigation of Subsurface Voids, Task III GPR Production Survey, I-40, Milepost 4.2 to 7.9, Mohave County, Arizona”, Geological Consultants Project No. 2001-105(3), June 6, 2001, 4 p. w/ Appendices.

Howard K. A., John B. E., Nielson Jane E., 1997, Preliminary Geologic Map of the Eastern and Northern Parts of the Topock 7.5-minute Quadrangle Arizona and California, U.S. Department of the Interior, U.S. Geological Survey, Open File Report 95-534.

Howard K. A., Nielson J. E., Wilshire H. G., Nakata J. K., Goodge J.W., Goode S. L., Reneau B. E., Hansen V. L., 2000, Geologic Map of the Mohave Mountains Area, Mohave County, Western Arizona, U.S. Department of the Interior, U.S. Geological Survey, Miscellaneous Investigations Series I-2308.

John, B.E., 1987, Geologic Map of the Chemehuevi Mountains Area, San Bernardino County, California and Mohave County, Arizona, U.S. Geological Survey Open-File Report 87-666.

Law Engineering and Environmental Services Inc., 2000, LTPP Manual for Falling Weight Deflectometer Measurements Operational Field Guidelines Version 3.1, August 2000, Prepared By; FHWA-LTPP Technical Support Services Contractors LAW PCS, A Division of Law Engineering and Environmental Services Inc., Prepared For: Office of Infrastructure R & D, Long term Pavement Performance Team HRDI-13, Federal Highway Administration.

Matsui K., Nishizawa T., Kikuta,Y.,1998, Time domain back calculation of pavement structure, Paper #: 3400-47 *in*: SPIE Proceedings Vol. 3400, pp 410 – 419, Structural Materials Technology III: An NDT Conference, Editor(s): Ronald D. Medlock, David C. Laffrey, The International Society for Optical Engineering, P.O. Box 10, Bellingham, Washington, 98227-0010, USA

Metzger D.L., Loeltz O.J., 1973, Geohydrology of the Needles area, Arizona, California and Nevada: U.S. Geological Survey Professional Paper 486-G, 130 p.

Missouri Department of Transportation, 1994, RDT 96-001, Evaluation of Falling Weight Deflectometer, Missouri Department of Transportation, Research Development and Technology Div. P. O. Box 270, Jefferson City. Missouri 65012.

Morey, R.M., 19989, Ground Penetrating Radar for Evaluation Subsurface Conditions for Transportation Facilities, NCHRP Synthesis of Highway Practice 255, Transportation Research Board, National Research Council.

Ninyo & Moore, 2001, consultants report entitled "Geophysical Evaluation, I-40 at Needle Mountain to State Route 95, Milepost 4.2 to 7.9, Mohave County, Arizona", Ninyo & Moore Project No. 600080005, July 3, 2001, 12 p. w/ appendices.

Rexford M. M., 1998, Synthesis of Highway Practice 255, Ground Penetrating Radar for Evaluating Subsurface Conditions for Transportation Facilities, Transportation Research Board, National Research Council, National Academy Press, Washington, D.C. 1998.

Wilshire H. G., Reneau S. L., 1992, Geomorphic surfaces and underlying deposits of the Mohave Mountains Piedmont, Lower Colorado River, Arizona, Zietschriftg Fur Gemorphologie, v. 36, p 207-226.

Wilson D., 2002, Virtual Tour Colorado River Boat Trip from Topock to Lake Havasu, Mohave Community College; http://homepages.mohave.edu/science/fieldtrip/topock.html.

Geologic Characterization for Bridge Foundations, Colorado River Bridge, Hoover Dam Bypass Project

Nicholas J. LaFronz, P.E.
AMEC Earth & Environmental, Inc.
3232 W. Virginia Avenue
Phoenix, Arizona 85009
Tel. 602-272-6848
email: nick.lafronz@amec.com

David E. Peterson, P.G.
AMEC Earth & Environmental, Inc.
3232 W. Virginia Avenue
Phoenix, Arizona 85009
Tel. 602-272-6848
email: david.peterson@amec.com

Robert D. Turton, P.E., S.E.
HDR Engineering, Inc.
2141 East Highland Avenue
Suite 250
Phoenix, Arizona 85016
Tel. 602-508-6600
email: rturton@hdrinc.com

Scott Anderson, Ph.D., P.E.
FHWA-CFLHD
Denver Federal Center
Building 52
P.O. Box 25246
Denver, Colorado 80225-0246
Tel. 303-716-2191
email: scott.anderson@fhwa.dot.gov

ABSTRACT

Increasing congestion on US 93 (a NAFTA route) at Hoover Dam resulted in the 3.5-mile Hoover Dam Bypass Project from Clark County, Nevada, to Mojave County, Arizona. CFLHD-FHWA, the managing partner responsible for project delivery, awarded the design to the Hoover Support Team, led by HDR Engineering with T.Y. Lin International, Jacobs Civil and subconsultants, including AMEC for geological and geotechnical engineering services. The 1,900-foot long Colorado River Bridge is planned in a rugged bedrock setting in Black Canyon, 1,500 feet downstream of the dam and 900 feet above river level, with 1,090-foot main arch and seven approach spans. Bridge components are to include steel box girders, composite concrete deck, and reinforced concrete arch rib and pier columns, foundations and abutments. Geology of the bridge site consists of Tertiary-age volcanic and sedimentary rocks. Foundation areas feature basalt flows, local breccia, massive tuff and highly irregular basalt dikes. Geotechnical investigation of the bridge site required a highly diverse and specialized approach to data collection in the rugged bedrock environment. Included were 3D laser scanning for topographic mapping of the canyon walls, geologic mapping by multiple methods (including with mountaineering techniques), core drilling with specialized rigs/access equipment, optical televiewer borehole logging to acquire fracture data, NX borehole jack testing and down-hole seismic surveys, and laboratory rock strength testing. Analysis of distribution of rock types and capacities in the various foundation areas, and excavation slope designs and stabilization designs, were performed using the collected surface mapping, rock quality and fracture information, and televiewer, in situ strength and seismic data.

INTRODUCTION

This paper presents a brief description of the design development and geotechnical investigative approach required in the rugged canyon terrain for the Colorado River Bridge, the principal component of the U.S. Highway 93 (US 93) Hoover Dam Bypass Project, located in Mohave County, Arizona and Clark County, Nevada (Vicinity Map – Figure 1). Also included is a summary of the geologic conditions in the canyon slopes and limited geotechnical findings at the two skewback sites.

Figure 1 - Vicinity Map

The Hoover Dam Bypass is designed to greatly enhance mobility in the vicinity of historic Hoover Dam; the bypass will relocate through-traffic off the dam and onto a new high-speed, four-lane roadway. The selected Sugarloaf alignment alternative of this facility carries the roadway approximately ¼-mile downstream of the dam, requiring nearly 3.5 miles of new approach roadway and a 1,900-foot-long bridge across the Black Canyon, a 900-foot deep gorge carved by the Colorado River. The bridge profile will peak at an approximate elevation of 1530 feet, about 885 feet above the normal Colorado River elevation of 645 feet. The proposed bridge will consist of a composite arch structure 1,896 feet in total length (total of 16 spans), with the main arch 1,090 feet in length spanning the central canyon. The structure includes five approach spans on the Nevada side and two on the Arizona side, which range in length from 100 to 120 feet. The overall bridge width is 88 feet. Arch skewbacks will bear at elevation 1200 feet within the canyon walls.

The effort is being led by the Project Management Team (PMT), which includes the lead agency, the Central Federal Lands Highway Division (CFLHD) of the Federal Highway Administration (FHWA), supported by the Arizona Department of Transportation (ADOT), Nevada Department of Transportation (NDOT), U.S. Bureau of Reclamation (BOR), National Park Service (NPS) and the Western Area Power Administration (WAPA). The project design team - designated the Hoover Support Team, or HST - is headed by HDR Engineering and includes major partners Jacobs Civil (Arizona approach roadway) and T.Y. Lin International (River Bridge). Geotechnical engineering services were provided to the HST by AMEC Earth and Environmental of Phoenix, Arizona and Reno Nevada. Information contained herein consists of a description of the investigative techniques utilized, a description of the geology and limited geotechnical details, and a bridge design overview. Presently, the project is in the final design stage, such that detailed design information is not available for general release.

GEOTECHNICAL INVESTIGATION

Geotechnical investigation of the River Bridge site followed a targeted and phased approach, and included a preliminary assessment and a final investigation. The preliminary assessment consisted of a review of available (published and unpublished) geologic and geotechnical data, aerial photograph interpretation, helicopter reconnaissance of the project corridor, geologic surface mapping and reconnaissance of the canyon walls, refraction seismic surveys, and limited laboratory testing. A topographic base map of the Black Canyon bridge site was also created at this phase utilizing a long-range laser scanning technique. The objective of the preliminary assessment was to provide preliminary geological and geotechnical data to the design team early in the design process to identify geotechnical project constraints. Subsequently, the preliminary assessment was supplemented by subsurface investigation consisting of seven core borings at the proposed skewback sites, accompanied by optical televiewer (OPTV) logging , in situ NX-borehole jack (Goodman Jack) testing in three borings, down-hole seismic surveys in four borings, and laboratory testing of rock core. The objective of the supplemental investigation at the preliminary phase was to collect subsurface geological and geotechnical data to supplement the surficial data, in order to permit assessment of foundation conditions at the short- and long-span bridge alternatives. Geotechnical issues of primary concern for all alternatives included evaluation of the rock quality at the arch foundation landings and the stability of the cut slopes required to create the skewback foundation excavations.

Final-phase characterization of the geological and geotechnical conditions at the River Bridge site consisted of drilling 22 core borings at bridge abutment, pier and skewback locations, OPTV logging of selected borings, in situ Goodman Jack tests and down-hole seismic surveys in selected borings, and an extensive program of laboratory testing of rock core samples. The objective of the final investigation was to collect additional geological and geotechnical data to supplement the preliminary assessment data. The final investigation was structured to permit further assessment of the foundation conditions at each of the skewbacks, piers and abutments, in order to provide recommendations for the foundations and excavation slopes/slope stabilization.

Surface Geologic Mapping

The geologic units exposed in the slopes of Black Canyon at the River Bridge site were mapped and characterized using multiple techniques due to the rugged terrain. The techniques included ground inspections by conventional foot access in less steep slopes, canyon wall inspections using mountaineering techniques with the assistance of a professional mountaineer/rigger, aerial photograph review (with limited success due to strong canyon shadows), three-dimensional (3-

D) laser scan point cloud image review for photo lineament identification, and helicopter reconnaissance for additional views of canyon walls and geology units in non-accessible areas. The mountaineering inspections included six rappels completed on the Arizona side and ten rappels completed on the Nevada side for the preliminary assessment. Initial efforts provided characterization of exposed rock units, including documentation of the lithology, degree of weathering and hardness, distribution of occurrence and the general frequency, character and orientation of primary discontinuities contained in the rock mass. Eleven additional rappels along the canyon walls (five on the Arizona side and six on the Nevada side), were performed during the final geotechnical investigation to further assess the global stability of the skewback foundations, considering their close proximity to the natural canyon walls, by a more focused inspection for potentially adversely-oriented discontinuities below the skewback foundations with the potential to form sliding wedges. Documentation of geologic data during rappels was recorded on canyon wall photographs. Potential structural features identified during the 3-D point cloud image review were ground proofed during the various field inspections and confirmed features were included on the photo and map bases. Photograph 1 depicts Richard Bansberg of AMEC rappelling on the canyon wall.

Photograph 1 – Rappeller inspecting Nevada Side Canyon Wall. Rappeller is next to white target in center of picture.

Topographic Base Map of Canyon Walls

A three-dimensional laser scanning technique was utilized to create a topographic base map of the Black Canyon slopes on the Arizona and Nevada sides at the River Bridge crossing, since the very steep to near-vertical to locally overhanging canyon walls precluded the use of conventional aerial mapping. A series of high-resolution laser target scans (the ground surfaces were scanned at a spacing of 6 to 8 inches between each scanner pass) of each canyon wall were performed using a long-range laser mirror scanner. Several linked laser scans of each side (a total of about 15 to 20) achieved an area of coverage extending for about 2,200 feet along bridge/roadway centerline, vertically from the crest of the canyon walls to the river level (a mapped vertical dimension of about 900 feet), and laterally for a distance of about 300 feet along the walls, approximately centered on the bridge centerline. Scanner setups in multiple locations for both sides were required to obtain complete coverage of the desired areas due to the rugged geometry of the walls, obstructions of view, and scanner range (distance vs. resolution) limitations. A helicopter was required to access one scanner set-up on the Nevada side, located about 500 feet south of the southernmost existing transmission line, in order to eliminate "scan shadows" on the south-facing Nevada-side surfaces. Temporary scan targets were set on the surfaces to be scanned and then surveyed for location and elevation.

The laser scan data (three-dimensional point clouds) with supporting target survey data were registered and translated onto the project coordinate/elevation datum, vegetation edited out, and the data processed to produce a file of three-dimensional faces (polygonal surfaces) and a digital terrain model (DTM) for the scan area at a 1-foot contour interval, layered into 1-, 5- and 10-foot intervals. The end product was topographic base map files for use by the HST in AutoCAD and MicroStation file format. The 3-D scan data also permitted creation of the renderings of steep to vertical or overhanging canyon wall surfaces, overlaid with excavation geometries, and with anticipated excavation conditions for the skewbacks, which are discussed in a later section of this paper. These renderings were utilized to visualize and assess skewback excavation and bearing conditions. More conventional presentations (topographic plan views, cross sections, and profiles) also were developed using the scan data and resulting base maps to support the bridge design, earthwork estimates and plans preparation.

The resulting DTM showed reasonably good agreement with the DTM developed from the aerial photogrammetric mapping at the edges of the laser-scanned area (within the areas of overlap between the two DTMs). Initially, it was anticipated that digitally-rectified photo overlays of the canyon walls could be derived from the laser scans and overlaid onto the wall topographic surfaces, for utilization in the surface mapping and reconnaissance of the canyon walls. However, the resulting product was distorted by “stretching” and was unusable, so good quality photographs of the walls were used as bases during surface mapping. The scanning was completed by DEI Professional Services, Inc., and surveying of targets was completed by Aztec Engineering, Inc.

Drilling

The steep and rugged terrain at the River Bridge site made conventional truck-mounted drill rig access impossible. The steep to vertical slopes, complicated with existing overhead transmission lines which coincide with the proposed alignment, required more innovative mobilization techniques with lightweight portable equipment. The techniques included using a helicopter, crane and articulated backhoe to lift the equipment into place. The helicopter was most commonly used; the road crane (operated from the US 93 “hairpin turn” on the Nevada side), and articulated backhoe (referred to as a “spyder”) were used to access sites where overhead power lines limited the use of the helicopter. Track-mounted drill rigs traveled to boring locations at the Arizona side abutment. All borings were advanced using either HQ- or NQ-size wireline diamond bit rock coring systems. The borings varied from vertical to inclined between 8 and 44 degrees from vertical in order to intersect critical subsurface areas associated with the skewback foundations from areas where the rigs could set-up. Some locations directly adjacent to steep cliffs required temporary construction of fencing for worker safety. The fences were installed by the drill crews suspended by ropes prior to placing the rigs. Drilling services were provided by CRUX Subsurface, Inc. (CRUX). Photographs 2 through 4 present drill rig set-ups using the various techniques.

Optical Televiewer Logging

Optical televiewer (OPTV) logging of selected borings was completed to obtain measurements of the orientation and dip of rock discontinuities penetrated in both vertical and inclined borings to assist in cut slope evaluations. The OPTV system was lowered down the boring and a 360-degree optical "slice" of the boring was imaged using a camera lens. The analog video signal from the camera was transmitted to the OPTV surface instrumentation, digitized, and presented

Photograph 2 – Helicopter setting drill rig components on Arizona canyon slopes above skewback.

Photograph 3 – Crane assisted transport of drill rig at knoll on Nevada side at skewback location.

Photograph 4 – Articulated backhoe (Spyder) assisting drill rig set up on Nevada abutment boring.

as a two-dimensional (2-D) unwrapped image of the boring wall. A digital fluxgate magnetometer was used to determine the orientation of the system and digital image. The OPTV image is presented as an oriented, 2-D picture of the boring unwrapped from south to south, and planar features (discontinuities) which intersect the boring appear as sinusoids on the unwrapped image (Figure 2). The angle of the dip of a given discontinuity is equal to the arctangent of h/d, where "h" is the amplitude of the sinusoid and "d" is the boring diameter. The dip direction is coincident with the trough of the sinusoid.

Each discontinuity was subjectively ranked from zero to 5 based on the observed characteristics of the feature, including appearance, distinctiveness, apparent aperture (width of opening) and interconnection to differentiate the more significant features. All discontinuity orientations were referenced to true north and corrected for boring inclination. It should be pointed out that use of the televiewer results in much larger quantity of data (strike and dip values) than would be obtained from a borehole with more conventional methods, such as oriented coring, or from measurements of exposures. Consideration of the discontinuity rankings (particularly aperture) and use of engineering judgment are required in distillation and application of the data. Also, the OPTV data were used as a quality control check during review of the boring logs. Pole plots of the OPTV data and surface discontinuity measurements obtained during canyon mapping were developed and stereographic analyses of the data performed to develop slope recommendations for the excavations. OPTV logging, data processing and reducing were performed by CRUX. Feature rankings and inclusion of comments were performed by AMEC.

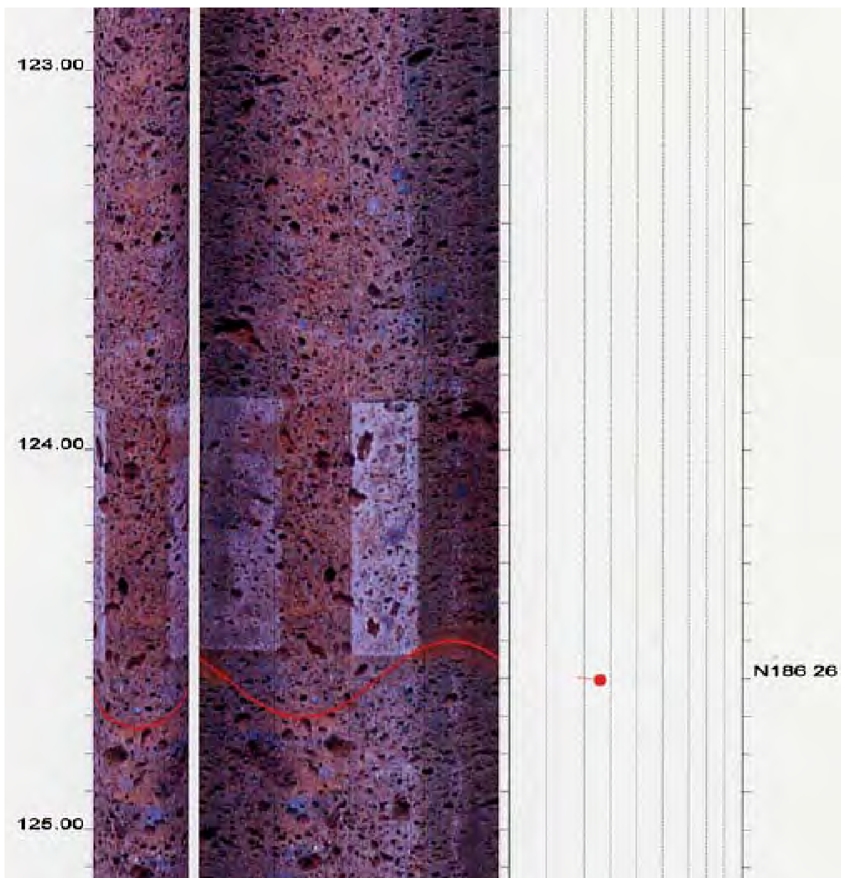


Figure 2 – Optical televiewer image of borehole wrapped – left, unwrapped – right, with fracture orientation and Goodman Jack plate impression in borehole wall..

In Situ Static Modulus (Goodman Jack) Tests

The in situ elastic modulus (deformation modulus) of the bedrock units at the skewback sites was evaluated using an NX-borehole jack (Goodman Jack) system and ASTM procedures. The system consists of a twelve-piston hydraulic jack with curved bearing plates designed for use in a 3.00-inch diameter (NX-size) boring, and includes measurement of the rock deformation under load using two linear variable differential transformers (LVDTs) integral to the jack unit. The system has a maximum capacity (in terms of rock modulus) of about 14 to 15 million pounds per square inch (psi), based on a maximum system operating (hydraulic) pressure of 10,000 psi. Generally, each tested interval included two measurements, one each approximately parallel and perpendicular to the canyon wall. Locations (depths) of the tests within the skewback borings were selected to bracket the planned skewback foundation bearing level, within an overall depth limitation (from top of boring) of approximately 150 feet for the jack hydraulic system. Estimation of static modulus was performed within a range in applied pressure of about 200 to 10,000 psi. Field data were reduced and static moduli estimated using the procedure recommended by Heuze (1980, 1984) and Heuze and Amadei (1985) and an estimated value of Poisson's ratio of 0.25 (the data reduction method is not overly sensitive to changes in Poisson's ratio), and graphed as displacement versus applied pressure (Figure 3). The Goodman Jack system was provided by Slope Indicator Co. and operated by CRUX personnel. Data was recorded, reduced and interpreted by AMEC.

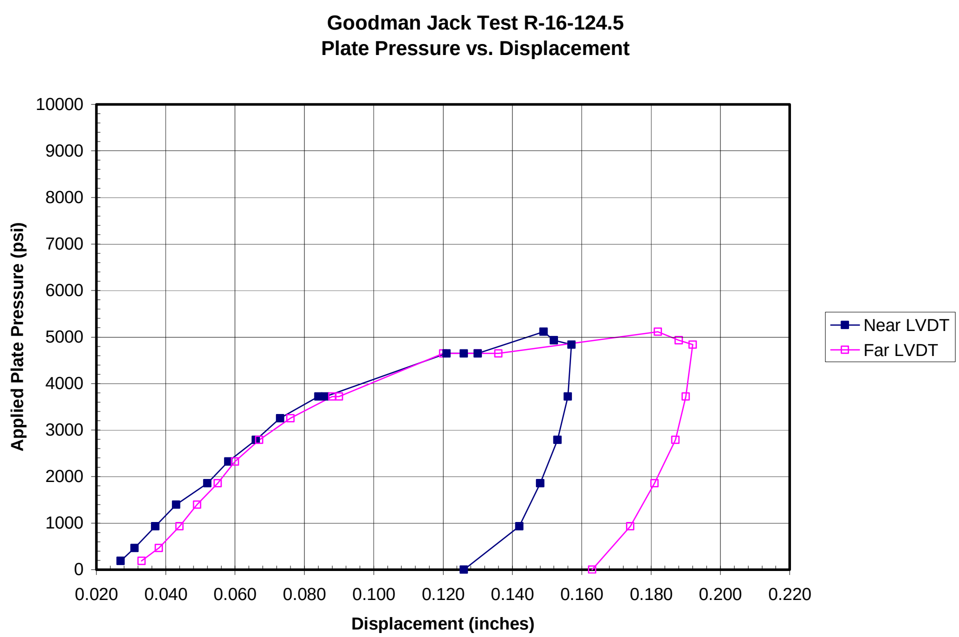


Figure 3 – Goodman Jack Test Results

In Situ Down-hole Seismic Surveys

In situ down-hole seismic surveys were completed in all borings for the preliminary assessment and in 18 borings for the final investigation, using a Geometrics S-12 Smartseis signal enhancement seismograph, fabricated down-hole probe designed to detect compression waves (p-waves), and sledgehammer energy source. In the course of the preliminary assessment, a signal from the sledgehammer energy source at a velocity of about 5,000 feet per second (ft/s) was identified in some of the data for one boring as having traveled along the PVC pipe string used to deploy the probe in the inclined holes. Bedrock seismic velocities in other borings were sufficiently high that useful signals arrived well before the PVC pipe signal. This potential interference was eliminated in the final investigation by suspending the probe using the cable and air tube. For the preliminary assessment, the probe consisted of two vertically-oriented geophones mounted five feet apart on a PVC pipe body, and fitted with pneumatic bladders to press each phone against the borehole wall. It was determined that the PVC pipe string was not necessary to deploy a probe down inclined holes, and the probe system was improved and simplified for the final investigation. The final probe system consisted of a single geophone/pneumatic bladder arrangement suspended at the top of the boring by the cable and air tube.

Down-hole readings were taken at 5-foot intervals throughout the depth or length (inclined borings) of each boring. Data trends were checked as needed by obtaining arrival times for other reliably interpretable portions of the seismic signal. Interpreted seismic velocities were over intervals of at least 5 feet, and more typically over intervals of 10 feet or more, and presented as plots of interpreted seismic velocity vs. depth, augmented with results of the Goodman Jack tests and laboratory determinations. Photograph 5 shows a test location on the canyon slopes. Each of the AMEC personnel was secured to the wall with ropes during the test. Figure 4 presents typical down-hole seismic survey results.

Photograph 5 – Down-hole seismic test in progress on Arizona slopes at skewback.

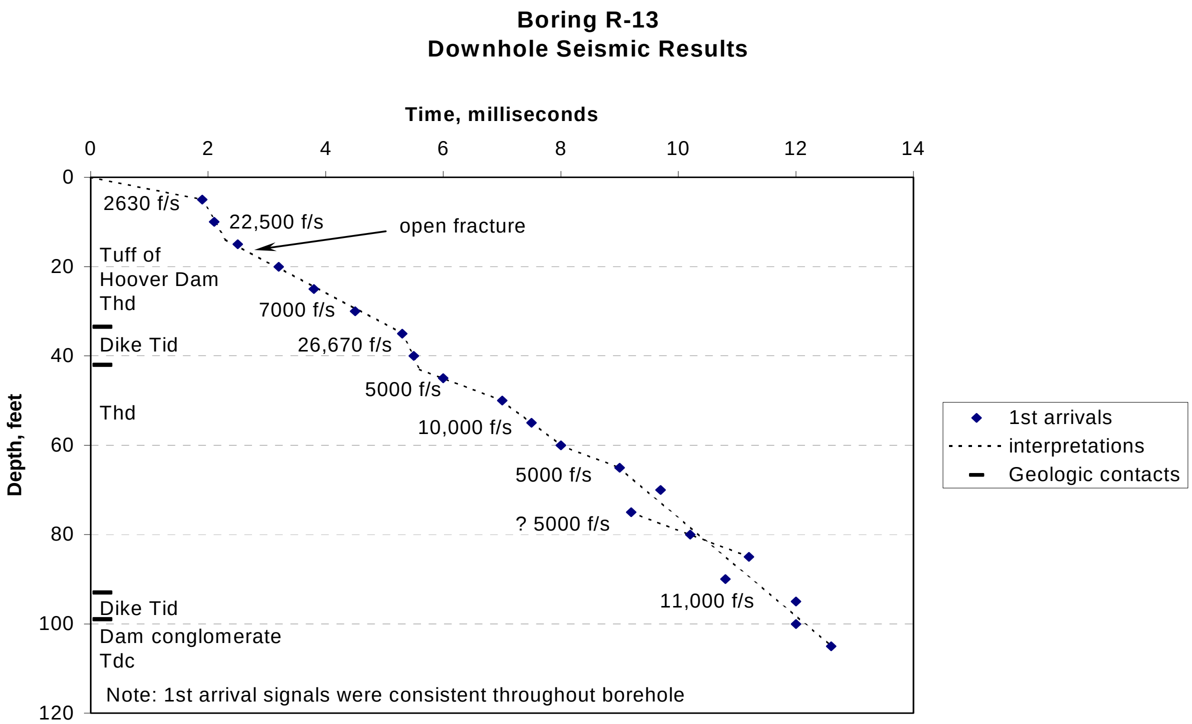
Laboratory Testing

Laboratory testing of representative samples of rock core recovered from test borings was performed to characterize the physical properties of the rock, including unconfined compression (213 tests); bulk density of rock core (water displacement method, 87 tests); bulk dry density of rock core (226 tests); splitting tensile strength (17 tests); dynamic (ultrasonic pulse velocity method) and static Young’s modulus and Poisson’s ratio (18 tests); static uniaxial compression with axial strain measurements (18 tests); and creep in unconfined compression (7 tests). The creep in unconfined compression (UC) tests were completed to evaluate the potential for local crushing of the rock in the softer layers of tuff identified at the Arizona skewback at anticipated applied load levels.

REGIONAL GEOLOGY

The Hoover Dam Bypass corridor of US 93 is located near the eastern margin of the Basin and Range Physiographic Province, approximately 45 miles west of the boundary with the Colorado Plateau Province. The physiography of the Basin and Range Province typically is characterized by a series of parallel and elongated mountain blocks that represent structural highlands separated by intervening down-dropped basins or valleys. The bypass project is located in a structural basin bounded by the Eldorado Mountains to the west and the Black Mountains to the east. However, the basin has been filled with Tertiary volcanic, volcaniclastic and sedimentary deposits, is further distorted from intrusion of Tertiary plutonic rocks, and thus typical valley topography is not preserved in the project area. As a result, the entire area is dominated by

Figure 4



bedrock conditions characteristic of rugged mountainous terrain. The bedrock units exposed in the area transition from Precambrian igneous and metamorphic rocks dominating the core of the two adjacent mountain ranges, to a dominance of Tertiary plutonic, volcanic and sedimentary rocks in the immediate area of Hoover Dam.

The volcanic rocks in the region immediately surrounding Lake Mead and at the project site were derived from several large volcanic centers that were active during mid-Tertiary time (12 to 25 million years ago {Ma}),(Longwell, 1936, 1963; Anderson, 1971; Anderson and others, 1972; Smith, 1982). Eruptions from these volcanic centers produced thick deposits of dacite, andesite, basalt, basaltic andesite, and dacite to rhyolite ash-flow tuff (Anderson, 1971; Bohannon, 1984). A compositional shift from rhyolitic and dacitic magmas to more basaltic volcanism occurred from about 10 to 15 Ma (Mills, 1985).

Extension and ductile thinning of the crust was initiated in late Oligocene to early Miocene time and was characterized by low-angle and listric-normal faulting (Mills, 1985). The volcanic activity was contemporaneous with and related to this crustal deformation. Approximately 10 Ma the regional strain rate slowed, resulting in the brittle deformation of the upper crust into large fault blocks bounded by low-angle, listric-normal, and high-angle normal faults (Anderson, 1971; Otton, 1982; Suneson and Lucchitta, 1983).

SITE GEOLOGY

The Colorado River Bridge will span Black Canyon about ¼ mile downstream of Hoover Dam. Black Canyon is a steep-walled bedrock gorge formed by erosion and progressive downcutting by the Colorado River. The topography of Black Canyon at the bridge site is rugged and characterized by steep relief, with near-vertical canyon walls with occasional small benches and local overhangs dominating the mid- to lower portion of the canyon, progressing upward to slightly less steep slopes characterized by an irregular steeply sloping surface with numerous but less continuous near-vertical cliffs.

The relevant bedrock units exposed in the canyon walls at the bridge site consist of a sequence of Tertiary volcanic and sedimentary rocks as part of the basin fill sequence. Unconsolidated deposits of Quaternary colluvium and recent man-made fill only locally overlie the bedrock within select locations of the canyon. The bedrock sequence includes the Switchyard basalt (Tsb), which outcrops in the uppermost cliffs, locally underlain by breccia (Tbr), which is more of a slope former directly below the upper cliffs, followed by massive deposits of tuff of Hoover Dam (Thd), which include alternating cliffs and steep benches in the mid-canyon slopes, and finally underlain by Dam conglomerate (Tdc) in the lower canyon walls. Intrusive basalt dikes and sills (Tid) irregularly dissect the Dam conglomerate, tuff of Hoover Dam and the breccia at various levels within the canyon walls and may have been feeder dikes for the overlying Switchyard basalt. This sequence is further underlain by an older tilted sequence of volcanic and volcaniclastic rocks that outcrop at the base of the canyon and downstream of the bridge alignment, but were not significant to the River Bridge foundations.

A brief description of the geologic units exposed in the canyon slopes at the bridge site is presented below.

Switchyard Basalt
Switchyard basalt is exposed only in the upper portion of the canyon slopes, at both the Arizona and Nevada abutments, and also at portions of the upper Nevada pier (Pier 1). The deposit consists of massive flows and flow breccias apparently fed by dikes and sills which intruded the Dam conglomerate and tuff of Hoover Dam. The basal portion of the Switchyard basalt is often characterized by a 5- to 30-foot thick flow breccia with a distinctive greenish (Arizona side) to purplish (Nevada side) hue. In general, the basalt is hard with some moderately hard zones, slightly weathered with some moderately to highly weathered zones, and light gray to brownish-gray with some greenish-gray to olive gray zones. The flow breccia is typically moderately hard to hard, moderately weathered, with occasional zones that are slightly or highly weathered, and greenish-gray to olive gray in color. Fractures within this unitvary from closely to moderately closely spaced, with some very closely spaced zones in the flows and more widely spaced in the flow breccias. Slickenside surfaces are present on some fracture surfaces.

Breccia
The breccia occurs locally between the Switchyard basalt and the tuff of Hoover Dam, and appears to occur as filled erosional channels cut within the top of the tuff. The breccia was encountered in borings completed at the two uppermost piers (Pier 1 on Nevada side and Pier 15 on the Arizona side). On the Arizona side, the breccia unit consists of a coarse-grained clast-supported conglomerate comprised of cobble- to boulder-sized particles ranging up to 10 feet in diameter. This deposit locally filled an erosional trough in the tuff of Hoover Dam, which occurred in a zone of weakness along the Sugarloaf fault. On the Nevada side, the breccia unit is

comprised of a finer-grained deposit of bedded siltstone and fine-grained tuffaceous sandstone that is within a fault block, such that the unit is tilted and locally in fault contact with Switchyard basalt and tuff of Hoover Dam.

The unit is slightly weathered with some moderately weathered zones, medium strong to very strong, and reddish-brown to pale red. Locally the unit appears to consist of re-worked ash-flow tuff. Fractures are rare and very widely spaced in the coarse-grained deposits and vary from very-closely to moderately-closely spaced in the finer-grained materials.

Tuff of Hoover Dam

The tuff of Hoover Dam underlies the Switchyard basalt and breccia unit and is the dominant rock type present at most of the planned pier and skewback foundations sites. It is extensively exposed in the mid- to lower-canyon walls and makes up a large portion of the near-vertical canyon walls in the skewback areas.

The basal contact of the tuff is conformable with the Dam conglomerate, and the contact is sharp to locally gradational in nature. The unit consists of a series of volcanic ash flows welded by their own heat. The unit generally is moderately welded with some poorly welded zones. The rock generally is fine to medium grained with locally abundant lithic fragments and varying amounts of phenocrysts that consist primarily of plagioclase and biotite and give the rock a porphyritic appearance. The unit locally contains vesicles, and is particularly vesicular at the planned Arizona-side skewback site. The rock has a massive to blocky fabric and generally is a cliff-former. The unit is locally a slope-former where the unit is poorly welded, such as the broad bench above the planned Arizona-side skewback site. The unit is locally weakly to moderately laminated. The tuff is slightly to moderately weathered, varies from moderately strong with some strong zones, and is grayish-orange pink.

Fractures within the tuff generally are moderately close to widely spaced, with some closely-spaced fractures and rare very closely-spaced fractures. A few fractures have slickenside surfaces.

Dam Conglomerate

The Dam conglomerate is exposed in the lower portion of the Black Canyon below the proposed skewback foundations. The Dam conglomerate consists of moderately- to well-stratified beds of medium- to coarse-grained sandstone and conglomerate. The conglomerate typically is matrix supported, but locally clast supported. The conglomerate generally consists of pebble- to cobble-sized clasts in a variable sandy, silty and clayey matrix, and the clasts are locally imbricated. In some areas the clasts are predominantly angular to subangular and in other areas the clasts are subrounded to rounded. The unit appears to have been deposited in an environment dominated by braided streams.

The conglomerate is slightly weathered, well indurated, and strong to very strong with a grayish-red matrix and light to medium gray clasts. Fractures within the conglomerate are widely to very widely spaced.

Dikes and Sills
Dikes and sills of basaltic andesite cut through the Dam conglomerate and the tuff of Hoover Dam, and are extensively exposed in portions of the walls of the Black Canyon. A single 1- to 4-foot thick dike occurs below the Arizona skewback site, whereas the Nevada skewback site is cut by several massive dikes and sills. The dikes are composed of porphyritic basaltic andesite with about 10 to 30 percent phenocrysts in an aphanitic groundmass. The phenocrysts generally are less than 1/16 inch in diameter and mostly consist of plagioclase, with occasional augite or olivine (altered to iddingsite) phenocrysts. In general, the dikes are slightly weathered to unweathered, strong to very strong, and grayish-red purple to brownish-gray. The rock commonly contains a trace to some vesicles up to 1/2 inch in diameter, locally lined or filled with calcite. Calcite stringers are locally common.

Fractures within the dike are moderately close to widely spaced, with some closely-spaced fractures and rare very closely-spaced fractures.

Unconsolidated Deposits
Unconsolidated deposits only locally occur in the canyon slopes and include colluvium and isolated man-made fill. The most extensive deposit of colluvium overlies the bedrock units at Abutment 1 and Pier 1 in the upper canyon slopes on the Nevada side of the canyon. This deposit is the result of an accumulation of debris at the base of the near-vertical cliffs of Switchyard basalt. The colluvium is comprised of silty gravel, cobbles and boulders with some large blocks derived from the Switchyard basalt ranging up to tens of feet in dimension and one block about 50 feet in length. The colluvium appears to have a maximum thickness of about 20 feet, excluding the large block at the surface at Pier 1.

Man-made fill is present at Pier 4 below the hairpin turn of US 93 and is comprised of silty gravel and cobbles with some clay and boulders. The fill appears to have a maximum thickness of about 12 feet.

RESULTS AND ANALYSIS

Conditions at the Nevada Skewback (Pier 5)
The foundation area for the Nevada skewback (Pier 5) is located behind the existing canyon wall below the rock knob at the hairpin turn of US 93. The canyon wall in the immediate area of the skewback foundation site consists of a near-vertical cliff. The elevation at the top of the rock knob is about 1320 feet; therefore, a cut of about 120 feet below the existing ground surface (bgs) will be required on the northeast side (left) of the foundation footprint adjacent to the hairpin turn to excavate to the skewback foundation level.

Borings completed at the Nevada skewback encountered volcanic rocks, including the tuff of Hoover Dam and basaltic andesite dikes which intrude the tuff. Three borings penetrated the Dam conglomerate beneath the volcanic rocks. Due to the mode of emplacement, the geometry of the dikes generally is irregular and unpredictable. A basaltic andesite dike forms the top of the rock knob at the hairpin turn and also drapes over the knob, forming a veneer along the existing canyon wall, particularly left of centerline. This capping dike, excluding the veneer along the canyon wall, is underlain by tuff, which is further dissected by additional, contorted, very irregularly-shaped dikes. The tuff ultimately is underlain by conglomerate. The distribution and irregularity of the geologic units along the canyon walls is shown on Photographs 6 and 7.

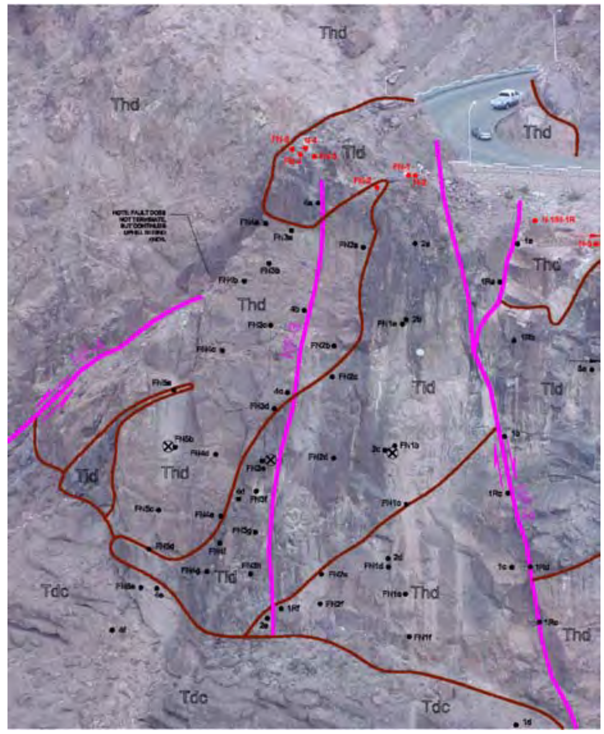
The Nevada skewback foundation will be supported on both tuff and dike rock, with the dike anticipated to be exposed primarily in the northern one-third of the floor and also in a smaller area in the southwestern (back) corner of the floor. The tuff will be exposed in the remainder of the floor, covering about two-thirds of the foundation area. The dike will form a majority of the northeast (left) wall, including an upper cap and the front (canyon side) portion of the wall. The tuff will be exposed below the capping dike along the back half of the left wall, including the corner section of the wall. The back (northwest) wall will consist of the upper cap of dike underlain by a thick section of tuff and a thin dike near the base of the wall. Figure 5 is a schematic of anticipated geologic conditions in the planned excavation.

Numerous faults cut through the tuff and dikes in the area of the knob as shown on Photographs 6 and 7. Most of the more significant faults miss the planned excavation. One fault, oriented roughly northeast-southwest but curved in an upward direction, will cross through the upper portion of the back wall and also may extend to the upper section of the left wall near the corner.

Goodman Jack tests in three skewback borings at depths corresponding to planned foundation level resulted in estimated low-strain modulus values from about $1.7x10^5$ to $1.0x10^6$ psi for applied plate pressures less than 1,000 psi, and from $4.3x10^5$ to $2.1x10^6$ psi for applied plate pressures of 1,000 to 10,000 psi. A trend in the difference between values for perpendicular- versus parallel-oriented tests was not apparent, and no difference in the magnitude of the values was noted relative to depth of the test interval. Down-hole seismic surveys in four borings indicated interpreted p-wave velocity within the skewback influence zone from about 3,300 ft/s to 21,000 ft/s in the tuff of Hoover Dam and the dike rock zones, respectively.

The bulk density of the tuff varied from about 121 to 152 pcf with most values in the range of 125 to 130 pcf, and UCS varied from about 1,400 to 9,000 psi with most values from 3,000 to 5,300 psi. Splitting tensile strengths of two tuff samples were 204 and 772 psi. Dynamic measurements of Young's modulus and Poisson's ratio (four tests) were about $3.3x10^6$ to $3.9x10^6$ psi and 0.17 to 0.26, respectively, and static measurements ranged from $2.4x10^6$ to $3.8x10^6$ psi and 0.19 to 0.34, respectively.

Bulk density of the dike rock varied from about 134 to 160 pcf and UCS varied from about 4,000 to almost 19,000 psi, with most values from 5,000 to 10,000 psi. The splitting tensile strength of four samples of dike rock varied from about 1,700 to 2,300 psi. Dynamic measurements of



Photograph 6 – Geology of canyon wall – Nevada skewback area (upstream view).

Photograph 7 – Geology of canyon wall – Nevada skewback area (cross canyon view).

Tid – Basalt Dike
Thd – Tuff of Hoover Dam
Tdc – Dam Conglomerate
Red Lines – contacts
Pink Lines – faults
Numbered Dots – rappel sites

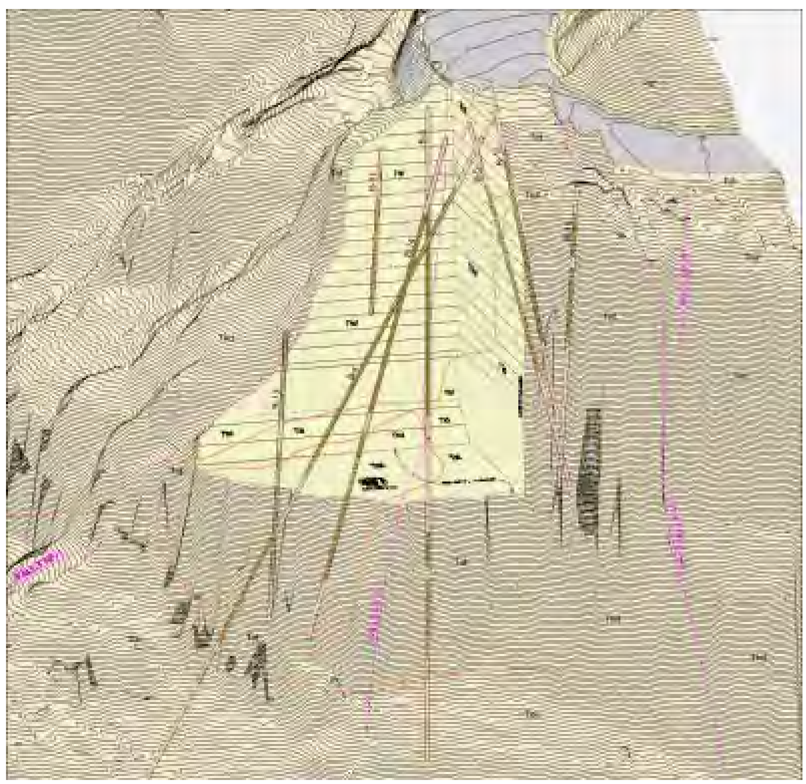


Figure 5 – Oblique view of planned Nevada skewback excavation showing geologic conditions anticipated with borehole data. Units same as in Photograph 6.

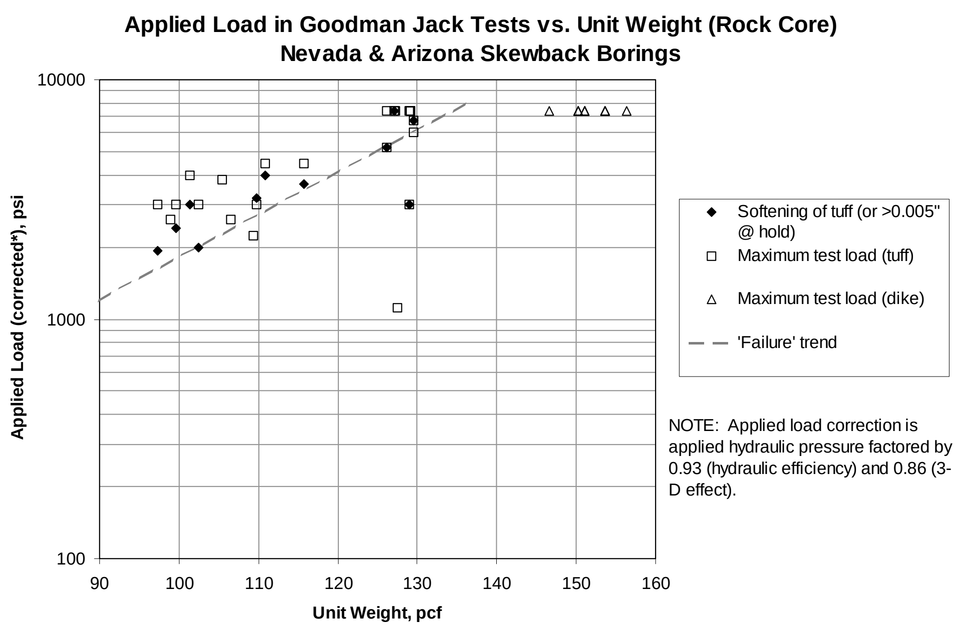
Young's modulus and Poisson's ratio (four tests) were $6.3x10^6$ to $7.6x10^6$ psi and 0.25 to 0.26, respectively, and static measurements were $6.3x10^6$ to $7.2x10^6$ psi and 0.28 to 0.30, respectively. The bulk density of the Dam conglomerate varied from 148 to 159 pcf, and the UCS of two samples of conglomerate were about 2,900 to 8,300 psi. Results of creep in unconfined compression tests indicate maximum, non-sustained creep rates of less than about 0.0001 inch/day at applied pressures of 250 and 300 psi.

Results of in situ Goodman Jack tests plotted against the bulk density of core samples from the in situ test intervals for both tuff and dike are presented on Figure 6. The Goodman Jack test results are plotted either as the maximum applied test load or the applied load at which more than 0.005 inch of plate movement occurred without load increase. Values of the rock mass modulus for the tuff of Hoover Dam, which forms the primary bearing unit, estimated from Goodman Jack tests are as follows:

> Applied pressures of 200 to 1,000 psi (non-linear segment of load-deformation curve) - average of $4.7x10^5$ psi;
>
> Applied pressures of 1,000 to 10,000 psi (linear portion of load-deformation curve) - average of $9.7x10^5$ psi.

The value of modulus at applied pressures of 1,000 to 10,000 psi, within the linear segment of the load-deformation curves for the tests, is thought to represent a reasonable modulus for the tuff. This value is essentially identical to the GSI-derived rock mass modulus for the tuff. The modulus at applied pressures of 200 to 1,000 psi, within the non-linear segment of the curves, is thought to represent a lower-bound modulus likely affected by borehole wall conditions. By comparison, static modulus values for the four core samples ranged from $2.4x10^6$ psi to $3.8x10^6$ psi, about an order of magnitude or so greater than the values from Goodman Jack tests and based on GSI.

Figure 6



The modulus of the rock mass underlying the skewback bearing level also was estimated using GSI criteria. The two material types, tuff of Hoover Dam and dike rock, have different GSI values, with the tuff being the weaker material. A GSI value of 55 and average intact sample UCS of 3,400 psi for the tuff results in an estimated modulus of 9.3×10^5 psi (after Hoek & Brown, 1997). The basis of the GSI are absence of groundwater, average values of UCS (3,400 psi), average RQD (87.5), discontinuity spacing (0.6 m or 2 feet), discontinuity conditions of length greater than 20 meters (66 feet) and width of 0.1 to 1 mm (0.004 to 0.04 inch), some slickenside fractures, hard filling and slight weathering.

Tuff of Hoover Dam and dike rock at the Nevada skewback site had measured, representative down-hole seismic compression wave velocities of about 10,000 ft/s and 15,000 ft/s. Using a Poisson's ratio of 0.25 and respective, representative densities of 120 pcf and 130 pcf results in estimated low-strain modulus values of about 2.2×10^6 and 5.3×10^6 psi, respectively for the tuff and dike rock.

Photograph 8 – Geology of canyon wall – Arizona skewback area (cross canyon view.
Tid – Basalt Dike
Thd – Tuff of Hoover Dam
Tdc – Dam Conglomerate
Red Lines – contacts
Pink Lines – faults
Numbered Dots – rappel sites

Conditions at the Arizona Skewback (Pier 14)

The planned foundation area for the Arizona skewback (Pier 14) is located behind the existing canyon wall, below a broad bench. The canyon wall geometry at the skewback foundation consists of a near-vertical cliff. The general elevation of the broad bench above the skewback foundation excavation is about 1340 feet; and a cut depth of about 140 feet bgs will be required to reach the skewback foundation.

The tuff of Hoover Dam is the dominant rock unit exposed in the canyon wall and encountered in borings at the Arizona skewback site. A thin dike of basaltic andesite also was encountered in three borings; this dike appears to be a continuation of a dike exposed in the canyon wall, and thins to the southeast. A more massive dike, which intruded the tuff in a near-vertical orientation, is exposed lower in the canyon wall below the planned foundation level. Photograph 8 shows the distribution of the geologic units in the canyon wall in the skewback area.

Several characteristics of the tuff encountered during the drilling program were markedly different than the characteristics exposed in the canyon wall. During the canyon wall mapping, the tuff was described as strong to very strong and slightly weathered with rare moderately weathered zones. By comparison, rock core extracted during the drilling program was moderately strong with some strong zones and slightly to moderately weathered. It is suspected that the large vertical dike that is exposed below the foundation level may have at one time extended upward to the broad bench at the top of the skewback site. The canyon wall below the bench may have been baked and hardened by the vertical dike, but the hardening probably did not extend more than several feet beyond the contact.

The planned walls and floor of the excavation will consist of tuff with a small area of dike rock in the back right corner as shown on Figure 7. The dike will likely be exposed in both walls and the floor at this location with tuff in the very corner of the excavation. The Sugarloaf Fault will not be encountered in the floor of the skewback excavation, but may be exposed near the intersection of the upper portion of the back wall and left wall of the excavation. The fault dips steeply and is oriented northwest-southeast. Fault gouge/breccia typically occurs along the fault and will be exposed in the walls. The fault gouge/breccia generally is thin (up to 2 to 4 feet wide), moderately soft and comprised of silty sand- and gravel-sized fragments.

Figure 7 – Oblique view of planned Arizona skewback excavation with geology and borehole data.

Tid – Basalt Dike	Red Lines – contacts
Thd – Tuff of Hoover Dam	Pink Lines – faults
Tdc – Dam Conglomerate	Numbered Dots – rappel sites

Goodman Jack tests were performed at selected depths in three skewback borings at planned foundation level. Estimated low-strain modulus values ranged from about 1.5×10^5 to 1.1×10^6 psi for applied plate pressures less than

1,000 psi, and from $1.6x10^5$ to $1.6x10^6$ psi for applied plate pressures of 1,000 to 6,000 psi. A trend in the difference between values of modulus for perpendicular- versus parallel-oriented tests was not apparent, and no difference in the magnitude of the values was noted relative to depth of the test interval. Down-hole seismic surveys in three borings resulted in interpreted p-wave velocity within the skewback influence zone from about 4,800 to 6,000 ft/s for the tuff, whereas the p-wave velocity of the thin dike was about 10,000 ft/s. Several zones within the tuff had compression wave velocities of 10,000 to 15,000 ft/s. Bulk density of the tuff samples varied from about 93 to 140 pcf with most values between 95 and 110 pcf. The UCS of the tuff samples varied from about 500 to 9,800 psi with most values between 1,000 and 2,500 psi. The splitting tensile strength of five tuff samples varied from 204 to 510 psi. Dynamic measurements of Young's modulus and Poisson's ratio on seven samples of tuff were about $1.2x10^6$ to $3.4x10^6$ psi and 0.08 to 0.33, respectively, and static measurements were $7.8x10^5$ to $4.2x10^6$ psi and 0.10 to 0.33, respectively. Bulk densities of two samples of dike rock were about 130 and 140 pcf, the UCS values were 3,200 and 5,000 psi, and a splitting tensile strength of 712 psi was determined from a single test. Results of creep in unconfined compression tests indicated maximum, non-sustained creep rates of less than about 0.0001 inch/day at the applied pressures of 250 to 350 psi.

Values of the rock mass modulus from Goodman Jack tests were as follows:

> Applied pressures of 200 to 1,000 psi (non-linear segment of load-deformation curve) - average of $2.8x10^5$ psi, standard deviation of $7.6x10^4$ psi;
>
> Applied pressures of 1,000 to 6,000 psi (linear portion of load-deformation curve) - average of $4.1x10^5$ psi, standard deviation of $3.0x10^5$ psi.

The value at applied pressures of 200 to 1,000 psi, within the non-linear segment of the tests, is thought to represent a lower-bound modulus likely affected by borehole wall condition. The value at applied pressures of 1,000 to 6,000 psi, within the linear segment of the tests, is thought to represent a reasonable modulus for the tested rock. This value is in the range of the GSI-derived rock mass modulus for the tuff. Static modulus values for seven core samples averaged $2.0x10^6$ psi with a standard deviation of $1.4x10^6$ psi.

The modulus of the rock mass, consisting primarily of tuff of Hoover Dam, underlying the proposed foundation to a depth of about 30 feet below bearing elevation, also was estimated using GSI criteria. A GSI value of 52 and average intact sample UCS of 1,400 psi for the tuff results in an estimated modulus of $5.1x10^5$ psi (after Hoek & Brown, 1997). The basis of the GSI are absence of groundwater, average values of UCS (1,400 psi), average RQD (90), discontinuity spacing (0.3 m or 1 foot), discontinuity conditions of length greater than 20 meters (66 feet) and width of 0.1 to 1 mm (0.004 to 0.04 inch), no filling and moderate weathering. It is possible that some fracture conditions might be better represented by a width of 1 to 5 mm with soft filling. In that case, the GSI value is 45 and the resulting estimated modulus is $3.4x10^5$ psi.

The tuff of Hoover Dam at the Arizona skewback site has a measured, representative down-hole seismic compression wave velocity of about 5,000 ft/s and a lower-bound velocity of about 2,500 ft/s in isolated zones. Given a Poisson's ratio of 0.25 and typical and lower bound densities of 105 and 95 pcf, resulting estimated modulus values (typical and lower-bound) are about 4.8×10^5 and 1.1×10^5 psi, respectively.

Elastic Modulus Determinations

The rock mass within a depth of about 30 feet below the proposed bearing elevation at each prospective skewback location was considered in determination of elastic modulus, as this depth (approximately equivalent to the skewback footing width) was considered to represent the zone of major influence for arch rib and column loads. A relevant large-scale rock mass deformation modulus for bridge design within this zone cannot be directly measured; therefore, in situ and laboratory measurements and empirical methods were used to establish a range in estimated rock mass deformation modulus. Modulus values were obtained at various scales and ranges of strain and then compared. Four basic means of estimating modulus from field and laboratory data were utilized, including:

- Laboratory high-strain deformation (UC tests) and low-strain dynamic modulus of intact samples
- In situ borehole deformation modulus (Goodman Jack) measurements,
- Empirical rock mass deformation modulus from rock mass and strength rating systems (RMR & GSI), and
- In situ borehole low-strain dynamic modulus (down-hole seismic velocity) measurements - compression wave velocity was used to estimate elastic modulus in order to check values determined with the other methods.

Laboratory test results and in situ down-hole measurements were utilized as checks on upper-bound values of rock mass modulus. Goodman Jack measurements at different pressure ranges provided lower bound to reasonable rock mass modulus values. Empirical rock mass modulus from rating systems provided a cross-check on the derived modulus values, as compared to the general body of rock mass modulus data. Low-strain modulus derived from the down-hole seismic measurements was utilized as a reasonable upper-bound check on the rock mass modulus, and as a check on the reduction in modulus between laboratory- and field-scale measurements.

Laboratory testing of intact core samples resulted in modulus values significantly greater than would be expected to be representative of rock masses affected by discontinuities and weathering, for both large-strain unconfined compression testing to failure and small-strain dynamic modulus testing by ultrasonic means. The unconfined compression test was used to obtain tangent modulus and Poisson's ratio at 50% of maximum strength. Lower modulus values would be calculated at lower-strain portions of the loading curves for most of the tested samples. It is noted that Heuze (1980) found that, in most cases, field-scale modulus values were within a range of 20% to 60% of the laboratory modulus values, although a much wider overall range was reported. Overall, laboratory modulus values averaged about 2½ times field-scale modulus values. Laboratory modulus values served as an upper-bound check on field modulus values.

Borehole deformation testing using the Goodman Jack provided a relatively high-strain in situ estimate of modulus through testing of a rock mass volume of about five cubic feet. The test has been found to be very sensitive to deviations from a borehole diameter of 3.00 inches, and initial Goodman Jack deformation measurements at low pressures likely were affected by the as-drilled borehole diameter. Minimum jack hydraulic pressures to assure full contact between the borehole wall and the jack plates can be estimated using procedures described by Heuze (1984). Borehole diameter deviations of about 0.01 to 0.04 inches from the theoretical 3.00 inches were suggested by the deformation data. Resulting hydraulic pressures up to several thousand psi theoretically could be needed to ensure establishment of full contact. Alternatively, Heuze (1984) suggests (but does not necessarily recommend) that full contact can be assumed if the load-deflection curve is linear. Higher load portions of the load-deflection curves are substantially linear even though the borings may possess potentially significant diameter variations from the theoretical.

Load-deflection curves and estimated static modulus at different applied loads were developed from the Goodman Jack tests. Modulus values at low loads comparable to anticipated bridge footing bearing pressures, and within the linear portion of the load-deflection curves, were utilized to develop understanding of the test and to establish both lower-bound and reasonable values of deformation modulus. It is expected that a lower-bound deformation modulus is influenced by borehole effects that reduce the apparent modulus at lower hydraulic pressure. Deformation modulus derived from the linear portion of the load-deformation curve is thought to reflect a reasonable static modulus value corresponding to a rock mass criterion of 50% of the maximum strength. This deformation modulus correlated well with the GSI estimate for rock mass modulus.

Low-strain Young's (dynamic) modulus was estimated from the down-hole compression wave velocities using equations presented by Viskne (1976), which have been found to be valid at the rock mass scale. Poisson's ratio is a function of the ratio of the compression and shear wave velocities, and was determined at the laboratory scale (intact samples) as described above. Field-scale dynamic modulus values obtained from measured seismic velocities on the rock mass scale, including effect of discontinuities, were lower than the values obtained from laboratory testing.

Skewback Excavation Slope Design

The discontinuity data collected from OPTV logs of borings and surface measurements were plotted on a stereonet to determine optimal slope angles for excavation walls and need for stabilization treatments at the Nevada and Arizona skewbacks. For the Nevada skewback excavation, the analysis indicated a dominant relatively steeply-oriented joint system and that untreated slopes could stand at angles of ½H:1V (horizontal:vertical) to ¾H:1V. However, since the left wall is located directly below the existing hairpin turn of US 93, a vertical slope is required to avoid interference with the road. Thus, the left wall will require application of patterned rock anchors on a relatively close pattern to prevent development of wedge failures at ¾H:1V in the vertical slope.

For the Arizona skewback excavation, the analysis indicated a dominance of a steeply-inclined joint system such that untreated slopes could stand at angles of about ½H:1V. However, since the back wall is located below Pier 15, it is required that the slope be steepened to ¼H:1V to avoid interference with the pier. Thus, the back wall will require application of patterned rock anchors on a relatively close pattern to prevent development of wedge failures in the ¼H:1V slope.

Earthquake Ground Motion for Bridge Design

A detailed seismic evaluation was completed, including development of seismic acceleration response spectra and synthetic seismograms, to support structural analysis and design of the River Bridge. For the evaluation, catalogs of historical earthquakes were examined for patterns of epicenters in proximity to active faults, and seventeen faults within 100 miles of the site were considered to be active based on an appropriate definition. Aerial and ground-based geologic reconnaissance within 100 miles of the River Bridge site was performed to supplement available seismotectonic data. Maximum earthquake magnitudes were determined for each active fault, and peak horizontal accelerations were estimated using three ground-motion attenuation relationships.

The results of a probabilistic seismic hazard assessment published by the US Geological Survey (USGS) in 1996 were used, in part, to select earthquake magnitudes and distances, and target ground motions for use in design of the River Bridge. The HST, in consultation with the Structures Management Group, selected ground motion corresponding to an exceedance probability of 10 percent in 100 years (recurrence interval of 950 years) as appropriate for design of the River Bridge. The USGS-published probabilistic ground motion for an exceedance probability of 5 percent in 50 years (recurrence interval of 975 years) was utilized, and a comparison was made with current procedures used for design of new buildings. This design criterion was specific to the River Bridge, as other project bridge structures were designed to a less stringent ground motion.

The River Bridge was designed on the basis of a nonlinear dynamic analysis using three-component seismograms at each abutment. A 1-Hz spectral acceleration of 0.139g was selected as the target ground motion on which to anchor design earthquakes and response spectra. The target 1-Hz spectral acceleration would be produced by a moment magnitude earthquake of 6.2 at a hypocentral distance of 16 km, or by a moment magnitude earthquake of 7.0 at a hypocentral distance of 36 km. The magnitude 6.2 earthquake would likely occur on the Mead Slope Fault, whereas the magnitude 7.0 earthquake would occur on the California Wash Fault.

Acceleration response spectra for both design earthquakes were calculated using an appropriate attenuation relationship. The smaller magnitude earthquake produced the maximum high-frequency motion, whereas the larger magnitude earthquake produced the maximum low-frequency motion. The recommended design response spectrum for the River Bridge was the maximum of the two motions. The shape of the recommended design response spectrum closely matched the required spectrum for design of new buildings.

A Composite Source Model was used to produce synthetic, three-component seismograms at each abutment for nonlinear dynamic analysis of the river bridge. Input parameters to the Composite Source Model included specific geographic fault location, parameters pertaining to the physics of fault rupture (length, width, average displacement, rake and rupture velocity), and seismological parameters of the source (seismic moment and stress drop) and site area (Green's functions). The acceleration time history records generated with the Composite Source Model were adjusted to bring their acceleration response spectra into close agreement with the design response spectrum, and the results compared to the design response spectrum to ensure compatibility.

Details and discussions of the seismic evaluation are presented in a companion paper (Keaton, 2003).

BRIDGE DESIGN OPTIONS

The key element of the Hoover Dam Bypass is the crossing of Black Canyon at Hoover Dam, a setting that requires a bridge that aesthetically blends into the form of the canyon. The new bridge must both be worthy of the natural beauty and drama of the rugged rock gorge setting and satisfy the functional needs for the transportation corridor. In addition to selecting a bridge that meets these aesthetic requirements, cost and technical suitability were the driving factors in determining the type selection for the crossing. A host of structure types – including deck arch, truss, segmental box girder, cable-stayed and suspension – were examined in the initial stages of the bridge evaluation program. Ultimately, the candidate bridge types were reduced to the deck arch, deemed the most feasible for the steep-walled canyon setting and overall cost of the structure. The deck arch alternative also meets the commitment of the Environmental Impact Statement to "design the bridge profile to be below the horizon line of Black Canyon as seen from Lake Mead, if feasible." Deck arch designs were initially developed for both a shorter (1,090-foot) and longer (1,325-foot) arch span; the preliminary geotechnical assessment and mapping studies undertaken during the study confirmed either span option as technically viable, reducing the decision between the two to a matter of economics and aesthetics. Six deck arch alternatives – three for each span length – were developed through preliminary engineering and cost estimating, in order to recommend a bridge type for final design.

Short-Span Bridge Alternatives

The short-span alternatives extend from the rock knoll at the Nevada-side switchback of US 93 (the "hairpin turn") to the canyon wall face on the Arizona side of the canyon. The elevation of the arch springing (skewback) is established by the rock knoll, being low enough to both provide adequate width for the springing foundation and to keep the thrust line of the arch within the rock mass behind the knoll. The rise of the arch varied slightly with the alternatives but is generally in the range of 285 to 290 feet, for a span-to-rise ratio of 3.8. Due to its proximity, the resulting excavation for the Nevada-side skewback would require rock anchors to preserve the existing roadway at the hairpin turn. Short-span alternatives considered included steel, concrete, and concrete-steel composite types.

The steel deck arch bridge alternative represents a traditional engineering solution for this type of canyon crossing. The alternative using a solid web rib was preferable over a trussed-rib alternative because of its superior aesthetics (Figure 8). The main span has a twin steel box arch rib configuration, with steel box spandrel columns and Vierendeel bracing, and a rib depth from 11 feet at the crown to 17 feet at the springings (skewbacks). The main span arch is inclined to match the plane of the bent-leg spandrel columns, resulting in a splayed rib configuration. The approach span columns are also a bent-leg, Vierendeel-braced configuration. The deck system consists of steel box girders spanning approximately 120 feet on the main span and up to 180 feet on the approach spans, with a composite concrete deck. The girders are integrally connected with the steel box pier caps at the spandrel and approach columns.

Figure 8 – Steel Alternatives - Short

The concrete alternative was a classic twin rib concrete box arch, with cast-in-place spandrel columns and concrete box girder deck (Figure 9). The arch tapers from 18 feet deep at the springing to 12 feet deep at the open spandrel crown, and is cast-in-place with stay support and form travelers. The twin box girder deck system is either incrementally launched or cast on a traveling formwork truss. The box girder is longitudinally post-tensioned as launched, and transversely post-tensioned within each box. While this alternative would increase the demands on lifting equipment, it would reduce the construction schedule for the critical path spandrel columns and deck construction.

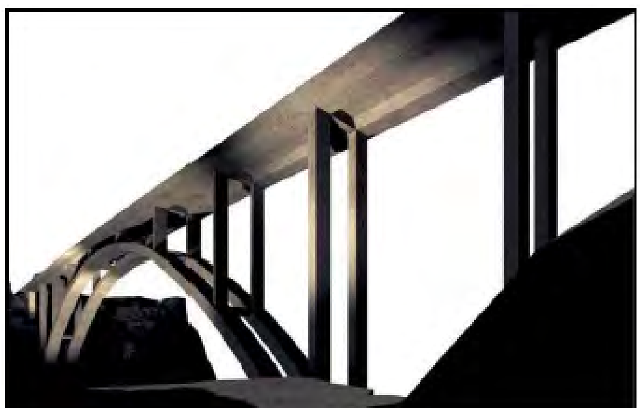


Figure 9 – Concrete Alternatives - Short

The short-span concrete-steel composite alternative combined a twin-rib concrete arch with steel or concrete spandrel columns and a conventional steel box girder with composite concrete deck (Figure 10). This option combined the most cost-effective components of concrete and steel. The spandrel and approach columns are comprised of either concrete box or steel tube sections. The deck boxes are conventional steel box girders, erected span-by-span using a girder launcher or by cableway (highline).

Figure 10 – Concrete – Steel Composite Alternative - Short

Long-Span Alternatives

The long-span layout for the arch lands on the Arizona canyon wall at approximately the same location as for the short-span layout. The long span would traverse the rock knoll at the switchback and be founded just beyond a fault behind the knoll on the Nevada side. The rise varied with each of the three long-span alternatives, and generally ranged from 276 to 290 feet, for a span-to-rise ratio of about 4.8 to 4.6. Long-span alternatives included the steel Vierendeel arch, concrete, and steel trussed-rib.

The steel Vierendeel arch alternative was advanced to provide the maximum opportunity for aesthetic expression (Figure 11). The variable depth arch ribs are vertical and parallel, and the spandrels consist of parallel columns inclined to be perpendicular to the slope of the rib. Both have Vierendeel bracing. The deck system is similar to that of the steel solid rib short-span alternative. The spans on the Nevada approach are supported by a bent similar to those on the main span, except that it is oriented vertically.

Figure 11 – Steel Vierendeel Arch Alternative - Long

The concrete arch for the longer span is a twin box rib, tapered in elevation and plan (Figure 12). The arch crown is integral with the deck, which lessens second-order effects and blends well with the aspect ratio for the long-span alternative. The deck system is similar to the short-span solid rib concrete. Due to the integral crown, a span-by-span, cast-in-place operation using a formwork truss would be used. The box girder would be longitudinally post-tensioned, and transversely post-tensioned within each box section.

Because of the longer span, the steel deck arch alternative is configured with a trussed rib (Figure 13). The variable-depth trussed ribs are vertically oriented with parallel and vertical spandrel columns. The deck system is comprised of steel plate girders spanning approximately 130 feet, with a composite concrete deck. The approach spans are similarly comprised of vertical column bents supporting a steel plate girder deck system with spans to 160 feet.

Figure 12 – Concrete Alternative - Long

BRIDGE TYPE EVALUATION PROCESS

Each of the six alternatives, as well as two additional alternatives (short-span trussed rib and long-span concrete composite) extrapolated from the original group to complete an array of choices, was evaluated for three major criteria: aesthetics, technical suitability and cost. In addition to the PMT, a Design Advisory Panel (DAP) was created to provide guidance, recommendations and input to the design team on cultural, historic and aesthetic elements of the project. The DAP includes FHWA, ADOT, NDOT, the Arizona and Nevada State Historic Preservation Offices, the Advisory Council on Historic Preservation, the National Historic Landmark Coordinator, NPS, BOR, WAPA, a Native American tribal representative, an independent architectural historian and an independent registered landscape architect.
For reviewing the technical elements of design development, a Structural Management Group (SMG) was established, comprised of the design team, FHWA and state bridge engineers from ADOT and NDOT.

The DAP's aesthetic review of the bridge options was separated into three categories: acceptable, unacceptable and controversial. The latter category was reserved for the Vierendeel alternative, which evoked strong commentary both pro and con. All solid-face rib options were considered acceptable by some or all of the DAP. The trussed arch was judged unacceptable by consensus among DAP members. With the exception of the trussed rib, all remaining concepts were acknowledged as appropriate candidates for the site. DAP input was factored into the decision process for the recommended bridge types for final design.

Figure 13 – Steel Trussed Rib Alternative - Long

The SMG technical review process involved the development of consensus ranking criteria for technical aspects of the type selection process. The SMG ranked the candidate bridge types within the type study for six technical criteria:

- Inspection requirements - to judge the difficulty and cost of routine inspections.
- Construction complexity - to judge the contractor's risk during construction for unforeseen items that arise during the course of routine work.
- Vulnerability - to judge the toughness of the structure as an impediment to terrorist threats, impacts, explosions, etc.
- Estimate volatility - to judge how stable and predictable market prices are for the construction cost estimate.
- Construction duration - to develop an anticipated schedule of main bridge construction activities.
- Serviceability - to measure the frequency and cost of routine repairs, including an assessment of the ultimate life span and ensuing life-cycle costs.

A detailed preliminary construction cost estimate was developed for each of the six alternatives based on the engineering work from the type study. Because this bridge site is quite unique and experience with long-span arch construction is rare in contemporary US bridge-building practice, the cost estimating effort goes beyond the conventional level of estimating. More detailed cost estimates, termed "build" estimates, were developed for two of the six alternatives. One alternative was selected from each of the long- and short-span layouts to obtain site-related costs for each configuration. To support these estimates, more detailed engineering drawings were necessary than would otherwise be developed for a traditional type study. To balance out a more complete family of alternatives, the steel truss rib was extrapolated to the short-span layout using a pro-ratio of steel weight for span length. The composite concrete was extrapolated to the long-span using a scale factor for the arch concrete and rebar adopted from the long-span concrete option. In this way, the full range of cost options across both span lengths could be included in the final evaluation of bridge types.

Scores from the aesthetic and technical evaluations were combined with estimated construction cost and scheduling scores to develop a total composite score. The concrete-steel composite alternative had the best composite score of all alternatives considered. Based on total evaluation, which included aesthetics, technical merit, cost and schedule, the PMT selected the short-span concrete-steel composite alternative as the final bridge type (Figure 14). Overall, the composite alternative offers many advantages, including an aggressive schedule (the composite nature of the bridge provides opportunities for major on-site concrete arch progress while steel fabrication for the deck system is underway) and cost efficiency with the greatest flexibility to design and construct the most cost-effective elements for each of the bridge components.

Figure 14 – Rendering of Selected Bridge Alternative

STATUS OF THE PROJECT

Final design of the Colorado River crossing began in the summer of 2002, and included modifications and further refinements to the selected bridge type. The pier and spandrel column type evaluation, arch and superstructure cross-sections, and integral bent cap details have been finalized. In addition, the subsurface investigation has been completed and the abutment and pier details are being finalized. The advancement of the design is being accomplished with continued coordination and guidance from the DAP, PMT, and SMG.

An advertise/award period will follow the design phase, which is scheduled for completion in September 2003. Pending availability of the required funding, advertisement for construction is scheduled for October 2003, and construction is scheduled to continue through the first quarter of 2007.

ACKNOWLEDGEMENTS

The authors would like to thank the FHWA Central Federal Lands Highway Division, the Arizona Department of Transportation, the Nevada Department of Transportation, and our respective employers for the opportunity to be associated with such an exciting, challenging, and notable project. We would also like to acknowledge the many professionals that are involved in the delivery of the Hoover Dan Bypass. The partner agencies and HST consultants are as follows:

AGENCIES – FHWA / Central Federal Lands Highway Division; Arizona Department of Transportation; Nevada Department of Transportation; U.S. Bureau of Reclamation; National Park Service; Western Area Power Administration.

CONSULTANTS – Hoover Support Team - HDR Engineering, Inc.; T. Y .Lin International; Sverdrup Civil; AMEC Earth & Environmental, Inc.; Saguaro GeoServices, Inc.; Aztec Engineering; Logan Simpson Design, Inc.; EcoPlan Associates, Inc.; Ryden Architects; H2L2 Architects/Planners, LLP; West Wind Laboratory, Inc.; John H. Clark; Walter Podolny; Art Hedgren; Scale Models Unlimited, Inc.; Kenney Aerial Mapping, Inc.; Archaeological Consulting Services, Ltd.; DEI Professional Services, Inc.; Paul Diefenderfer/Phoenix Rock Gym; Call & Nicholas, Inc.; Ropes That Rescue, Ltd.; Kaneen Advertising & Public Relations.

REFERENCES

Anderson, R.E., C. R. Longwell, R.L. Armstrong, and R.F. Marvin, 1972, Significance of K-Ar ages of Tertiary Rocks from the Lake Mead region, Nevada-Arizona: GSA Bulletin, v. 83, p. 273-287.

Anderson, R.E., 1971, Thin Skin Distension in Tertiary Rocks of Southeastern Nevada: GSA Bulletin, v. 82, p. 42-58.

Anderson R. E., 1977, Geologic Map of the Boulder City 15-Minute Quadrangle, Clark County, Nevada, USGS, Map GQ-1395.

Anderson, R. E., 1978, Geologic Map of the Black Canyon 15-Minute Quadrangle, Mohave County, Arizona, and Clark County, Nevada, USGS, Map GQ-1394.

Gans, P. and Faulds, J., 1994, Written Communications Regarding Age of Patsy Mine Volcanics, in Mills, 1995.

Goodman, R.E. and Shi, G., 1985, Block Theory and Its Application to Rock Engineering, Prentice-Hall Inc., Englewood Cliffs, New Jersey.

Green, W.E., 1985, Geologic Design Data Report for Hoover Dam Visitor and Parking Facilities, Transmission Tower Relocation, Roadway Realignment, Elevator Shaft, U.S. Bureau of Reclamation, Lower Colorado River Dams Project.

Heuze, F.E., 1984, Suggested Method for Estimating the In-Situ Modulus of Deformation of Rock Using the NX-Borehole Jack, Geotechnical Testing Journal GTJODJ, Vol. 7, No. 4, pp. 205-210.

Heuze, F.E., 1980, Scale Effects in the Determination of Rock Mass Strength and Deformability, Rock Mechanics, Vol. 12, No. 3-4, pp. 167-192.

Heuze, F.E. and Amadei, B., 1985, The NX-Borehole Jack: A Lesson in Trials and Errors, International Journal of Rock Mechanics Min. Sci. & Geomech. Abstr., Vol. 22, No. 2, pp. 105-112.

Hoek, E. and Brown, E.T., 1997, Practical Estimates of Rock Mass Strength, International Journal Rock Mechanics Mining Sciences, Vol. 34, No. 8, pp. 1165-1186.

Hoek, E. and Brown, E.T. (1980), Underground Excavations in Rock, The Institute of Mining and Metallurgy, Stephen Austin and Sons Ltd., Hertford, England.

Hoek, E., 1998, Reliability of Hoek-Brown Estimates of Rock Mass Properties and their Impact on Design, International Journal Rock Mechanics Mining Sciences, Vol. 35, No. 1, pp. 63-68

Keaton, J.R., 2003, Earthquake Ground Motion for Design of Hoover Dam Bypass Bridge (US Highway 93), presented at Highway Geology Symposium 54, Burlington, VT, September.

Longwell, C.R., 1936, Geology of the Boulder Reservoir Floor, GSA Bulletin, v. 47, no. 9.

Longwell, C.R., 1963, Reconnaissance Geology between Lake Mead and Davis Dam, Arizona-Nevada, USGS Professional Paper 374-E.

Mills, James G. Jr., 1994, Geologic Map of the Hoover Dam Quadrangle, Arizona and Nevada, Nevada Bureau of Mines and Geology, Map 102.

Mills, James G. Jr., 1985, The Geology and Geochemistry of Volcanic and Plutonic Rocks in the Hoover Dam 7½-Minute Quadrangle, Clark County, Nevada and Mohave County, Arizona, unpublished Master's Thesis, University of Nevada, Las Vegas.

Otton, J.K., 1982, Tertiary Extensional Tectonics and Associated Volcanism In West-Central Arizona: in Frost, E.G. and Martin, D.L. (eds.), Mesozoic-Cenozoic Tectonic Evolution of the Colorado River Region, California, Arizona, and Nevada, Cordilleran Publishers, San Diego, CA, p. 143-157.

Smith, E.I., 1982, Geology and Geochemistry of the Volcanic Rocks in the River Mountains, Clark County, Nevada and Comparisons with Volcanic Rocks in Nearby Areas, in Frost, E.G. and Martin, D.L. (Eds.), Mesozoic-Cenozoic Tectonic Evolution of the Colorado River Region, California, Arizona, and Nevada: San Diego, California, Cordilleran Publishers, p. 41-54.

Suneson, N.H. and Lucchitta, I., 1983, Origin of Bimodal Volcanism, Southern Basin and Range Province, West-Central Arizona, GSA Bulletin, V. 94, p. 1005-1019.

Viskne, A., 1976, Evaluation of In Situ Shear Wave Velocity Measurement Techniques: REC-ERC-76-6, Division of Design, Engineering and Research Center, U.S. Dept. of the Interior, Bureau of Reclamation, Denver, Colorado.

A Risk-Consequence Hazard Rating System for Missouri Highway Rock Cuts

Ahmed Youssef, University of Missouri-Rolla, 1006 Kingshighway, Rolla, MO, 65409-0660, Tel: (573) 341-6714, Fax: 573 341-4368, Fax: 573 341-4729, Email: youssef@umr.edu

Norbert H. Maerz, University of Missouri-Rolla, 1006 Kingshighway, Rolla, MO, 65409-0660, Tel: (573) 341-6714, Fax: 573 341-4368, Email: norbert@umr.edu

Mike A. Fritz, Missouri Department of Transportation, 1617 Missouri Boulevard, Jefferson City, Missouri 65109, Tel: 573-526-4346, Fax: 573-526-4345, Email: fritzm@mail.modot.state.mo.us

Abstract

A new method for the analysis of rockfall hazards along roads of the Missouri State Highway System is described here. Existing rockfall hazard rating systems focus on the risk of failure and ignore the consequence of failure, or they lump the ratings for risk and consequence together. In this new method, risk and consequence factors are given equal weight and isolated from each other. The ratings for the categories that related to risk or consequence are easy to determine and are more objective. The risk – consequence rating system can be used by DOT's to cost effectively determine the need, priority, and design of maintenance on rock cuts, in order to provide the safety and convenience of the motoring public demands and also to reduce the consequence that will affect the falling rocks on the road by decreasing the risk of vehicle damage and traffic delays.
The risk–consequence system named the Missouri Rockfall Hazard Rating System (MORH RS) utilizes two phases; 1) Identification of the most potentially problematic rock cuts, by using mobile digital video logging, and 2) Using the system to characterize and prioritize remediation for the potentially problematic rock cuts identified in phase 1. In phase 2 three types of parameters are evaluated; 1) parameters such as slope height, slope angle, ditch width, ditch depth, shoulder width, block size, number of lanes, ditch capacity, and expected rockfall quantity can be measured on computer scaled video images, parameters such as weathering, face irregularities, face looseness, strength of intact rocks, water on the face, and design sight distance are descriptive, and 3) parameters such as average daily traffic and average vehicle risk are obtained from the Department of Transportation for each section of road. The system has been tested on sections of Missouri Highways 63 and 65, and Interstate Highway 44.

Introduction

Construction and maintenance of highways and railways in rocky and mountainous regions presents a special challenge to geologists and geotechnical engineers. Because there are thousands of highways, and hundreds of thousands of highway miles it is difficult do sufficient stability assessments for each of the rock cut along the routes.

For that reason most highway cuts tend to be designed, constructed, and maintained on the basis of rather rudimentary geotechnical analyses concerning the stability of the slopes against major sliding or toppling failures. Only the populated areas in highly developed countries receive even this type of care and analysis (1).

Rockfalls take place every year during the rainy seasons in both natural and man made slopes, especially along the road cuts of the hilly areas. These rockfalls block roads, damage infrastructure, and cause injuries and fatalities to occur. According to the Department of Highway in Washington State a significant number of accidents and nearly a half dozen fatalities have occurred because of rockfalls in the last 30 years … and…. 45 % of all unstable slope problems are rockfall related (2). In Canada almost 13 people died because of rockfall in the last 87 years, most of them on British Columbia highways (3).

Because of the difficulty of carrying out detailed investigations and analyses on the miles of highway in the United States and Canada, most of the Department of Transportation's try to design a good rating system to save them time and money. These systems are designed to be simple, relying primarily on visual inspection and simple calculations. The importance of these rating systems is to identify slopes which may be particularly hazardous and which require further more detailed study.

Analyses that are used for slope stability

Planar and wedge sliding and toppling mechanisms

In this type of failure mechanism the discontinuities are oriented in such a way that they contribute to create wedges, planar sliding blocks, or toppling blocks. Franklin and Senior report that of 415 analyzed cases of failure in Northern Ontario, Canada, only 33% of failures involved these mechanisms (23% toppling, 8% planar sliding, 2% wedge sliding) (4). These types of failures are however easy to analyze, and can range from limiting equilibrium analysis to numerical modeling (5).

Raveling type failure mechanism

Previous studies in Northern Ontario, reported that 65% of the failures were of the raveling type (4). These included raveling, overhang/undercutting failure, ice jacking, and rolling blocks. In other terrains, most notably flat lying sedimentary rock with vertical jointing, where planar and wedge slides are unusually not found, the predominant failure mechanism being of the raveling type is even greater. These raveling failures whether slow, time-dependent or fast and catastrophic are much more difficult to analyze. Analytical techniques for prediction are non-effective, and remediation judgments are typically made with on-site engineering judgment of an experienced specialist, who must then balance the risk in terms of probability of failure and consequence of failure, against the cost of effective remediation. The use of empirical design and rock mass classification become important (6).

Empirical Design and Rock Mass Characterization

Empirical design is a design methodology that does not use formal design methods, and calculations or analytical equations or modeling or such. Instead it relies on experience and judgment of the engineer. The realization of empirical design that uses not only individual experience, but also the cumulative experiences of many comes from the following principles:

1. Description of ground quality,
2. Description of ground performance,
3. Correlation of the above two based on a study of case histories.

Design schemes like this are common in the mining and tunneling industries, and are described by Singh and Goel (7). Examples of such classification systems that include elements of design include several different classes of systems.

Systems that consider geological factors only

Deere's RQD (rock quality designation) system (8), Franklin's Size-Strength system (9), Franklin's Shale Rating System (10), Bieniawski's RMR (rock mass rating) system (11), and Barton Q system (12) consider only geological factors. In addition there are several schemes for slopes. Romana's SMR system is for rock slopes, based on Bieniawski's RMR system (13).

Systems that consider rainfall as well as geological factors

There are two systems that consider the geological factors and the rainfall effect as Rock Engineering system (RES) (14) and rock mass instability index RMIIj developed by Ali, M. K and Hassan, (15).

Rock hazard rating systems

Rock hazard systems consider not only geological factors but also highway factors such as ditch capacity. The Oregon RHR (rock hazard rating) system is designed specifically for highways cuts (16). The Ontario RHRON (Rock Hazard Rating ONtario) system is a modification of the Oregon system (4).

Limitation of existing systems

1. The systems that apply easily to analyses of planar, wedge and toppling failure types are not useful for other types of failures.
2. Some of them consider geological factors only and essentially classifying risk only without considering the consequence of failure.
3. It is hard to distinguish between stable slopes from unstable slopes by using a field inspection as the rock engineering system.
4. The rock hazard rating system developed in Oregon is not very sensitive to low rock cuts. It is not a universal system.

5. The Ontario RHRON is somewhat arbitrary. There is no actual separation between risk factors and consequence factors. It is time consuming to measure such a large number of factors. Some factors need laboratory analysis and this adds time and cost.
6. The New York Rock Slope Rating System does not adequately distinguish between risk and consequence. The system is insensitive for small slopes. The connection between the rated GF, and more analytical SF and HEF is ambiguous and may be tenuous.

A New Method for Rockfall Hazard Rating

A risk-consequence rating system is currently under development for the Missouri Department of Transportation (MODOT). The system named Missouri Rockfall Hazard Rating system (MORH RS) has several unique and progressive attributes

Concept

The rock cut (rockfall) hazard rating system being designed for Missouri highways is designed to cost effectively determine the need and priority of maintenance on rock cuts. This is in order to safeguard the motoring public and also to reduce the risk of vehicle damage and traffic delays as a result of rock falls, especially in light of the potential of rock fall disruption of transportation if major activity on the New Madrid fault should occur.

A three phase approach to mitigating rock fall hazards is proposed, to utilize resources efficiently:

1. Identification (over the entire road network) of potentially problematic rock cuts, using mobile digital video logging.
2. Characterization and prioritization of remediation (for the potentially problematic rock cuts identified in phase 1) using a purpose designed rock mass rating system.
3. Detailed analysis and design methodologies for final remediation (for the prioritized rock cuts identified in phase 2).

The efficiency will come from rapid, low cost screening of problematic areas, an effective relatively efficient characterization scheme to prioritize remediation, leaving most of the resources for the task of detailed characterization, design, and implementation of final remedial measures on slopes deemed to be high priority.

This research specifically addresses the first 2 phases. It is here that new methods can be developed to introduce efficiencies and comprehensiveness into the process. Phase 3 methodologies have been developed and are widely implemented. A flow chart for the MORH RS has been prepared (Figure 1).

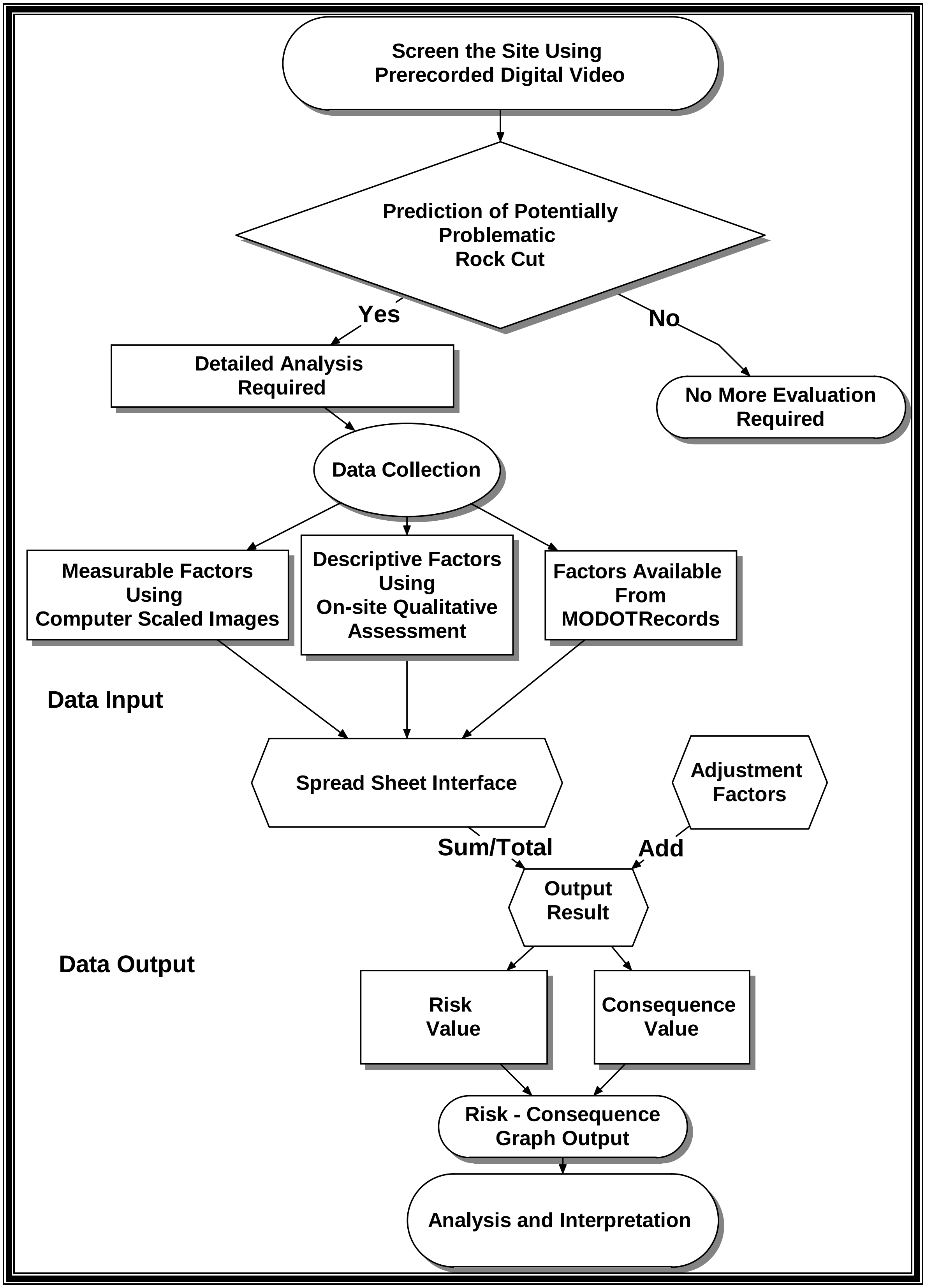


Figure 1. Flow chart

Video logging

A digital video logging system (Figure 2), described previously (3) is used as a screening tool to identify problematic highway rock cuts. Video images of highway right-of-ways are routinely done for inventorying of highway assets and measurements of such attributes as sign placement (18). A specific subset or the entire network of Missouri highways can be video logged, using a video camera equipped with GPS (global positioning system) coordinate overlays, using DOT personnel to do the driving. Trained geotechnical engineers or geologists can review the video footage at a computer workstation in their office to identify problematic cuts and then decide which sites warrant more detailed investigation.

Measurements on scaled video images

The same images that can be used for video logging can also be used to measure some of the parameters required for the rating system (18). Measurements can be made on single images without extensive vehicle instrumentation and modifications. Although not as accurate as manual measurements in the field, the measurements are sufficiently accurate to provide input data for a rock hazard rating system. At the University of Missouri-Rolla a prototype of such a system has been developed (Figure 3).

The system uses video logging hardware, which includes a simple camera setup, scale calibration, and appropriate manual identification of object endpoints to enable quick and easy measurements of blocks. Typical measurements include slope heights, lengths, and angles; ditch widths, depths, and volumes; mass volumes; and other linear measures.
In a recent study by Maerz et al. (18), video measurements were compared to manual measurements for specific parameters that would be required in any of the rock hazard rating systems mentioned previously. The measurement errors, defined as the percentage difference between manual and image measurements, on average were found to be less than 10% Table 1.

Risk vs. consequence system

The MORH RS is predicated on separating risk from consequence (Figure 4). While other rating systems may consider both risk of failure and consequence of failure factors, they tend to lump them together. This is incorrect, as some parameters affect risk and consequence in different ways. For instance, the larger the block size, the lower the risk of failure but the higher the consequence of failure. A 90° slope would present the highest risk of failure, while perhaps a 30° and 85° slopes would present the highest consequence of failure for large rolling blocks and small bouncing rocks respectively.

Figure 2. Digital camcorder mounted on vehicle dashboard.

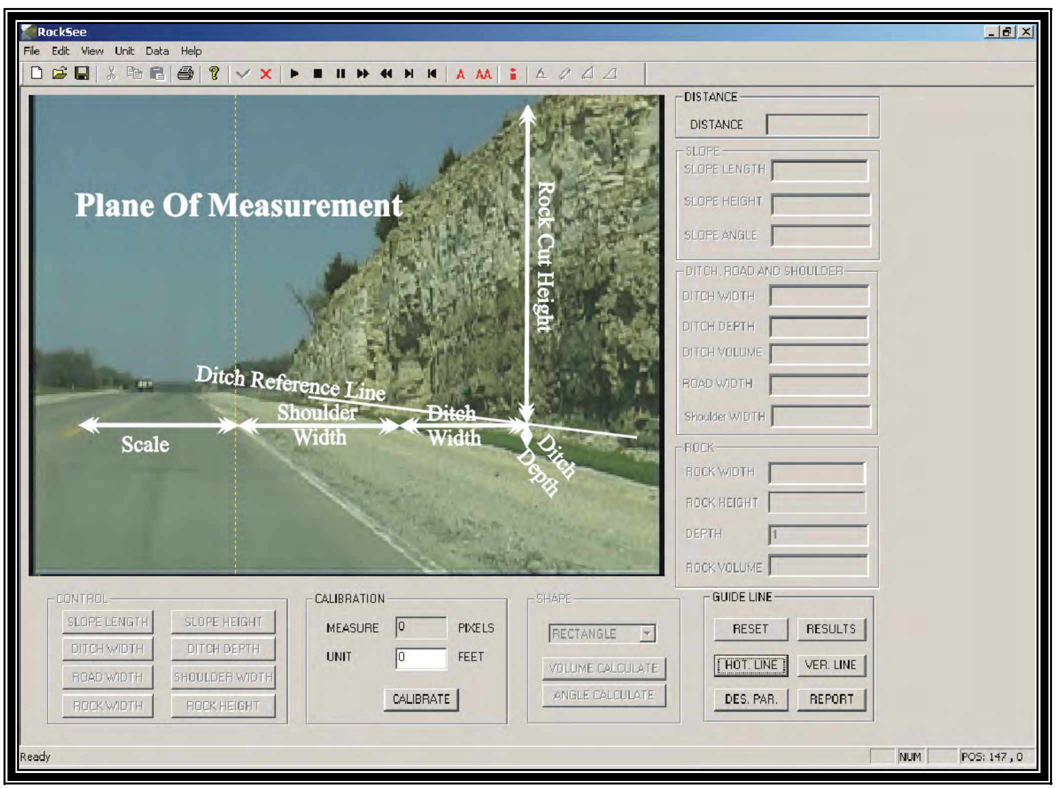


Figure 3. RockSee program to measure various parameters needed for the hazard rating system.

Table 1. The average error in % for each type of scaled video measurement.

Measurable Factors	Error %
Ditch Width	6.0%
Ditch Depth	8.6%
Slope Length	4.2%
Slope Angle	2.7%
Cliff Height	3.9%
Shoulder Width	7.6%
Road Width	2.7%
Rock Cut Length	6.8%

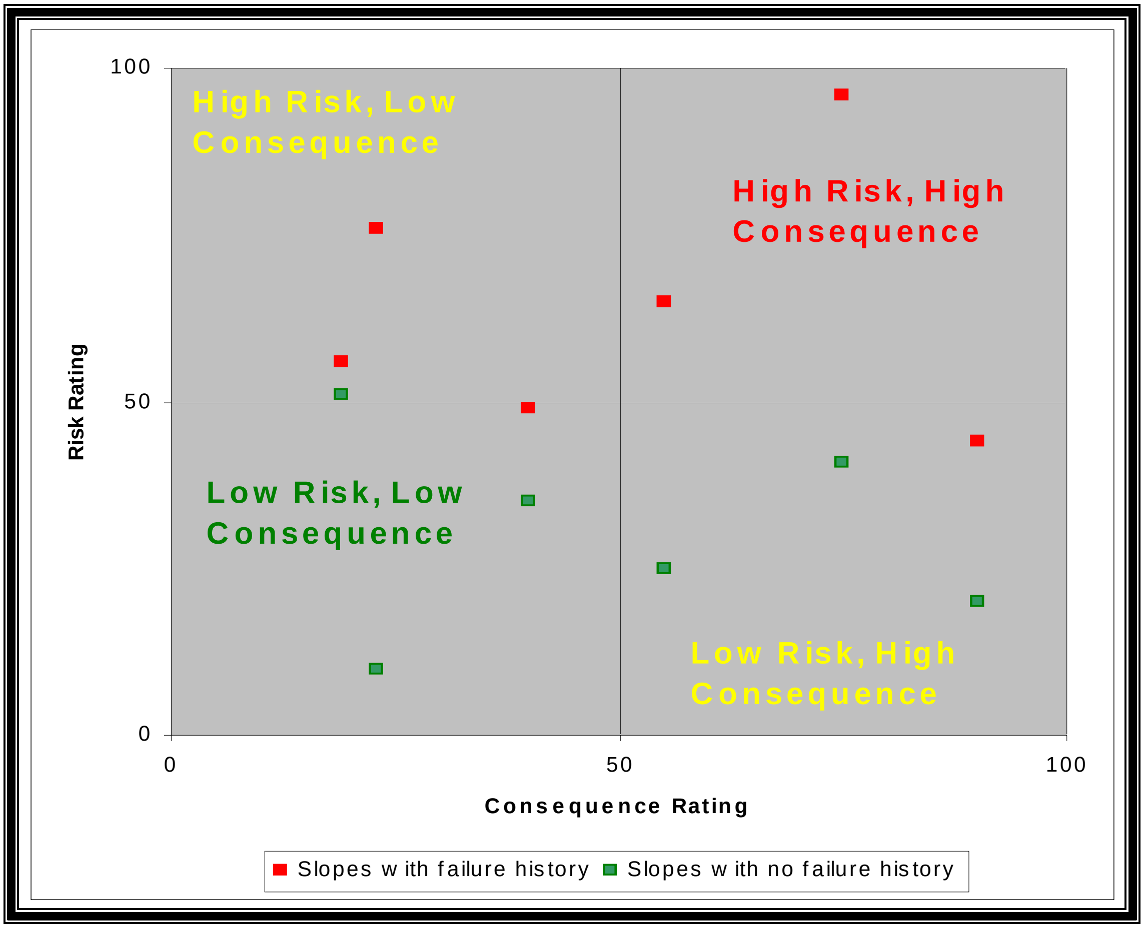


Figure 4. Conceptual example of risk/consequence assessment. Case histories of failed/stable slopes can be plotted on this graph to determine threshold action levels.

In any case, separating risk and consequence seems useful, because it may be possible to concern ourselves only with high risk, high consequence rock cuts. Low risk rock cuts need not worry us because there is small chance of failure, and low consequence cuts need not worry us because the fallen rock is not likely to reach and affect the highway traffic.

The Missouri Risk – Consequence Rating System (MORH RS)

The current iteration of the MORH rating system includes 22 factors (These factors are still under evaluation). The system includes 9 factors for risk, 10 factors for consequence, and 3 adjustment factors as described below. These factors have been organized into risk (of failure) and consequence (of failure) categories, and identified based on how the factors are evaluated:

A-Risk Factors	**Rating**
1- Slope Height*	0-12
2- Slope Angle*	0-12
3- Rockfall Instability (History)**	0-12
4- Weathering Factor***	0-24
5- Strength of the intact rocks***	0-12
6- Face Irregularity***	0-12
7- Face Looseness***	0-12
8- Block Size*	0-12
9 -Water On Face***	0-12

B-Consequence Factors	**Rating**
1- Ditch Width*	0-12
2- Ditch Volume*	0-12
3- Rockfall Quantities (Expected)*	0-12
4- Slope Angle*	0-12
5- Shoulder Width*	0-12
6- Roadway Width*	0-12
7- Average Daily Traffic (ADT) **	0-12
8 -Average Vehicle Risk ****	0-12
9 -Decision Sight Distance (DSD)*	0-12
10- Block Size*	0-12

C-Adjustment Factors/Risk	**Rating**
1- Dip angle of discontinuities***	0-12
2- Filled sinkhole size ***	0-12

D-Adjustment Factors/Consequence	**Rating**
1- A- Ditch Capacity Exceedence****	0-15

* Factors that can be measured on computer scaled images
** Factors that can be made available by MODOT
*** Factors that require on-site qualitative assessment
**** Factors that are calculated based on other input values

Table 2. Risk – Consequence Factors
Risk Factors

Slope height (ft)	10	20	30	40	50	60
Rating	2	4	6	8	10	12

Slope angle	30^o	40^o	50^o	60^o	70^o	80^o	90^o
Rating	0	2	4	6	8	10	12

Rockfall Instability	Completely unstable	Unstable	Partially stable	Stable	Completely stable
Class Number	4	3	2	1	0
Rating	12	9	6	3	0
Weathering	High	Moderate	Low	Slight	Fresh
Class Number	4	3	2	1	0
Rating	12	9	6	3	0
Intact rock strength	Very strong	Strong	Moderate	Weak	Very weak
Class Number	4	3	2	1	0
Rating	0	3	6	9	12
Face Irregularity	Very high	High	Moderate	Slight	Smooth
Class Number	4	3	2	1	0
Rating	12	9	6	3	0
Face Looseness	Very high	High	Moderate	Low	No
Class Number	4	3	2	1	0
Rating	12	9	6	3	0

Block Size	Massive (> 5 ft)	Moderately blocky (2.5 ft)	Very blocky (1 ft)	Completely crushed (< 0.5 ft)
Rating	0	3	9	12

Water on the Face	Dry	Damp	Wet	Dripping	Flowing
Class Number	0	1	2	3	4
Rating	0	3	6	9	12

Consequence Factors

Ditch width (ft)	0	5	10	15
Rating	12	8	4	0

Ditch volume (cu ft/ ft)	0	5	10	15	20	25	30
Rating	12	10	8	6	4	2	0

Expected Rockfall Quantity (cu ft/ ft)	< 5	10	20	30	> 40
Rating	0	3	6	9	12

Slope Angle	20^o	30^o	40^o	50^o	60^o	70^o	80^o	85^o	90^o
Rating	0	12	10	6	3	2	4	12	0

Shoulder Width (ft)	0	3	6	9	12
Rating	12	9	6	3	0

Number of Lanes	One lane	Two lanes	Three lanes	Four lanes
Rating	12	6	3	0
Average Daily Traffic	5000 Cars / day	10000 Cars / day	15000 Cars / day	20000 Cars / day
Rating	3	6	9	12
Average Vehicle Risk	Low Risk 25% of the time	Medium Risk 50% of the time	High Risk 75% of the time	Very high Risk 100% of the time
Rating	3	6	9	12
Design Sight Distance	Very Limited	Limited	Moderately Limited	Adequate
Class Number	3	2	1	0
Rating	12	8	4	0
Block Size	Massive (> 5 ft)	Moderately blocky (2.5 ft)	Very blocky (1 ft)	Completely crushed (< 0.5 ft)
Rating	12	8	4	0

Table 3. Risk – Consequence Adjustment Factors.
Risk factors

Adversely Oriented Discontinuities	**Favorable**	**Fair**	**Unfavorable**	**Very Unfavorable**
Dip angle of discontinuities, Daylighting into cut	< 20	20 – 45	45 - 65	65 – 90
Rating	0	4	8	12

B- Sinkhole effect

Filled sinkhole size	**Small** **50 ft wide**	**Medium** **100 ft wide**	**Large** **150 ft wide**
Rating Value	4	8	12

Consequence factors
A- Ditch Capacity Exceedence (ERFQ/DV)

Ditch Capacity Exceedence (RFQ/DV)	1	2	3	4
Rating Value	0	5	10	15

Ditch Capacity Exceedence (Expected Rockfall Quantity/Ditch volume) (ERFQ/DV)

If ERFQ/DV = 1 that means the ditch will contain all the fallen rocks.
If ERFQ/DV = 2 that means the ditch will completely fill and a large amount spill over.
If ERFQ/DV = 3 that means the fallen rock will spill over to the shoulder of the road.
If ERFQ/DV = 4 that means the fallen rocks will spill over to the road.

Details of the rating system factors can be seen in Table 2, with the adjustment factors given in Table 3. In this system there are many different methods used to determine the rating values for each parameter by using tables, graphs, and equations.

For each of the risk and consequence factors, the ratings are summed, and divided by the maximum total ratings to give a value in percent. Adjustment factors must be added afterward. These range from 0-12 (0-15), and are added directly to the rating system (i.e. not averaged in with the rest of the parameters).

MORH RS (User input vs. Internal Calculations)

MORH RS is designed to be as complex as required, but have as simple as possible a user interface. The current version uses a Microsoft Word® user interface, with and embedded Microsoft Excel® OLE® object. Figure 4 shows the one page report, which consists of:

1. Site location information (Road name, site number, and GPS coordinates),
2. Picture,
3. Rating chart, and
4. Rating graph.

The site location information is manually entered; the picture is pasted in. The rating chart is interactive and linked to the graph. Changes can be made anytime to the rating system, and the changes are reflected in the graph. The user needs only to enter the white fields in Figure 4, and the ratings are calculated automatically and the plot will appear on the graph. Where real measurements are available, they are entered directly. For descriptive parameters the ordinal values 0-4 are entered:

A- Risk Factors	**Values**
1- Slope Height	0 - 60’
2- Slope Angle	30 - 90°
3- Rockfall Instability (History)	0 - 4.0 (class number)
4- Weathering Factor	0 - 4.0 (class number)
5- Strength of the intact rocks	0 - 4.0 (class number)
6- Face Irregularity	0 - 4.0 (class number)
7- Face Looseness	0 - 4.0 (class number)
8- Block Size	0.1 - 5’
9- Water on Face	0 – 4.0 (class number)

B- Consequence Factors	**Values**
1- Ditch Width	0 - 15’
2- Ditch Volume	0 - 30 cubic feet/foot
3- Rockfall Quantities (Expected)	0 - 40 cubic feet/foot
4- Slope Angle	20 – 90o
5- Shoulder Width	0 - 12’
6- Roadway Width	1 - 4 lanes
7- Average Daily Traffic (ADT)	0 - 20,000 cars per day
8- Average Vehicle Risk	calculated from:
Speed Limit	40 - 70 mph
Hazard rock cut length	100 - 600’
9- Decision Sight Distance (DSD)	0 - 4.0 (class number)
10- Block Size	0.1 - 5’

Road and Site Data

GPS Data

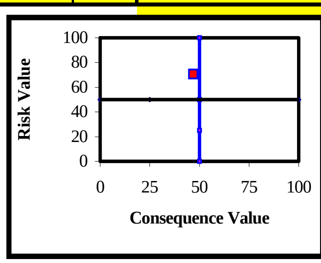
Site No. 1

HYW	Elevation	Latitude	Longitude
63	1225 ft	N 37- 32.591	W 091-51.745

Site Picture Interface

Risk	Value		Rating	Consequence	Value		Rating
Rock Cut Height	30		6	Ditch Width	9		4.8
Slope Angle	65		7	Ditch Volume	12		7.2
Rockfall Instability	4		12	Slope Angle	65		2.5
Weathering	3		18	Shoulder Width	9		3
Rock Strength	0		12	Lanes Number	1		12
Face Irregularities	4		12	Average daily Traffic	5500		3.3
Face Looseness	4		12	Rockfall Quantity	10		3
Block Size	5		0	Average Vehicle Risk	60	968	8.4
Water on Face	2		6	Design Sight Dist.	0		0
	Joint	Sinkh.		Block Size	5		12
Adjust. Factor	**0**	0	0	**Adjust. Factor**	1		0
Total			**71**	**Total**			**46.8**

Risk Consequence parameters, Values, and calculations Interface



Risk - Consequence Graph Interface

Figure 4. Single page report to show results of evaluation. White (un-shaded) fields are user inputs.

C-Adjustment Factors/Risk	**Values**
1- Dip angle of discontinuities	0 – 3.0
2- Filled sinkhole size	0 – 3.0

D-Adjustment Factors/Consequence	**Values**
1- A- Ditch Capacity Exceedence	1 – 4.0

Application and Results of MORH RS to Missouri Rock Cuts

Highway 63 near Rolla Missouri

Figure 6 shows an example of a rockcut along Highway 63 north of Rolla MO. Figure 5 shows the results for 58 sites that have been studied along Highway 63. The distribution of the data shows that the data fall in three zones: high risk-high consequence, high risk-low consequence, and low risk-low consequence. Significantly there are many in the high risk-high consequence section and relatively few in the low risk-low consequence section.

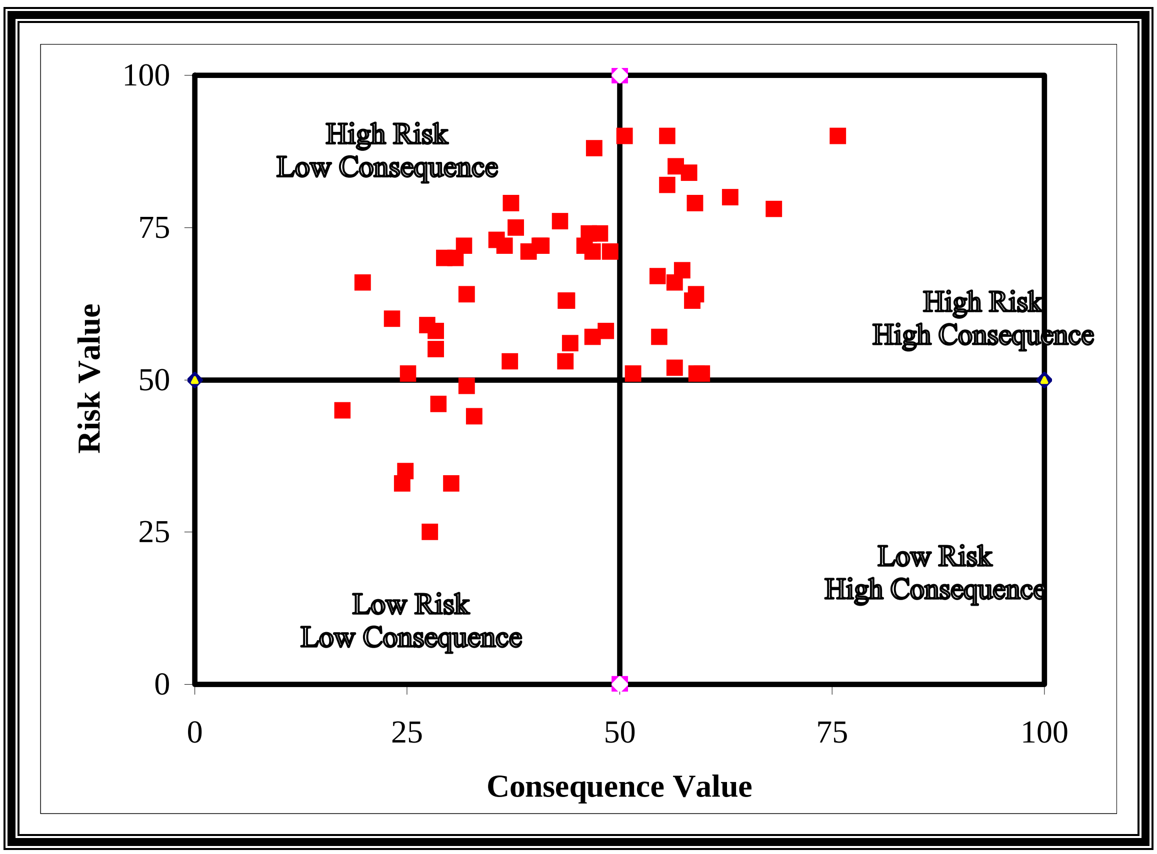


Figure 5. Risk – Consequence diagram for the data from Highway 63.

Site No.	HYW	Elevation	Latitude	Longitude
58	63	799 ft	N 38-08.974	W 091-53.517

Risk	Value		Rating	Consequence	Value		Rating
Rock Cut Height	45		9	Ditch Width	13		1.6
Slope Angle	90		12	Ditch Volume	13		6.8
Rockfall History	4		12	Slope Angle	90		0
Weathering	4		24	Shoulder Width	16		0
Rock Strength	0		12	Lanes Number	1		12
Face Irregularities	4		12	Average daily Traffic	5500		3.3
Face Looseness	4		12	Rockfall Quantity	100		12
Block Size	5		0	Average Vehicle Risk	65	500	4.0
Water on Face	1		3	Design Sight Dist.	0		0
	Joint	Sinkh.		Block Size	5		12
Adjust. Factor	**0**	3	12	**Adjust. Factor**	7.7		15.0
Total			**90**	**Total**			**55.6**

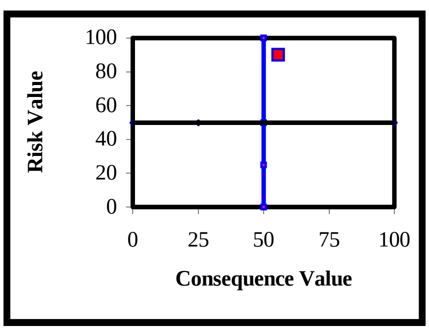


Figure 6. Report for site No. 58 on Highway 63.

Highway 65 between Springfield and Branson

Figure 8 shows an example of a rockcut along Highway 65 between Springfield and Branson MO. Figure 9 shows the results for 50 sites that had been studied along Highway 65. The distribution of the data shows that the data fall in three zones: high risk-high consequence, high risk-low consequence, and low risk-low consequence. Significantly there are many in the low risk-low consequence section and relatively few in the high risk-high consequence section.

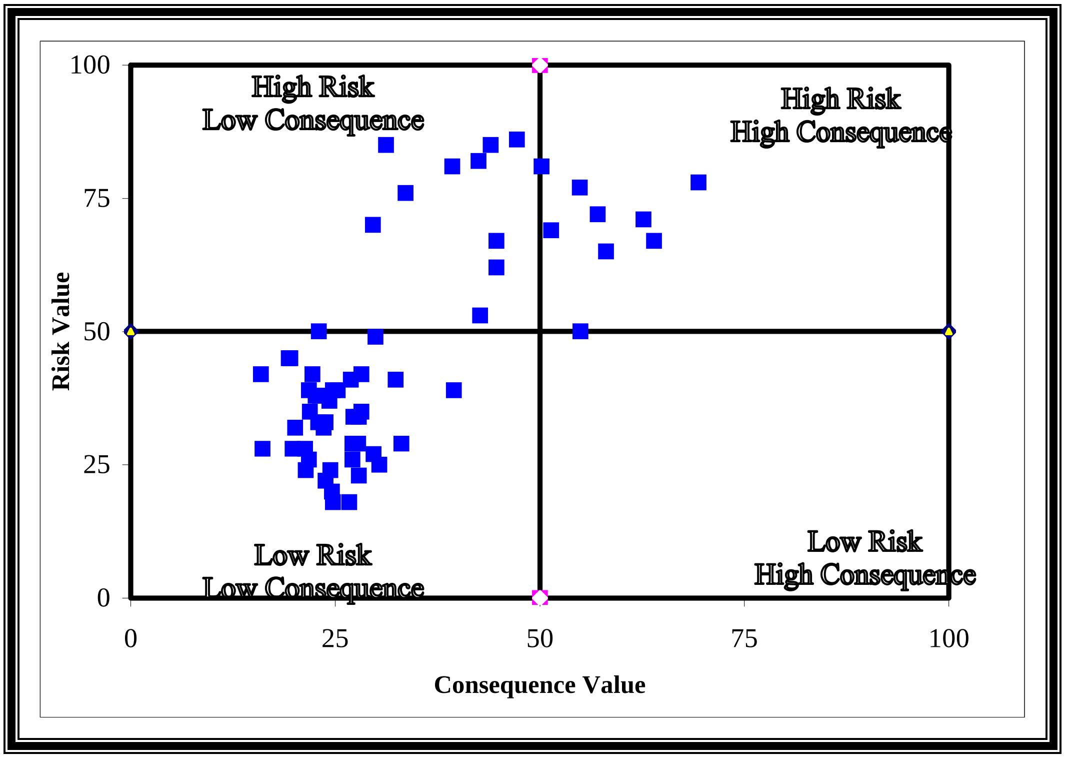


Figure 7. Risk – Consequence diagram for the data from Highway 65.

Site No.	HYW	Elevation	Latitude	Longitude
30	65	965 ft	N 36- 41.169	W 093-13.279

Risk	Value		Rating	Consequence	Value		Rating
Rock Cut Height	30		6	Ditch Width	26		0
Slope Angle	90		12	Ditch Volume	52		0
Rockfall History	1		3	Slope Angle	90		0
Weathering	1.5		9	Shoulder Width	10		2
Rock Strength	3		3	Lanes Number	2		6
Face Irregularities	1.5		4.5	Average daily Traffic	24000		12
Face Looseness	1		3	Rockfall Quantity	5		1.5
Block Size	3.5		1	Average Vehicle Risk	65	0	0.0
Water on Face	1.5		4.5	Design Sight Dist.	0		0
	Joint	Sinkh.		Block Size	3.5		6
Adjust. Factor	0	0	0	**Adjust. Factor**	1.0		0.0
Total			**38**	**Total**			**23.1**

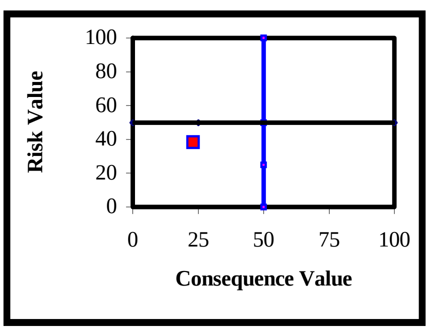


Figure 8. Report for site No. 30 on Highway 65.

Highway 44 between St. Louis and Springfield

Figure 10 shows an example of a rockcut along Highway 44 from St. Louis to Springfield west and east of Rolla MO. Figure 11 show the results for 49 sites that had been studied along Highway 44. The distribution of the data shows that the data fall in three zones: high risk-high consequence, high risk-low consequence, and low risk-low consequence. Significantly there are many in the high risk-high consequence section and relatively few in the low risk-low consequence section.

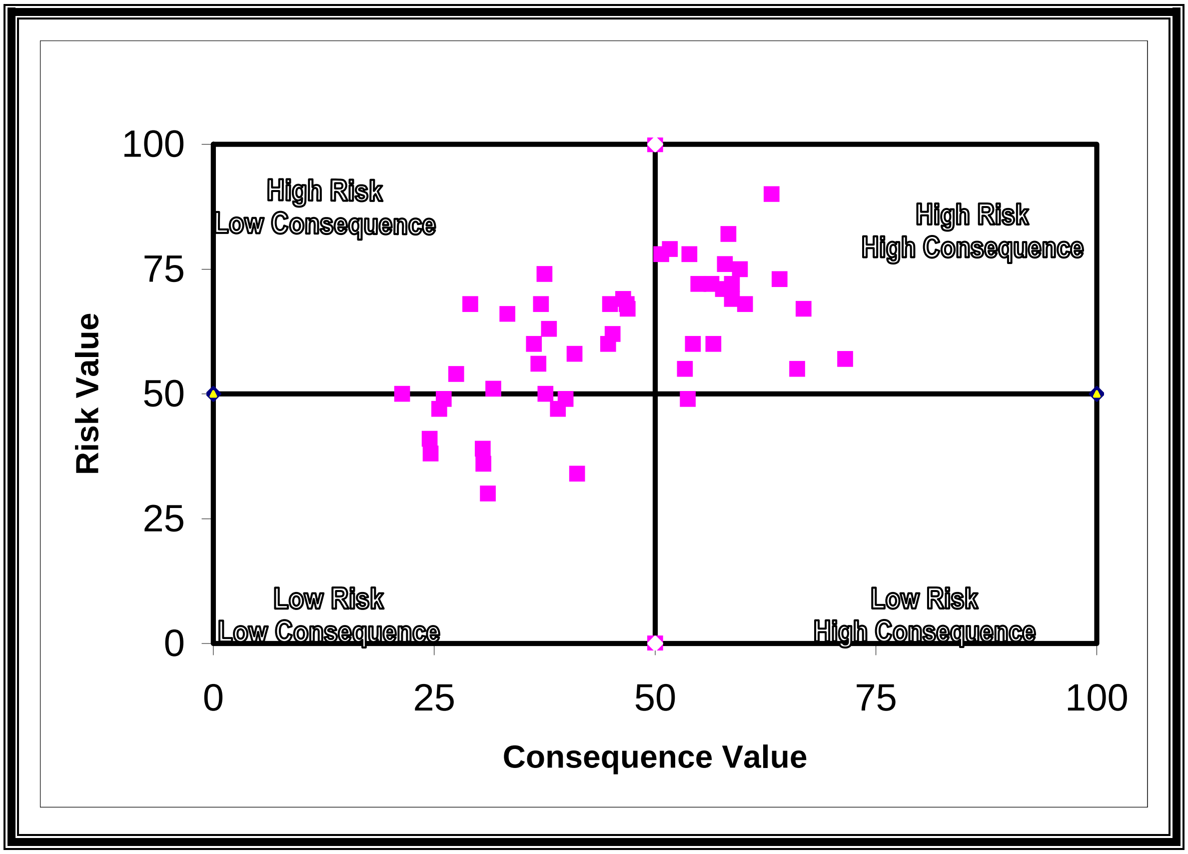


Figure 9. Risk – Consequence diagram for the data from Highway 44.

Site No.	HYW	Elevation	Latitude	Longitude
30	44	975 ft	N 37- 51.777	W 092-02.820

Risk	Value		Rating	Consequence	Value		Rating
Rock Cut Height	25		5	Ditch Width	6		7.2
Slope Angle	75		9	Ditch Volume	16		5.6
Rockfall History	4		12	Slope Angle	75		5
Weathering	4		24	Shoulder Width	10		2
Rock Strength	0.5		10.5	Lanes Number	2		6
Face Irregularities	3		9	Average daily Traffic	24000		12
Face Looseness	4		12	Rockfall Quantity	30		9
Block Size	3.5		1	Average Vehicle Risk	70	850	12.0
Water on Face	3		9	Design Sight Dist.	0		0
	Joint	Sinkh.		Block Size	3.5		6
Adjust. Factor	**0**	0	0	**Adjust. Factor**	1.9		4.4
Total			**76**	**Total**			**57.9**

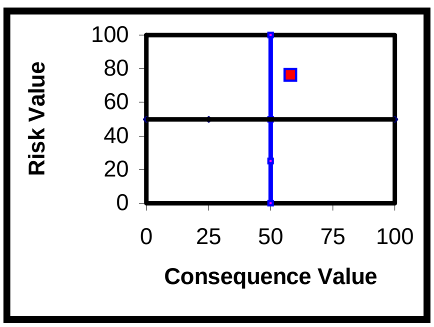


Figure 10. Report for site No. 30 on Highway 44.

Comparison of Highway 63/44 and Highway 65 sites

A comparison of the sites shows that the rating for the Highway 65 rock cuts are considerably lower for both risk and consequence than those of Highway 63 and 44. This is most likely because the Highway 65 cuts are new and have superior design and better blasting. The Highway 63 and 44 rock cuts are older cuts, and have had additional weathering over time.

Conclusions

This new rockfall hazard system (MORH RS) is designed to very efficiently and effectively determine the risk and consequence values for any site and to identify, which sites need further attention by plotting the risk consequence values on a risk-consequence diagram. The data are prepared for the system from three different sources, factors that can be measured on computer scaled images, factors that can be made available by MODOT and factors that require on-site qualitative assessment.

Currently this system works with an excel spread sheet file embedded in a word document. The spreadsheet is interactive, the risk-consequence plot changes in response to changes in the values of the parameters.

Preliminary work on Missouri highways has demonstrated that the system can effectively be used to prioritize the severity of potential rockfalls.

Acknowledgements

The authors would like to acknowledge the Missouri Department of Transportation (MODOT) for funding this research. We would especially like thank Tom Fennessey, Ray Purvis and Rob Lauer of MODOT for their help in providing guidance for this project. We would like to thank the University of Missouri System Research Board for the original seed funding of this research, especially the development of the RockSee program.

References

1- Hoek, E. 2000. **Practical Rock engineering**. 299 pages
2- Badger, T.C. and Lowell, S. 1992. **Rockfall Control Washington State**. In Rockfall Prediction and Control and Landslide Case Histories, Transportation Research Record, National Research Council, Washington, No 1342, pp 14-19.
3- Hungr, O. and Evans, S.G. 1989. **Engineering aspects of rockfall hazard in Canada.** Geological Survey of Canada, Open File 2061, 102 pages.
4- Franklin, J. A., And Senior, S. A., 1987. **Outline of RHRON, the Ontario Rockfall Hazard Rating System.** Proceedings International Symposium on Engineering Geology and the Environment, Athens, Greece, 1987, pp. 647-656.
5- Hoek, E. and Bray, J.W. 1981. **Rock Slope Engineering** . 3rd edn. London: Institution of Mining and Metallurgy 402 pages

6- Franklin, J. A., and Maerz, N. H., 1996. **Empirical design and rock mass characterization.** Proceedings of the FRAGBLAST 5 Workshop on Measurement of Blast Fragmentation, pp. 193-201.
7- SINGH, Singh, B. and Goel, R. K., 1999. **Rock Mass Classification**. A Practical Approach in Civil Engineering: Elsevier, 267 pp.
8- Deere, D. U., Merritt, A. H., And Coon, R. F., 1969. **Engineering Classification of In Situ Rock**. Air Force Systems Command, Kirtland Air Force Base, Report AFWL-64-144.
9- Franklin, J. A., 1986. **Size-Strength System for Rock Characterization**. Proceedings of the Symposium of Rock Characterization to Mine Design, American Society of Mining Engineers Annual Meeting , pp. 11-16.
10- Franklin, J. A., 1983. **Evaluation of Shale's For Construction Projects – An Ontario Shale Rating System.** Research Report RR229, Ontario Ministry of Transport and Communication, 99 pp.
11- Bieniawski, Z. T., 1984. **Rock Mechanics Design in Mining and Tunneling**. A. A. Balkema.
12- Barton, N. M., Lien, R., and Lunde, J., 1974. **Engineering Classification of Rock Masses for the Design of Tunnel Support.** Rock Mechanics 6, 1974, pp. 189-236.
13- Romana, M., 1985. **New Adjustment Ratings for Application of Bieniawski Classification to Slopes**. International Symposium on the Role of Rock Mechanics, Zacatecas, pp. 49-53.
14- Hudson, J. A., 1992. **Rock engineering systems: theory and practice**. Ellis Horwood, Chichester.
15- Ali, K. M, and Hasan, K., 2002. **Rock Mass Characterization to Indicate Slope Instability at Bandarban, Balgladesh.** A Rock Engineering Systems Approach. Environmental & engineering Geoscience, vol. VIII, No. 2, pp. 105-119.
16- Pierson, L. A., and Van Vickle, R., 1993. **Rockfall Hazard Rating System – Participants' Manual**. FHWA Report – FHWA-SA-93-057, 102 pp.
17- Maerz, N. H., 2000. **Highway Rock Cut Stability Assessment In Rock Masses Not Conducive to Stability Calculations**. Proceedings of the 51st Annual Highway Geology Symposium, Aug. 29-Sept. 1, Seattle, Washington, pp. 249-259.
18- Maerz, N. H., and McKenna, S., 1999. **Mobile Highway Inventory and Measurement System.** Transportation Research Record No. 1690, pp. 135-142.
19- Maerz, N. H., Youssef, A., and Xiang, Q., 2003. **Digital imaging for screening and making measurements of features on highway rock cuts.** Transportation Research Board, 82nd Annual Meeting, Washington, D.C., Jan. 12-16, 2003, 20 pp.

Calibration and Accuracy of Rockfall Simulation Programs

Robert J. Watters, Kurt W. Katzenstein, and Weston A. Thelen. Department of Geological Sciences, MS 172, University of Nevada, Reno, NV 89557

ABSTRACT

The calibration and accuracy of rockfall simulation programs CRSP (CDOT) and RocFall (Rocscience) were compared to observations and measurements of rockfall taken inside the Mount St. Helens crater. The interior of the crater provides an excellent opportunity to obtain plentiful "real" rockfall data as input to simulation programs. The 1980 eruption crater has rock slopes between 45 – 60 degrees over 550 m (1800 ft) high and talus slopes at 27 – 40 degrees and over 330 m (1000 ft) high. The crater slopes consist of dacitic dome rocks, andesite lava, and pyroclastic flow deposits. Almost all the rockfall produced originates from the andestic-dacite-basaltic lava and pyroclastic flow rocks overlying the older domes.

Since crater formation, the crater flanks have undergone small and large failures through copious and dramatic rockfall events. Rockfall varies from individual boulders to aggregates of boulders comprising all size ranges with volumes up to about 20 cubic meters. Thermal rock expansion appears to be the trigger for rockfall initiation. In the morning sunlight, east facing slopes undergo almost continuous instability whereas other slopes with different slope aspects are essentially silent and rockfall free. By late afternoon rockfall is occurring from all faces. Rockfall trajectories are clearly identified from sequential down slope rock impacts on talus slopes over 600 m in length. Data collected included; slope length and height, slope roughness, rockfall bounce length, rock shape, and rock volume. Results differed in comparing the output from the two computer programs, although both were good in predicting rockfall run-out for similar input.

Introduction

Research for other projects on the volcano required assessing the risk posed from rock fall to people working on and beside the dome, near and at the toe of the headwalls, and at the breach within the crater. The projects involved rock sample collection, rock discontinuity assessment, and deployment of micro-seismic recorders (MSR's). It rapidly became apparent that rockfall is related to slope aspect with the east facing slopes producing the first day's significant rockfall after receiving early morning sunlight. By late afternoon rockfall was produced from all sides of the crater in dramatic fashion, with rock accumulation surrounding the crater dome, south, east, and west headwalls. As rock fall trajectories were clearly identified from rock-slope impacts on the slope, a side project was initiated to correlate the observed rock fall trajectories with trajectories generated by CRSP and RocFall the two most common rockfall simulation programs and compare the simulations with actual rockfall.

The crater was formed at Mount St. Helens volcano as a result of the May 18, 1980 eruption (Fig. 1). The eruption was a catastrophic event and was watched and captured by eyewitness photographs. The edifice (summit cone) failed as a complex rockslide comprising three slide blocks which was accompanied by a lateral blast and emplaced 2.5-km^3 (0.6 mi^3) of debris landslide-avalanche deposits (Voight et al, 1983; Glicken, 1996). The summit elevation of 2950 m (9677 ft) changed to 2550 m (8365 ft).

Figure 1. Mount St. Helens Crater and dacite dome (copyright Smith-Western Co.)

The crater is breached to the north as a result of the lateral blast and catastrophic landslide-avalanche. The crater slopes were initially over-steepened due to the depressurizing of the cryptodome which produced small explosive failures and debris slides after the failure of the last slide block. The andesite-dacite volcano is presently in a semi-dormant state since the eruption; with the dacitic dome not experiencing any measurable growth since 1986.

Crater Morphology

The crater has a diameter of approximately 2 km east-west and 4 km north-south, and is breached completely on its northern flank (Fig. 2). The true summit of Mount St. Helens has an elevation of 2550 m (8365 ft) that forms the southernmost highpoint of the crater headwall which continues to the east and west forming steep crater walls. The snow/ice/rock debris covered crater floor has an approximate maximum elevation of 2000 m (6560 ft) south of the dacite dome below the summit. The crater floor slopes downwards to the north with the floor elevation dropping on both sides of the dome to a low of about 1800 m (5900 ft). Headwalls on the south, east, and west have rock slopes from 45 – 60 degrees with the southern most headwall almost 550 m (1800 ft) in height. Extensive talus slopes at angles varying from 27

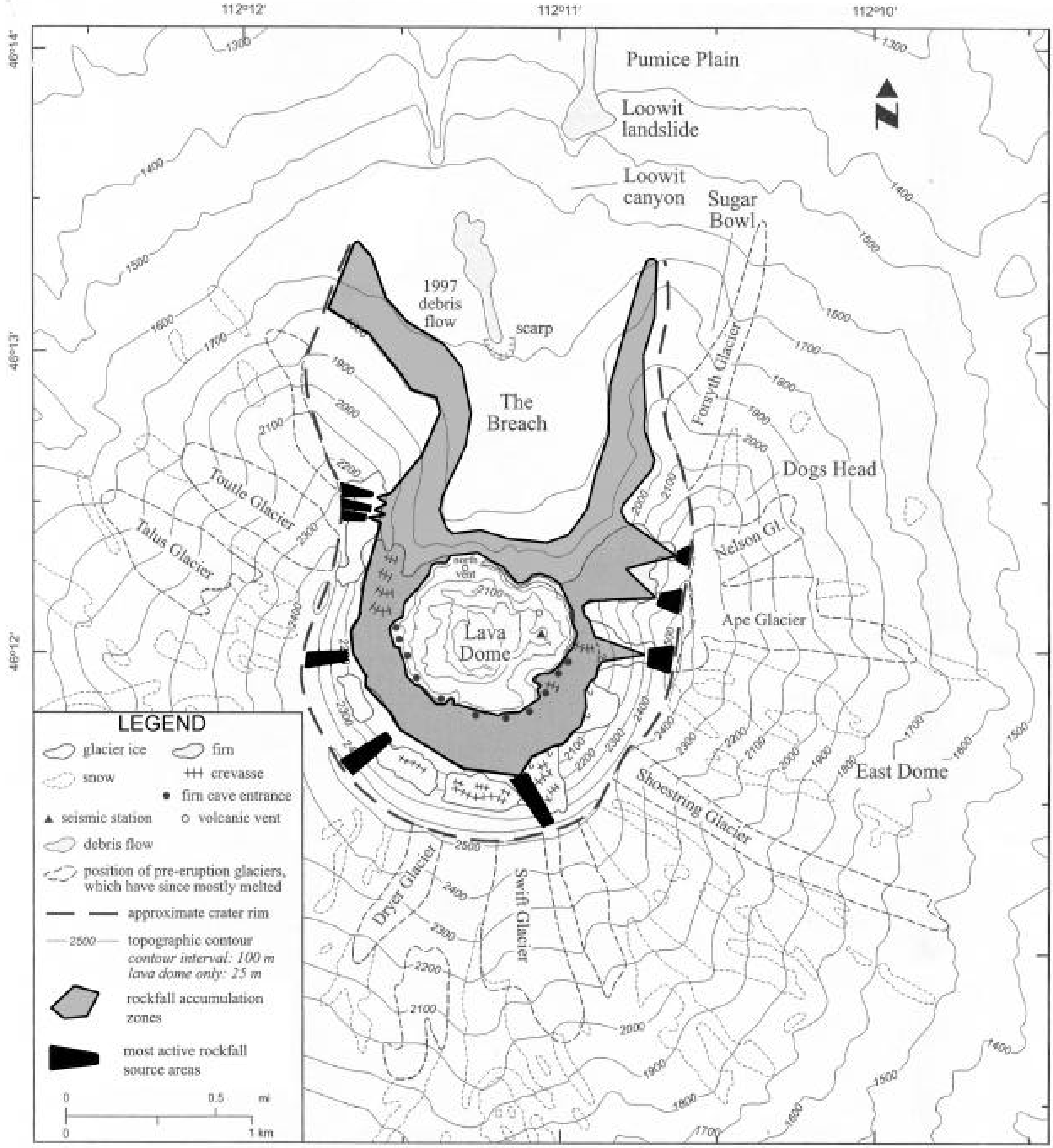


Figure 2. Topographic map of Mount St. Helens showing crater rim, dome, debris flow, rockfall accumulation zones and active rockfall areas (base map from Anderson and Vining, 1999).

- 40 degrees and over 330 m (1000 ft) exist on the crater slopes developing from debris flows and rockfall from dacite dome rocks, basaltic flows, basaltic andesite flows; and pyroclastic deposits. The dome is approximately 1 km wide and rises over 265 m (861 ft) above its vent and about 350 m (1137 ft) above its northern base (Swanson, 1988). A perennial snow and ice field "moat" interbedded with rockfall debris surrounds the dome on its south, east, and west sides. Summer sun illuminates the "moat" which is slowly transforming from snow and firn into a glacier with active crevasses (Anderson and Vining, 1999).

Rockfall Assessment

Ritchie (1963) performed the first systematic assessment of rockfall for highway design. The study showed that rockfall trajectory was highly dependent on slope geometry. Significant advances have been made in the 40 years since Ritchie's pioneering research. The two most significant advances developed contemporaneously, the **R**ockfall **H**azard **R**ating **S**ystem (**RHRS**) and computer rockfall simulation programs. The **RHRS** (Pierson, 1991) is a numerical method (although somewhat subjective) of assessing risk at rockfall sites, which rank sites according to the level of perceived rockfall risk. The rock cuts which have the greatest risk can be studied by two rockfall simulation programs, CRSP and RocFall. The programs have manuals and/or help programs which contain theoretical and rockfall information and should be consulted for additional details (Jones et al, 2000; Rocscience, 2002). Both programs permit the trajectory behavior of falling rock to be investigated. The programs permit graphical interaction with the rock/soil slope to be observed.

Rockfall Behavior

Numerous factors influence rockfall behavior. These include slope geometry parameters of inclination, length, slope surface variability, and surface roughness. The interaction of the falling rock with slope irregularities is the most important factor, after slope inclination, in determining rockfall behavior. The slope surface roughness (irregularities) changes the rock impact angle with the slope surface and determines bounce characteristics. Slope material properties affect the rock rebound behavior from the slope and are represented as a normal coefficient of restitution (Rn) and the tangential coefficient of frictional resistance (Rt). Frictional resistance causes energy attenuation and reduces motion parallel to the slope. Rock elasticity governs the motion normal to the slope (rock bounce).

The rockfall dimensions play two important roles. Rock size relative to the slope roughness is important, where the roughness may reflect slope "grain size" of boulders, cobbles, etc. of slopes composed of unconsolidated deposits. Bedrock slope roughness reflects joint spacing, benches, blasting procedures (highway slope), and detritus on the bedrock. Rock shape (e.g. round rocks, tabular rocks, or columnar shapes) produces different rock behavior characteristics as they fall down slope. Lastly, rock strength or durability is critical as to whether rock breaks apart on impact with the slope and also influences rock rebound.

Program Assessment

Program Similarities:
Neither of the programs utilizes values from the **RHRS**, although important differences exist in how the programs input details on falling rock. Slope input geometry is similar and both programs can assess energy, velocity, and bounce height for the entire slope with specific graphical output. A falling rock is assumed to stay intact and does not break into more rocks. Both programs show rockfall trajectories, horizontal extent of rock movement and rockfall statistics including bounce height and velocity.

Colorado Rockfall Simulation Program (CRSP):
CRSP models surface irregularities by randomly varying the slope angle between limits defined by the rock size and surface roughness. The falling rock can have three different shape types; spherical, cylindrical, or discoidal. The dimensions and density of the rock can be imputed. Rock friction can not be imputed. Graphical zooming on selected portions of the trajectories is not possible.

RocFall:
This program differs from CRSP in the types of material properties input data and the statistical analysis. Additionally, it can determine types of remedial measures. The shape of the rock cannot be changed and is assumed to be a circular boulder, the worst situation. The density of the rock is not considered in the analysis although the rock weight is utilized. Rock friction is an input parameter, though it is not used to calculate bounce characteristics, it is used only when the rock is sliding; consequently a rolling or "skipping" rock can be modeled. Different rock material parameters can not be inputted for various types of rock. Different rock behaviors can be assessed by increasing or decreasing the standard variation of the material. Energy loading of a barrier in the toe region of the slope can be calculated and an assessment of the mitigation made for highway design.

Field Work

Field work was performed in late July and August of 2002 and 2003. Observation of rockfall was made from sunrise to sunset on the east, south, and west headwalls and rockfall was sequentially photographed and videotaped by digital equipment from safe locations within the crater. Rockfall initiation and impact locations on the slope, toe, and where movement ceased were surveyed by laser measuring equipment and from its relationship to mapped topographic features. Individual rockfall at rest (when safe to do so) was dimensionally measured, rock type assessed, and Schmidt hammer rebound values obtained.

Results

Rockfall photography:
Inspection of digital and video photography showed that two major types of rockfall source areas existed. The first source type is from block and ash flow and pyroclastic materials which contain

a range of sizes (gravel – boulders) in an ash (sand size) matrix. These falls produce large dust clouds and numerous small sized rock fall (Fig. 3). This type of event often failed to show

Figure 3. West crater side, pyroclastic rockfall produces large dust cloud, slope height 455 m (1600 ft).

Figure 4. East crater side, stronger rock produced excellent rockfall slope impacts.

Figure 5. Numerous rockfall trajectories reflect the curved nature of talus topography.

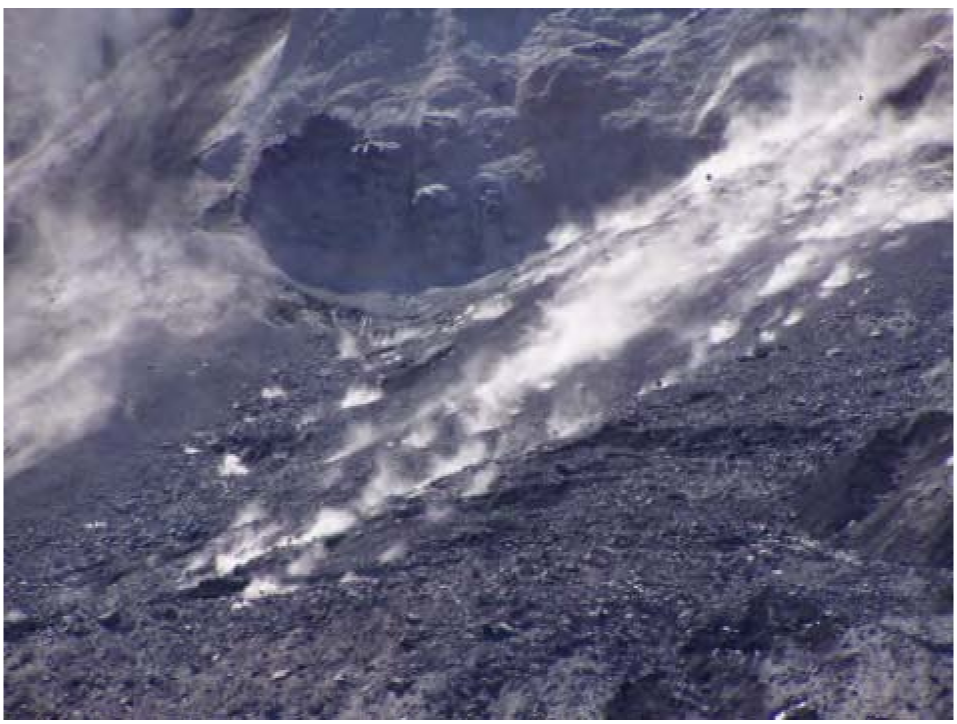


Figure 6. Individual rockfall boulders can be identified as the lower slope is inundated.

significant rock fall in the accumulation area as the individual rocks were fragmented as a result of rock to rock interaction or from slope contact. The second source appeared to be from stronger rock; andesite-dacite lava in a weaker matrix of finer grained materials. Individual rockfall trajectories are clearly identifiable from this source type but still produce dust clouds (Fig. 4).

The more dramatic aspects of the rockfall events are shown in the photographs of Figures 5 and 6. Figure 5 illustrates numerous rockfall appearing to emanate from a single source, analogous to a firework display starburst. Closer inspection and observation of other similar events suggest it is a result of either triggering movement of other rock in the slope and/or rock fragmentation of an individual boulder. Individual rock trajectories and bounce distances are identifiable and measurable. Figure 6 contains the lower portion of a rock slope impact accumulation zone. Individual rockfall shape, trajectory, and bounce characteristics can be ascertained.

Rockfall material characteristics:

Rockfall dimensions measured varied from cobble to boulder sized fragments, with the largest boulder measuring 2 x 3 x 3 m. Individual rockfall weight between 200-1000 kg, although occasionally some were as high as 20,000 kg. The majority of the rockfall measured was angular to sub angular reflecting the volcanic nature of the in situ deposits with rockfall occurring from all rock types, andesite, dacite, basalt, cinder, block and ash fall, and pyroclastic materials. Specific gravities of these materials varied from between 1.1–2.4 g/cc. Schmidt hammer tests on unaltered boulders had ranges in UCS for the andesite-dacite of 8–12,000 psi and tangent modulus of 5-7 x 10^6 psi, and UCS of 11-16,000 psi and tangent modulus of 6-9 x 10^6 psi for basaltic-andesite.

Rockfall behavior:

To assess the rockfall behavior predicted from both programs, similar values for the normal and tangential components and rock weight were inputted. Other factors which were used by only one program like rock shape (CRSP), rock dimensions (CRSP), or rock friction (RocFall) could not be directly compared between programs. Limited field data on rock height bounce meant that this parameter was not compared. The rockfall run-out (accumulation zone) simulation was the parameter which affected safety; consequently the accuracy of this simulation was paramount.

A slope profile and rockfall trajectories for similar slope geometries and rockfall input parameters are shown in Figures 7 and 8. Both figures are comparable, although RocFall allows actual elevation to be plotted whereas CRSP only permits elevation to be plotted from a common datum. Results are similar in assessing rockfall run-out distance. Rock shape does affect the run-out distance (CRSP), though circular boulders were used for a “worst case” situation. Slope roughness varied on the slope with the lowest values in the slope mid-portion and the highest value in the slope toe region, reflecting large at rest rockfall. The high slope roughness combined with low slope angles rapidly attenuated the rockfall.

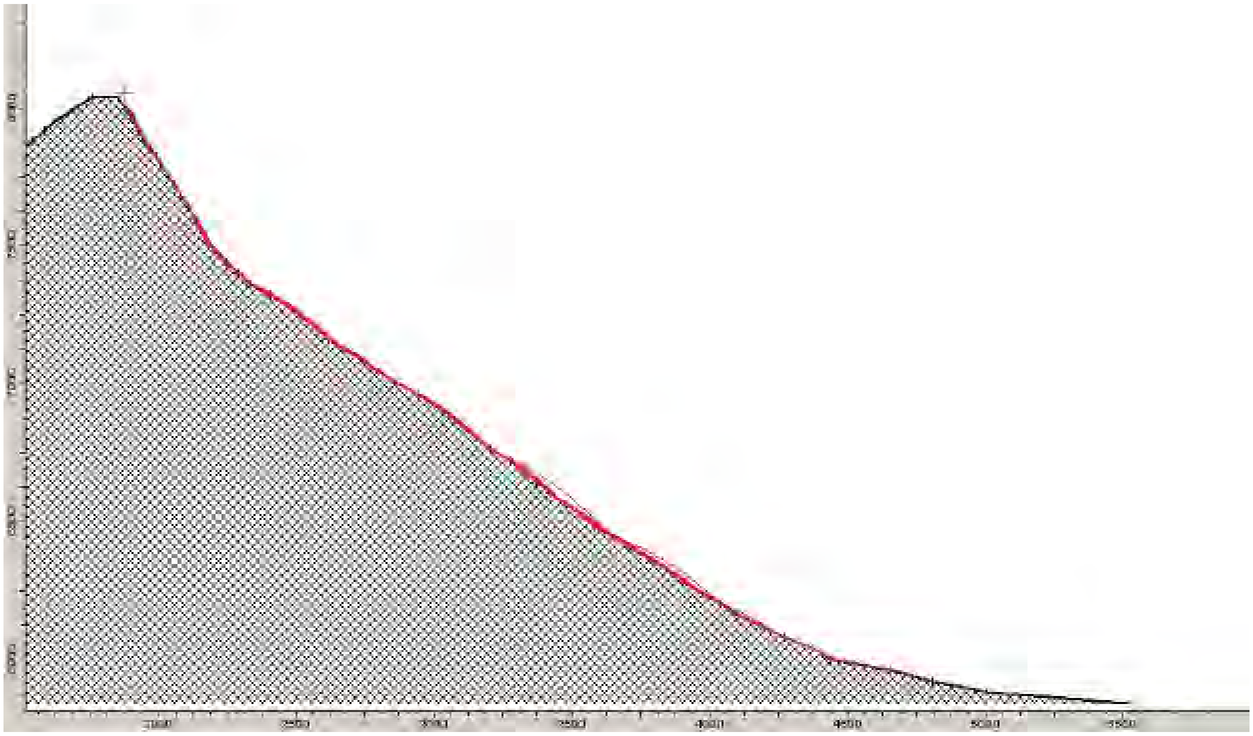


Figure 7. RocFall plot of slope profile, true scale in feet, showing rockfall trajectories.

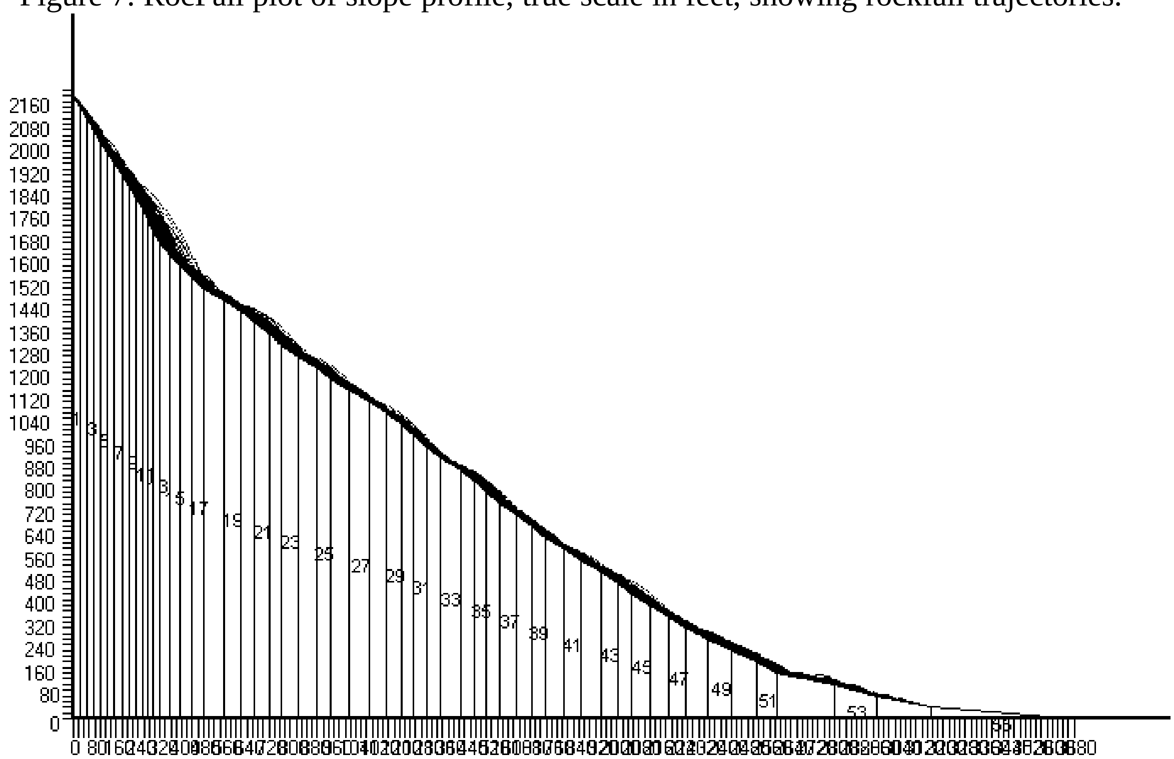


Figure 8. CRSP plot of slope profile, normalized true scale, in feet, showing rockfall trajectories.

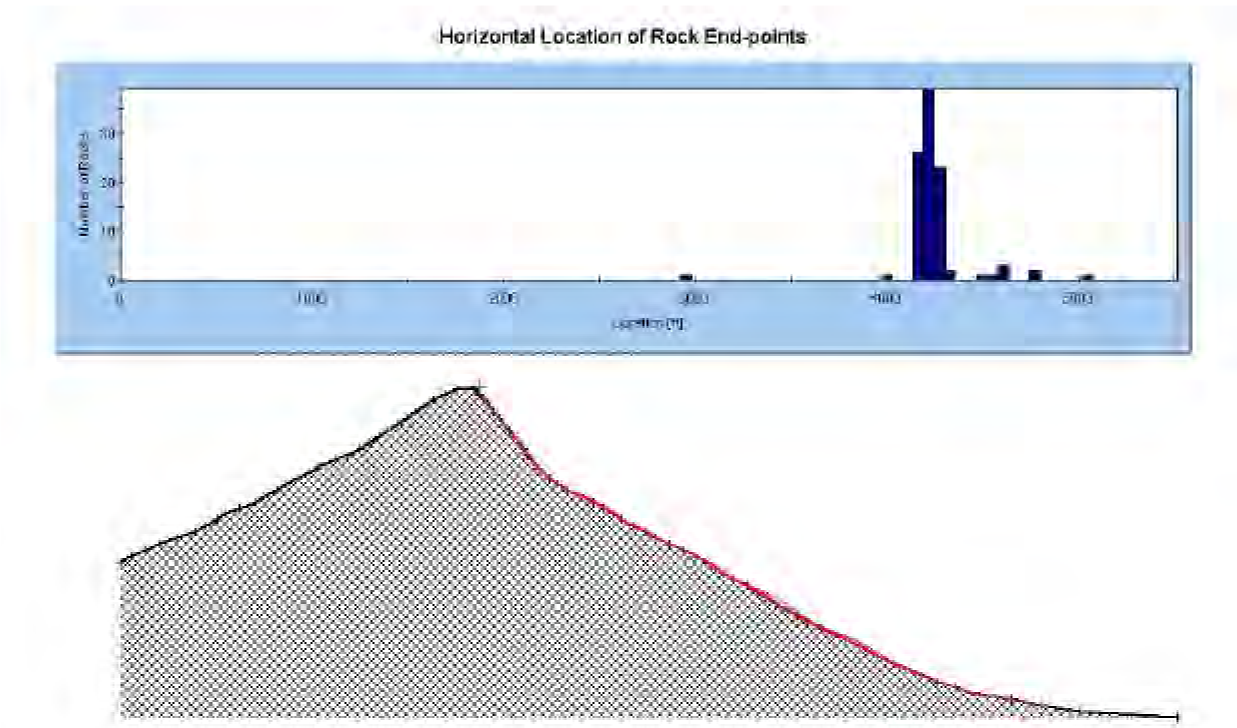


Figure 9. RocFall output illustrating crater rim, slope profiles, rockfall trajectory and rock end-points (rockfall cessation locations).

Both programs provide excellent output and statistics for assessing the run-out distance for individual rockfall. RocFall output is presented in a graphical form (Fig. 9), whereas CRSP output is tabular.

Conclusions and Recommendations

Field work was safely performed inside the crater. The use of the rockfall simulation programs enabled work to be carried out in a potentially hazardous environment, risk free. This was possible as both programs showed that rockfall run out and accumulation zones were comparable to those observed from actual rockfall events. However, given the large scale of Mount St. Helens, slopes over 600 m in vertical height, and the large number of daily rockfall events, a significant buffer zone (100 m plus) was added to the rockfall accumulation zones for safety.

The programs were not ideally suited to our application as the vertical slope heights were over 600 meters. Highway slopes generally have heights of between 25 – 100 meters. The RocFall program has excellent graphical and statistical ability, with the benefit to zoom in on specific portions of the slope. CRSP by comparison, is significantly more limited in graphical capability, although the statistics in both programs are comparable, with RocFall albeit presented differently. The ability to input different rock shapes appears to be less critical than was first thought for our specific application. This possibly reflects the exceedingly high slopes and long rockfall trajectories. Given the long rockfall distances terminal velocity was found to vary from observing individual rockfall. Rockfall events from an initial “single” event, produced many individual trajectories with similar velocities, as can be seen in Figure 5. However, rogue boulders within the rockfall cluster could be seen moving at velocities significantly faster that other boulders. The rogue boulders often had movement mannerism different than those boulders surrounding them. The boulder movement mannerisms often appeared to “skip” down-slope, analogous to children “skipping” flat stones at the lake or sea shore.

Mount St. Helens provides an excellent natural laboratory for rockfall study. Our preliminary investigation shows that rockfall source areas within the crater are plentiful and can be monitored safely. DEM combined with more numerous digital imaging equipment would provide improved coverage of rockfall and consequently better characterize the slopes. Small-scale slopes (100-200 m high) could also be studied which would be more appropriate for highway slope rockfall.

Acknowledgments and Hazards

This work was partially supported by NASA grant NAG5-9498, and NSF grant EPS 0132556. Many thanks are extended to the personnel of the US Forest Service and Mount St. Helens National Volcanic Monument from which an access permit is required to conduct research within restricted areas at the Monument. Caution must be exercised at all times within the crater as a consequence of the objective dangers of rock fall, crevasses, and the possibility of a phreatic eruption from the dome.

References

Anderson, C.H, and Vining, M.R. (1999). Observations of glacial, geomorphic, biologic, and mineralogic development in the crater of Mount St. Helens, Washington. Washington Geology, vol. 27, No. 2/3/4, pp 9-19.

Glicken, H. (1996). Rockslide-debris avalanche of May18, 1980, Mount St. Helens volcano, Washington. USGS Open File Report, No. 96-677, p.90.

Jones, C.L, Higgins, J.D, and Andrew, R.D. (2000). Colorado rockfall simulation program, user’s manual, version 4.0. Colorado Department of Transportation, Denver, Colorado, p. 127.

Pierson, L.A. (1991). The rockfall hazard rating system. In Highway and Railroad Slope Maintenance, Hambley, D.F. (editor), Association of Engineering Geologists, Symposium Series No. 6, pp. 1 – 22.

Rocscience, (2002). Rocfall – Risk analysis of falling rock on steep slopes, program and user’s manual, version 4.0. Rocscience, Toronto, Canada, p.59.

Ritchie, A.M. (1963). The evaluation of rockfall and its control. Highway Research Record, National Academy of Sciences-NRC, Washington, DC, No. 17, pp.13 – 28.

Swanson, D.A. (1988). Volcanic activity at Mount St. Helens since May 1980. Kagoshima International Conference on Volcanoes, Kagoshima, Japan, pp 33-36.

Voight, B, Janda, R.J, Glicken, H, Douglass, P.M. (1983). Nature and mechanics of the Mount St. Helens rockslide-avalanche of 18 May 190. Geotechnique, No 33, pp. 243-273.

ROCK SLOPE FAILURES: A LEGAL CASE HISTORY

By: Joseph A. Fischer and James G. McWhorter: Geoscience Services

ABSTRACT

In March 1995, a local soils engineer (***1***) was hired to investigate foundation conditions for a large building to be placed over deep glacial lake sediments. A roadway and parking lane was planned between the building and a 335-meter (1,100-foot) long by 21- to 28-meter (70- to 90-foot) high cliff. Another 300-meter (1,000-foot or so) long by some 15- to 21-meter (50- to 70-foot) high cliff extended along a proposed parking area. As a result, an extensive cut into reddish brown shales, siltstones and mudstones of the Triassic/Jurassic-aged Passaic Formation was required.

As project planning continued, the soils engineer was asked to develop an estimate of a safe slope angle for the cliff as the roadway continued to encroach upon its base. The Municipality's geotechnical consultant (***2***) and the site engineer developed an investigation protocol with two test borings, site visits and perhaps, bedding dip measurements. The results were included in a "Cliff Geologic Investigation".

During construction, the cliff was steepened and rock falls occurred. Another geotechnical firm (***3***) was hired. They looked at the cliff, but apparently did not map it, and selected grouted rock anchors (some tensioned, some not) for remediation. Netting and bolting were installed by an experienced contractor, albeit with some difficulties. Field inspection was provided by the designers with oversight by the Municipality.

Additional minor failures occurred and two additional consultants (***4*** and ***5***) were hired to review the situation. Both recommended a retaining wall. One firm (***5***) was hired to develop the design with, again, review by the Municipality. *The cliff face remained unmapped*. About 185 meters (±600 feet) of rock face now has a sheet pile and lagging wall. Minor rock falls and weathering continue outside the walled area and lawsuits abound. Next, the first design firm (***4***) was eventually hired to review the original soils engineer's (***1***) work.

The authors' firm (***6***) was retained by the soils engineer's insurance company. The interplay between the owner, designer, constructor, site engineer, Municipality, and various geotechnical consultants was a roadmap for the ensuing lawsuit. As discussed in the paper, the lessons learned are numerous; one of the most important is that the state-of-the-practice is not up to the state-of-the-art in rock slope stability evaluation.

INTRODUCTION

As first contemplated in early 1995, a huge 300- by 100-meter (1,000- by 325-foot) warehouse was planned for fast-track construction on a large tract bordered by wetlands on one side and a 21- to 28-meter (70- to 90-foot) high, vegetated cliff on the other. A picture of the cliff, chain link netting, and adjacent roadway is provided as Figure 1. The corrugated PVC shown on Figure 1 handles cliff-top drainage. The major concern was how to support the warehouse and associated paved parking "field" across a subsurface that encompassed fill, colluvium, residual soils and glacial lake deposits. A local soils engineer (***1***), well-versed in similar geotechnical environments, was hired by the design/build contractor in conjunction with the site engineer. The choice was rational in that the firm was experienced in supporting structures over local glacial lake deposits and was not known for inflated invoices.

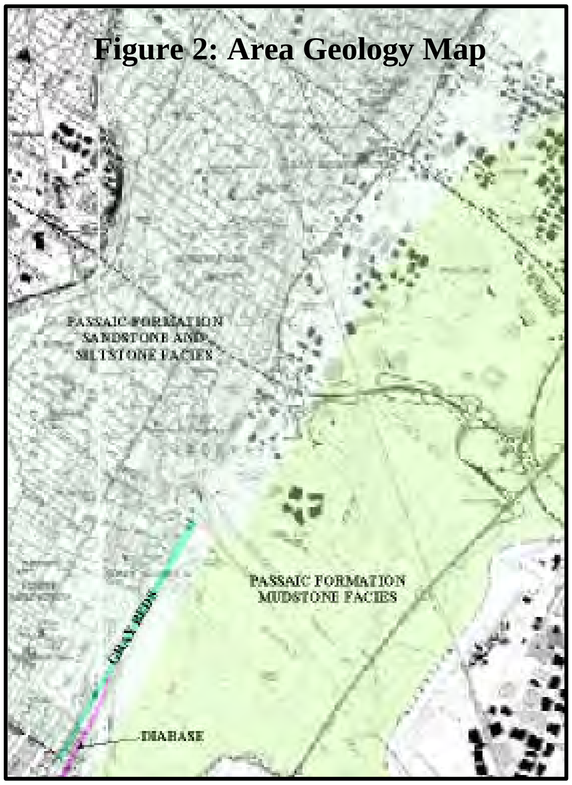
Figure 1: Cliff and Roadway.

Various environmental, space, and building code restrictions began to intrude upon the available space planned for the roadway areas between the proposed structure and the cliff. As the project changed in nature, the involvement of the original players changed, the geotechnical problems increased, and a number of new firms arrived on the scene; followed in order, by rock falls, remedial efforts (as a result of the rock falls), new owners, more rock falls, then attorneys.

GEOLOGY

The site is located within the glaciated portion of the Triassic Lowlands, just west of New York City, and on the periphery of the Hackensack Meadows (see Figure 2, Area Geology Map). The cut slope essentially marks the natural boundary formed by the more resistant sandstone and minor siltstone facies (to the west) with the less-resistant mudstone facies (to the east) of the Passaic Formation. That is why there is a natural cliff trending southwest and northeast of the site for a considerable distance. Parker (1993) described the stratigraphic relations between the four mappable lithofacies of the Passaic Formation below the first of the Watchung Mountains, the Orange Mountain Basalt.

Figure 2: Area Geology Map



Sandstone and minor siltstone facies actually make up the bulk of the cut slope. It best can be described as a series of interbedded greenish-gray to grayish-red, medium- to fine-grained, medium- to thick-bedded sandstone with minor brown to purplish-red coarse grained siltstone The unit described as "green sandstone" within the cut slope in several reports reviewed by the authors actually forms the resistant unit within the cut slope, with fissile siltstone occurring below it. This sandstone layer is the "green line" of Figure 3.

Bedding strikes about N35°E and dips about 5° northwest, with the principal joints striking west-northwest dipping near vertical, and N45-50°E dipping 75° to 80° to the southeast. A secondary set of joints strike about east/west, dipping 70° to the southeast with zeolite mineralization in the joint surfaces. The west-northwest set and the N45°E set dipping 75-80° southeast intersect on the face of the slope which, along with the bedding orientation, forms the general boundaries of the wedge-shaped blocks that have failed after formation of the cut slope. An example of this is shown on Figure 1; the cut slope orientation is about N30°E. For comparison, see Figure 3.

Lessons to be learned are:

- Gather and analyze the available geologic information for the site. The site geologic references quoted in the firm (***6***) report (2003) for the attorneys had never been reviewed by any of the five geotechnical firms involved.
- Get a qualified engineering geologist involved early in the project.
- Use borings (continuous soil and rock sampling) to provide coverage of subsurface conditions behind the rock slope face.
- Map and understand the geology as exposed in the slope face.
- Map and analyze the discontinuities present in the slope face **prior to** making any design decisions regarding slope stability.
- All joints are not necessarily vertical and not all wedge failures are like those pictured in textbooks.
- Don't confuse "global stability" issues of soil slope failures with issues related to rock falls.
- Understand the difference in the effects of basic Newtonian forces and weathering on rock slope stability.
- Evaluate the effects of water within the rock slope face and the need to divert or drain the water from the rock mass to increase the stability of the rock slope.

MUTATIONS AND PERMUTATIONS

IN THE BEGINNING

The project changed in nature as the building was forced closer to the cliff by environmental constraints related to the wetlands. To maintain the desired width of the roadway, it now appeared necessary to cut the slope back somewhat and the soils engineer (who had performed a bare-bones, but essentially successful foundation design study) was asked for an opinion as to what the slope angle should be. Upon the basis of his lack of rock mechanics knowledge, he should have refused the work or qualified his opinion heavily. He did neither and was paid a small sum for the opinion expressed in a short (less than one page) letter recommending a slope design of 6 (Vertical) to 1 (Horizontal) without a disclaimer. The time is now April 1995 and it all starts moving downhill.

At this time, it appears that the soils engineer was primarily interfacing with the site engineer, with the authorization of the construction firm running the project for the owner. Payment to the soils engineer for this work was by the owner after a review and approval of the site engineer and constructor. Communication between the interested parties was diminishing.

The first response by the soils engineer, that the face could be cut at 6 V to 1 H, was quickly superceded by a 3.5 V to 1 H suggested cut after inspection of several (at least two) cut slopes in the general vicinity of the site in question.

The lesson to be learned is:

- **Stay within your field of knowledge.**

One of the inspected cuts had been evaluated by the authors' firm (***6***) in 1994. That cut was a 4.5 V to 1 H (maximum), 15- to 22-meter (50- to 70-foot) high cliff with heavy netting bolted into the rock at the top of the slope and a low, "Jersey Barrier" type wall for rock fall protection. Rock bolts had been recommended for several potential weak zones (to prevent wedge failures) identified by mapping and analyses (under gravity and seismic loading). The owner of that slope has not yet put in the recommended rock bolts, but minor maintenance has been performed. Rock "fall and bounce" tests were run at the site and later checked using an early version of the Colorado Rock Fall Simulation Program (Colorado Geological Survey, 1995). Another slope visited by the site engineer was at an adjacent waste transfer station. The only protection was rock bolting and netting behind a generator pad. The slope looked to be cut at about 5 V to 1 H and appeared to be in good shape. About a ¼-mile of slope, lined with industrial buildings, existed along the entrance road to the warehouse site, all without any rock fall protection.

Thus, the 3.5 V to 1 H recommendation appeared reasonable and a preliminary recommendation of a 2-meter (6-foot) high fence to handle talus was given. About this time, the Municipal engineer evidenced concern about cutting the slope back and requested assistance from a geotechnical engineering firm (***2***) that had a long work history with the Municipality and their engineer. The same civil engineer/geotechnical consultant had oversight of the aforementioned project that the writers' firm had worked upon probably six months prior to the project in question. Shortly, the proposed roadway encroached even further into the cliff as a result of the need to maneuver large emergency vehicles between the cliff and the warehouse and the Municipality obtained permission to drill borings on two properties at the top of the slope.

The site engineer negotiated the new scope of work with the Municipal geotechnical consultant. Two test borings were to be drilled by a local drilling firm. Conventional auger holes to rock hard enough to core (about 6 meters [20 feet] below the surface) with Standard Penetration Testing (SPT) blow counts in soil to 3.7 meters (12 feet) in one hole. A single-tube, Nx-size core barrel was used to sample the rock. Core recovery was reportedly good, with a Rock Quality Designation (RQD) greater than 50%. The soils engineer was hired to make a short inspection during drilling operations and eventually log the core. The site engineer also provided drilling inspection services. Subsequently, a question that remained unresolved arose about the number of core runs actually made.

A report, apparently coauthored with the site engineer (although signed only by the soils engineer), was submitted in August 1995. The soils engineer's invoice was $1,425; the site engineer's invoice was $6,500 for the work. That August report represented the soils engineer's last involvement with the project.

<u>Lessons to be learned are</u>:

- Stay within your field of knowledge (again) <u>and</u> be aware of what you do <u>not</u> know.
- Looking at nearby examples of cut slopes is good, but is much more informative if some engineering geologic knowledge is used in the comparison.
- Prepare reports that clearly present any recommendations and assumptions.
- Write defensively with an understanding of your future role in a project.
- Developing and maintaining a good relationship with a client does not mean that one should not question project scope and submit low-cost invoices.
- Do not accept work that does not allow a reasonable amount of time for completion.
- Typical foundation test boring practices generally do not tell you enough about a rock subsurface to perform an appropriate slope stability evaluation. Consider oriented core or television logs, the use of split double-tube core barrels, geophysical logging, etc. to obtain 3-dimensional information on rock discontinuities.
- Get adequately paid for what you do.
- Low budget projects can create high budget insurance premiums.

<u>THE FALL FROM GRACE</u>

At this point, the roadway was encroaching even further upon the cliff face and the earlier 3.5 V to 1 H slope recommendation became 3.5 V to 1 H <u>or greater</u> in the August 1995 report. The soils engineer (***1***), in deposition, stated that he thought "or greater" meant some small zones could be cut at slightly greater than the generic 3.5 V to 1 H during construction. Some large areas of the slope were in the 5 V or 6 V to 1 H range. During deposition, the site engineer said that he thought the 6 V to 1 H recommendation in the earlier report still held when he changed the slope to meet design considerations. The soils engineer said that no one informed him of any changes. The site engineer indicated that he would not have made changes without input.

<u>The lesson to be learned is</u>:

- There are no friends in a lawsuit situation. The most you can hope for is a modicum of integrity.

The soils engineer, site engineer and constructor seemed to be in general agreement that no remediation or slope protection needed to be considered until the cliff face could be examined after construction. Aside from economic considerations, this was likely a reasonable technical decision because of the existence of so many stable cuts into the same bedrock both adjacent to the site and along a major nearby highway, as well as the relatively overgrown nature of the slope itself. However, it is obvious that some preliminary mapping of the face could have helped conceptual remediation design and overall construction planning.

Work on the cut started in the fall of 1995, stopped for the winter, and did not start again until June 1996. A hoe-ram was used for excavating; blasting was not employed at any location.

As the cut progressed, minor rock falls and, of course, some weathering occurred. Technical inspection by the owner or constructor was not provided other than occasional observations made when the site engineer's representatives were visiting for other reasons. The Municipal geotechnical consultant monitored the cut. In August 1996, a large piece of reportedly wedge-shaped rock about 10 meters (35 feet) long, up to some 3 meters (10 feet) wide and 1-meter (3 feet) deep fell. This large rock fall heightened the Municipality's concerns about slope stability.

As some environmental work was ongoing at the site, the constructor hired his environmental consultant (***3***), which advertises itself as having geotechnical/environmental expertise, to evaluate cliff stability. The constructor made the decision that the soils engineer was not suitable to handle design and remediation within the current environment of concern. Consultant (***3***) apparently <u>looked</u> at the slope from a crane-held box for about a week. This "mapping" effort resulted in establishing the location of the "green line", the location of the rock falls to date and the "approximate locations of primary discontinuity system". A section of the "Rock Slope Reinforcement Drawing" showing the results of that "mapping" effort are provided as Figure 3, Rock Slope Reinforcement Section. Bedding dip and direction are not provided, lithology is not recorded, the joints and the variations in bedding plane thicknesses are not noted, the apparent zeolite filling within many joints was not recorded, and the water carrying potential of the bedding partings (which can be seen evidenced by the icicle collections on Figure 4) was not reported in either the drawing or the associated reports.

The "green line" was a resistant sandstone bed. "Slope Stability" failures were believed to occur only below this "green line". The authors are not privy to the rationale behind the analysis used to design the rock bolts, but we suspect that the bolting analyses were directed toward stabilizing blocks of rock below the "green line" in order to avoid a cantilevered sandstone layer that could fail from the weight of the rock above. In deposition, one of the firm (***3***) individuals who performed the calculations identified them as a means of establishing a "factor of safety" for *global stability* (emphasis added). The other individual involved also indicated in deposition that, although the calculations were an analysis of "global stability", they were "onion peel" failures. By this, we presume they mean toppling failure. Throughout their investigation, bolting operations, depositions and subsequently, their "expert's" reports, consultant (***3***) never realized what type of rock fall was occurring, although "onion peel" failures became "orange peel" failures during the process.

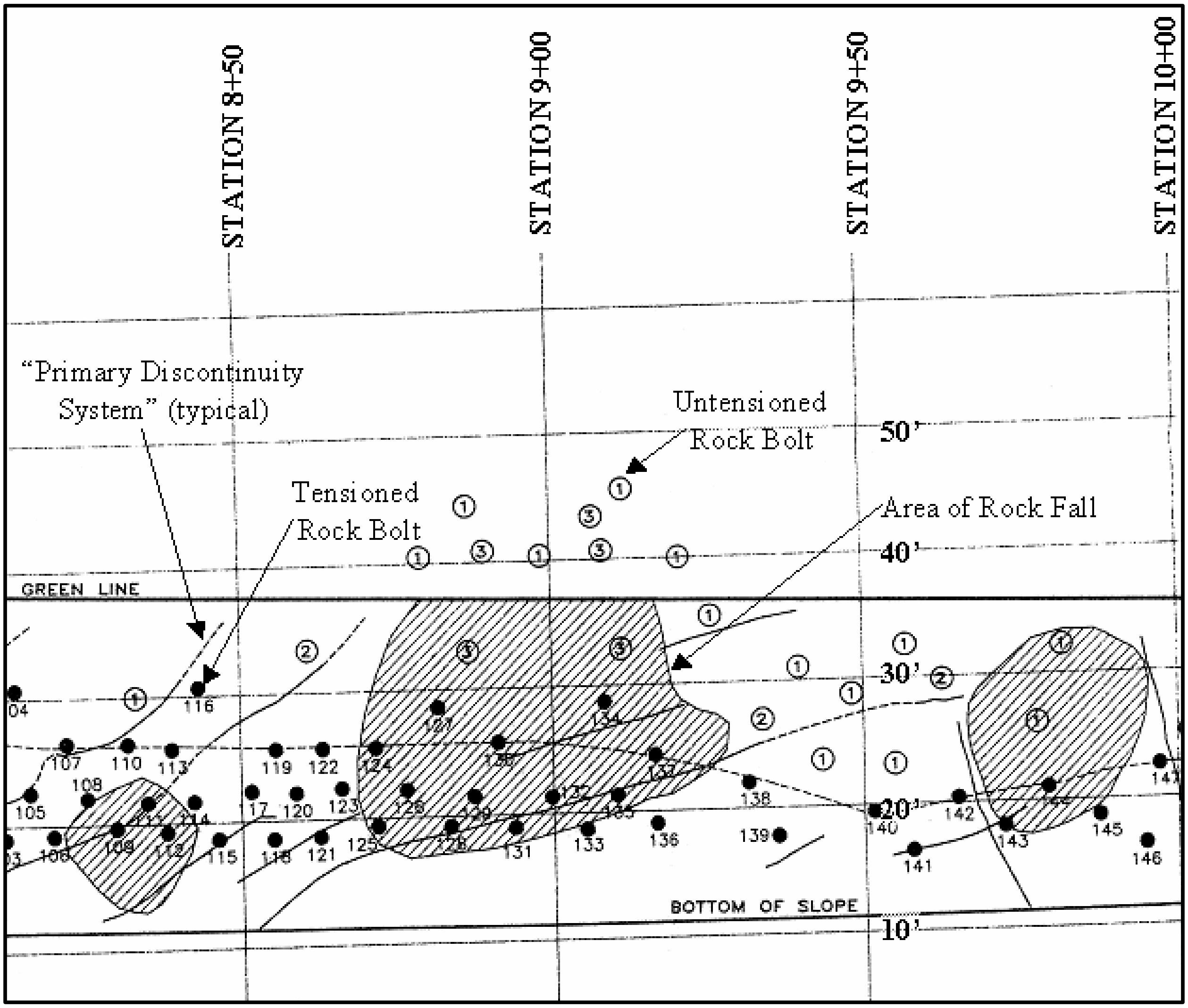


Figure 3: Rock Slope Reinforcement Section.

The hydrostatic pressure effect of water moving though bedding planes was not considered in any document available to the authors during the lawsuit.

Lessons to be learned are:

- A letterhead that says "geotechnical engineers" may be misleading.
- Read the reports and depositions of other technical witnesses, including their references.
- Wedge-shaped rock falls are not toppling, orange peel failure, onion peel failure, or accelerated weathering.

During the design process, the constructor was offered several alternate wall systems by consultant (***3***). They chose the lowest cost option, rock bolting, with an agreement to do maintenance and monitoring of the "weathering" slope.

Upon the basis of the aforementioned delineation of the "primary discontinuity system", firm (***3***) selected a number of areas where rock bolting was to be undertaken. Apparently, tensioned and non-tensioned rock bolts were used (Figure 3). Tensioned bolts were employed only where the firm thought the nature of the rock was such that tensioning of the bolts were required. The Municipal geotechnical consultant (***2***) added some 10% more in the way of bolts after inspecting the slope and reviewing the remediation plan.

Somewhere around this time, damage occurred to the building and its roof as a result of a major rainstorm coupled with the possibility of poor structural and civil design. The attorneys circled and legal action was planned.

Figure 4: Evidence of Water Movement Through Bedding Partings

A well-recognized specialty contractor installed the bolts under interim monitoring by the geotechnical/environmental firm and occasional observation by the Municipal geotechnical consultant. During the progress of the rock bolting operations, a number of problems with the set-time and/or holding ability of the resin grout occurred. The Municipal geotechnical consultant raised concerns and the rock bolts were tested. A number of bolts failed. After several additional rounds of regrouting, the test program was deemed satisfactory.

The originally estimated price for the rock bolting (about $250,000) was cut by more than half as a result of "bargaining" between the constructor and the specialty contractor. The environmental/geotechnical firm (***3***) failed to collect some $40,000 worth of invoiced charges. This firm wrote an acceptance report for the submittal to the Municipality before the final remediation work was complete in order for the owner to receive an occupancy permit, likely because at this time they were owed a large sum of money by the constructor.

Of course, inspection by firm (***3***) was never authorized and maintenance of the slope did not figure into the budget. Neither the designers nor the Municipal geotechnical consultant ever followed up on this missing link.

Lessons to be learned are:

- Cutting costs without understanding the ramifications, and as a result, performing and/or accepting a second rate job, can be very costly.
- It may not be a good idea for a consultant to allow a design/build contractor to get too far in debt for the services rendered.
- Short-term advances in the client relations gained by stating untruths can be detrimental to long-term professional reputations.
- If part of the design is an agreement to inspect and this does not happen, the geotechnical firm(s) should confirm in writing that the inspections are not being performed.

THE END GAME

It is now 1999. Rock falls continue, building ownership changed, and the current owners contacted two large geotechnical firms to suggest a means of repair. At that time, lawsuits were pending against the original soils and site engineers for the cliff as well as various contractors (including the site engineer) for roof drainage and structural problems.

Firm (***4***) was an old-line, reputable soil mechanics/civil design operation. Firm (***5***) was a geotechnical/environmental/civil design consultant. Both recommended a large retaining wall be constructed to cover the area of greatest rock fall. Firm (***5***), the lower-priced firm, was hired.

The unsuccessful firm (***4***) was later hired by the new owner's law firm to critique the original soils engineer's (***1***) work.

Meanwhile, the rock bolt designers and the Municipal engineer were added to the lawsuit. There was some discussion of enjoining the specialty contractor as well, but that apparently passed into disfavor.

Thus, involved in the geotechnical aspects of the lawsuit were: A) The original soils engineer (***1***); B) The site engineer; C) The second geotechnical/environmental consultant (***3***); and later D) the Municipal geotechnical consultant (***2***). Hired by lawyers to aid in their prosecution of the case were: E) A large soils/civil engineer firm (***4***) attacking the original soils engineers (***1***) work; and F) A small geotechnical firm (***6***) to review the degree of liability of the original soils engineer. In addition, the plaintiff wanted a soldier beam and lagging retaining wall along the entire length of the cliff, so the existing wall designer (***5***) developed a cost estimate for an expansion of the existing wall. The authors' firm (***6***) was later asked to develop a cost estimate for rock bolt and netting protection such as used at the non-failing site to the north.

Depositions abounded. Reams of paper were generated. Lawyers made a lot of money. The various geotechnical personnel for the defendant firms put in a lot of unpaid defense time. Lawyer-hired geotechnical reviewers made money, but not as much as the attorneys.

The firm (***4***) critique of firm (***1***) still did not mention wedge failures and hydrostatic effects as the only result of water contribution was weathering, "the greatest cause of rock falls at the site". The lack of slake testing on representative sections of the cliff was decried as the primary dereliction in firm (***1***)'s duties and the use of limiting equilibrium analyses to evaluate a cliff composed of quickly weathering "soft rock" such as sandstone and shale was not considered appropriate. We are not sure their minds were changed by examples from Hoek and Bray, 1981.

Three forensic engineering firms joined the fray, hired by the lawyers of sued firms (including the specialty contractor). Their work lacked geotechnical credibility and to the writers' eyes, appeared to have obvious biases. Hence, the various critiques were lacking in overall credibility.

The lessons to be learned are:

- The problems were mainly a result of the lack of technical expertise and the lack of coordination between the various consultants and contractors.
- Charge more for legal work; the anguish is worth it.
- The state of the practice in rock slope stability is not good, even by firms that have a good reputation in soil mechanics.
- Some attorneys are good, some are evil (at least evidencing a different ethic than professional engineers and geologists), and some are completely ignorant of any of the technical aspects of the work with which they are involved as it appeared they believed it had little bearing on a technical case. The authors' believe that it is useful to provide attorneys some technical information related to the subject that they are arguing. It certainly helped during the authors' involvement in this case.
- Technical people should provide the technical questions to a reasonably intelligent attorney for best use during a deposition or trial.
- It may be useful and interesting to be an expert witness occasionally, but apparently even attorneys look down their noses at forensic engineering specialists.

Eventually, the case was settled out of court. The soils engineer (***1***) was considered the least responsible in the series of errors that resulted in the lawsuit. Of greater blame, from a geotechnical standpoint, were the site engineer, the constructor, and Municipal geotechnical consultant (***2***), and the rock bolt designers (***3***). The firm (***4***) critique of firm (***1***) was not considered to be a viable attack. The authors recommend that any "geotechnical" firms doing rock mechanics or critiquing the work of other technical organizations should first go to a few Highway Geology Symposiums.

REFERENCES

1. Colorado Geological Survey, 1995. *Colorado Rockfall Simulation Program*, v. 3.0a.
2. Hoek, E., and J.W. Bray, 1981. *Rock Slope Engineering*, E&FN Spon, London.
3. New Jersey Geological Survey, 2000. *Bedrock Geology and Topographic Base Maps of New Jersey*, CD Series CD-00-1.
4. Parker, R.A., 1993. *Stratigraphic Relations of the Sedimentary Rocks Below the Lower Jurassic Orange Mountain Basalt, Northern Newark Basin, New Jersey and New York*, Misc. Field Studies Map MF-2208

Selected Case Histories of Rock Slope Stabilization in New Hampshire

Richard Lane, PG, CPG
Engineering Geologist
Bureau of Materials & Research, Geotechnical Section
New Hampshire Department of Transportation

Abstract

Difficult access for equipment, restrictions on traffic delays, changing site conditions and environmental constraints are often encountered on rock excavation work. A review of three case histories on NHDOT projects will examine how these challenges were overcome.

A project in Manchester, N.H. involved the excavation of 100,000 cubic yards of rock for a 70-foot high rock slope in close proximity to one of the highest traffic volume highways in the state without ever delaying traffic more than 10 minutes. The successful completion of the rock excavation was accomplished by a complex traffic control plan, by controlled blasting methods and by closely coordinated excavation activities within very limited space. The longest delay to traffic was 9 minutes and 57 seconds.

A rock cut project on U.S. Forest Service land in Harts Location, N.H. consisted of excavating a 50-foot high rock slope within 45 feet of the Saco River while allowing no rock to enter the water. Controlled blasting techniques to eliminate fly-rock and provide a stable rock slope were implemented. The rock excavation work was completed during the winter with no rock in the river and with minimal disruption to traffic.

Remediation of an existing 80-foot high rock cut in Colebrook, N.H. involved very difficult access for drilling equipment, extensive site work and construction of a temporary detour. A quick assessment of the problems encountered during construction and an immediate redesign of the cut slopes for changing site conditions were critical on this project. Stabilization and protection measures included resloping, a wider catchment ditch, drainage and energy absorbing stone.

Introduction

Excavation of a rock slope to reduce the driving forces, to flatten the slope angle or to remove unstable rock is a common rock slope stabilization method. Rock excavation for highway construction sometimes provides unique challenges to include difficult access for equipment, restrictions on traffic delays, changing site conditions and environmental constraints. The experience level of the contractor often determines the construction techniques utilized on the project. The contract documents must clearly state what the owner (DOT) wants, the design requirements, the required results and any restrictions on the project. It is important in rock excavation work that the contractor be given the flexibility, under review and oversight by the owner (DOT), to select the construction method that best fits the conditions at the site. The contractor's rock excavation plan must address the construction schedule, the sequencing of the work and the method(s) of operation. For a successful outcome, there must be close coordination between the contactor, the blaster and the State DOT personnel. The rock type, the degree of weathering and fracturing of the rock, the orientation of the structural features in the bedrock and the hydrology can rapidly change within the limits of the project.   A quick assessment of problems encountered during construction and an immediate redesign of the rock slope for changing conditions can be critical on some projects.   Three case histories on NHDOT projects examine how challenges encountered during rock excavation were successfully overcome.

Manchester, N.H. – I-93, Roadway Rehabilitation and Widening

The project was located along interstate 93 in the town of Manchester (Figure 1). This project consisted of reconstructing four interstate bridges, constructing a retaining wall, installing a sound wall and widening 1.8 miles of I-93. The roadway work included excavation of more than 100,000 cubic yards of rock on both sides of the northbound and southbound barrels.   The merge/diverge ramps with I-293 join the I-93 mainline within the rock cut. This section of roadway has one of the highest traffic volumes in the state with an average annual daily traffic greater than 90,000 vehicles. The challenge on this

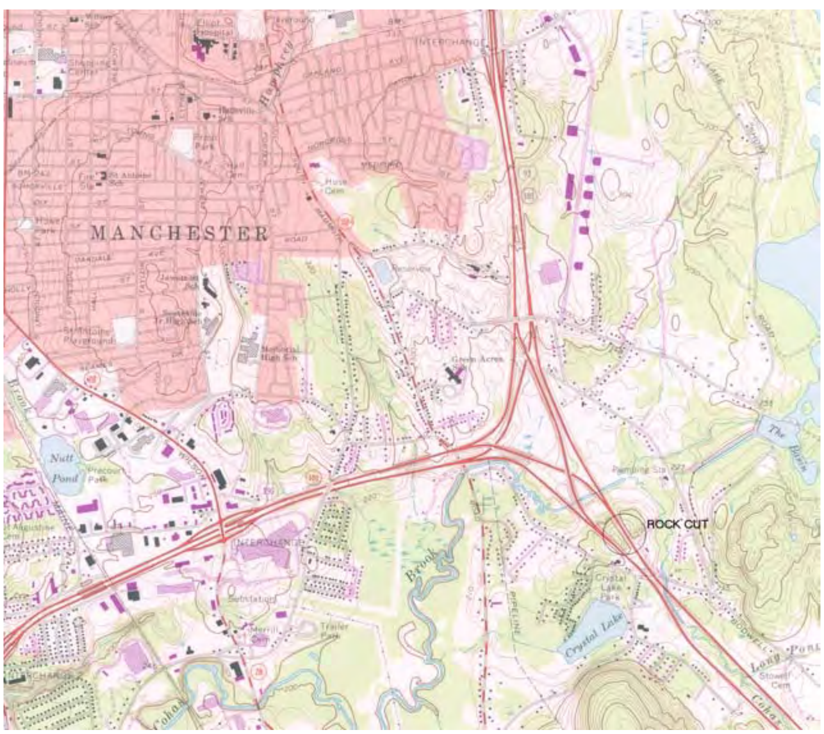


Figure 1. Location Map – Manchester Project

project was to excavate the rock, within very limited space, without delaying traffic more than 10 minutes.

Both the northbound and southbound barrels had two travel lanes with paved shoulders (total width of paved roadway per barrel was 38 feet). The two barrels extend through an existing rock cut and are separated by a rock median reaching approximately 70 feet in height. The existing rock slopes were originally excavated by production blasting methods resulting in rough and jagged rock faces (Figure 2). The ditches at the toe of the rock slopes ranged from 6 to 12 feet in width and were not adequate for catching rock fall. The bedrock is primarily banded gneiss with intrusions of granite and pegmatite. There were planar joints throughout the rock cut that dipped at approximately 5 degrees from horizontal. Other natural discontinuities within the rock trended in a northeast direction, dipping either to the southeast or northwest. The existing rock slopes were oriented in a northwest direction with a slope angle varying between vertical and 60 degrees from horizontal. In general, the primary planes of weakness had a favorable orientation to the roadway alignment.

A number of rock excavation techniques and methods of operation were considered in an effort to minimize traveling delays to the public while completing the work in a safe and timely manner. Several options for rock removal work were considered to include conventional blasting methods, mechanical removal of the rock and chemical expansive breaking of the rock. Methods for improving safety and minimizing delays included median crossovers to divert roadway traffic away from the rock removal site, use of restraints and/or anchored mats, and physical barriers to shield the roadway from

Figure 2. Jagged rock slope with horizontal jointing, I-93 Manchester, N.H.

probable fly rock. Removal of the upper portion of the rock slopes by a mechanical method would greatly reduce the risk of displacing fly rock, but would be expensive and time consuming. The cost and duration of this method was roughly estimated at 3 to 5 times that of blasting. Breakage of the rock by chemical expansive methods would be costly and the duration long, roughly estimated at 10 times that of conventional blasting methods. Median crossovers could divert roadway traffic away from the rock excavation site on one barrel until the rock excavation work was completed, and then traffic could be switched for the rock removal work on the other barrel. Traffic flow would have to be reduced to one lane in each direction either on the NB or SB barrel during the day with the intent of having both barrels open prior to evening traffic peaks. In addition, I-293 traffic would require detour routes. The impact of reducing I-93 to one lane for long durations was deemed unacceptable. A temporary, moveable rock catchment barrier was considered as a method to contain the blasted rock and to protect the public during the rock excavation operation. There was concern that the barrier could be buried under blasted rock and slow down the rock removal operation. It was decided that the NHDOT would set "criteria" for the rock removal, but would not specify the "means".

A separate bid item was added to the contract for any rock removal work conducted above a specified elevation shown on the contract plans. Rock removal work conducted in this upper level was designated the "Limited Operations Method". This would help compensate the contractor for the extra effort, care and additional time that would be needed to excavate the upper half of the rock slope. To minimize traffic delays, an incentive/disincentive program was established (Figure 3).

DISINCENTIVE PROGRAM	
ACTION	**FINES**
First occurrence of a delay over ten minutes - Warning	$5,000 lump sum plus $1,500 for each 3 minutes delay beyond 10 minutes
Second occurrence of a delay over ten minutes - Warning	$10,000 lump sum plus $3,000 for each 3 minutes delay beyond 10 minutes
Third occurrence of a delay over ten minutes – Blasting Contractor of record shall be removed from job	$10,000 lump sum plus $3,000 for each 3 minutes delay beyond 10 minutes

INCENTIVE PROGRAM	
ACTION	**AWARD**
No occurrence of a delay over ten minutes	$100,000 lump sum payment
One occurrence of a delay over ten minutes	$60,000 lump sum payment
Two occurrences of a delay over ten minutes	$20,000 lump sum payment
Three or more occurrences of a delay over ten minutes	No lump sum payment

Figure 3. Disincentive/Incentive Program

The Incentive Program would be a one-time cash award, which would be made upon the completion of all I-93 rock excavation. At that time the total number of delays greater than 10 minutes would be summarized for determination of the lump sum award.
If the first blasting contractor of record had been removed, no lump sum payment would be awarded. For the purpose of calculating the incentive/disincentive payments, the time delay would start when the last vehicle ahead of the rolling roadblock passed perpendicular to the location of the intended blast. The time of delay would end when all obstructions and equipment have been removed from the traveled way and the first vehicle behind the rolling blockade passed the blast site.

Rolling roadblocks were used to control traffic on both the NB and SB barrels during blasting. Moving vehicles were slowed to approximately 15 miles per hour by state police cruisers traveling in front of the traffic. Timing was critical, requiring extensive planning and coordination to work effectively. The process was further complicated by nearby ramps that merged with the mainline. Mock roadblocks were initiated before the actual blasting operations took place. This allowed for fine-tuning of the traffic control plan without any hazards in the roadway. A full time coordinator was designated for communications with the state police vehicles, the contactor and the blaster. Blasts were shot at the same time each day to simplify the coordination efforts. It was critical that the contractor have an adequate amount of the proper equipment at the site to quickly remove any blasted rock that landed in the roadway.

Guide bits and other special drilling accessories were used to minimize drill hole deviation and to maintain the required drilling accuracy for the pre-split operation. A borehole deviation survey system was used to measure the inclination of all pre-split holes greater than 30 feet in length. The borehole measurement data was used in determining the placement and amount of explosives to be used in the blast holes.

The production blasting was accomplished in two lifts. The pre-splitting operation was accomplished in a single lift extending the entire height of the rock slope. The pre-split holes were drilled at 18 inches on center with alternating holes drilled to the full depth of the cut. The shorter pre-split holes extended to the bottom of the upper lift (Limited Operation Method pay line) and were loaded with 7/8 inch continuous pre-split explosive @ .32 lbs. per foot with 25 grain detonating cord, 2.5 lbs. of 2X16 gelatin dynamite and 3 feet of crushed stone stemming. The bottom portion of the full depth pre-split holes were loaded in a similar manner and stemmed for 3 feet with the remaining top portion left unloaded. The unloaded upper portion of the holes served as gas relief (Figure 4).

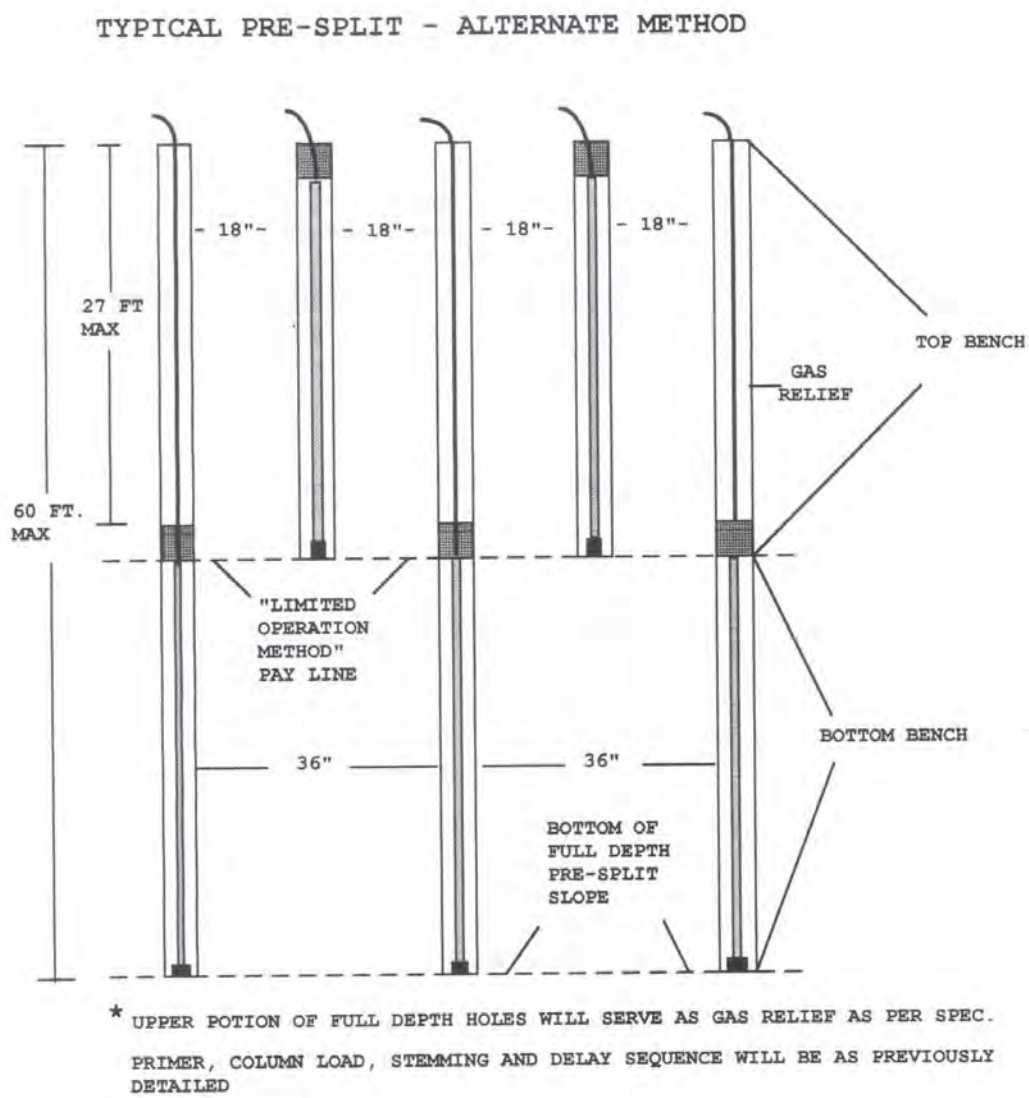


Figure 4. Typical loading of pre-split holes

Overlapping and anchored blasting mats were used to contain fly rock (Figure 5). The contractor utilized concrete jersey barriers with a 6-foot high chain link fence erected on top as a protective barrier for the traveling public during the rock excavation activities (Figure 6). All production shots were designed with a delay pattern that moved the rock parallel with the roadway alignment. Test shots were conducted with drill patterns, delays and loading procedures adjusted as conditions warranted.

Figure 5. Overlapping and anchored blasting mats help contain fly rock

Figure 6. Concrete barriers with chain link fence

The rock was excavated on a 1H:2V pre-split slope.   The resulting rock slopes were uniform, stable and free of loose rock (Figure 7). In general, most of the pre-split holes varied less than a foot in any direction from their proposed slope. The maximum measured drill hole wander was 1.7 feet in the highest section of the rock cut. The largest shot size on the project was 1,000 cubic yards. A total of 180 blasts were conducted with the longest delay to traffic at 9 minutes and 57 seconds. The contractor and blaster satisfactorily met all the requirements stated in the contract documents and were awarded the $100,000 incentive award.

Figure 7. Completed northbound median rock slope

<u>Harts Location - Route 302, Roadway Rehabilitation and Widening</u>

The project site is located in the town of Harts Location along Route 302 within the boundaries of the U.S. Forest Service land (Figure 8). The project consisted of widening, reclaiming the existing pavement, repaving, rehabilitation of a bridge, drainage work and rock excavation. The rock excavation involved cutting further into an existing rock cut located on the south side of the road. The Saco River, a designated sensitive river, runs parallel and in close proximity to Route 302 on the north side of the road (Figure 9). The river was within 45 feet of the existing rock face. The rock slope was located on the inside of a curve with no ditch and with the highest section overhanging the roadway (Figure 10). The existing rock slope was approximately 50 feet in height and was composed of coarse grained, syenite. There was a zone of fractured rock that extended behind the existing rock slope, and a joint set that ran nearly parallel to the alignment and dipped toward the road. In addition, there were several joint sets oriented transverse to the alignment.

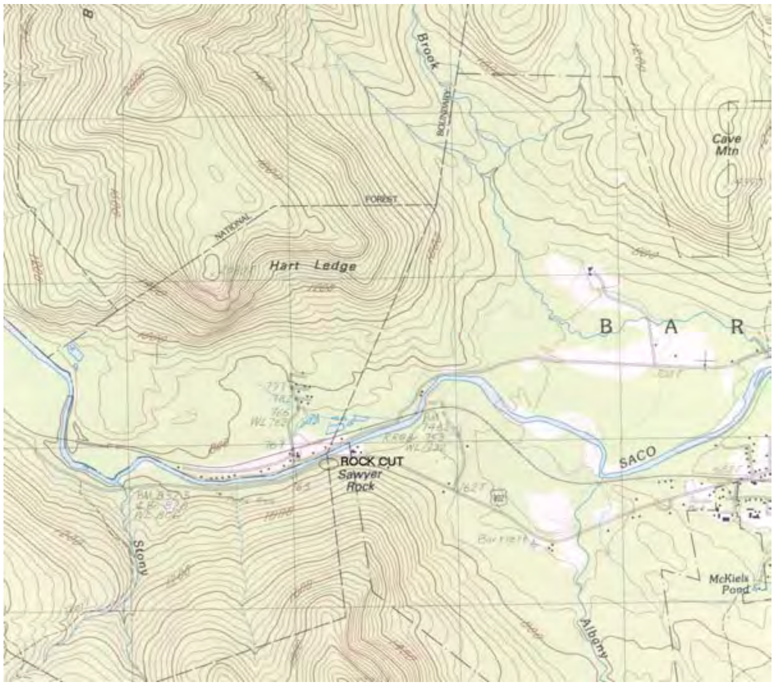


Figure 8. Location Map – Harts Location

Although production blasting was the method originally used to excavate the rock slope, its overall stability was good. Portions of the slope were covered with a thick layer of ice during the winter months. The challenge was to excavate the rock slope while allowing no rock to enter the river or its associated banks. The rock excavation had to provide a stable rock slope with minimal disruption to traffic and limited impact to Forest Service land.

Figure 9. Saco River located in close proximity to Route 302

Figure 10. Rock cut overhangs road

The blaster and the contractor were required to submit a Blasting Plan, and a Sequencing and Positioning Plan for Rock Removal. The contractor's plan called for containment of the blasted rock with blasting mats and moveable steel framed trench boxes (8'HX6'WX20'L) placed along the toe of the rock slope. The blasting mats were overlapped and secured with anchor cables to keep them from landing in the road and becoming entangled with the blasted rock rubble. Each shot was small in size with delays sequenced to move the rock in a direction parallel with the roadway alignment. The pre-split holes were carried the full depth of the rock cut, while production blasting was accomplished in two lifts. Each production shot was limited to a zone with horizontal dimensions no greater than 15 feet (Figure 11).

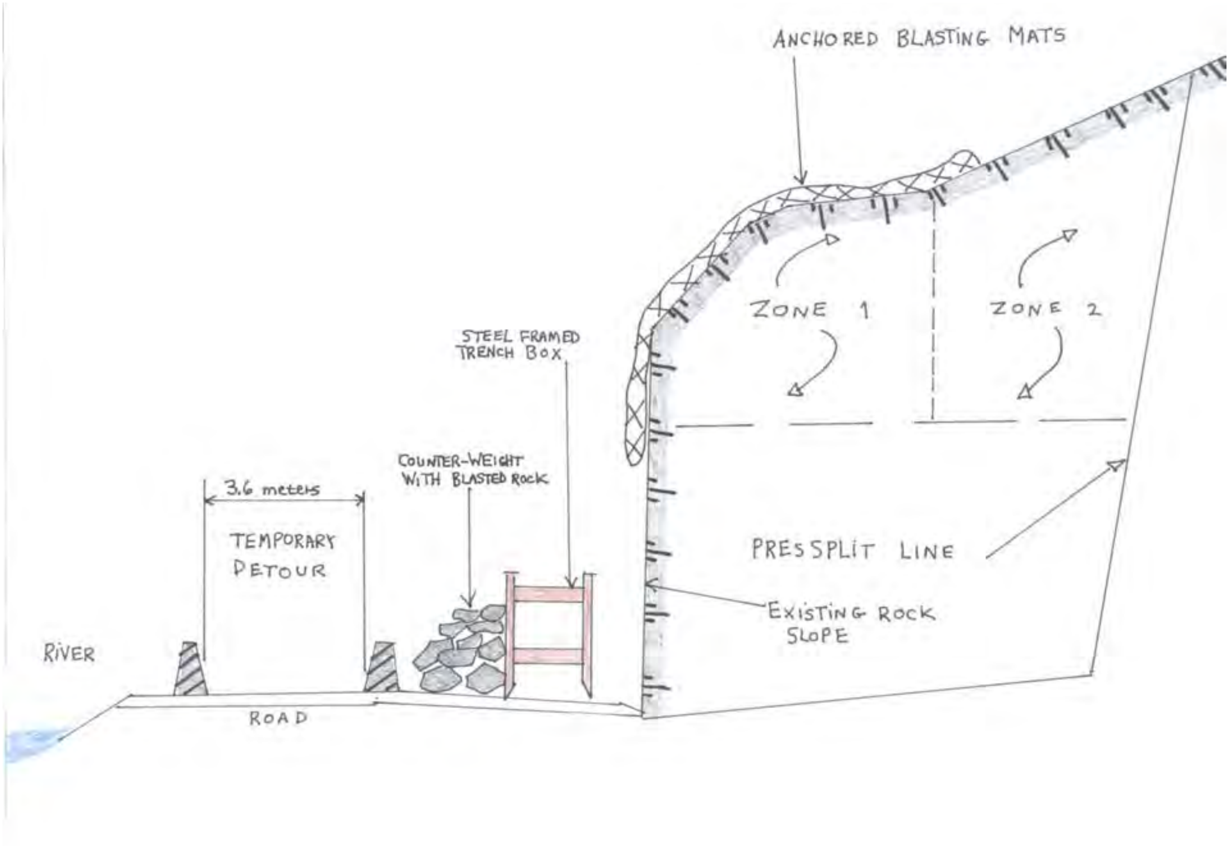


Figure 11. The rock slope was pre-split in one lift and excavated with small production shots in two lifts

The rock slope was excavated on a 1H:6V pre-split slope with a 21-foot wide ditch at the toe. The widened ditch was to provide a rock fall catchment zone and storage for snow/ice removal during the winter.

The Forest Service requested that the controlled perimeter blasting be eliminated to avoid pre-split drill hole imprints on the final rock slope. This request was considered, but denied for the following reasons:

- There would be a greater risk of blast damage to the new rock face, which could result in freeze-thaw damage and increased rock fall.
- There would be a higher risk of the rock breaking beyond the proposed slope limits, causing a greater impact to the area behind the rock slope.
- It would be more difficult to control the shots and keep blasted rock out of the river.

The overall stability of the rock slope was assessed after excavation was completed. A potential failure surface, extending the entire height of the rock face, was oriented within 15 degrees of the rock face and dipped toward the road at 47 to 55 degrees from horizontal (Figure 12).

Figure 12. Potential failure plane dipping toward road

Nine rock bolts (lengths ranged from 22 to 35 feet) were installed to secure the potentially unstable rock.   The exposed portions of the rock bolts were painted to blend with the surrounding rock (Figure 13).

Figure 13. Installed rock bolts

The rock excavation work was conducted without any blasted rock entering the nearby river or its associated banks. All the rock excavation work was completed during the winter when temperatures were consistently below zero and the top of the rock slope was covered with a 1 to 2 foot thick layer of ice. The work was completed with minimal impact to the surrounding US Forest Service land. The final rock slope was stable and was excavated with minor delays to traffic.

<u>Colebrook – U.S. Route 3, Rock Slope Stabilization</u>

The project site is located along U.S. Route 3 in the northern portion of the state, approximately 1.0 mile north of Colebrook, N.H. (Figure 14). This segment of Route 3 runs along the Connecticut River and has deep side hill cuts on its east side. An inactive railroad is located on the west side of the road between Route 3 and the Connecticut River. A 700-foot long segment of the river curves toward the east in close proximity to the railroad and Route 3. The banks of the river on the inside of the curve have eroded and threaten to undermine both the railroad and Route 3. The terrain on the east side of the road is heavily wooded, rugged and steep with a series of natural bedrock cliffs. The existing side hill cuts consist of three rock cuts separated by steep soil slopes.

Figure 14. Location Map – Colebrook

The soil cut slopes were wet and reached a maximum angle of 1.25H:1V with some failure scarps occurring along the upper portion. The rock cuts were constructed in 1969 using pre-split techniques. The quality of the pre-split was generally poor with drill holes that wander and sometimes cross each other. The three rock cuts ranged from 20 to 70 feet in height and had a total combined length of 2100 feet. The slope angle of the existing rock slopes ranged from 60 degrees to vertical. The existing ditch was 4 to 9 feet in width and not adequate for catching rock fall (Figure 15).

Figure 15. Narrow ditch at the toe of the rock slope

The profile of the rock slopes was irregular with numerous overhangs and unstable wedges. The bedrock was foliated, phyllitic schist with quartz intrusions and occasional basalt dikes. In general, the foliation was oriented in a northwesterly direction and dipped 15 to 35 degrees toward the northeast. In some areas, the foliation was severely contorted and folded. The bedrock was highly fractured and jointed with severely weathered pockets where the rock had deteriorated to a soil consistency. Over-blasting had resulted in extensive back breakage and highly shattered sections. Vertical discontinuities provided a path for the infiltration of surface water, leading to accelerated weathering and ice jacking. Water continuously flowed over the rock face and seeped from open fractures in the rock slope. Heavy ice build-up occurred on sections of the rock slope during the winter period. Slides (Figure 16) and individual rock fall were common with the most recent event occurring in April 2001. The latest slide, which reached the road, consisted of several cubic yards of rock and a large block (8 feet in diameter) of ice (Figure 17).

Figure 16. Site of rockslide that occurred April 2001

Figure 17. Eight foot diameter block of ice

The overall stability of these rock slopes had been deteriorating at an accelerated rate. The challenges in stabilizing these rock slopes included difficult access, no existing detour for Route 3, variable orientation of the structural features, changing rock conditions, steep terrain and an environmentally sensitive river in close proximity to the rock cut.

Investigations at the site included mapping the orientation and inclination of the discontinuities, graphical representation of the discontinuities on stereonets, utilization of the Colorado Rock Fall Simulation Program, exploratory borings taken to assess the quality of the rock, and an extensive ground survey of the terrain to develop accurate plans and cross-sections. A total of 17 test borings with recovered rock cores were taken upslope from the existing rock cuts. No borings were taken along a 650-foot long segment of the middle rock cut, due to access problems for the drill rig.

In general, the foliation trends in a direction parallel with the roadway alignment and dips into the rock slope at 15 to 35 degrees. At several locations the foliation is folded and contorted. There are joint sets oriented nearly parallel with the alignment and dipping 47 to 85 degrees from horizontal toward the road. Several potential combinations of intersecting discontinuities form wedges that dip toward the road at 5 to 87 degrees from horizontal. The orientation and inclination of the discontinuities were measured, and the data presented graphically on stereographic projections. Wedge and toppling failures were identified as the primary rock slope failure modes.

The spacing between parallel joint planes ranged between 2 inches to more than 10 feet. This had a direct relationship with the size of the individual blocks that could be expected from a potential rock fall event. Some of the discontinuities had infilling composed of weathered rock and soil.

The project consisted of rock slope stabilization and rehabilitation of the eroded riverbank. Remediation of the rock slopes included cutting the rock further back from the roadway on 1H:2V to 1H:4V pre-split slope, constructing surface drainage, installing horizontal drains and placing energy absorbing stone in selected areas of the ditch along the toe of the rock slopes. Rock slopes greater than 40 feet in height were excavated in two lifts (Figure 18).

Berm ditches were constructed along the top of the cut slopes to intercept and to divert surface water. Horizontal drain holes were drilled into the rock face to reduce the cleft water pressure within the rock slopes. The drain holes were spaced 25 feet apart and extended 40 feet into the rock slopes at a 5 to 10 degree incline. The soil slopes were excavated on a 1.5H:1V angle and covered with a three foot thick blanket of stone fill (angular rock fragments at 1 to 3 cubic feet in size).

Figure 18. Rock slope excavated in two 40 foot lifts

To provide room for the rock excavation activities in the area of the middle rock cut, a temporary detour was constructed on the west side of the road by filling over the existing railroad. The stabilization work along the river consisted of rock fill placed the full height of the riverbank along the inside of the bend in the river for a distance of approximately 1000 feet (Figure 19). Both the riverbank rock fill and the temporary detour were constructed with blasted rock from the roadway cuts.

Figure 19. Stabilization of riverbank and construction of detour

The condition of the rock and the orientation of structural features were closely monitored during the construction phase. The middle rock cut was initially designed for a flatter 1H:2V angle, due to the lack of subsurface information behind the existing rock slope. During construction, the slope angle for the middle rock cut was changed to a steeper angle (1H:4V) to match favorably oriented discontinuities that were uncovered during the removal of the overburden soils. In addition, test blasts indicated the rock quality was better than expected and could accommodate a steeper slope. This modification to the rock slope design resulted in lower rock excavation quantities and improved alignment of the pre-split holes.

The rock excavation and river stabilization work were completed successfully with minimal delays to traffic. Sampling of the water down stream from the construction site on a daily basis confirmed that water quality requirements were met. A total of 65,000 cubic yards of rock was excavated with no rock thrown into the river during the blasting operation. The expected completion of the project is during 2003, which is one year ahead of schedule.

Conclusion

The specifications should be tailored for the conditions at the project site, should provide guidance to the contractor and should clearly state the required results. The contract documents must provide enough flexibility to deal with the challenges and unknowns that are sometimes encountered on rock stabilization projects. This information combined with an experienced contractor and knowledgeable DOT personnel is critical for a successful project. It is also important that the engineer or engineering geologist responsible for the rock stabilization design be actively involved in the supervision of the work and in any changes made during construction.

References

Chassie R., Sculptured (“Ragged”) Rock Slopes Verses Effective Rockfall Mitigation, Federal Highway Administration

Jones C. L., Higgins J.D., Andrews R.D., March 2000, Colorado Rockfall Simulation Program, Version 4.0, Colorado Department of Transportation, Colorado Geologic Survey, Colorado School of Mines

New Hampshire Department of Transportation, 2000, Contractor’s Blasting Plan, Manchester 12159 project

Norrish, N.I., Lowell S.M., Aesthetic and Safety Issues for Highway Rock Slope Design, Golder Associates Inc. and Washington State Department of Transportation

Rock Slopes: Design, Excavation, Stabilization, 1989, U.S. Department of Transportation, Federal Highway Administration, Publication No. FHWA-TS-89-045, McLean, Virginia

Rock Slope Failure Case History, Yanacachi, Bolivia, S.A.

Theodore von Rosenvinge, P.E.[1] Robert D. Powell, P.Eng., R.G.[2]

INTRODUCTION

This paper summarizes the results of an evaluation of a construction phase, rock slope failure which took place during construction of a hydroelectric project. The construction site was situated at the base of steep bedrock-controlled terrain along the Rio Unduavi (River) near the remote village area of Yanacachi, Bolivia, South America (Figures 1 and 2).

Traversing the terrain in this area are narrow one-way roadways notched into the steep hillsides. These roads include the so-called "most dangerous road in the world" according to travel guide literature. Road travel in this rugged terrain can take several hours to go distances covered in half an hour in the United States. Photographs of the terrain and roadway conditions are shown as Figures 3, 4 and 5.

The authors' firm (At the time Mr. Powell was Senior Geologist/Project Manager at Geo**Design**, Inc.) was retained subsequent to the rock slope failure to independently observe and evaluate the nature and cause of the incident. The project commenced with a review of available data followed thereafter with our site visit. The site visit started with air travel to La Paz by the authors. La Paz is situated at an Elevation of over 3500 m above sea level, the world's highest capital city.

At La Paz, research was conducted at civil and military agencies including the Servicia Nacional de Geologica y Mineria and the Instituto de Geografico Militar. These institutions provided geologic maps, topographic maps and aerial photographs of the area. The research was followed by travel to the construction site to collect information, conduct interviews, carry out strike and dip measurements and observe post-failure conditions.

Failures, of course, provide opportunities to learn. In our opinion, two lessons emerged from this case history. First, the case provides a reminder of the importance of design phase geological site evaluations to help anticipate potential slope construction conditions and risks, and to provide opportunities to mitigate risk before construction. Second, follow-up construction phase observations are important in order to check initial assumptions and permit adjustments to slope stabilization designs.

1. Principal, Geo**Design**, Inc., 984 Southford Road, Middlebury, Connecticut 06762
2. Senior Geotechnical Engineer, Klohn Crippen Consultants Ltd., Mississauga, ON, Canada L5K 2P8

PROJECT DESCRIPTION and BACKGROUND

The overall project consisted of construction of a hydroelectric facility (85 MW) located about 40 km east of La Paz in the Yungas Province in Bolivia, South America. Estimated total project cost is approximately $100 million (US). The subject portion of the project pertinent to this paper is a powerhouse, located at the base of a 30-meter high steep rocky, colluvium-covered slope (Figure 6). The powerhouse is situated along the southern banks of the Unduavi River, north of the Village of Yanacachi.

Site elevation at the powerhouse is approximately 1450 m above sea level.

Water for the hydroelectric system is to be diverted from the Rio Taquesi to the powerhouse via a tunnel to a penstock pipe. (Figure 7)

Construction of the Yanacachi Powerhouse entailed the following:

1. Construction of an access road
2. Construction of a cofferdam between the Rio Unduavi and hillside
3. Excavation of a near vertical penstock trench into the slope
4. Excavation for the powerhouse for a total temporary slope height of 50 meters
5. Stabilization of a steep (79 degree), 30-meter-high rock slope associated with the general excavation for the penstock and powerhouse
6. Construction of the powerhouse and ancillary equipment

REGIONAL SITE GEOLOGY

Several geologic maps published for the area including those listed below and the excerpt shown as Figure 7 were obtained prior to visiting the site. The maps indicated that the site area bedrock consisted of Ordovician sedimentary to metamorphic rocks (metapelite/slate).

Summarized below are average mapped strike and dip (in degrees) selected from representative published map data for the area along the Rio Unduavi.

Before 1994	Carta Geologica De Bolivia, Hoja Chulumani (6044), 1:50,000, Preliminary Map (average strike/dip within general region of Yanacachi Powerhouse = 299/51)
1994	Carta Geologica De Bolivia, Hoja Chulumani (6044), 1:100,000, I-CGB-30 (average strike/dip within general region up and downstream of the Yanacachi Powerhouse = 294/49)
1997	Thematic Maps of the Mineral Resources of Bolivia, Hoja La Paz Y Copacabana, 1:250,000, II-MTB-7B (average strike/dip within general region of Yanacachi Powerhouse = 311/31)

Based upon a review of the smaller scale Chulumani sheets, the average regional strike/dip was noted to be about 297/50 in the vicinity of the Yanacachi Powerhouse for the slopes leading down to the Rio Unduavi. Range of dip angles from the representative data set was 20 to 75 degrees.

Typical strike and dip measurements are also indicated on Figure 7, together with regional structural trends.

PRE-FAILURE SLOPE DESIGN

Initial project plans indicated the top of slope at approximate elevation 1485 m and bottom of powerhouse excavation to be at 1434.3 m. Figure 6 shows a typical profile of the slope, powerhouse and penstock excavation, based on project plans. Up to about 2 to 3 m of colluvium was shown blanketing the bedrock. The pre-construction slope angle was about 1.17 vertical to one horizontal.

Plans called for steep cuts into the bedrock for a final slope of 5 vertical to one horizontal (79 degrees). In addition, a 3 m wide penstock trench was to be notched into rock at a slope of 3.27 vertical to one horizontal (73 degrees). (Figure 6).

Prior to commencement of construction, the site was not readily accessible by roadway. As such, construction drawings were developed based solely on available geological mapping, seismic profiling, site reconnaissance including two strike/dip measurements of rock surface exposures, and excavation of shallow test pits above and below the slope.

Equipment access to the Yanacachi Powerhouse site was first available in March 2000 when an access road was completed (Figure 8 photograph). The roadway construction exposed a considerable length of rock slope contiguous to and roughly parallel to the failed slope. At that time, a test pit was excavated at the powerhouse site in order to assess the depth to bedrock and the permeability of the alluvial soil terrace at the base of the slope. Figure 9 is based upon a project file sketch of the slope made at the time of the test pit program.

No deep test pits, test borings or rock coring were available for the design and construction of the 30-meter high rock slope.

Initial belief of the designers was that there was a favorable orientation of planar discontinuities within the rock mass based upon the two strike and dip measurements. These measurements suggested bedding with a strike roughly parallel to the hillside and a dip of 70 and 75 degrees into the hillside (opposite to actual conditions as indicated on Figure 9 and favorable to construction). Nevertheless, a minimal pattern of the rock bolts was specified in project plans reportedly for safety and protection from small surface failures.

The pattern and sizes of the rock bolts were reportedly based on experience and were to be adjusted in the field, as directed by an on-site geologist. The bolts were to be installed on a 3-meter spacing of alternating horizontal rows of 4 m and 6 m long bolts. Bolts consisted of 25 mm diameter, grade 60 steel bars, placed in grouted holes set at 15 degrees below horizontal and post-tensioned. In addition to bearing plates, nuts and washers, a surface treatment of wire mesh or welded wire fabric was also noted.

CONSTRUCTION and FAILURE

Following construction of about 500 meters of new access road to the powerhouse site, excavation of the rock slope and powerhouse commenced and continued between March and June 2000, including initial stabilization of the rock slope.

On June 24, 2000, the rock slope failed, infilling much of the powerhouse excavation. Fortunately, the failure occurred very early in the morning and no one was injured. The contractor estimated the amount of slide rock to be about 4,000 cubic meters. Figures 10 and 11 are “before and after” photographs of the slide area from project files. The depth and pattern of the rock bolting was not sufficient to hold the rock in place.

After the failure, mapping was conducted at the failure zone by the designers to assess the cause and to provide input to the design of remedial stabilization measures. Based on this work, the designers concluded the failure was the result of a sliding block; with the block defined by two sets of intersecting rock joints. These joints included a 50-degree joint plane and a near vertical joint belonging to a set of joints striking nearly parallel to the slope. Seepage was also reported along the 50-degree joint after the failure and cited as a possible contributing factor.

Remedial measures along the rock slope were subsequently designed and implemented by the design-build team. The measures included the installation of numerous 6 to 11 meter long rock bolts, loose rock removal, rock overhang removal, flattening the overall slope’s inclination and grouting along the crest of the slope in order to limit the infiltration of surface water.

To our knowledge the remedial measures have been successful.

SITE VISIT

On May 2001, Geo**Design** personnel visited the site during construction, reviewed project records and interviewed project personnel regarding events and conditions. The post-failure, stabilized rock surface was still exposed and was observed at the penstock and powerhouse areas. Observations were also made along the access road of similar cut rock slope geometry (five vertical to one horizontal) to the powerhouse/penstock slope. Figures 12 and 13 are photos of the overall slope after stabilization.

Strike and dip measurements of rock discontinuities were made at the construction site and along the rock slope of the access road. Figure 14 is a photo at the failure plane behind the powerhouse to the right side of the penstock. A pattern of discontinuity planes with similar strike and dip angles to the failure plane was also found along the access roadway. At one notable location a large exposed rock face was found in the nearby road cut with a similar strike and dip to the 50-degree failure plane (Figure 15).

Project personnel indicated that in January 2000, initial joint measurements were made during preliminary stages of road cutting and that the joint measurements suggested favorable orientation (into the hillside), which was believed to be part of an anticline feature. As roadway excavation progressed below the level of these initial joint measurements; there is no indication that additional joint set measurements were made after the initial set. Including at the powerhouse and penstock excavations.

DISCUSSION

Published geologic maps indicated an average strike/dip of about 297/50 in this area, which is very close to actual strike and dip at the failure site.

Preconstruction measurements of strike/dip within area of the Yanacachi Powerhouse were however 130/70 to 75 degrees. This orientation was opposite to published mapping for the area and unfortunately the implications not recognized. Based upon our limited field work and office study, it appears that these measurements were likely performed upon rock outcroppings which had experienced “slope creep” e.g., surficial slabs of rock rotated down slope 45 degrees or so, creating the appearance of intact rock with bedding angled into the hillside. Figure 16 depicts a schematic of this concept. Evidence of this type of movement is shown in the photograph of Figure 17 taken along the access roadway cut.

CONCLUSIONS

To design and construct a permanent excavation of a high, steep slope in bedrock such as the subject slope, adequate information about the orientation of discontinuities is needed, or undue risks will be taken (e.g., risk of slope failure). The risk of slope failure can be mitigated in advance by: obtaining subsurface information, collecting surface measurements, researching and comparing results with published geology and ultimately, and incorporating this information into a stabilization design. Even with such information, contingency plans are prudent to deal with actual conditions as they emerge during construction. In this regard, field observations must be transferred to the designer so the implications can be properly assessed.

Where conditions in remote areas can make collection of adequate field information in advance difficult and costly, more emphasis can be placed upon collection of information during construction and following the Observational Approach as proposed by Karl Terzaghi. Provisional designs and contingency plans can be made for probable and potential rock bedding and slope stabilization scenarios.

When site mapping appears to vary significantly from regional geologic mapping, the difference should be investigated. Slope creep may also be a factor. If so, apparent bedding and/or discontinuity angles from mapping may be seriously misleading.

ACKNOWLEDGEMENTS

The writers would like to thank the numerous and various parties involved with design and construction for their complete cooperation during our site visit and investigation. We also wish to express our thanks to our client for the opportunity to complete this assignment.

Special thanks to the project owner for safe transportation and outstanding Bolivian hospitality during the visit. Our hosts informed us that “there are bold drivers and old drivers in Bolivia, but there are no old bold drivers.” They provided an engineer who was an expert driver.

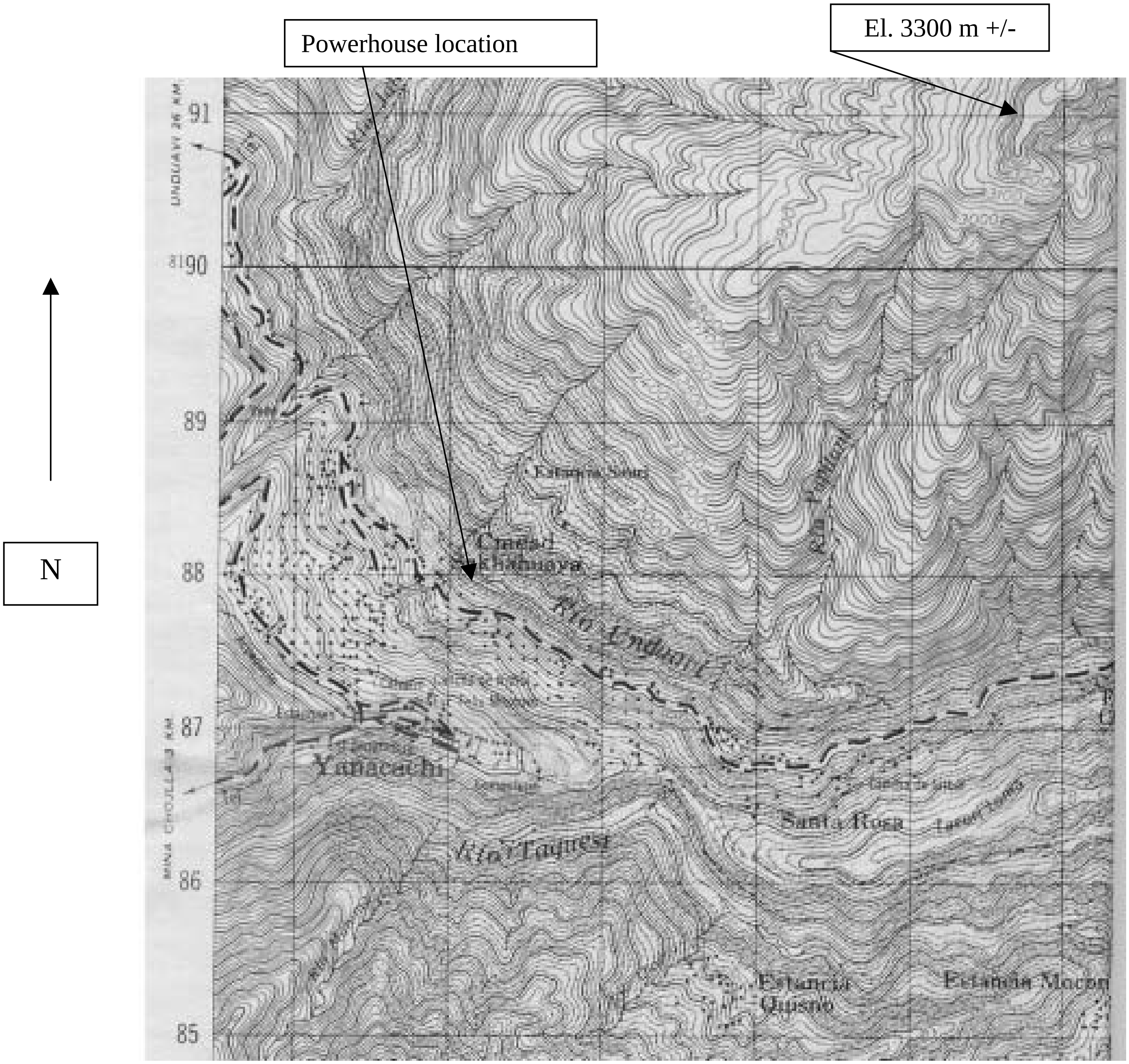
Acknowledgement and thanks to the writers' colleague, Mr. Randall T. States, P.E. who assisted with the production of the figures and editing.

LOCUS PLAN
Yanacachi Powerhouse
Yanacachi, Bolivia

FIGURE 1

TOPOGRAPHIC MAP
Original Scale 1:50,000
Yanacachi Powerhouse
Yanacachi, Bolivia

FIGURE 2



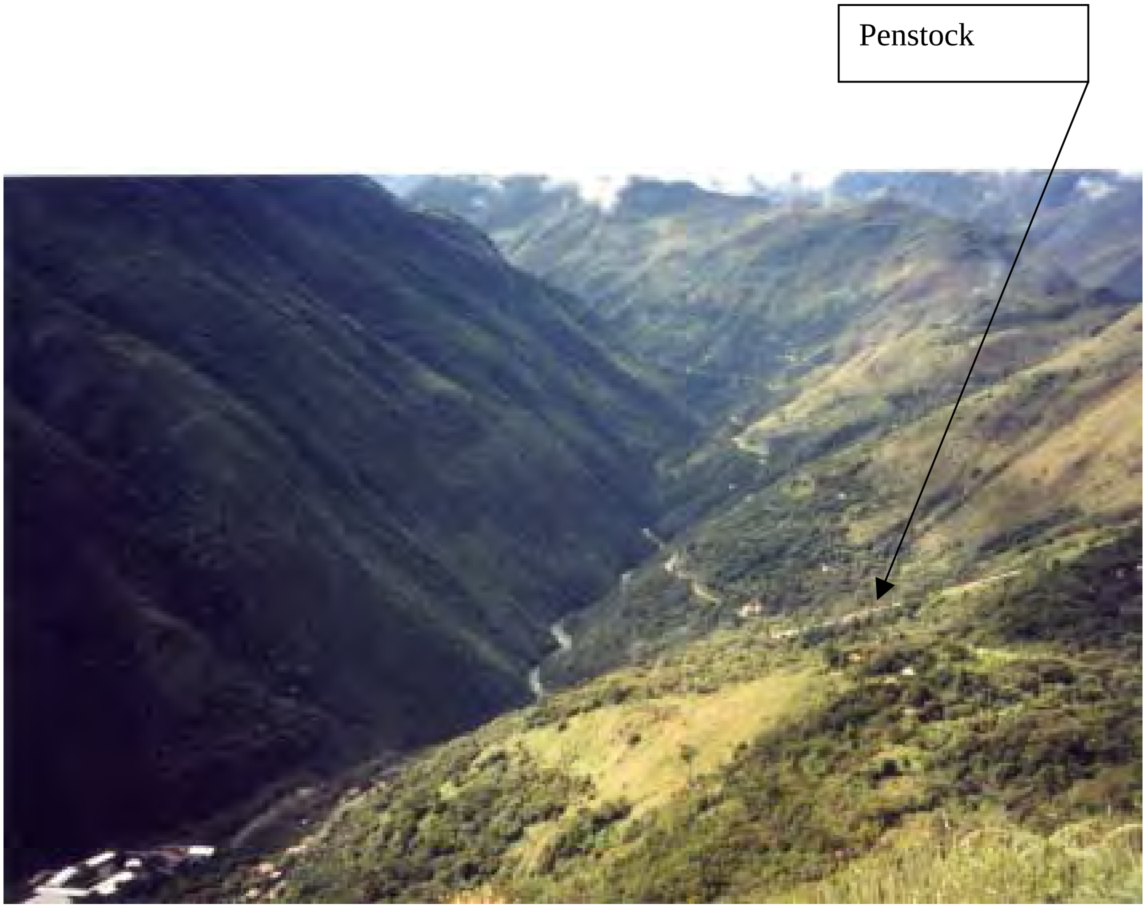


FIGURE 3

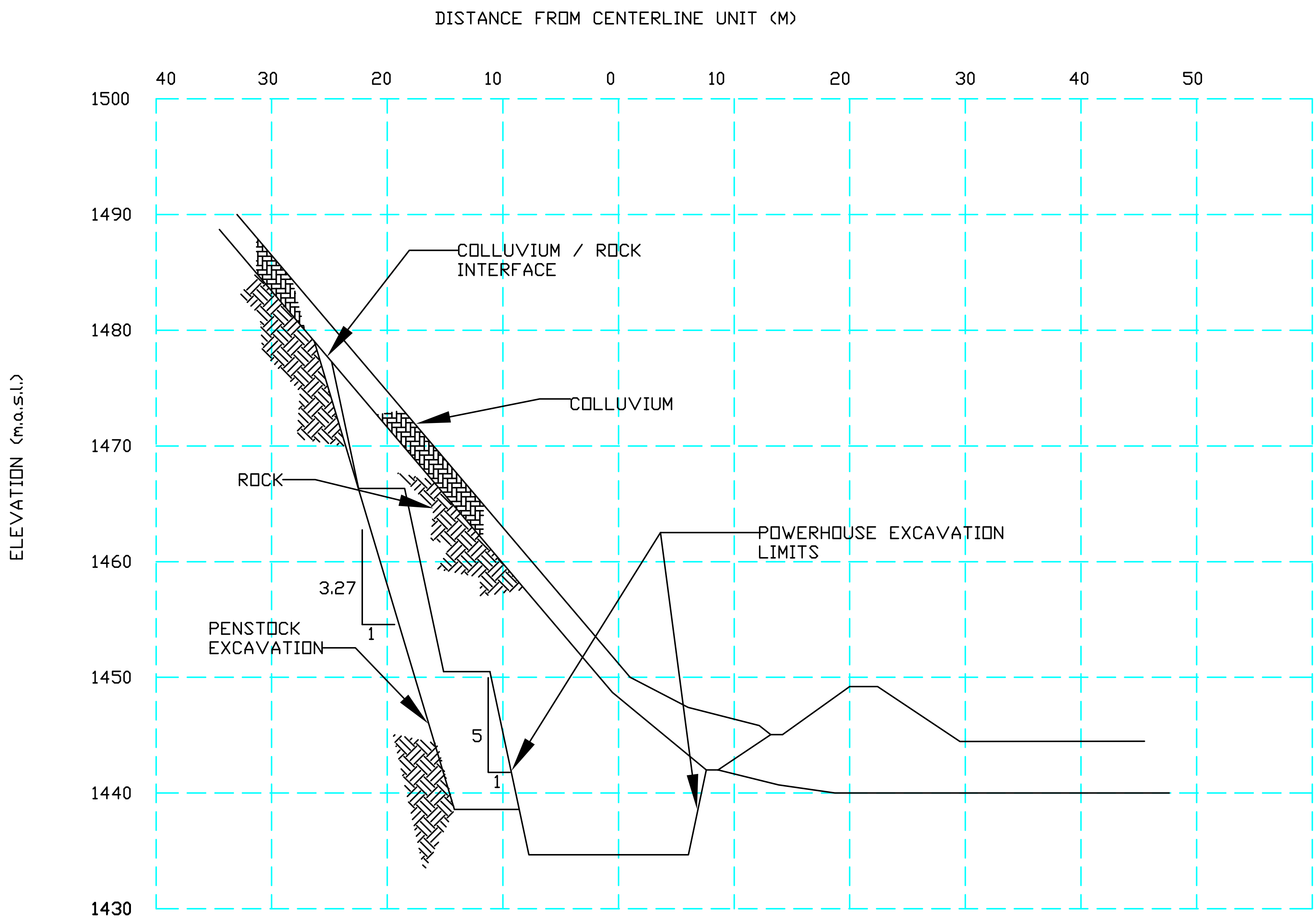
Looking south east toward powerhouse site and Rio Unduavi

FIGURE 4

Typical hillside roadway (note slide areas)

FIGURE 5

Typical Roadway Conditions



NOTES: PROFILE BASED ON PROJECT PLANS.
SCALE 1:500

FIGURE 6
POWERHOUSE SLOPE PROFILE

GEOLOGIC MAP
Scale 1:100,000
Yanacachi Powerhouse
Yanacachi, Bolivia

FIGURE 7

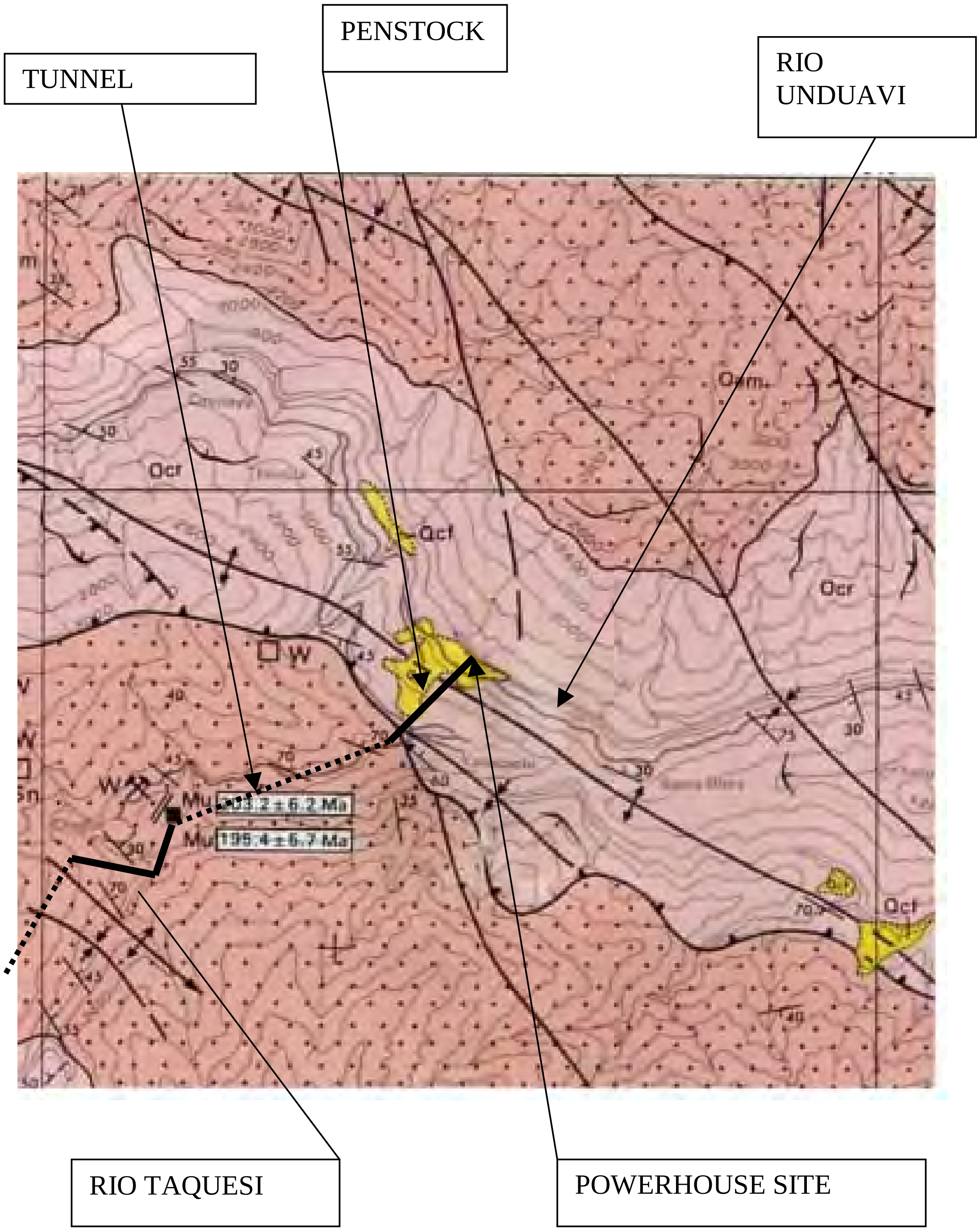
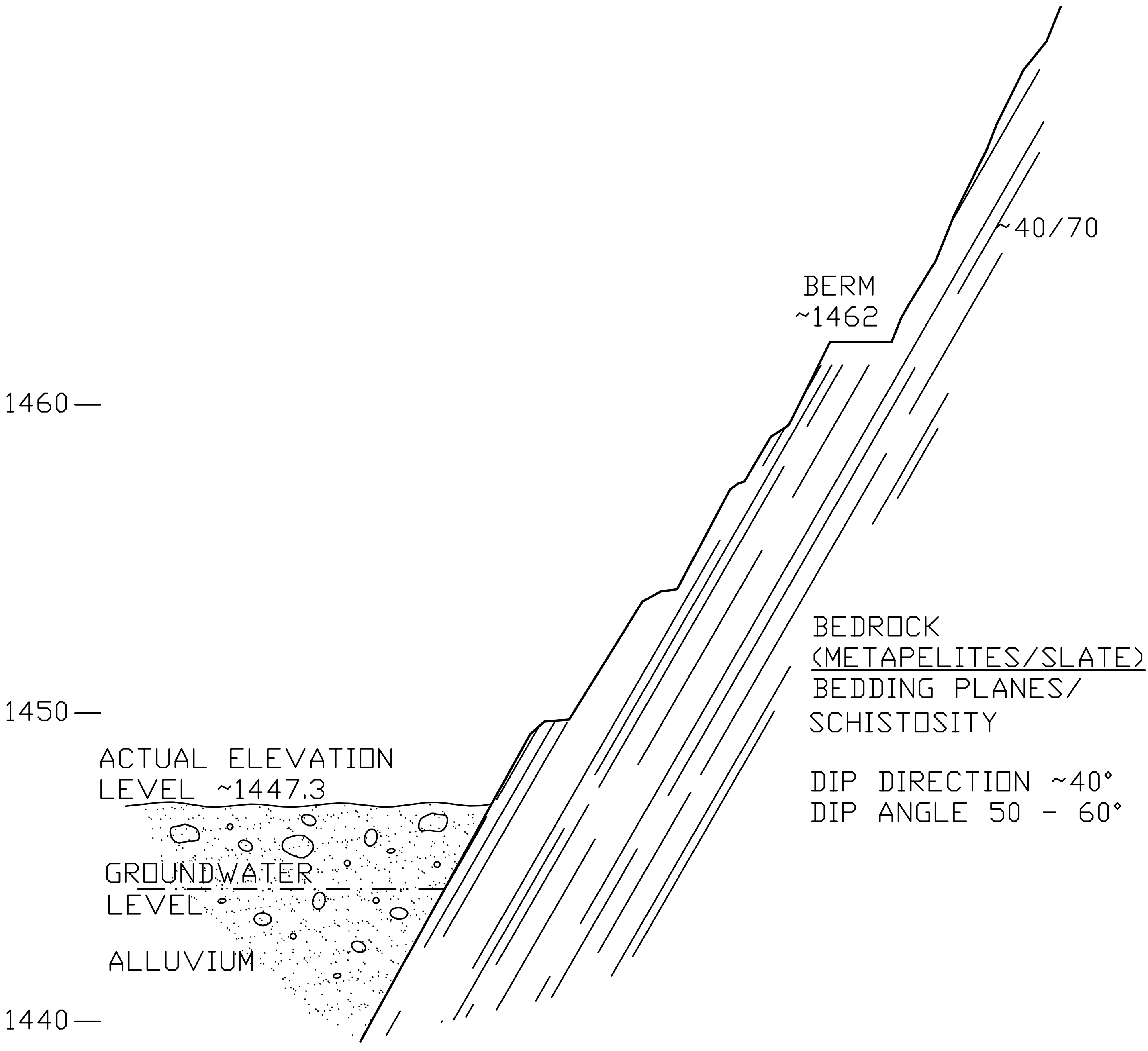


FIGURE 8

View of Access Road from Crest of Slope



GEOLOGIC SECTION - YANACACHI POWERHOUSE

SCALE 1:200

FIGURE 9
CROSS SECTION

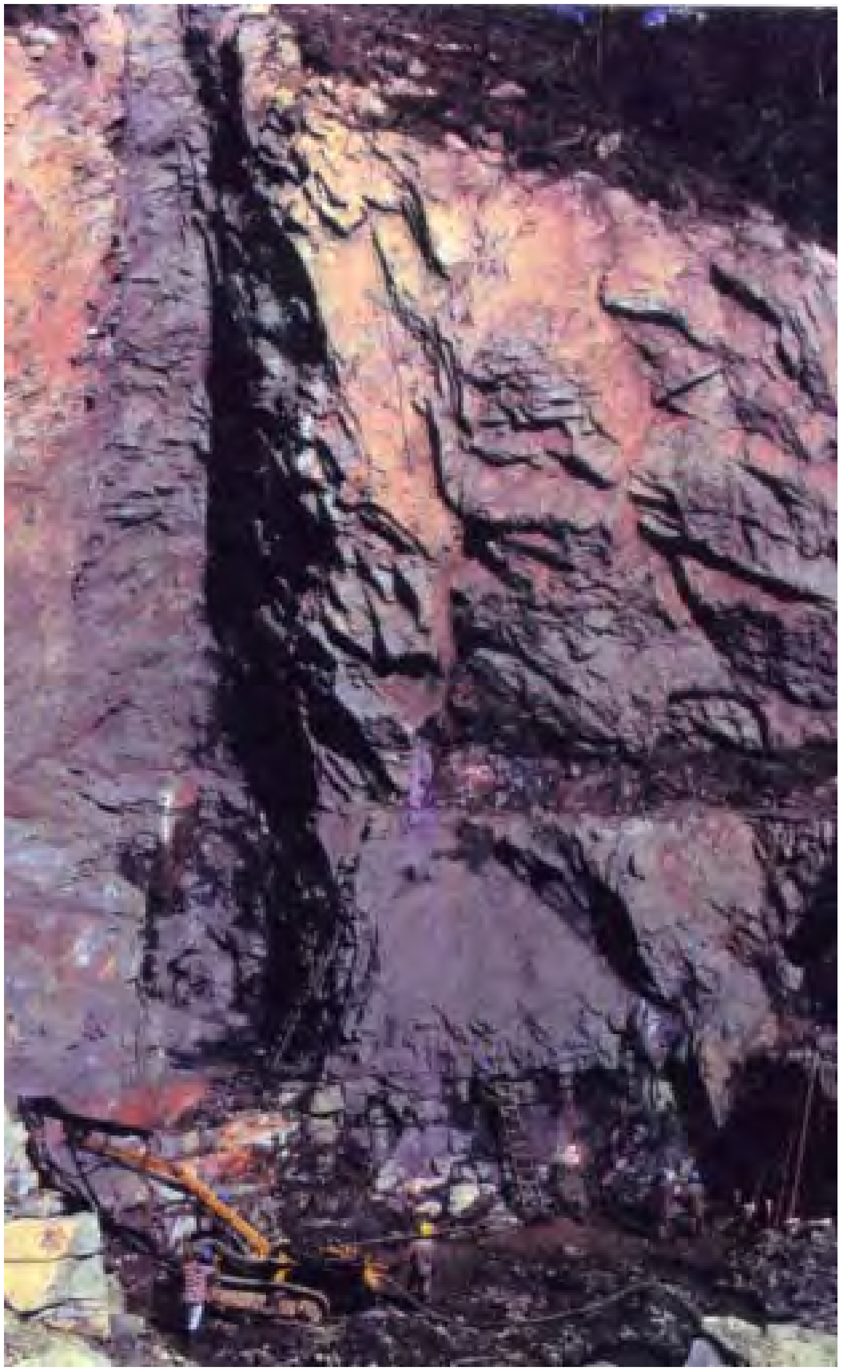


FIGURE 10

Before June 24, 2000 Rock Slide

FIGURE 11

After June 24, 2000 Rock Slide

FIGURE 12

Stabilized Slope May 2001

FIGURE 13

Crest of stabilized slope looking downstream
(May 2001)

FIGURE 14

Failure Surface
Strike 316° Dip 52°
(May 2001)

FIGURE 15

Access Roadway Bedrock Surface
Strike 318° Dip 50°
(May 2001)

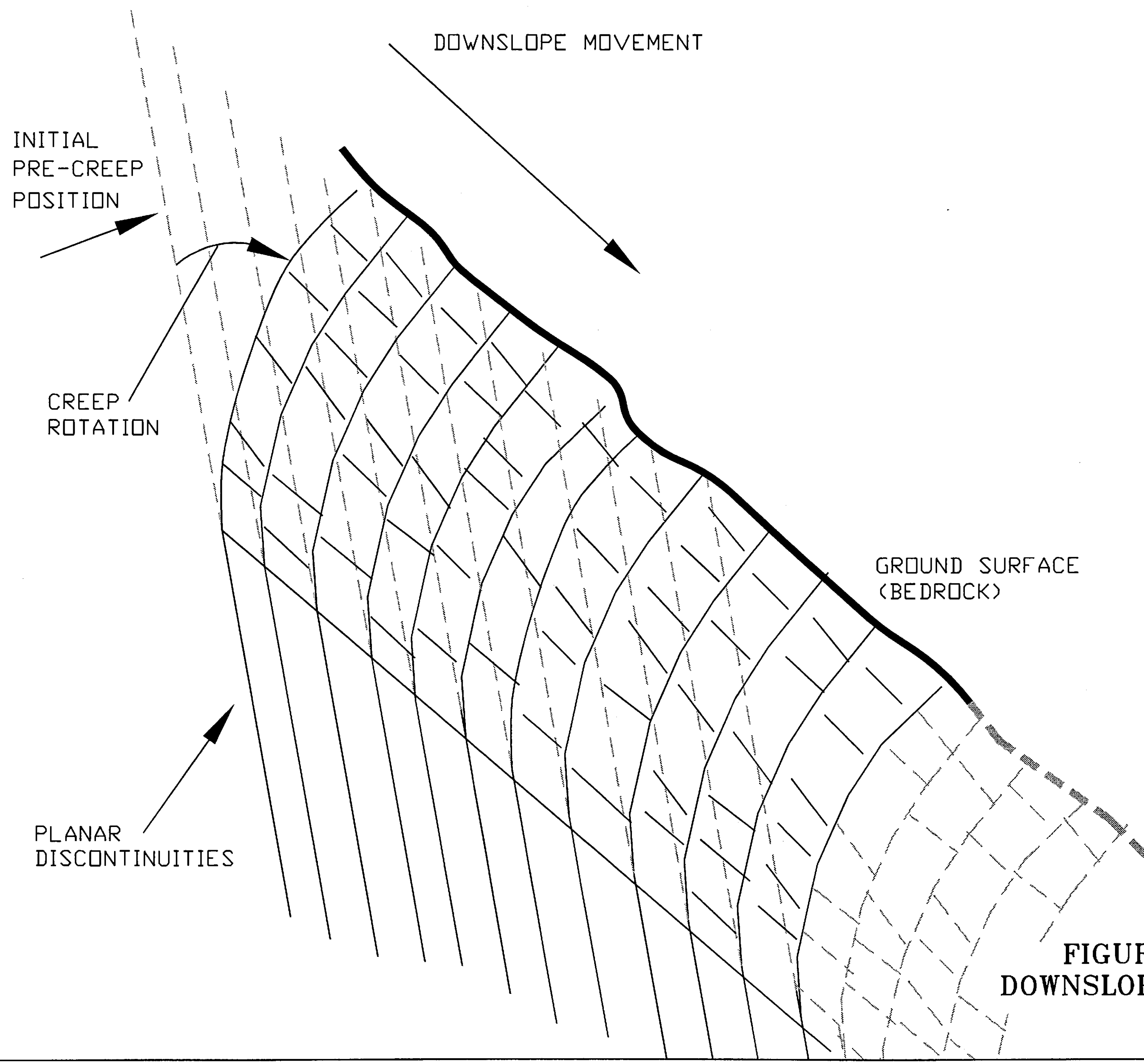


FIGURE 16
DOWNSLOPE CREEP

ACAD NO.:M:\CL\CAD\BORDERS\8.5x11h_SI.dwg

FIGURE 17

Toppling and Creep
Access Road (May 2001)

DIGITAL MAPPPING ASSISTANTAND LOGGER: TWO PALM APPLICATIONS FOR DIGITAL COLLECTION OF GEOLOGIC DATA USING A PDA AND A GPS RECEIVER AND A GEOTECHNICAL BOREHOLE LOGGING APPLICATION

by Randy L. Kath[1], Katie Ross[2], and Deana S. Sneyd[2]

[1]Department of Geosciences, State University of West Georgia, Carrollton, Georgia 30118
[2]Golder Associates Inc., 3730 Chamblee Tucker Road, Atlanta, Georgia 30340

Abstract-

Traditional geologic mapping employs US Geological Survey topographic maps and a field notebook. Once data are collected in the field, the data are transferred to a base map and structural data can be analyzed using a stereonet or other application. To expedite data entry, Digital Mapping Assistant (DMA-Palm) has been coded for Palm OS. Lithologic, structural, and positional data are collected using the personal digital assistant and a GPS receiver. Field data that are collected in the PDA are stored in an MS Access® database for easy access, analysis, and preparation of GIS datasets for ArcInfo® and/or AutoCAD Map/Land®.

DMA, developed by the author, consists of three separate forms. DMA-Palm is the main form to which all data are referenced. This form is used to store lithologic, positional, and datum information. DMA-Palm consists of fields which contain unique information that is collected at each mapping station. The other forms allow users to collect structural data in either quadrant or azimuth format. The relationship of the sub-forms to the master form allows users to collect structural data using the UserName and MapStation as the primary keys.

The DMAStructure sub-forms allow handheld users to collect information for both planar and/or linear structural features and store this information for each mapping station. The structure sub-forms use common lookup lists for planar and linear features, which can be customized on the host PC and uploaded to the PDA during synchronization for immediate use. The DMAStructureQuad sub-form consists of fields which contain unique quadrant-formatted structural information that is collected at each mapping station. The DMAStructureAzimuth sub-form consists of fields which contain unique azimuth-formatted structural information that is collected at each mapping station.

The use of the structural sub-forms allows entry of as many structural measurements at an individual mapping station as desired. Whether measuring one or five fracture(s), one or two foliation(s), one or multiple lineation(s), the user maintains complete flexibility for the number of structural measurements required. A sketch field has been included which allows digital field sketches. These sketches are stored in the database as bitmaps that can be edited or exported for other applications. DMA-Palm can be downloaded at http://www.westga.edu/~geosci/dma.

Following development of the digitial-based geologic mapping application, a palm-based geotechnical soil logging application was developed. The earliest version of the application was called SoilLogger; however, the application has been modified to accept piezometer/monitoring

well installation and rock core logs. The new application is called Logger and contains modules for soil, rock and well installation. Logger directly interfaces with a MS Access® database that can be imported into any borehole logging application. The soil module in Logger has been coded to directly interface with WinLog® via a PC-based application called SoilLogger-PC (see Kath and Williams, 2002). The PC application allows the user to populate the WinLog® database with a few mouse clicks. Once the WinLog® database has been populated, the user simply opens WinLog® and can print the finished logs. With the newest version of WinLog®, one can directly import and export to the gINT® logging application. Because this requires two logging applications (WinLog® and gINT®), the author is currently interfacing Logger with gINT®. A new version of Logger will soon be available with this interface.

The DMA-Palm was initially used by structural geology students on mapping projects at the State University of West Georgia. As a part of the instructional use, the author created a series of MS Access® queries and visual basic applications to automate a bridge between the DMA-Palm database and GIS point data. Creation of GIS point coverage and links to the original palm database allow the user to quickly plot digital geologic maps including structural and positional data collected with the DMA.

The DMA-Palm has also been used by Golder Associates Inc. for geologic mapping on two Tennessee Department of Transportation projects: State Route 40 in Polk County (Blue Ridge) and State Route 35 in Jefferson and Sevier Counties (Valley and Ridge). The DMA-Palm was further customized during and following these projects to allow input of geotechnical information and greater flexibility in entering structural and lithologic data. These projects provided the first real test of the dynamic dataset method due to the great variability in type and amounts of structural data. Because structural measurements were recorded digitally and attributed to the database, these data were quickly and easily processed into a stereonet file for use in analysis of structural stability for cut slopes in rock. Additionally, redundancy in data entry and consequently, human error, were essentially eliminated by collecting the data in a digital format.

Following the successful use of digital data collection for geologic mapping on DOT projects, Golder Associates decided to branch out and begin to collect soil boring and rock coring information in a digital format. Because the Logger developed by the author had not been interfaced with gINT® when highway projects with the Kentucky Transporation Cabinet and TDOT were awarded to Golder Associates this past summer, Golder Associates decided to purchase a digital logger that advertised itself as being easily customizable and fully interfaced with gINT®. Unfortunately, the digital logger purchased by Golder Associates required significant customization to allow real-time plotting of digital logger data into gINT® logs. Numerous iterations were required during the customization process because the developer did not have a good understanding of the flexibility required to enter datasets that are highly variable and complex. The time-consuming debugging of the software and significant training effort prohibited use of the digital logger on the TDOT projects; however, Golder Associates has worked through most of the problems and is ready to use the digital logger on future highway projects.

Introduction-

Traditional geologic mapping has been conducted for highway, tunnel, and other geotechnical projects using a topographic base map, commonly a USGS 7.5 minute topographic series, and a field notebook. Data are collected in the field and transferred to a digital format for processing once back in the office. Because of the potential for error in transferring field data from a paper format to a digital format, many workers have developed digital mapping applications. These applications range from pen-based laptop applications to palm-based digital field notebooks.

Traditional Mapping Gear

Digital Mapping Gear

During the annual meeting of the Geological Society of America (GSA) in Denver, Colorado (1998), an entire theme session was dedicated to digital geologic maps and digital collection of structural geologic data. Many different applications were presented at this meeting including the Geological Survey of Canada's Fieldlog® (see Brodaric 1997) and UC Berkley's GeoMapper®. However, after sitting through the entire sessions, the primary author became convinced that an efficient, cost-effective method to collect digital geologic data did not yet exist. GeoMapper® required carrying a pen-based laptop in the field which is expensive and extremely bulky for most mapping applications in the Southeastern US. Fieldlog® is a CAD-based application that requires data to be collected using a field notebook or Apple Newton and later transferred to the application for data compilation. Another aspect that was unclear from the GSA theme session was the platform for digital collection: Palm-based, Newton-based, Windows CE-based, or ArcPad®-based. Even though there was no clear consensus of what was best, a presenter from the USGS in Reston talked about collecting digital data using a palm-based platform. The advantage of the palm platform was its simplicity and efficiency of battery use. The palm system that was presented by Greg

Walsh with the USGS, used a simple series of forms to provide the basis of the digital mapping application.

After trying to develop different applications using a HP Jornada® running Windows CE® that contained digital topographic base maps, the primary author went back to his notes from the GSA theme session and was interested in the overlooked presentation by Walsh and a subsequent publication by Walsh, Reddy, and Armstrong (1999) on "Geologic Mapping and Collection of Geologic Structural Data with a GPS Receiver and a Personal Digital Assistant (PDA) Computer." Walsh and others (1999) used a form-based fully customizable application to collect structural data in a digital format using PDA. Their application is coded using Pendragon Forms® and is specifically designed to collect structural data for building ArcView® point coverage.

Walsh sent us the form-based application for review. Although this application was only designed for collection of structural data, the form-based techniques seemed to be the way to proceed. Using forms to build a digital geologic collection application allow for complete customability by the user. Once the base application is coded, any user running Pendragon Forms® can edit, modify, add, delete and change the application. Walsh and others application was the start of Digital Mapping Assistant (DMA-Palm). A detailed description of DMA-Pam was presented by Kath (2003) and the application and users manual are available for download.

In addition to developing a digital-based geologic mapping application, a palm-based geotechnical soil logging application has been developed. The earliest version of the application was called SoilLogger; however, the application has been modified to accept piezometer/monitoring well installation and rock core logs. The new application is called Logger and contains modules for soil, rock and well installation. Logger directly interfaces with a MS Access database that can be imported into any borehole logging application. The soil module in Logger has been coded to directly interface with WinLog® via a PC-based application called SoilLogger-PC. The PC application allows the user to populate the WinLog® database with a few mouse clicks. Once the WinLog® database has been populated, the user simply opens WinLog® and can print the finished logs. With the newest version of WinLog®, one can directly import and export to the gINT® logging application. Because this requires two logging applications (WinLog® and gINT®), the author is currently interfacing Logger with gINT®. A new version of Logger will soon be available with this interface.

DMA-Palm and SoilLogger-

The first versions of DMA and SoilLogger were coded to have a fixed number of field-entry data points within a data-record. This was somewhat problematic and was very restrictive to the handheld user. Also, this made the associated database bigger than necessary in that it contained a field for each record whether it was populated or not. For example, when collecting structural data at a single map station/outcrop the user was restricted to two foliations/beddings, four joint measurements, one fault measurement, and one lineation measurement. If the user needed to collect more than four joint orientations, the handheld user would have to create a new map station (i.e., MS1.0 and MS1.1).

Also, this was very problematic and restrictive with the Logger applications because the handheld user did not know how many split spoon samples, for example, that would be collected from a single borehole. To overcome this limitation, the early versions of Logger contained up to 15 samples to be collected. However, if auger refusal was very shallow, most of the 15 fields were empty thus creating a larger than necessary database.

The newest versions of DMA-Palm and Logger use a dynamic dataset method to collect multiple data at any given location or borehole. The handheld user simply adds a new data field to the record as necessary. For example, while conducting geologic mapping, the handheld user can enter as many planar or linear features as necessary. The records for each structural data entry are created on the fly. This allows the handheld user to measure one or 100 beddings, joints, faults, lineations. If no structural data is collected at an outcrop, then no structural records are created in the relational database for that particular map station, thus creating a smaller and more efficient database.

Structural data is added to the DMA-Structure Module by selecting the Add button. The user is then prompted for data type, either Planar or Linear. Because the structure form is dynamic, as much or as little data can be added to the handheld database.

Advantages and Disadvantages-

Advantages-

The major advantages of DMA-Palm are the quick access to data in a digital format. Once the data are entered in the handheld in the field, the data are interfaced with a database upon synchronization with a desktop computer or network server. This greatly reduces redundancy and increases accuracy of the processed data, i.e., field notebook copy and data re-entry errors.

Additionally, the data can be quickly interfaced with a GIS using automated scripting and querying. This allows collaborators to share data quickly and efficiently. The handheld can also interface with a GPS receiver; however, DMA-Palm is does not currently support direct interface with a GPS receiver (see disadvantages below).

User and User Groups- The Pendragon Forms® Manager maintains a list of active handheld units that are to be synchronized during a Hot-Sync data transfer. When the Pendragon Forms® Manager is first installed on the PC or server, the installation process prompts the user to enter a handheld user name, and this user name is added to the User list. If a multi-user license for Pendragon Forms® is activated, additional users can be added by clicking on the Users button in the Pendragon Forms® Manager. Once a handheld unit has been listed in the Users list, the Groups button is used to specify which forms should be sent to which user. Through the use of Users and User Groups, it is possible to centrally select a form to be sent to several handhelds. Each handheld will receive the form whenever next that handheld performs a Hot-Sync data transfer. It is also possible to centrally select a form for removal from various handhelds, by removing the form from a User Group. The next time that the handhelds in that group synchronize, the form will be removed.

Entering data on the Handheld-
Data are entered on the handheld using either record view or field view. Record View is a two-column format that displays field names in the left-hand column and responses in the right-hand column. The handheld user can tap in the right-hand column to enter a response for a given field. Field View displays the full field name and allows the handheld user to see only one field at a time on screen. Right and Left arrow buttons allow the user to move to the next or previous field on the form. The handheld user can switch from record view to field view at any time during data entry.

Field View

Record View

Disadvantages-
The disadvantages of DMA-Palm are relatively minor. The handheld user needs minor training, generally less than one hour, and specialized equipment, a handheld unit. The biggest disadvantage is the potential for loss of data using any electrical equipment in a field environment. This can be minimized by synchronization with a laptop at the end of each day. Then the most data one could lose is one days worth. To minimize the potential loss of data, the newest DMA-Palm writes all data to a backup memory card if it is available on the handheld. Handheld units are also susceptible to climate extremes, as geologists are! During data collection in very cold or hot weather, the handheld should be kept in a pocket to keep the screen warm/cool, respectively. Another disadvantage is the use of batteries. The early versions of DMA-Palm directly interfaced with a GPS receiver using the serial port on the handheld and the NEMA string output by the GPS receiver. Using the serial port on the handheld unit greatly diminishes the life of the batteries. Also, there is a need for cabling between the units which tethers one to the outcrop during GPS data collection.

Case Histories -
<u>Education</u>

DMA-Palm has been used for the past three years as a learning tool for beginning structural geology students at the State University of West Georgia. Undergraduate students have used palm hand held computers to digitally collect lithologic and structural data for classical geologic mapping. The version of DMA-Palm used does not contain any of the geotechnical attributes that are generally collected, including: weathering index; strength index; layer thickness; and joint dilation. As part of the instructional use of DMA-Palm, the primary author has created a series of MS Access® queries and visual basic applications (VBA) to automate a bridge between the DMA-Palm database and GIS point data. The output from these queries and VBA are used to create GIS point coverage and links to the original handheld database.

For example, structural data that is exported as a point coverage can be imported into a GIS. The import coverage uses the GPS coordinates, horizontal datum (hdatum), vertical datum (vdatum), and structural attitudes to place the appropriate structure symbol on a digital base map. The symbol is rotated based on the Strike/Trend angle and then labeled using the Dip/Plunge angle attributes. These data are uniquely identified in the DMA-Palm database using the Map Station attribute. By generating a link from the Palm database to the GIS, using the Map Station attribute, the coverage becomes fully linked to the palm database. Any changes to the external database will be automatically updated whenever the GIS is loaded.

In addition to automated point coverage's, once the rock/lithologic polygons are created for the final geologic map, a link between the polygon coverage and Palm database is made using the LithCode attribute in the database. Although these lithologic codes can be edited and changed, the author has developed a standardized code-color scheme at the State University of West Georgia. This code-color scheme allows standardization of data sets collected by various students, staff, and professors; yet another advantage of digital data collection.

TDOT SR40 - Ocoee River Gorge Project

The first use of the DMA-Palm for a DOT Highway project by Golder Associates was for the alternative alignment analysis for Tennessee State Route 40 near the Whitewater River Center constructed for the Olympics in 1996 in the Ocoee River Gorge. This project involved mapping a ½-mile wide, __-mile long corridor in extremely rugged and vegetated country in the Blue Ridge Physiographic Province. Geotechnical drilling was not performed; consequently, all geotechnical recommendations made were based on data collected from geologic mapping within the project corridor. Because of the ruggedness, this project was also an excellent use of GPS technology to accurately locate the outcrops where structural and lithologic data were collected. Additionally, the strong development of cleavage in the slates combined with joints, faults and bedding planes warranted collection of variable amounts of structural data at each exposure; the first real test of the dynamic data set method.

The module used for this project was not Geotech enabled. Because the strength index, weathering index, layer thickness, and joint dilation were not pre-programmed as lookup lists, this data was collected using the comments fields in the main DMA-Palm and DMA-structure modules. Because of the need for quickly separating these engineering parameters when estimating rock quality (either RMR and/or Q), the DMA-Palm Geotech Module was coded and made available for download.

Geologic mapping within the corridor was accomplished utilizing three different geologists. Generally this would require preparing photocopies of each others mapping notes and maps. Also, there is the potential for duplication of map stations which could be problematic when preparing a final base map. By using DMA-Palm these problems were eliminated. Each handheld users palm has a name, usually the person's name. Each time a record is written to DMA-Palm, the user name, date/time stamp, and Map Station become part of the primary key in the handheld database.

The use of these as the primary key allows for multiple users to have the same map station number. When plotting on field maps or the final map, one can create a dynamic dataset query to concatenate the user name with map station for presentation on the map.

Also, another advantage is the ability to load other peoples mapping data into your handheld. When working in a multi-user environment, each mapper can load the other mappers' data into their own handheld and plot all of the mapping data each night onto a common digital field base map. This greatly increases the efficiency of mapping and reduces redundancy. The field base map can then be plotted each night and made available for mapping teams the next day.

For the SR40 project, ____ map stations were evaluated for positional, lithologic, and structural data. At the end of the project these data were plotted as a georeference point file in a GIS in about 10 minutes. Additionally, ___ structural measurements were recorded using DMA-Palm.

Because these measurements were recorded digitally and attributed in the database using the Structure Type attribute (i.e., joint, cleavage, bedding, foliation, fold axis, slickensides, etc…) the data were processed into a stereonet file in approximately 10 to 15 minutes. Using the Structural Type attribute, stereonet analysis of all of the data or subsets of data for rock slope stability was easily implemented. Another interesting feature of using the GIS linked to the handheld database was the ability to perform domainal analysis of the structural data using spatial filters. This allowed us to evaluate local variations in the structural data that could be masked by regional trends.

TDOT SR35 - Sevier-Jefferson County Tennessee

Because of the successful use of the DMA-Palm for geologic mapping for the SR40 project, Golder Associates elected to use the DMA-Palm for geologic mapping along the proposed realignment of SR35 in Sevier and Jefferson Counties. This project involved mapping along a __-mile long alignment in the Valley and Ridge Physiographic Province of Eastern Tennessee. The alignment traverses shales, calcareous shales, and solutioned and competent limestone, and dolostone. The Geotech module was used for this mapping so that engineering properties of the various rock types could be easily separated. As with the SR40 project, structural measurements were recorded digitally and attributed to the database; consequently, these data were quickly and easily processed into a stereonet file for use in analysis of structural stability for cut slopes in rock. Additionally, redundancy in data entry and consequently, human error, were essentially eliminated by collecting the data in a digital format.

Following the successful use of digital data collection for geologic mapping on the SR40 and SR35 projects, Golder Associates decided to branch out and begin to collect soil boring and rock coring information in a digital format. With over 200 soil and rock borings planned for the SR35 project, collecting soil and rock borings data digitally for this project would result in great savings to the client. Because the Logger developed by the author had not been interfaced with gINT® when the SR35 project was awarded to Golder Associates this past summer, Golder Associates decided to purchase a digital logger that advertised itself as being easily customizable and fully interfaced with gINT®. Unfortunately, the digital logger purchased by Golder Associates required significant customization to allow real-time plotting of digital logger data into gINT® logs. Numerous iterations were required during the customization process because the developer did not have a good understanding of the flexibility required to enter datasets that are highly variable and complex.

For example, the customized digital logger initially would not allow for multiple soil or rock lithologies to be described in a single split-spoon sample or core run. Many fields (such as soil consistency) required data entry; otherwise the user could not proceed to the next or previous field. Ranges of moisture content (dry to moist) or ranges in grain size (fine to medium grained) could not be accommodated in the logger. One of the most cumbersome issues was the inability of the user to review all of the SPT data at a glance. If the user wanted to review N-values for an entire borehole, the user would have to review each sample or run and scroll through each of the fields to get the data. Significant thicknesses of soil encountered during rock coring (such that would occur in solutioned carbonate rocks) were difficult to describe and then plot in a representative way on the gINT® log.

Problems also occurred with the way all of the fields were keyed by depth. For instance, if a water level was measured at 10 feet below ground surface, and drilling conditions of note were encountered at 10 feet below ground surface (i.e. lost circulation), these fields would overprint on the gINT® log when the user printed it out. If groundwater was not encountered, the user could not record this information in the groundwater field because it is associated with depth. Consequently, the user would have to key this information into the comment field. Other problems included run times defaulting to current time. If the run started at 2:00 pm and ended at 2:21 pm, the run time field defaults to 2:21 pm as if you never entered the run start time as 2:00 pm. Other field would default as well, such as weathering and rock type. If these fields weren't specifically checked, inaccurate information would be recorded by default.

Problems also occurred in the way the digital logger is coded by using drop down lists for each physical property (color, moisture, consistency, etc.) which causes it to store these properties in separate cells in the database. Because these are not concatenated together into a single comprehensive description, editing the description in the database requires editing individual fields within a single record. The digital logger writes to a MS Access® database which is then used to populate the gINT® database. However, no internal checking exists to determine whether the hand held database or the PC database are the most current databases; consequently, this causes the synchronization to be unidirectional based on user input. This could cause edited fields in the MS Access® database to be overwritten by the handheld database during synchronization. Additionally, if the hand held database is appended to the MS Access® database duplicate records are created. For example, during the first synchronization, a record for a sample from 1 to 2.5 feet will be written in the MS Access® database. During resynchronization, if the same record is appended to the database, it creates a duplicate record, not a revised record.

The time-consuming debugging of the software and significant training effort prohibited use of the digital logger on the SR35 project; however, Golder Associates has worked through most of the problems and is ready to use the digital logger on future highway projects.

Lessons Learned

1. Be sure that project specific base maps contain appropriate metadata, i.e. zone, units, hdatum, vdatum, etc. Most DOT projects that we have worked on use: State Plane, feet, and NAD 83. If projects have site specific coordinates, the surveyors must provide at least two known points in State Plane or UTM to allow for reprojection.
2. Be sure that the GPS data collected in the field is collected in the same coordinate system as the USGS topographic base being used. This allows for plotting positional data in the field. Most all GPS units come from the factory in WGS84 using latitude and longitude

formats. The easiest units to plot in the field are UTM units because they are Cartesian coordinates and the map has UTM grid ticks every 1000 meters.

3. No matter how much preparation in the office, each application must be field tested to determine its limitations and what is missing!
4. DMA-Palm has continued to prove itself as an efficient, cost-effective and time saving application for digital geologic and geotechnical mapping.
5. When considering purchasing a digital logger, be sure that the software engineer has practical experience in the field in which the logger will be used. The digital logger purchased by Golder Associates was not ready to use "off the shelf". Unanticipated time was required to customize the logger to comply with Golder Associates' technical procedures for soil and rock logging.

References-

Brodaric, Boyan, 1997, Field data capture and manipulation using GSC Fieldlog V3.0, in D.R. Soller, editor, Digital Mapping Techniques '97, Proceedings of a Workshop on Digital Mapping Techniques: Methods for Geologic Map Data Capture, Management and Publication: U.S. Geological Survey Open-File Report 97-269, p. 77-81.

Kath, R.L., 2003, Digital Mapping Assistant: A Palm Application for Digital Collection of Geologic Data using a PDA and GPS Receiver: Geological Society of America, Abstracts with Programs, Combined Southeastern and North-Central Section Meeting.

Kath, R.L., and Williams, L., 2002, Digital Collection of Geologic and Geotechnical data using a Personal Digital Assistant (PDA) and a GPS receiver: Southeastern Section of the Society for Sedimentary Geology (SEPM) short course, combined North-central and Southeastern sections of the Geological Society of America meeting in Lexington, Kentucky, April 3-5, 2002.

Walsh, G.J., Reddy, J.E., and Armstrong, T.R., 1999, Geologic Mapping and Collection of Geologic Structural Data with a GPS Receiver and a Personal Digital Assistant (PDA) Computer: Digital Mapping Techniques '99-Workshop Proceedings, US Geological Survey Open File Report 99-386.

A large volume trim blast for slope stabilization, US 97A near Wenatchee, Washington

Tom Badger[1]

ABSTRACT

The recent removal of a roughly 4,000 cubic yard mass by trim blasting was successfully completed for a 170 foot high rock slope located on US 97A just north of Wenatchee, Washington. The mass was bound by a very persistent discontinuity that dipped toward the highway at about 55 degrees. The large bolting forces required to stabilize the mass coupled with the apparent instability were determining factors to remove the mass. Overhead high voltage transmission and fiber optic lines, a railroad, and the Columbia River were immediately adjacent to the slope and highway. The blast design entailed drilling 130 blast holes, during which time the mass was remotely and continuously monitored with tensiometers. Blast holes were carefully located and logged, and decked explosives and stemming were tailored to each hole. Each hole was individually delayed in a pattern to optimize the movement and fragmentation of the debris. The mass was draped with geotextile and chainlink fabric to control flyrock, and a temporary earthen berm between the highway and railroad helped contain the debris. The design efforts resulted in near complete containment of debris behind the berm, no damage to the railroad or utility lines, very little flyrock, and excellent fragmentation. The slope was scaled, and the highway was reopened to two-way traffic within 8 hours of the blast.

INTRODUCTION

Like many transportation agencies in the United States, the Washington State Department of Transportation (WSDOT) has programmatically identified slope hazards that impact the highway system. Additionally, WSDOT has a prioritization and programming system to mitigate for these hazards, which is currently funded at around $15 million annually. As part of this program, a large rock cutslope near Wenatchee, Washington (Fig. 1) was recently programmed to address previously identified rockfall hazards.

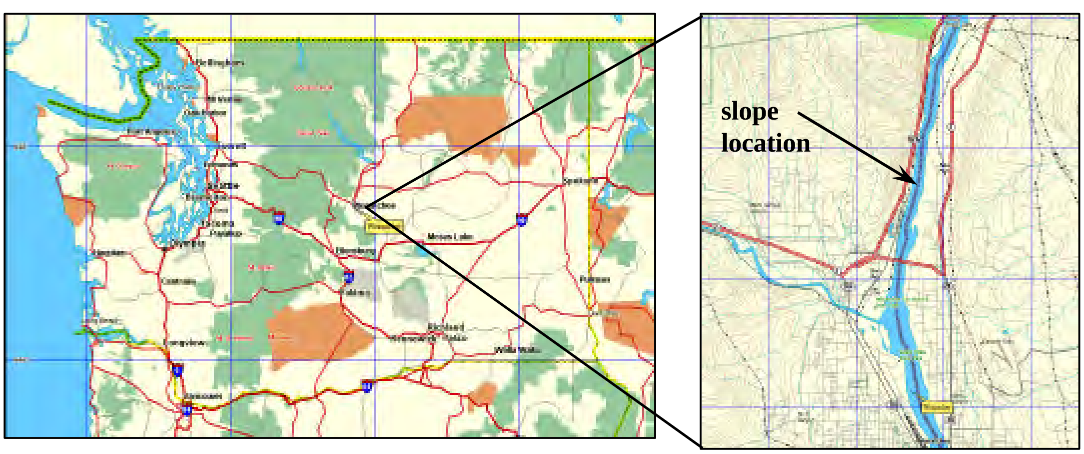


Figure 1. The slope is located at Mile Post 202.4 on U.S. Route 97A, situated on the west bank of the Columbia River just north of Wenatchee, Washington.

The geotechnical investigation identified a large, potentially unstable mass in the upper portion of the slope (Badger, 2002). After evaluating stabilization alternatives, it was

[1] Washington State Department of Transportation, P.O. Box 47365, Olympia, WA 98504-7363
badgert@wsdot.wa.gov; 360-709-5461

determined that the safest, most cost-effective alternative would be to remove the mass using trim blasting methods. The proximity to critical facilities, the precariousness and large size of the mass, access difficulty, and significant highway and railroad impacts complicated the task. This paper summarizes the site conditions, design and results of the shot, and contract provisions for this large volume trim blast.

SITE CONDITIONS

The slope is located about 2.5 miles north of Wenatchee, Washington in the central Columbia River valley. The steep, east-facing cutslope is approximately 400 feet in length (measured along the highway) and exceeds 170 feet in height. The overall slope angle is about 70°; at the crest of the rock cut, the slope flattens to a large, gently sloping bench. A flat ditch section between 4 to 12 feet in width provides minimal rockfall catchment at the base of the cutslope. The highway is an important two-lane facility that carries more than 6,000 vehicles daily. Railroad tracks adjacent to the highway service several freight trains per week, and overhead high voltage and fiber optic lines are located just downslope of the tracks (Fig. 2). The Columbia River is situated within several hundred feet of the cutslope.

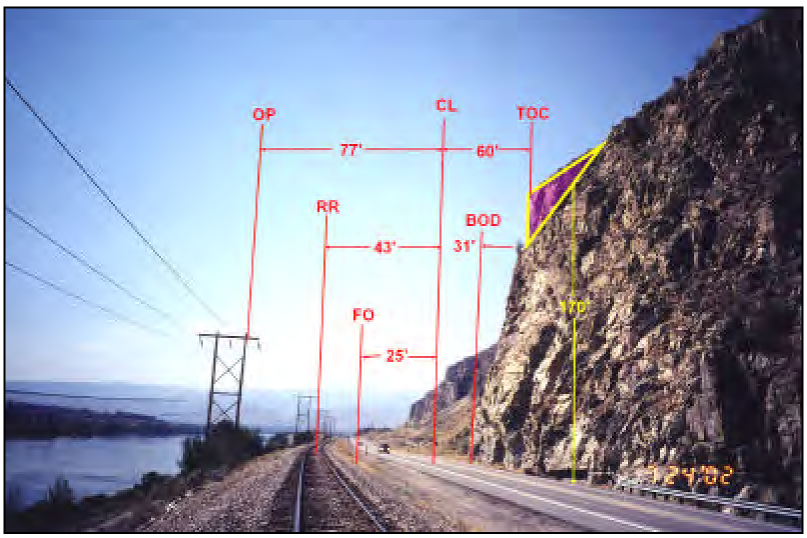


Figure 2. View of slope looking south; unstable mass is shaded triangular section at the top of cut. Horizontal distances reference the top of cut (TOC), back of ditch (BOD), centerline (CL), fiber-optic cable (FO), railroad (RR), and overhead power (OP).

Tabor and others (1982) mapped bedrock as the Swakane Biotite Gneiss. The gneissic rock mass is strong and mostly fresh to slightly weathered, but it also contains some degraded, highly weathered dikes that are mostly sub-horizontal and trend north-south. The rock mass is dissected by numerous discontinuities of variable persistence. The more persistent discontinuities define many potential planar, wedge and toppling failures. One such notable discontinuity was identified in the southern, upper third of the cut.

Figures 3a and 3b show a very persistent planar feature that bounds a large mass with dimensions of roughly 70 feet in length, 50 feet in width and 30 feet thick; the volume of the mass is about 4,000 yds^3. It also contained numerous open fractures indicating that it had experienced significant strain. The bounding discontinuity is planar, mostly smooth, very persistent and comprised of a highly weathered/degraded mafic dike. This discontinuity dips out of the slope around 55°. This large mass rested on a 6- to10-foot thick, highly fractured zone (pink shaded area in Fig. 3b).

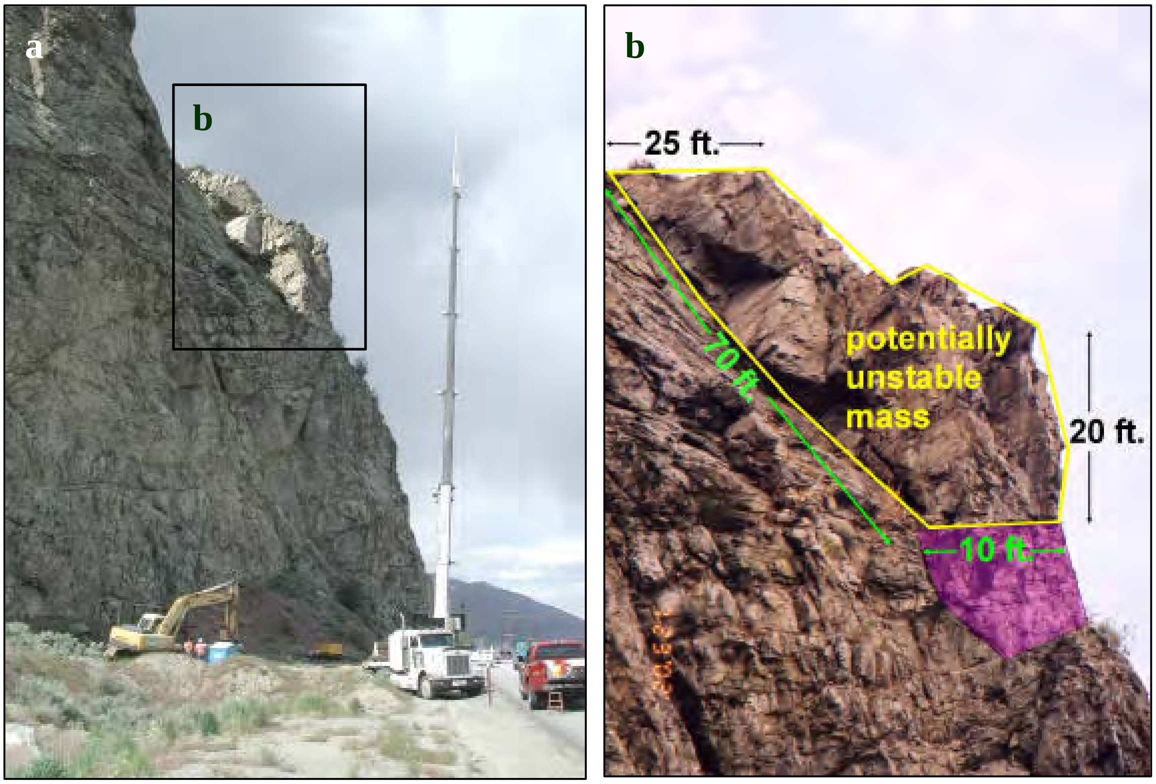


Figure 3. a) View looking north of potentially unstable mass; crane has 220 foot boom for rough scale.
b) Enlargement of mass showing dimensions. Shaded pink area that supports large mass is highly fractured, and also contains numerous open discontinuities.

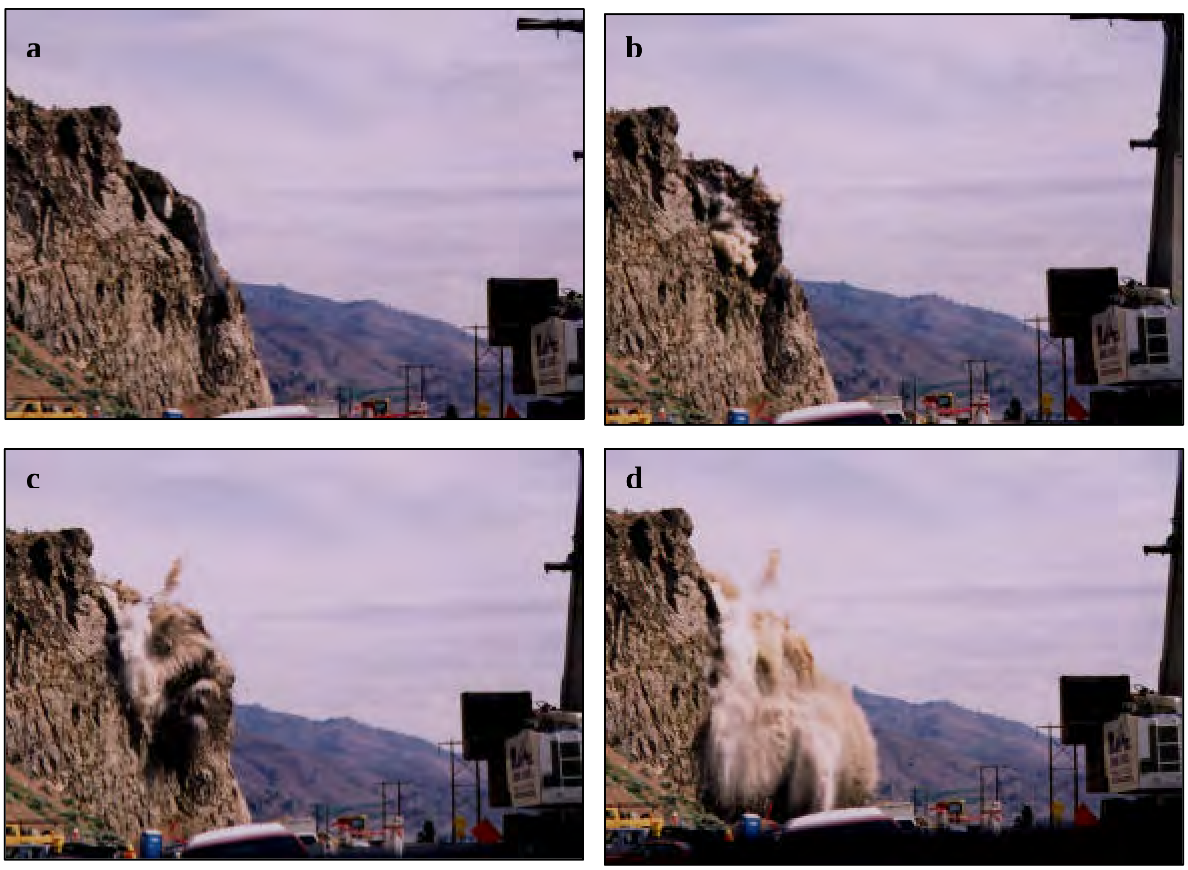
TRIM BLAST DESIGN AND RESULTS

To mitigate this slope hazard, two alternatives were considered; they included reinforcing the mass with high capacity rock anchors and removing it by trim blasting. Analysis for the reinforcement assumed a factor of safety near unity based upon the observed strain, and yielded a required anchor force to achieve a FS≥1.25 of around 1.9×10^6 lbf, or around 95–200 kip anchors. Given the estimated high cost to install the anchors (estimated to be around $250,000) and safety concerns over constructing the anchors in a fractured and strained mass, trim blasting was selected as the preferred alternative.

The removal of potentially unstable rock masses on slopes using trim blasting is an effective mitigation measure, but its use is often avoided or not considered. Trim blasting is typically considered where: 1) masses are bound by well-defined, adversely oriented discontinuities; 2) masses are too costly or potentially unsafe to stabilize; and, 3) adjacent facilities and structures can be safeguarded. To help ensure the third criteria, the contract required the contractor to retain a blasting consultant and continuously monitor deformation of the large mass during the drilling operation. The monitoring was accomplished using multiple “string pot”-type potentiometers and an electronic data acquisition system wired to a warning system. A ¼” deformation threshold was agreed upon by the Contractor and WSDOT.

The blast design was prepared by the contractor and his blasting consultant, and reviewed by WSDOT. It entailed drilling 130 blast holes using a crane-supported drill boom for the deeper horizontal holes and hand drills for the shorter vertical holes. Typical hole spacing and burden was 4 to 5 feet, and hole depths ranged from 3 to 29 feet. Blast holes were carefully oriented, drilled and logged to ensure adequate burden between holes and confinement of blast energy. Blast holes were located entirely within the mass; holes that inadvertently penetrated through to the bounding discontinuity were backfilled several feet. Decked explosives and stemming were then tailored to each hole. Stemming lengths ranged from 4 to 9 feet. Holes were individually delayed in a pattern to optimize the movement and fragmentation of the debris. The initiation pattern planned for moving the rock to the south, parallel to the slope, into a wide area at the base of the slope.

The mass was draped with geotextile and chainlink fabric to control flyrock, and a temporary earthen berm between the highway and railroad was provided for in the contract to contain the debris. Timbers were also placed between the railroad tracks to protect them from damage. The highway was to be flooded with 2 feet of sand to help protect the pavement, but the blasting schedule was accelerated due to strong winds, and the sand blanket was omitted. An overhead fiber-optic cable on the adjacent utility poles was encased in pvc pipe to mitigate potential flyrock damage. Figures 4a through 4f show a photo sequence of the blast.



With the exception of a roughly 100-yd^3 mass that was not drilled due to its instability (previously detected by the deformation monitoring), the blast successfully removed the entire 4,000 yd^3 mass. Most of the debris was contained in the shoulder area to the south as planned; less than ¼ of the debris volume covered the highway, and only several small rocks reached the railroad tracks (Figs. 5a and 5b). There was no damage to the overhead or buried utilities, and only minimal damage to the roadway. The blast also produced excellent fragmentation, which greatly expedited the cleanup. Eight scalers rapidly removed the small amount of debris remaining on the slope, and the highway was reopened to two-lane traffic within 8 hours following the blast.

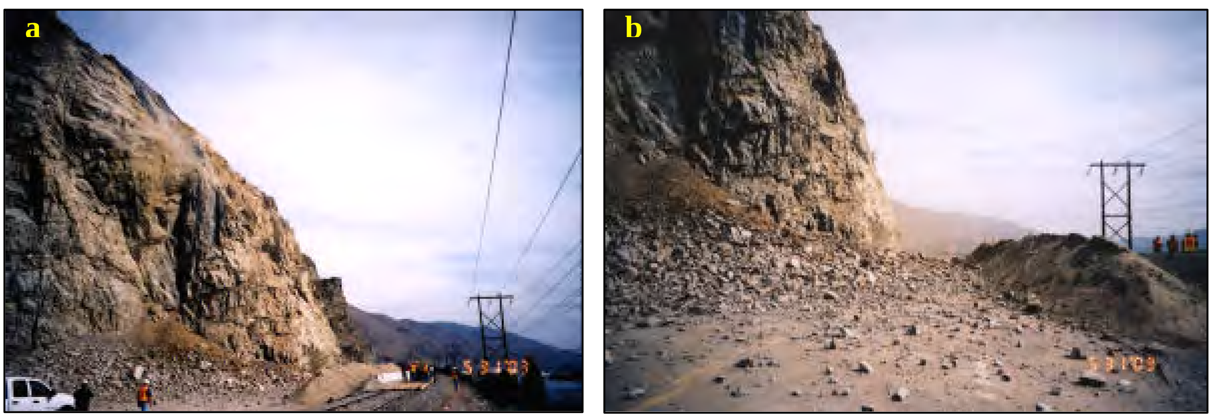


Figure 5. a) Post-blast photo of slope and bounding discontinuity. b) Debris from blast; note fragmentation.

SUMMARY OF TRIM SHOT

Mitigation of the potentially unstable mass using trim blasting proved successful. The containment of the debris, minimal flyrock, and high degree of fragmentation were primarily due to a well-designed blast. The large number of carefully located and logged blast holes, as well as the initiation pattern, provided excellent control on debris movement. Deformation monitoring detected movement of a large block during drilling exceeding the ¼" threshold, and the monitoring system successfully alerted the Contractor. Drilling of this area was terminated, and no other deformation was detected during the remainder of the drilling. Advanced notification of the stabilization work and highway closure was provided to the public. The utilities and railroad companies, as well as the public, were very cooperative.

The geotechnical report recommended that the work be bid on a per cubic yard basis to achieve the most competitive bid and encourage innovation. Instead, the work was set up as a Force Account item for a total of $100,000, because it was felt that there was considerable uncertainty in the work and WSDOT would have more control over the Contractor's operations. The Contractor that was awarded the work elected to access the slope with a large crane at considerable expense ($95,000), which was not fully anticipated in WSDOT's design estimate. Consultant costs for the blast design and monitoring totaled around $50,000. Because the slope was not accurately surveyed, the volume of mass was underestimated by 50-100%. More scaling was also needed to prepare the slope for drilling than what was anticipated. The final cost for removing the roughly 4,000 yd^3 mass was around $300,000, which equates to a unit cost of around $75/$yd^3$. This unit cost is about 25% less than another recently completed WSDOT project for a large volume trim blast with similar difficulties with slope access, traffic, and proximity to critical features.

REFERENCES

Badger, T.C., 2002, *Geotechnical Report: Wenatchee North Unstable Slopes, SR-97A MP 202.40 to 202.45*, Washington State Department of Transportation, Olympia, WA, 8 figures and 6 p.

Tabor, R.W., V.A. Frizzell, Jr., J.T. Whetten, R.B. Waitt, Jr., D.A. Swanson, G.R. Byerly, D.B. Booth, M.J. Herrington, and E.R. Zartman, 1987, Geologic map of the Chelan 30-Minute by 60-Minute Quadrangle, Washington: *U. S. Geological Survey Miscellaneous Investigation Series Map I-1661*, 1 sheet and 33 p.

Design of Passive Dowel Systems and Perimeter Control Blasting Measures for Stabilization of Excavated Rock Slopes.

Danny J. Van Roosendaal, P.E., P.G
Nicholas H. Strater, P.G.
Andrew F. McKown, P.E

Haley & Aldrich, Inc.
465 Medford Street, Suite 2200
Boston, MA 02129
617-886-7400
djvr@HaleyAldrich.com

Introduction

Rock slope failures are costly to clean up and may leave owners vulnerable to lawsuits in the event of injuries or property damage. Design of new rock cuts or the remediation of existing rock slopes is a vital step toward reducing the risk of rockfall or rock slope failures.

There are a number of strategies engineers can employ to prevent rock slope failures, or rock falls. For remediation of existing slopes, geologic characterization is the first step in understanding the likely failure modes in the rock slope. Two broad types of remedial measures can then be employed, those that reduce the driving forces that destabilize the slope and those that increase the resisting forces in the slope. To reduce driving forces, engineers can do the following:

- Scale the rock slope face. Removing loose rock from the slope face reduces the potential for rock fall and slope failure.

- Trim the slope with blasting. Careful drilling and blasting can remove portions of large, overhanging rocks along the rock slope, preventing future slope failures.

- Drain the slope. Holes drilled at an upward angle from the face of a rock slope can intercept and collect water from discontinuities in the rock mass behind the slope face. The drain holes, which are left open and uncased, stabilize the rock slope by relieving water pressure that can build up in the discontinuities.

Engineers can also remediate potentially hazardous slopes by increasing the resisting forces using the following methods:

- Rock anchors consist of steel bars or steel strands anchored to the bottom of drilled holes either mechanically or with grout or epoxy resin. Once installed, the anchors are pre-stressed and locked off with a plate and nut at the rock face. They act in tension to resist block movement and increase frictional resistance along the joint surface.

- Like rock anchors, rock dowels are installed in drilled holes but are not tensioned and are fully enclosed in grout or resin. These elements will be discussed in more detail later in this paper.

- Shotcrete or concrete can perform similarly when applied to areas of highly fractured rock by preventing progressive erosion of the rock or filling relatively large voids.

- Netting or mesh draped over a slope can keep small blocks in place, slow falling rock, or channel it into the ditch.

For new rock slopes, characterization of geological conditions can be used to understand potential failure modes and optimize the slope angle to minimize the risk of failures. Perimeter controlled blasting measures are then implemented to excavate a stable final rock cut. If the slope must be cut steeper than the optimal angle, then slope stabilization measures, such as rock anchors or dowels, can be used to reinforce the slope.

This paper will highlight the use of two particular design measures that the authors have used successfully on several rock slope projects: perimeter control blasting and rock dowels. When used separately or in concert, perimeter control blasting measures and passive dowel systems can provide cost-effective design alternatives for new or rehabilitated highway rock cuts. Because of the key role of understanding failure modes in the rock slope, geologic characterization is also discussed in the context of the design and implementation of these measures.

Geological Characterization and Stability Analyses

The first step in the design of stable rock slopes is the comprehensive field characterization of the subject rock mass. If present, existing rock slopes are examined for evidence of previous failures or current instability, and factors impacting the behavior of the slopes are identified. Such factors may include existing slope geometry, lithology, weathering profile, drainage characteristics, ice wedging, vegetation, and active surcharge loads. This initial examination is followed by detailed structural mapping of the exposed bedrock, where the orientation, spacing, extent, and surface characteristics (roughness and waviness) of discontinuities are recorded. It is also important to note how existing discontinuities (e.g. joints, faults) interact to form blocks and wedges.

In areas where no exposed bedrock is present, or where deep excavation is planned, the rock mass is investigated through sampled geotechnical borings, drilled to depths of at least 5 feet below the toe of the proposed slope. This allows verification of the rock type, weathering profile, rock quality designation, and relative orientation of existing joints. However, it should be noted that standard borings do not allow determination of specific discontinuity orientations due to the uncontrolled rotation of the sample in the coring apparatus. Where specific orientations are necessary, oriented coring or downhole imaging may be implemented, although often at considerable cost. Sampling of rock core also provides specimens for laboratory testing to determine the shear-strength properties of discontinuities.

Following field characterization, the optimum slope angle is evaluated with a kinematic analysis using stereographic projections of the discontinuity orientation data. Three types of potential block failure are typically considered; sliding, wedge failure, and toppling. By definition, sliding includes all block failures occurring along a single plane dipping out of the slope face, and is particularly important when dealing with bedded or foliated rocks. Wedge failure includes all blocks released by the intersection of two or more discontinuities, where the block slides along a line of intersection. Toppling occurs where closely spaced discontinuities dip steeply into the slope, allowing blocks to fall away from the face. An example of a kinematic analysis (stereographic projection) used for an evaluation of planar block failure susceptibility is shown in Figure 1.

If present, blocks which may be kinematically susceptible to failure are evaluated individually by limit equilibrium analysis. This involves calculation of a safety factor against failure using the methods outlined by Hoek and Bray, 1981. In general, the limit equilibrium calculations require the following input parameters;

- Slope height
- Slope angle
- Orientation of the contributing discontinuity planes
- Unit weight of the rock type
- Cohesion of each plane
- Friction angle of each plane
- Uplift forces due to water pressure
- External driving forces (e.g. existing surcharge)
- External resisting forces (e.g. proposed reinforcement).

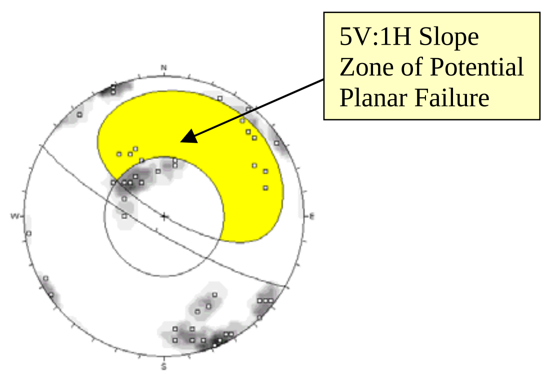


Figure 1 Stereographic projection of rock joint orientation data used for kinematic analysis of the planar failure mode

In rare instances, the actual friction angle and cohesion of the contributing planes can be determined through direct shear testing of rock core or block samples. However, most limit equilibrium analyses utilize friction angles obtained from published literature. Cohesion is comparatively difficult to estimate, and is often considered to be negligible to ensure a conservative evaluation.

Perimeter Control Blasting

Perimeter control blasting involves the control of the spacing and loading of blast holes along the perimeter of a blast round in order to minimize overbreak and damage to rock beyond the limits of the excavation. When properly implemented, perimeter control blasting will provide smooth, straight, and stable rock slopes and minimize the need for other stabilization measures. Perimeter control blasting measures best suited for use on highway rock cuts include pre-splitting and cushion blasting. A third method, line drilling, can be used when excavating near existing structures such as bridge abutments or pipelines. When properly specified and implemented, these measures can limit the blast-induced damage to the final cut rock slope. Each of the specific perimeter control blasting methods are summarized below.

- With pre-splitting, a line of holes is drilled along the perimeter of the excavation, loaded with light charges, and detonated prior to blasting and removal of the primary excavation. It should be noted that pre-splitting can cause higher vibration levels because the blasts are confined. Also, fractures in the rock can provide a pathway for explosive gases and result in opening of joints and damage to the rock beyond the perimeter. This method is appropriate for new excavations distant from existing structures. The use of pre-splitting for a building excavation is illustrated in Case History 1 at the end of this paper.

- Cushion or 'trim' blasting is similar to pre-splitting, but the lightly-loaded perimeter charges are initiated at the end of the production blast sequence. Cushion blasting can be used to effectively remediate existing rock slopes by cutting the slope back to a more stable angle, or to increase the width of rockfall catchments. Case History 2, which is presented at the end of this paper, illustrates the use of cushion blasting to remediate a rock slope on the New Jersey Turnpike.

- Line drilling involves the pre-drilling of closely spaced, unloaded holes along the final perimeter of the excavation that provide a plane of weakness to which the production blast can break. This method is relatively expensive due to the close spacing of drill holes required (approximately 6 to 9 inches on center), and should be considered if blasting is close (within 50 ft) to existing structures or if large seams or voids are encountered in the rock during drilling of the blast holes. The use of line drilling adjacent to and beneath the Maine State House in Augusta Maine is illustrated in Case History 3 at the end of this paper.

Several factors affect perimeter control blasting results and therefore should be considered in the design and addressed in the contract documents. These include geology, drilling accuracy, hole spacing and loading, and delay timing. For the benefit of the reader, Figure 2 is provided to define typical bench blasting terminology.

A thorough understanding of the slope geology is required for proper design and implementation of a perimeter control blasting program. Because of the potential for deviation during drilling, joint orientations that are sub-parallel to the finished slope present greater difficulties with drilling and blast hole loading than joints oriented perpendicular to the finished slope. Weathered and highly fractured rock generally requires a closer hole spacing and lighter charge loading to prevent flyrock and to improve fragmentation. Highly weathered seams and gouge zones should be observed during drilling and require stemming to prevent the rapid escape of explosive gases. The presence of such zones also precludes the use of more gassy explosives such as ANFO.

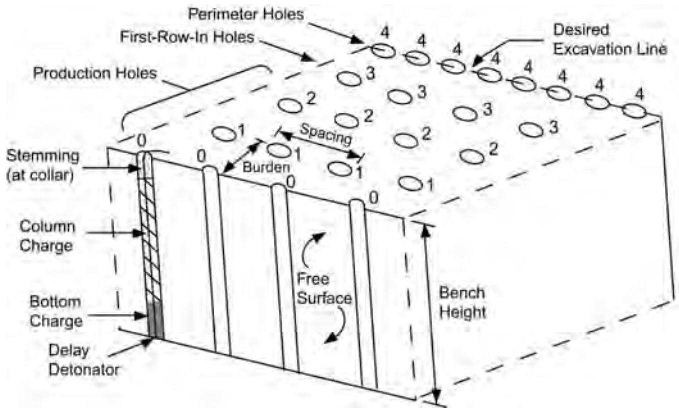


Figure 2 Schematic block diagram showing bench blasting terminology

Drilling accuracy is important to ensure uniformity of hole spacing and burden for the entire height of the blasted lift. Drilling accuracy can be improved by the use of a clinometer device on the drill rig, as well as stiff drill rods and drill guides. Also, the use of a borehole deviation measurement device along with specified deviation tolerances, improves the drilling accuracy and blasting results. For cushion blasting used to trim an existing slope, borehole deviation measurements are used in concert with laser profiling of the existing slope surface to ensure proper loading of holes adjacent to the face.

Perimeter hole spacing and loading have a significant impact on perimeter control blasting results. It has been our experience that blasters will generally drill perimeter holes at a spacing wider than optimum and compensate by increasing the charge loading of the holes, resulting in blast damage to the finished slope. Therefore, it is our practice to include loading guidelines in specifications for pre-splitting and cushion blasting (Tables 1 and 2). These guidelines provide an acceptable range of hole spacing and hole loading but still allow for adaptation by the blast designer.

Table 1. Pre-splitting Guidelines used in specifications

Hole Diameter (in)	Hole Spacing (ft)	Column Load Charge Concentration (lb/ft)
1.5 to 2.5	1.0 to 1.5	0.06 to 0.15
3.0 to 4.0	1.5 to 2.5	0.10 to 0.20

Table 2. Cushion Blasting Guidelines used in specifications

Hole Diameter (in)	Hole Spacing (ft)	Burden (ft)	Column Load Charge Concentration (lb/ft)
1.5 to 2.0	1.0 to 1.5	2.5 to 3.0	0.06 to 0.15
2.5 to 4.0	1.5 to 2.0	3.0 to 3.5	0.10 to 0.25

The orientation, loading, and spacing of the first row of production blast holes adjacent to the perimeter holes also should be controlled in order to minimize damage to the final cut face. These holes should be drilled parallel to the perimeter holes so that the larger production charges are not too close to the perimeter. The charge loading should be reduced by about 25 to 50 percent from the typical production loading to avoid large charges adjacent to the perimeter. To ensure adequate fragmentation in this zone with reduced charges, the spacing and burden of the first production row should be reduced by about 25 percent from the typical production hole pattern.

It has also been our experience that blasting results are optimized if the typical production hole spacing is less than the burden.

Perimeter control blasting results also are dependent on the design of the blast initiation or delay timing. Groups of adjacent perimeter holes should be initiated on the same delay (as permitted by vibration considerations) to enhance the uniform shearing between perimeter holes.

Performance of perimeter control blasting can be specified using the Half-Cast Factor (HCF). Half casts are the remnants of perimeter control blast holes that are visible on the final cut slope. The Half-Cast Factor is defined as:

$$HCF = \frac{\text{Total Length of Half Casts Visible}}{\text{Total Length of Perimeter Holes}}$$

Other recommended specification provisions for perimeter control blasting include:

- Pre-qualification: the blaster should demonstrate similar experience with perimeter control measures.
- Blast Plans should be submitted that detail all aspects of the blast design.
- Test Blasts should be required prior to full scale blasting. Perimeter control results should be evaluated and changes to the blast design should be made if needed.
- Payment provisions: Perimeter holes should be paid by the linear foot as a separate pay item. This provision fairly compensates the contractor for adaptations to the perimeter control blast design.
- Traffic delays: For remediation of existing slopes, a penalty clause can be added to stipulate that the contractor will be fined if blasting results in excessive traffic delays.

Rock Dowels

Rock dowels consist of steel bars or cables that are placed in boreholes and fully encapsulated in resin or cement grout. Dowels are considered “passive” support systems because, unlike rock bolts, they are not tensioned during installation. Therefore, displacements of the rock mass or joint surface are required to mobilize the shear resistance provided by the dowel.

Spang and Egger (1990) completed a thorough laboratory and field study of the mechanical behavior of rock dowels. Their work included shear tests and computer modeling of dowels installed across shear planes. Major conclusions pertinent to this paper are:

- Only small displacements on the shear plane are required to mobilize shear resistance from the dowel.

- Shear resistance from the dowel is mobilized very close to the point of shear. The dowel yields and bends and therefore quickly mobilizes axial tensile strength. Inclined dowels provided more shear resistance than dowels oriented normal to the shear plane, because the axial strength is mobilized without significant bending.

- Dowels are more effective on wavy or rough joints. The dowel limits displacements normal to the joint surface (dilatency) that are needed for shear displacement on wavy and rough joints.

- The shear resistance provided by the dowel increases as a function of the joint friction angle.

This work verified the common observation that passive dowels in many applications often perform better than tensioned rock anchors to provide “reinforcement” of a jointed rock mass. The key advantage to passive dowels is that shear resistance is provided at every point along the grouted length of the dowel. Also as pointed out by Spang and Egger (1990), due to factors such as dilatency and the correlation with joint friction angle, fully grouted dowels can provide additional shear resistance to a joint plane that equals and sometimes exceeds the tensile strength of the dowel bar.

An additional benefit of using fully grouted dowels rather than tensioned rock anchors is cost savings. Dowels are less costly to install because they do not require the multi-step grouting and tensioning process needed for anchors. Although horizontal dowels typically have face plates affixed nuts to stabilize the immediate rock face, these components are much less complex and costly than the head assemblies required for rock anchors.

Two general types of rock dowels systems, rock dowel pre-support (or vertical dowels) and horizontal dowels, have been successfully used by the authors for rock excavations. Figure 3 schematically shows the use of these dowels in a rock excavation adjacent to a building or bridge abutment. Case Histories 1 and 2, which are provided at the end of this paper, illustrate the use of vertical and horizontal dowels in association with pre-splitting and line drilling, respectively. These dowel systems are explained further below:

- Rock dowel pre-support (or vertical dowel support) is typically installed with a near vertical orientation behind the rock face prior to excavation. These dowels are often installed with a spacing of 5 ft or less. The shear strength of these dowels is quickly mobilized by small displacements that occur as the excavation is progressed. Rock dowel pre-support is often used together with perimeter control blasting measures and horizontal dowels to protect structures close to the excavation. The grouting of the closely-spaced dowels prior to excavation fills open joints connected to the hole, which cuts off pathways for explosive gasses to enter and damage the remaining rock slope.

- Horizontal dowels are installed on the face of the rock cut much like tensioned rock bolts or anchors. They can be used to support individual rock wedges or blocks, as is typically done for remediation of existing slopes, or can be installed in horizontal rows as each lift of the excavation progresses. When installed at each lift of the excavation, the shear strength of the dowels is mobilized by small displacements caused by excavation of subsequently lower lifts, much like soil nail systems.

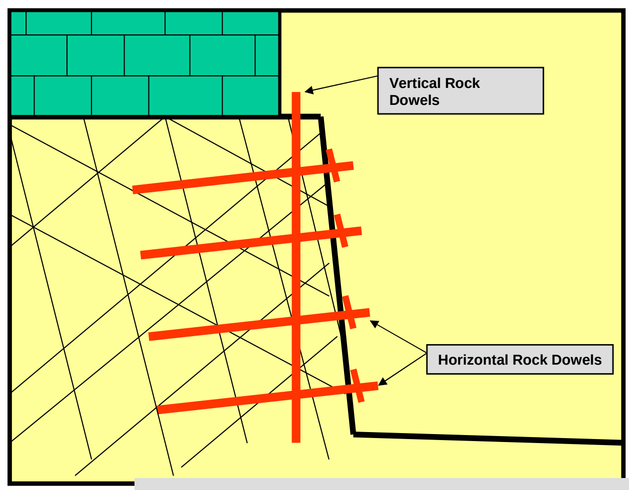


Figure 3. Schematic section of a rock slope illustrating the use of rock dowel pre-support (vertical dowels) and horizontal rock dowels.

The design of horizontal dowels is conducted using limit equilibrium wedge and planar analyses typically used for rock anchors. To account for the passive nature of the reinforcement, the support force provided by the dowel is added to the resisting force when determining the safety factor. Limit-equilibrium design of vertical dowels is problematic, because a support force applied parallel to a dowel with a near-vertical inclination actually acts to destabilize the wedge using typical limit-equilibrium analyses. One alternative design approach is to recognize that the dowel provides increased shear resistance to the sliding plane and include this as either an upward force parallel to the sliding plane or as an increased frictional strength of the joint plane. When using vertical dowels together with horizontal support, we sometimes take the conservative approach of neglecting the contribution of the vertical dowels.

Our typical design practice is to use high-strength pre-stressing steel (150 ksi threaded rebar) for the dowel rods. The design capacity of individual dowels is determined as 60 percent of the ultimate tensile strength of the bar, following the guidelines set forth by the Post-Tensioning Institute. (PTI, 1996). The use of Grade 150 steel rather than Grade 60 or 75 allows for a wider dowel spacing. The cost savings from the wider spacing generally more than offsets the increase cost from the higher grade steel. Epoxy coating is typically specified for all dowel components used for highway rock cut stabilization projects.

Grouting is a key aspect of the design, affecting rock mass strength, the mobilized capacity of the dowels, and the corrosion resistance of the installed reinforcement. Specification for rock dowel installations typically have provisions for water-pressure testing of drill holes prior to dowel installation. Such testing assures that the grout will not bleed from the drill hole. For holes that show leakage or high water takes, an initial grouting step is required to fill fractures and joints that intersect the hole, and the hole is re-drilled.

Specifications also include provisions for performance testing of randomly selected installed dowels. Performance testing typically involves loading the working dowels to 133 percent of design load through one or more cycles while recording displacements. These tests may also involve testing for creep at the design load or greater. Simple pull tests may also be used on short test dowels to confirm grout mix designs prior to installation of working dowels.

Conclusions

Perimeter control blasting measures and passive dowel systems have been used individually or in concert to provide a cost-effective approach to preventing rock slope failure. When properly implemented, perimeter control blasting measures (pre-splitting, cushion blasting, or line drilling) will provide smooth, straight, and stable rock slopes and minimize the need for other stabilization measures.

Passive dowel systems consist of fully grouted, high-strength steel bars, and offer a low cost reinforcement alternative to tensioned rock anchors or rock bolts. Vertical dowels can be installed to provide ‘pre-support’ for new rock excavations. Horizontal dowel systems, likewise, can be used to remediate existing slopes or can be installed in rows (much like soil nails) to provide pattern reinforcement for the cut slopes of new excavations.

Geological characterization is needed to understand with confidence the potential failure modes for individual rock slopes.

References

Hoek, E and J.W. Bray, 1981, Rock Slope Engineering, London, Institution for Mining and Metallurgy, 358 pp.

PTI, 1996, Recommendations for Prestressed Rock and Soil Anchors, Pheonix, Arizona, Post Tensioning Institute, 70 pp.

Spang, K, and P. Egger, 1990, Action of Fully-Grouted Bolts in Jointed Rock and Factors of Influence, Rock Mechanics and Rock Engineering V. 23, p. 201-229

Case History 1 – Pre-Splitting and Rock Dowels used for Excavation of a Rock Slope Adjacent to New Building Site, Lexington, Massachusetts.

Pre-splitting was used for perimeter control blasting of this rock excavation directly adjacent to a new multi-story office building. Note the high percentage of visible blast hole half casts on the slope.

Prior to excavation, vertical dowels were installed at a 2 ft spacing to pre-support the slope.

Horizontal dowels were used to provide additional support against sliding of specific rock wedges.

Case History 2 – Cushion Blasting used for Rock Slope Remediation, New Jersey Turnpike, Secaucus, New Jersey.

The toe of this steep, 80-ft high rock slope was directly adjacent to the travel lanes of the NJ Turnpike, presenting a rockfall hazard to motorists.

Cushion blasting of the hard diabase rock was used to lay back the slope to a 1H:2V angle and create a catchment ditch.

Blasting and excavation was accomplished with minimal disruption to traffic.

Case History 3 – Line Drilling and Rock Dowels used for Rock Excavation and Underpinning of the Maine State House, Augusta, Maine.

Construction of new facilities beneath the historic Maine State House involved excavating a 20 ft in cut in granite directly adjacent to the foundation and blasting beneath the west wing of the structure.

Vertical rock dowels (pre-support) and horizontal rock dowels provided support to the excavation and were an integral part of the foundation underpinning structure.

Line drilling was used for perimeter control blasting.

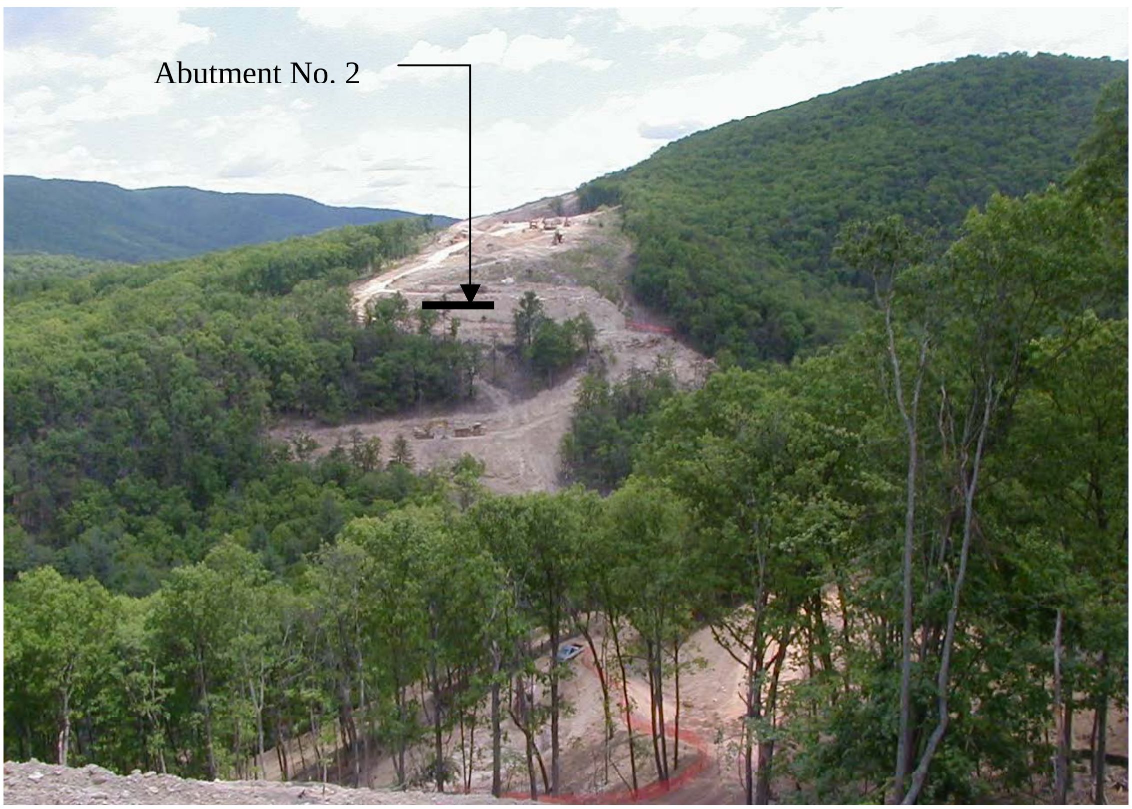
Evaluation of Adverse Bedding Orientation on the Clifford Hollow Bridge Foundations

Robert L. Dodson, P.E.
Project Geotechnical Engineer
HDR Engineering, Inc. Weirton, WV

James M. Sheahan, P.E.
V.P. and Chief Geotechnical Engineer
HDR Engineering, Inc. Pittsburgh, PA

Introduction

The Clifford Hollow Bridge is located in western Hardy County, West Virginia approximately 3 miles northeast of Moorefield. This structure will carry US 33 over the valley as part of West Virginia's Corridor H improvements program. The structure has six spans with a total length of 464 meters (1,522 feet). The superstructure consists of continuous steel girders supported on piers ranging in height from about 30 to 82 meters (98 to 270 feet) above the valley floor of Clifford Hollow. Photograph No. 1 was taken across the site from Abutment No. 1 toward Abutment No. 2. Due to their significant height, the piers were designed and constructed with hollow stems to provide a substantial reduction in the dead load. HDR performed the final design of the superstructure and foundations while preliminary design, initial subsurface investigation, and preliminary geotechnical recommendations were developed by others.



Photograph No. 1 – View from Abutment No. 1 to Abutment No. 2

The project site is located in the west-central Valley and Ridge Province, which is relatively complex. The sedimentary rock formations are folded and broken with some possible faulting from tectonic activity. In general, the thickness of soil overburden is relatively thin (i.e. less than 2m) and its impact on foundation evaluations was inconsequential. The alignment of the structure runs southwest to northeast while the strike of the rock formations in the immediate vicinity of the structure were measured at N83°E. The dip of the formations is about 25° to the NW. As a result, the dip-orientation of bedding planes in the southwest slope of the valley is nearly parallel to the ground slope and slightly skewed to the structure alignment. The plan and profile views of the bridge included in the attached Figure 1 shows the general layout of the area, structure alignment, and substructure locations as well as the approximate strike and dip of the rock formations. The presence of shale formations made the potential for instability along the bedding planes in the southwest slope a major concern. This potential appeared to be greatest in the upper section of the slope, west of Station 199+000, due to the presence of significant siltshale beds which are interbedded with sandstone. Formations encountered near the base of the slope are predominantly sandstone and sandy shale. Bedding in the northeast slope dips into the hillside, which presented a more stable condition.

Design Phase

Several foundation alternates were considered to support the massive pier and superstructure loads. These included spread footings founded on rock, drilled shafts socketed into rock, and driven steel H-piles. Due to the limited thickness of soil overburden encountered, spread footings were determined to be the most feasible and economical option.

Bearing capacity analyses were performed for each substructure unit using the methods presented in AASHTO Section 4.4.8. The rock mass below the footings was typically broken and jointed. Average RQDs at individual substructure units ranged from 16 to 59 percent with an overall average across the project site of approximately 36 percent. No unconfined compressive strength test data was available for rock encountered at the site and the allowable loads, based on the range of presumptive values presented in Table 4.4.8.1.2B of AASHTO, were believed to be unreasonably low given the RQDs reported in the borings. As a result, conventional bearing capacity analysis (i.e. equation 4.4.7.1.1.4-1 in AASHTO) was used to compute the allowable loads for spread footing foundations. The analysis treated the rock as a granular mass having a friction angle (ϕ) of 45° with no cohesion. Computations accounted for the footing size, angle of the slope (β), and groundwater conditions observed in the borings by others. The computed allowable bearing capacities ranged from 860 to 1,150 kPa (9 to 12 tsf).

As mentioned previously, the presence of shale formations dipping at 25° out of the southwest slope made the potential for instability along the bedding planes a major concern. Therefore, stability analyses were conducted at each substructure location concentrating heavily on the units founded in the southwest slope. Applied foundation loads were also included in the analysis based on estimated superstructure and substructure data available at the time of analyses. Stability analyses were performed using the computer program STABL6H.

The stability model was based on a wedge-shaped failure with the principal surface oriented along the interface of bedding planes. The potential direction of movement would be approximately perpendicular to the ground contours. The strength at the interface along these bedding planes was selected based on methods presented by Hoek and Bray, 1981. If the conservative assumption is made that no cohesion exists along the full length of the interface of these surfaces, the strength is based on friction alone. The total friction angle (ϕ_t) used for this strength is derived from a “base friction angle” (ϕ_b) resulting from intergranular friction, plus a roughness friction angle (i), which is related to the interlocking of surface projections along the interface between the bedding planes (Hoek and Bray,1981); therefore, $\phi_t = \phi_b + i$. No shear tests were made on samples from this project, but based on a review of data from a number of sources, a base angle (ϕ_b) of 30° was selected for the shale to shale interface. Based on a review of core samples and literature, a value of 5° was selected for “i”. It was believed that this was justified since no slickensides or clay seams were reported in the borings at these foundation units or observed during HDR’s review of the core samples. Nor is there apparent evidence at the site of previous movement in the ground mass. This supported the conclusion that the interface friction angle is greater than the base angle alone. Therefore, stability analyses were conducted using a total friction angle, $\phi_t = \phi_b + i = 35°$ for the interface friction between bedding planes. Hoek and Bray (1981) report $\phi_b + i$ values along bedding plane interfaces of 65° to 75° from tests by others at “low” normal stresses, even along shale/limestone interfaces. Reported normal forces in these tests of various rock types varied between 2.1 kPa (44 psf) and 68 kPa (1,420 psf), which would represent a depth of about 3 meters (10 feet). This suggests that the use of 35° for the total friction angle at this site could be conservative.

The strength of the rock in a direction other than parallel to the bedding planes was assumed to consist of friction only, with $\phi = 45°$ and no cohesion. It is believed this represents a conservative condition where the rock is very broken. Analyses at locations in the east slope used this strength throughout the rock mass since the bedding planes dipped into the hillside.

One other factor was considered significant in the analysis of these slopes -- groundwater. No groundwater monitoring wells were installed at the site and it is HDR’s opinion that groundwater monitoring data in rock can be very difficult to assess due to the influence of localized joints and other discontinuities on groundwater flow; however, core samples did exhibit staining indicative of some flow. As a result, the potential impact of groundwater was assessed. Water readings in borings at each substructure unit were reviewed. Little long-term data was available from these readings. Only those in borings BB-46, BB-48 and BB-9 were significantly longer than 24 hours after boring completion; however, the following observations were made:

- The “24-hour” water levels were typically lower than the “Zero” hour reading taken at completion of the boring.

- Some depth of water remained in most borings for the “24 hour” reading and in those with longer term readings as well.

While the conclusions that could be made from this information are fairly limited, it appeared that groundwater pressure was not artesian. Also, the levels may have continued to drop after 24-hours, if monitoring had continued. Finally, it appeared reasonable to assume that a groundwater regime exists. Therefore, based on recorded water levels and the staining of rock core, a groundwater level was selected for the stability analysis at each unit. Table 1 summarizes the key results of these stability analyses.

Table 1
Summary of Minimum Factors of Safety for Design-Phase Stability Analyses

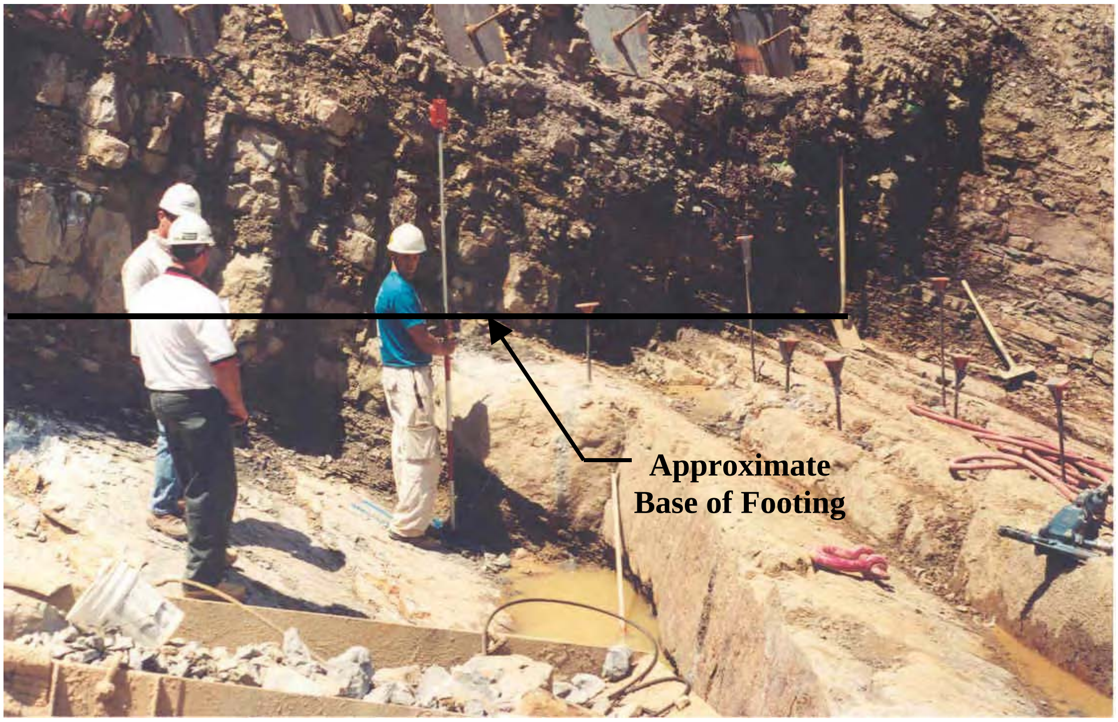
Substructure Unit	**Bottom of Ftg Elev. (m)**	**FS_{min} (No GWL)**	**FS_{min} (w/ GWL)**	**FS_{min} (w/ GWL+Anchors)**
Abutment 1	489.5	1.47	**1.23**	1.50
Pier 1	463.5	1.51	**1.30**	1.50
Pier 2	432.5	2.16	1.80	---
Pier 3	414.5	1.89	1.73	---
Pier 4	406.0	2.18	1.80	---
Pier 5	453.0	2.18	1.69	---
Abutment 2	481.0	3.74	---	---

The target minimum factor of safety for the analyses was 1.5. Results in Table 1 show that the minimum safety factor was obtained in all of the analyses without the inclusion of groundwater. However, with groundwater included, all analyses met the minimum safety factors except Abutment 1 and Pier 1. Two methods considered to improve the factor of safety in those areas included rock anchors and inclined drains. Rock anchors would increase resistance in the slope while drainage of the slope would prevent groundwater level buildups. Stability analyses indicated that two-1,100 kN (250-kip) anchors were required per meter of slope width below the Abutment 1 and Pier 1 to adequately increase the safety factor. Figure 2 illustrates the stability model used at Pier 1, which also includes the rock anchors. It was determined that the anchors would be placed in the slope rather than through the abutment since two rows of anchors would be difficult to incorporate into the footing design and installing the anchors in the slope would simplify construction of the foundations. The anchors were installed through bearing blocks poured against bedrock. As an alternative, inclined drains could have been drilled at a shallow angle, upwards into the slope below the two foundations, to relieve potential water pressure. However, clogging of these drains over the long-term could reduce their performance and dependability. Therefore, the rock anchors were recommended for construction. Photograph No. 2 shows the completed anchor blocks below Pier 1. The anchors were installed through the visible holes in the blocks followed by backfilling the area to the original ground line.


Photograph No. 2 – Completed Anchor Block Below Pier 1

Construction Phase

Construction of the Clifford Hollow Bridge began in the spring of 2001. During the excavation for the Pier 2 foundation, at least four clay seams were observed on the back and sidewalls of the excavation. These clay seams were present along bedding planes at the base of relatively thin shaley layers within more massive sandstone and siltstone units. These formations and their positions relative to the footing base are shown in Photograph No. 3. Selected points along these seams were marked for location by survey. Elevations from survey indicated the dip of the rock bedding planes to be approximately 26° to the north which correlated well with previous dip measurements of 25° on a N7°W heading. A skewed section (Figure 3), depicting the conditions shown in Photograph No. 3, was then developed through design phase borings BB-7 and BB-26. This section is also parallel to the dip of the bedrock. The elevations and locations of the clay seam survey points were plotted on this section and the clay seams were extended down the slope on a 26° angle.



Photograph No. 3 – Pier 2 Foundation Excavation at Bottom of Footing Elevation

This section was then used to evaluate the potential impacts of the clay seams on the stability of the Pier 2 footing. Stability analyses performed during design resulted in acceptable factors of safety here due to the rock dipping into the toe of the slope and the strengths used in the analyses. However, these analyses did not account for the potential presence of clay seams along the bedding planes since none were noted on the boring logs or observed by HDR in the cores. Next, supplemental stability analyses were performed in this area to model potential effects of the clay seams observed within the foundation excavation using the planned bottom of footing elevation of 430.5. Lower strengths were used along the bedding planes in these analyses to account for the potential of continuous clay seams within the slope. Based on a review of West Virginia geologic literature and discussions with the Department's geologist, a friction angle of 20° was used for the strength along the bedding planes. These analyses indicate a minimum factor of safety of approximately 1.3 for a friction angle of 20° as long as the material between the clay seams (i.e. across the beds) is as competent as previously anticipated (ϕ=45° & c=0). While the analyses indicate that the slope will be stable, the required long-term factor of safety of 1.5 was not obtained which stressed the need to verify subsurface conditions in the slope below the footing. Additionally, analyses were also performed to evaluate the potential stability benefits of lowering the footing. These analyses indicated that a 1.5 factor of safety would still not be obtained unless the footing could be placed below the observed clay layers, which would require significant and costly excavation. Therefore, it was concluded that additional borings were necessary to verify bedrock conditions down-slope of the Pier 2 foundation.

Based on the stability analyses results, four borings were drilled in sections of two at distances of 20 and 40 meters down-slope of the pier in the direction of the rock dip (Refer to Figure 4). Depths of these borings were to be at least 15 meters (50 feet) to penetrate the anticipated interval containing the clay seams visible in the Pier 2 foundation excavation. Additionally, it was determined that the clay seams encountered at Pier 2 could adversely effect the stability of Pier 1, if they extended through this area. Therefore, two borings were also planned at Pier 1. These borings were to be drilled at distances of 25 and 40 meters downslope of the pier foundation along the direction of rock dip (See Figure 5) to a depth of about 25 meters. Piezometers were installed in all of the borings to obtain extended groundwater level readings in the slope beyond 24 hours for use in evaluating conditions and supplemental stability analyses.

Pier 2 Evaluation

Bedrock encountered below Pier 2 in borings P2-1 through P2-4 consisted of sandstone and shaley siltstone. The sandstone was typically described as hard (HCSI=4) while the shaley siltstone was average to hard (HCSI=3 to 4). Core recoveries ranged from 26 to 100 percent with an average of 87 percent. RQDs ranged from zero to 100 percent with an average of about 50 percent. In zones of low recovery, there were no noticeable changes observed in the rate of drilling which suggests that the rock was likely of competent quality, but may be broken or a joint may have been encountered. Varying degrees of clay infilling were observed on several fractures and bedding planes. Some of these discontinuities contained measurable amounts of clay, but the majority were too thin to measure. These were typically described as having a "trace" or "little" clay present. Where clay was present in measurable thicknesses, variable amounts of rock fragments were typically present within the clay. Readings obtained in the borings through July 25, 2001 indicated that water levels continued to decrease after their completion. However, rain on July 25, 26 and 29 appeared to cause an increase in the water levels. The rain on July 25 produced 10mm (0.39") of precipitation at the site and appeared to cause only a minor increase in the readings taken on July 26. Next, the rain received on July 26 and 29, 2001 was more substantial causing the July 30 water level readings to be the highest since completion of the borings. Precipitation received on July 26 and 29 was 20mm (0.79") and 40mm (1.57"), respectively. This is a total of 60mm (2.36") in four days.

The results of these borings indicated that there were some zones of broken rock, but the majority of the bedrock was in good condition. Additionally, the groundwater level readings obtained in the recent borings through July 25, 2001 were lower than used in the stability analyses while water levels in borings P2-3 and P2-4 on July 30, 2001 were slightly higher. Additionally, the dip of the bedding planes was slightly steeper than the ground surface as well as the estimated surface of the top of rock. As a result, potential sliding surfaces that would follow the bedding planes must also cross these laterally, through a significant thickness of rock to reach the ground surface, if a failure were to occur. This is believed to be a significant feature in the analysis of this pier.

Based on the quality of the rock, it was believed that a nominal amount of cohesion would be justified for the rock strength between the bedding planes. In Hoek and Bray, 1981, ranges of cohesion for sedimentary rock including shale, sandstone, and siltstone are presented for both soft and hard materials. These ranges of cohesion are summarized in Table 2.

Table 2
Range of Sedimentary Rock Cohesion

Rock Hardness	Cohesion Range	
	(kPa)	**(psf)**
Soft	1,000 to 20,000	20,000 to 400,000
Hard	10,000 to 30,000	200,000 to 600,000

(Ref.: Hoek & Bray, 1981)

Based on the cohesion values presented in Table 2 and conditions encountered in the borings, a nominal cohesion of 48 kPa (1,000 psf) was used in the attached stability analyses. This value is approximately 5 percent of the low value for soft rock in Table 2 and is believed to be conservative. Re-analyses of the slope with this cohesion included in the rock between the bedding planes resulted in a minimum factor of safety of 1.47, which is essentially 1.5. This analysis included groundwater levels used in the original design analyses. An analysis using the water level readings obtained on July 30, 2001 after significant precipitation resulted in a minimum factor of safety of 1.46 (Figure 6). Finally, an analysis using the water levels from the July 25, 2001 readings resulted in a minimum factor of safety of 1.65. Based on the results of the investigation and these analyses, it was concluded that Pier 2 should have an acceptable long-term factor of safety; therefore, no modifications to the construction plans were necessary.

Pier 1 Evaluation

Subsurface conditions encountered in borings P1-1 and P1-2 were similar to those at Pier 2. Bedrock consisted of sandstone and shaley siltstone that were typically described as average to hard (HCSI=3 to 4). Core recoveries ranged from 66 to 100 percent with an average of 94 percent. RQDs ranged from zero to 100 percent with an average of 44 percent. Some fractures and bedding planes were observed to contain clay infilling, but their occurrence was significantly less than in the borings below Pier 2. This infilling was typically described as a “trace” or “little” clay since they were too thin to measure. The only two occurrences of measurable clay were encountered above a 26° plane extended from the exterior corner of the footing which placed them outside of the interval influenced by the structure loads. Water level readings obtained in the borings through July 25, 2001 continued to decrease while readings obtained on July 30, 2001 exhibited an increase due to the precipitation discussed in the evaluation of Pier 2.

The results of borings P1-1 and P1-2 indicated that the majority of the bedrock was in good condition. Groundwater level readings obtained in the borings before and after the recent precipitation were significantly lower than those used in the design stability analyses. Additionally the anticipated dip of bedding planes was almost parallel to the ground surface. Therefore, potential sliding surfaces that would follow bedding planes would have to cross these beds laterally to reach the ground surface, if a failure were to take place. While the thickness of rock through which the surface would pass was not as great as at Pier 2, this was again a significant feature in the analysis of stability at this pier as well.

Supplemental stability analyses were performed for the slope in this area using an equilibrium analysis (i.e.; FS=1.0) to determine conservative strength values that would account for the presence of continuous clay seams along the bedding planes. Two sets of equilibrium analyses were conducted. The first used water levels taken from the recent borings and the second used a water levels that were significantly higher than those measured in the borings. This second water level was developed from observations of stained fractures in the boring core samples, which might suggest that higher water levels have existed at some time in the past. This was believed to represent a conservative assumption. The analyses with the water level taken from the borings indicated that if a friction of angle of 20° was assumed along the bedding planes, in conjunction with a friction angle of 45° and a cohesion of 48 kPa (1,000 psf) across the bedding planes, the factor of safety was about 1.0. Additionally, the analyses with the higher water level developed from stained fractures in the core indicated that the friction angle along the bedding planes would need to be increased to 25° to provide a factor of safety of about 1.0. Since no signs of movement have been observed in the slope, these strengths can be considered as reasonable minimums with their associated water levels. Stability analyses were then performed for each of the water level conditions with the Pier 1 foundation loads and planned slope anchors included. These analyses indicated that factor of safety of 1.5 or greater was maintained for failure surfaces along the bedding planes with lengths of about 60 to 90 meters (200 to 300 feet) along the slope (Figure 7). Longer failure surfaces tend to exhibit progressively lower factors of safety approaching the equilibrium state as the magnitude of the hillside forces becomes much more significant and the foundation loads and slope anchors have a negligible impact on the global stability of the slope. It should also be noted that the probability for continuity in any clay seams present in the hillside is likely to decrease as the length of the surface increases.

Based on the quality of rock encountered in borings, the low measured groundwater elevation, and sporadic occurrence of observed clay seams, it was believed that the material strengths and conditions modeled in the design stability analyses remained applicable. In addition, the supplemental equilibrium analysis using parameters that are believed to be conservative indicated a factor of safety of 1.5 or greater would be anticipated for a reasonable length of slope below Pier 1. Therefore, an acceptable long-term factor of safety of 1.5 or greater would be obtained at Pier 1 with the installation of the proposed permanent ground anchors.

Current Project Status

As of the spring of 2003, the construction of the piers and abutments has been completed as well as the launching of the superstructure. Currently, the contractor is working on completing the pouring of the concrete deck and parapet walls. Photograph No. 4 is a view from the side of Abutment 2 looking across the valley toward Abutment 1 during the construction of the hollow-stem piers. Photograph No. 5 shows the launching of the steel superstructure.

Photograph No. 4 – View from Abutment 2 during Pier Construction

Photograph No. 5 – Launching of Steel Superstructure Approaching Pier 5

References

AASHTO, 1996 – "*Standard Specifications for Highway Bridges*", American Association of State Highway and Transportation Officials, Sixteenth Edition, 1996.

Hoek and Bray, 1981 – "*Rock Slope Engineering*", Hoek, E. and Bray, J.W., revised 3rd Edition, The Institution of Mining and Metallurgy, 1981.

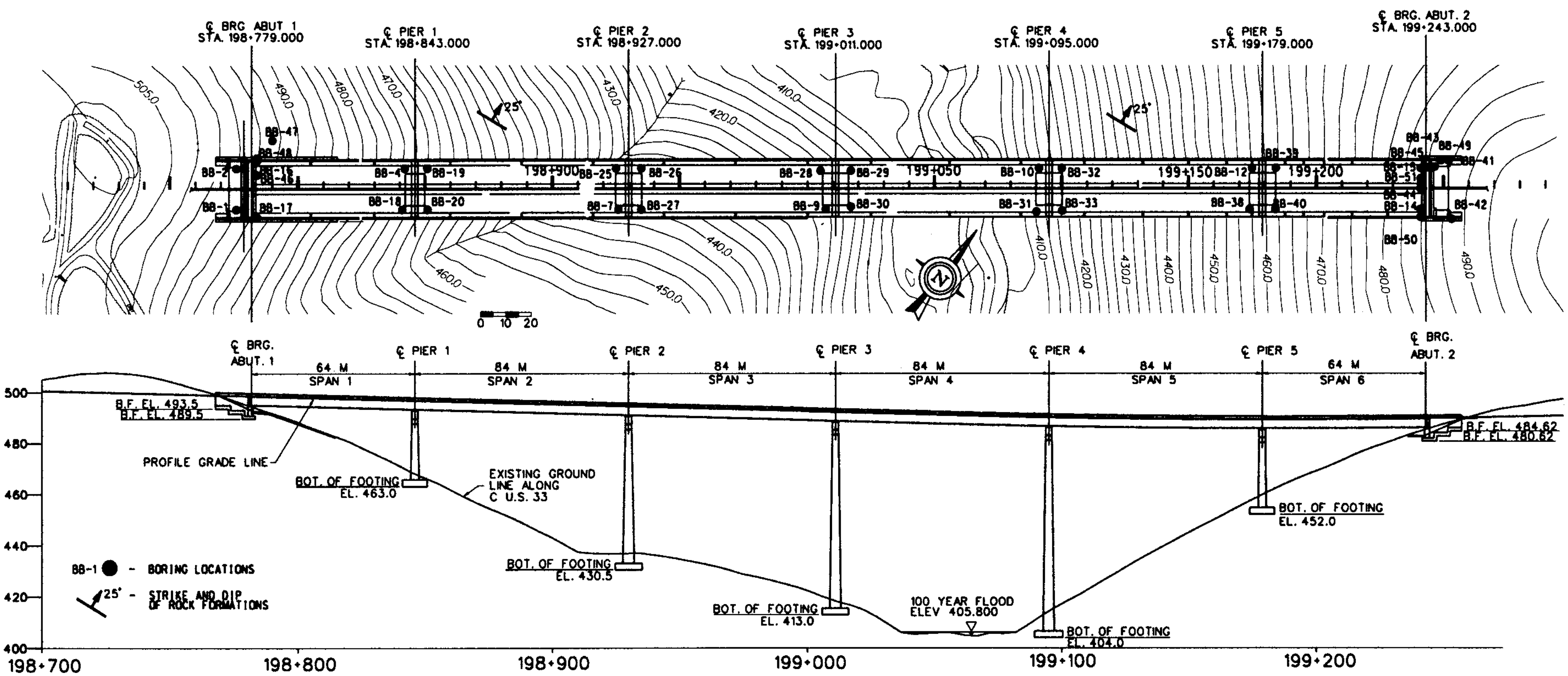
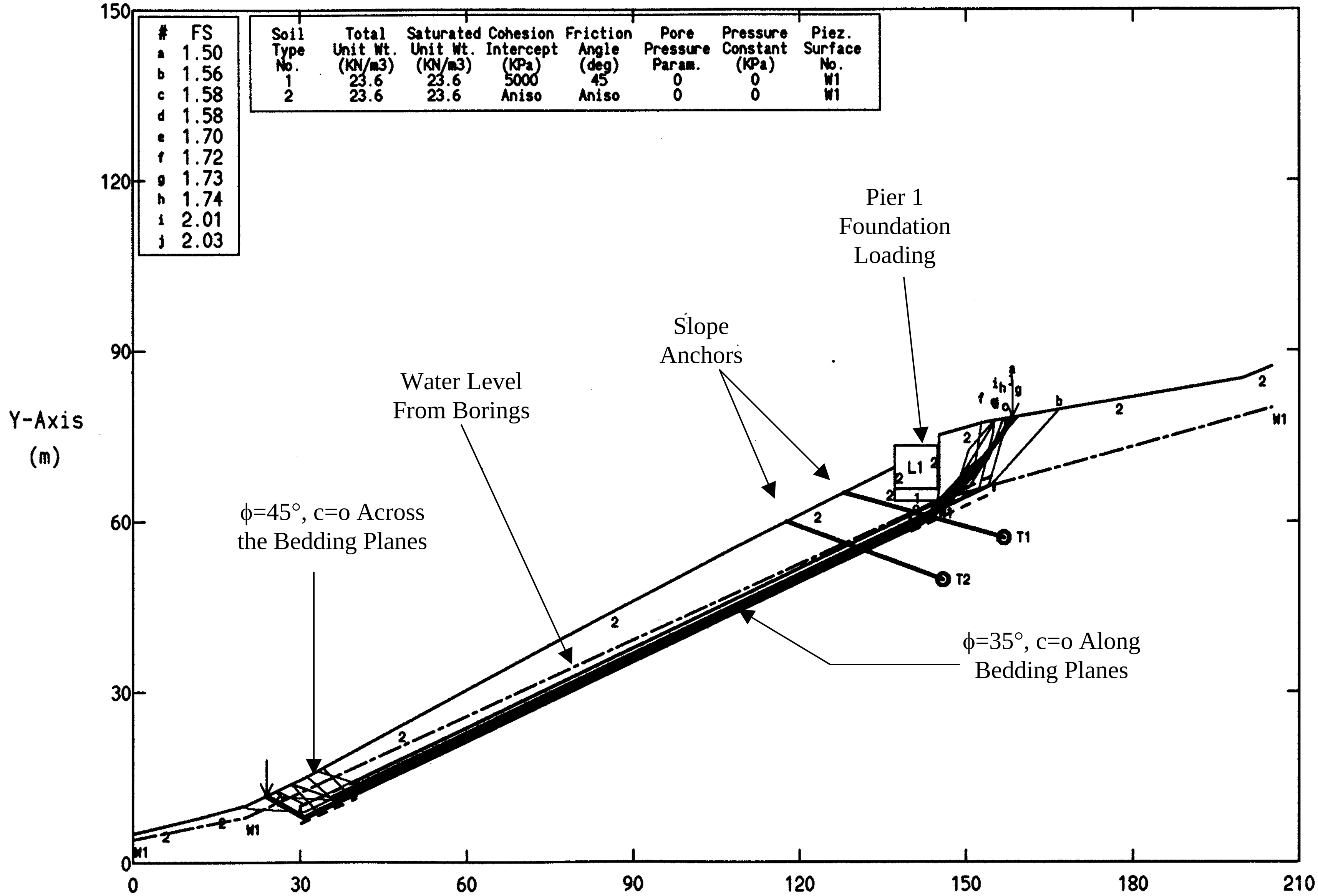


FIGURE 1
CLIFFORD HOLLOW BRIDGE
PLAN AND PROFILE VIEWS

HDR ENGINEERING, INC.

CLIFFORD HOLLOW STL PIER 1, RUN WT2A

Ten Most Critical. I:P1SWT2A.PLT By: Robert Dodson 8/01/2003 9:55am

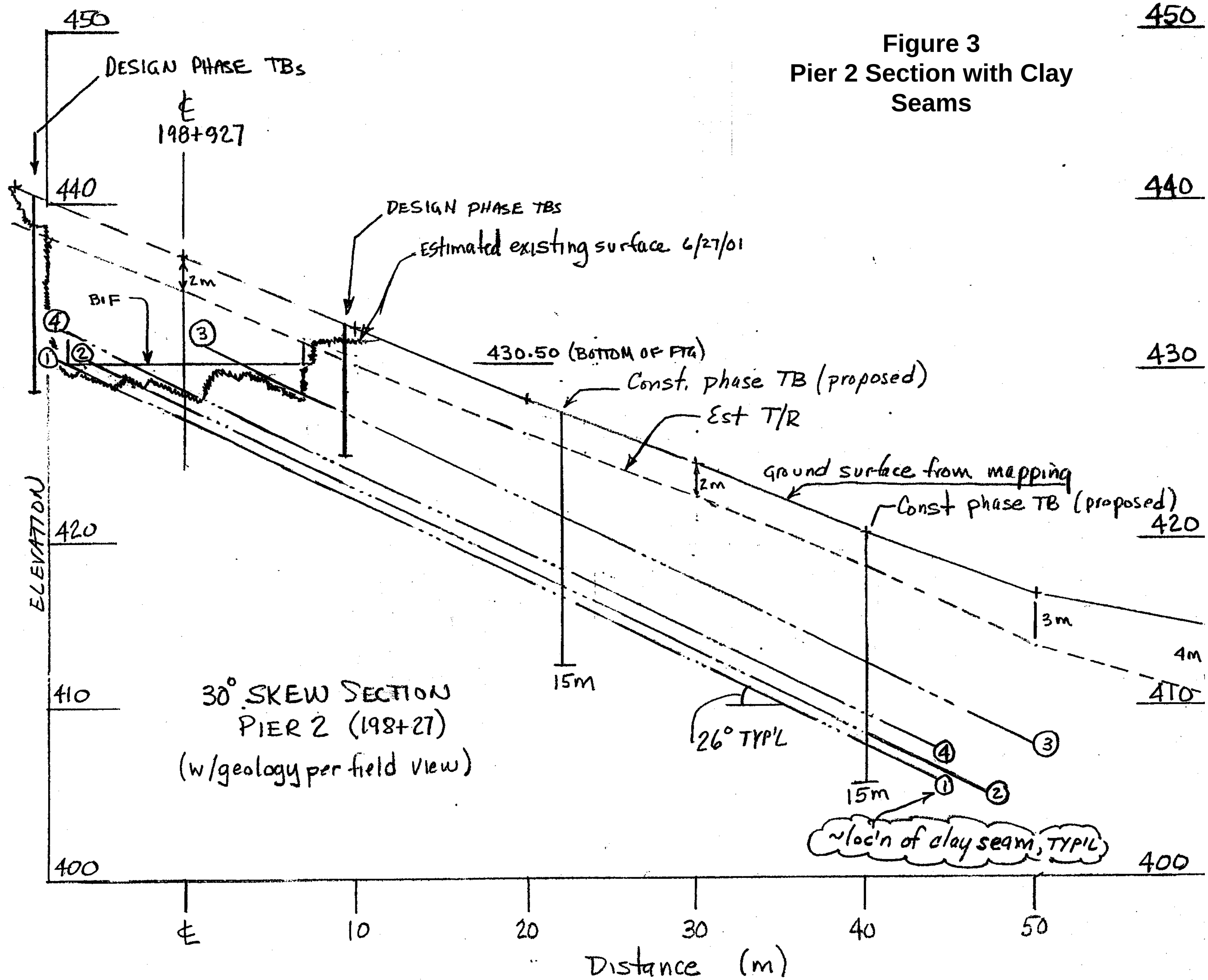


STABL6H FSmin=1.50 X-Axis (m)

Factors Of Safety Calculated By The Modified Janbu Method

Figure 2

Figure 3
Pier 2 Section with Clay Seams



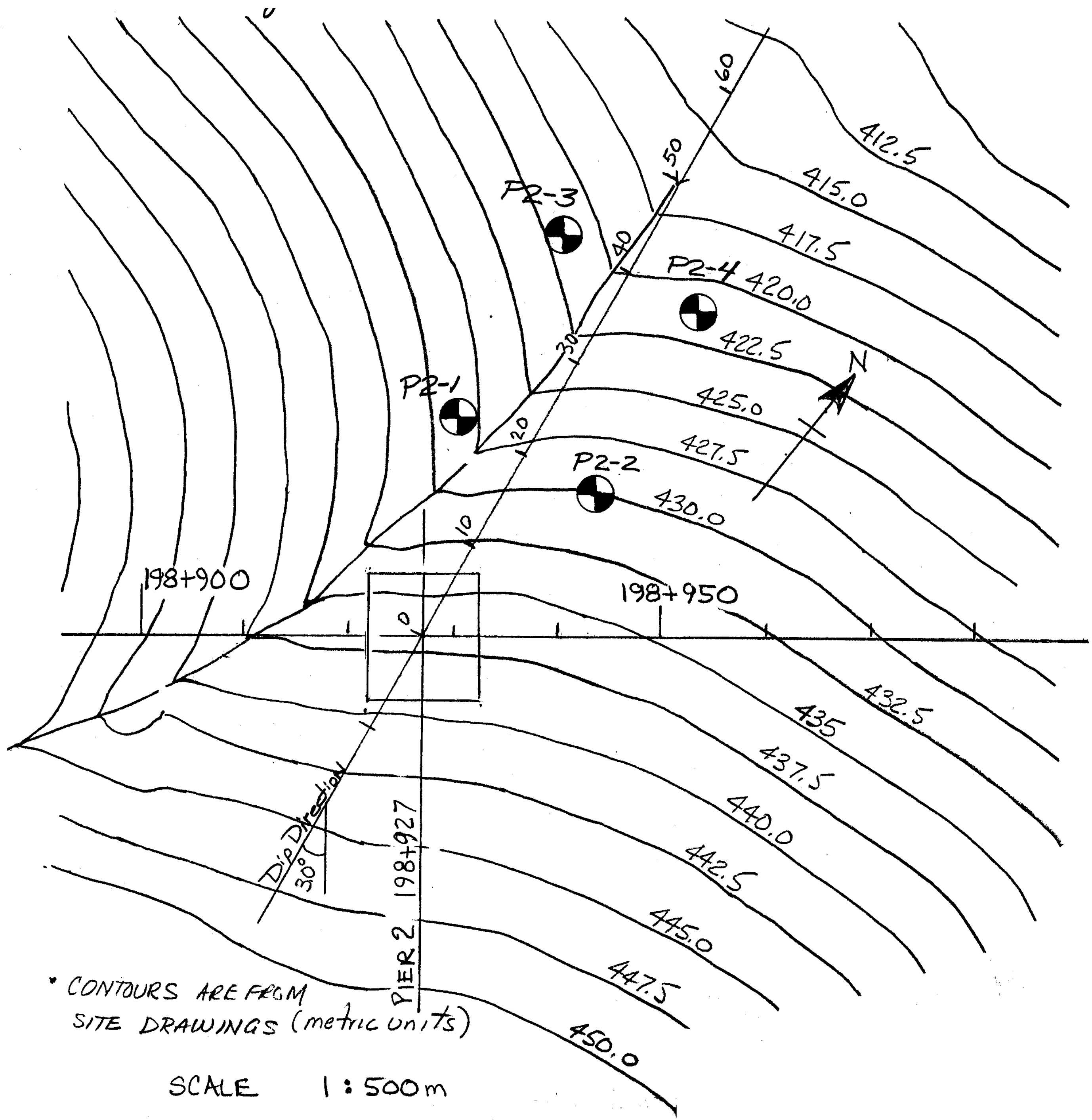


Figure 4
Pier 2 Test Boring Location Plan

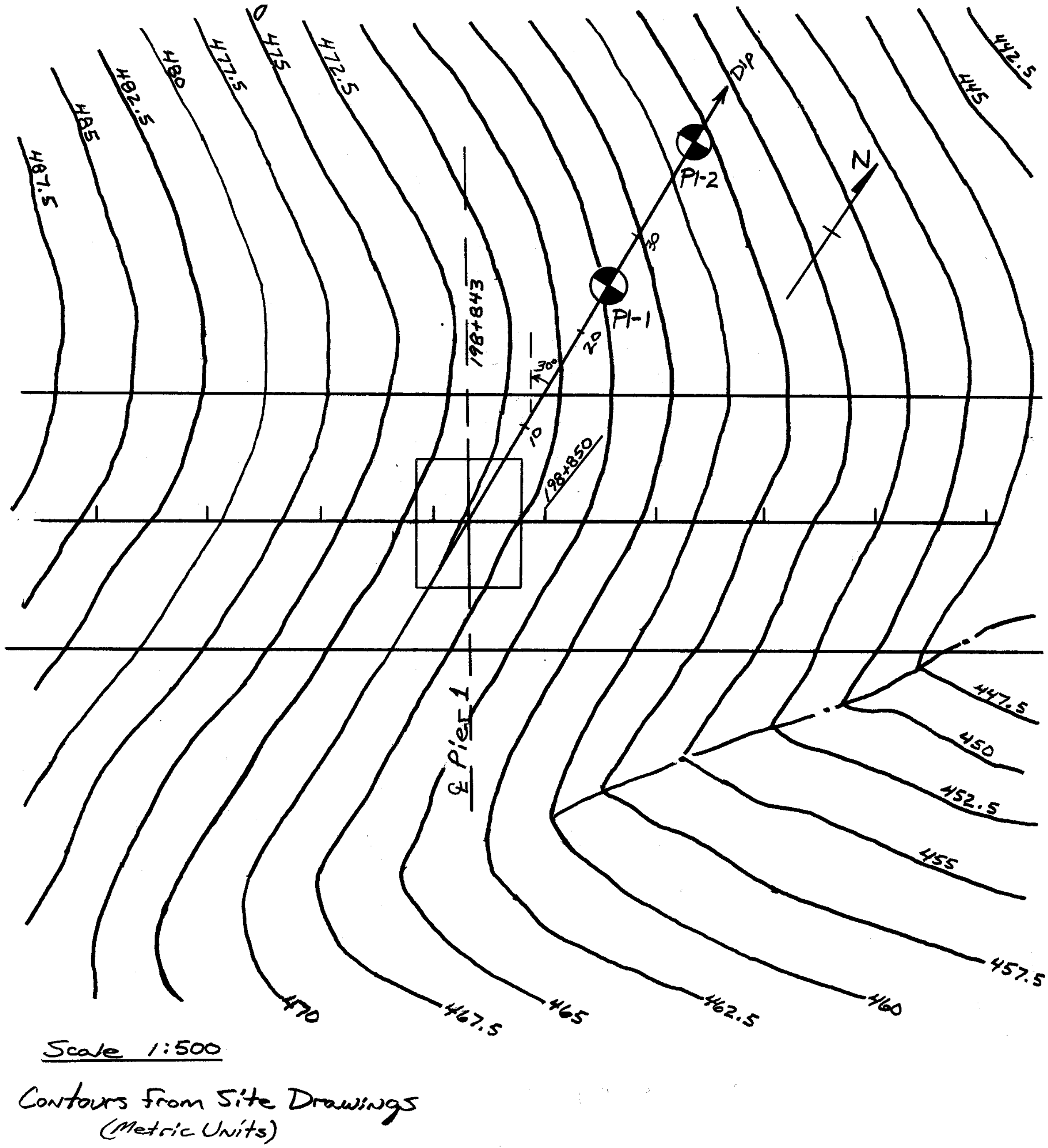
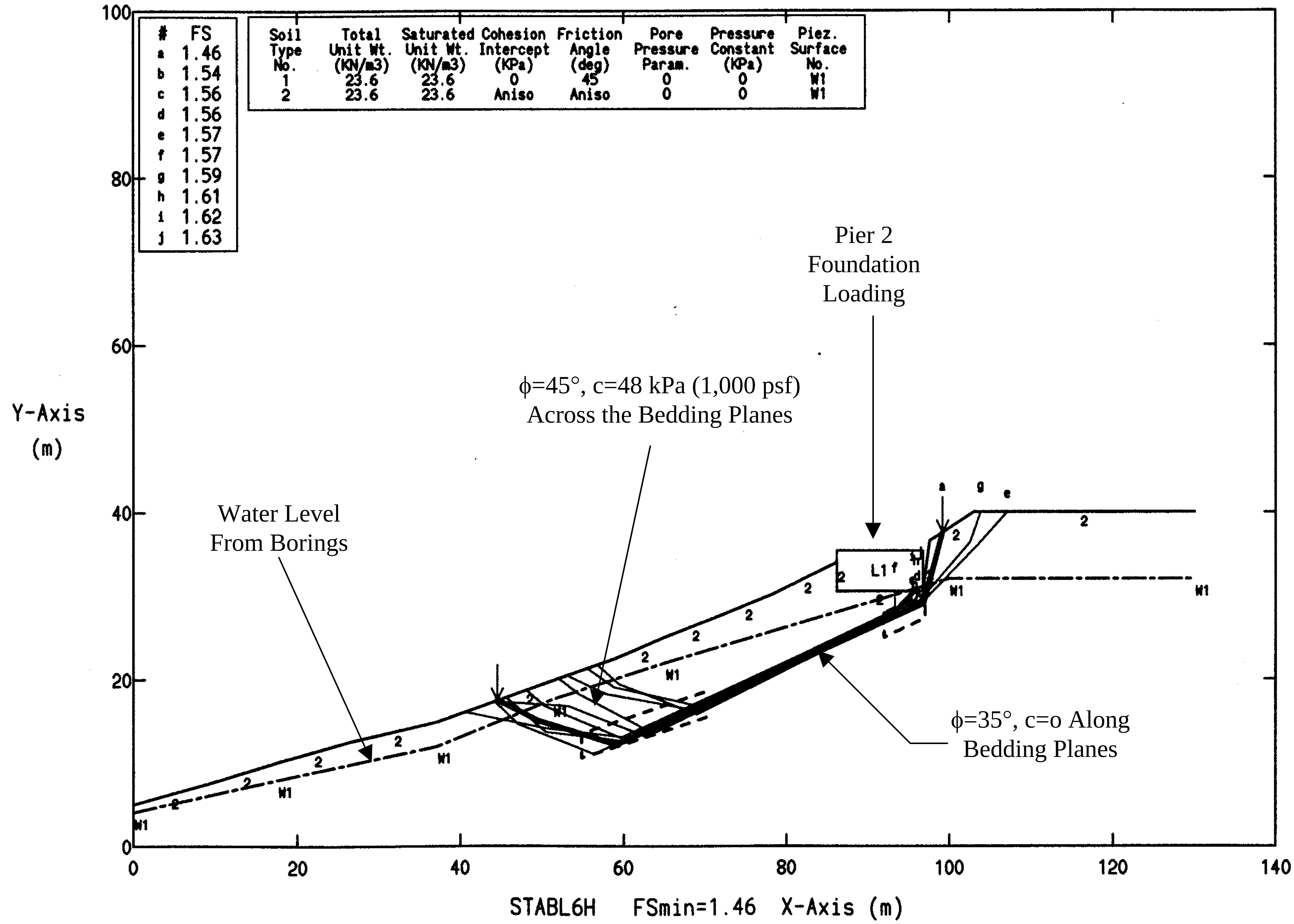


Figure 5
Pier 1 Test Boring Location Plan

CLIFFORD HOLLOW - Pier 2; 20 deg. Frict. and 48kPa (1000 psf) in Rock

Ten Most Critical. I:P2Z2A.PLT By: Rob Dodson 8/27/2001 4:49pm



Factors Of Safety Calculated By The Modified Janbu Method

Figure 6

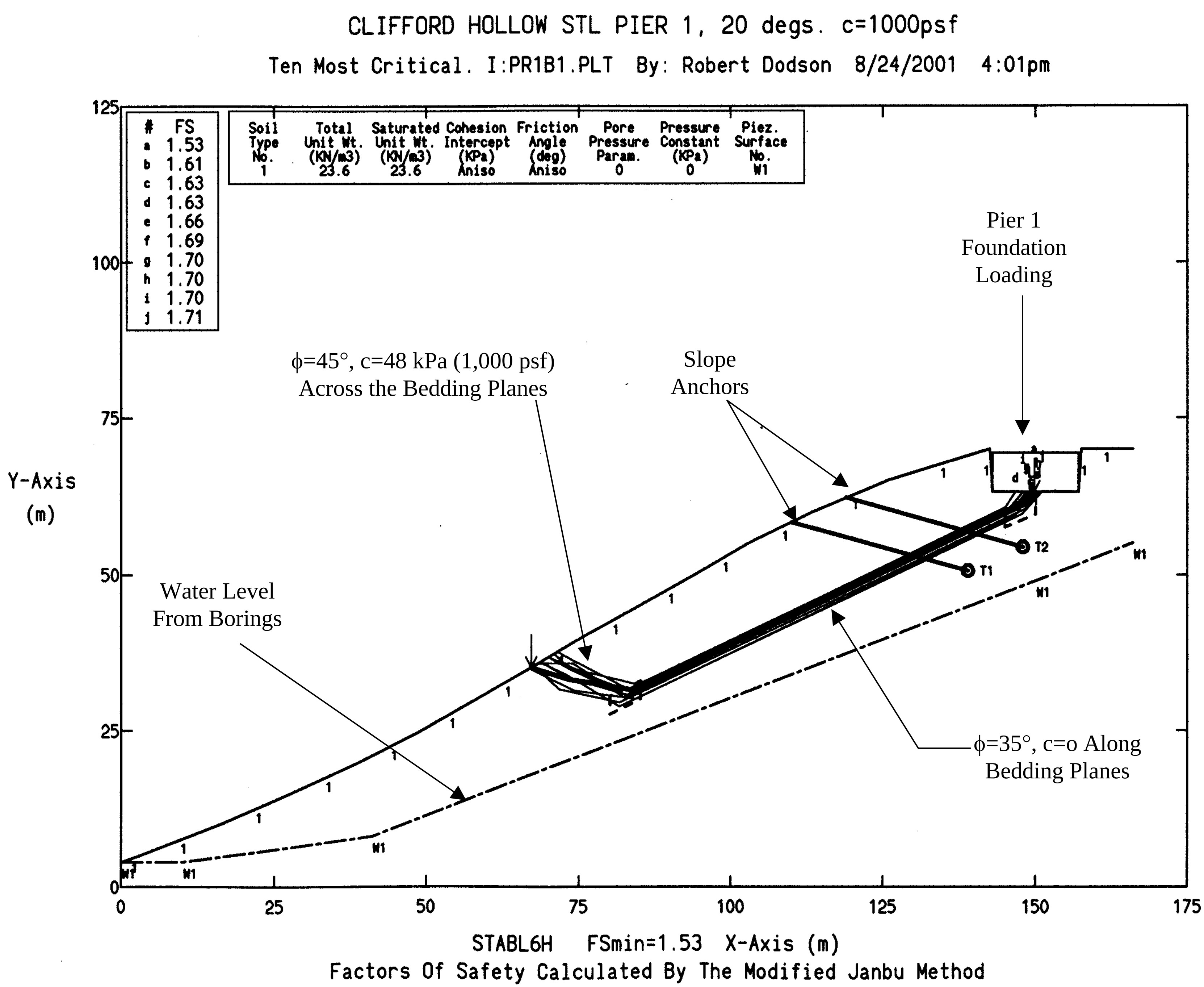

CLIFFORD HOLLOW STL PIER 1, 20 degs. c=1000psf
Ten Most Critical. I:PR1B1.PLT By: Robert Dodson 8/24/2001 4:01pm
FS
a 1.53
b 1.61
c 1.63
d 1.63
e 1.66
f 1.69
g 1.70
h 1.70
i 1.70
j 1.71
Soil Type No. 1
Total Unit Wt. (KN/m3) 23.6
Saturated Unit Wt. (KN/m3) 23.6
Cohesion Intercept (KPa) Aniso
Friction Angle (deg) Aniso
Pore Pressure Param. 0
Pressure Constant (KPa) 0
Piez. Surface No. W1
Pier 1 Foundation Loading
ϕ=45°, c=48 kPa (1,000 psf) Across the Bedding Planes
Slope Anchors
Water Level From Borings
ϕ=35°, c=o Along Bedding Planes
T1
T2
W1
Y-Axis (m)
STABL6H FSmin=1.53 X-Axis (m)
Factors Of Safety Calculated By The Modified Janbu Method

Figure 7

DISPERSIVE CLAY EMBANKMENT EROSION

A CASE HISTORY

Jeff Dean, P.E., Oklahoma Department of Transportation

ABSTRACT

A fine-grained soil mapped as the Cupco soil series by the USDA Soil Conservation Service was used as embankment material on a recent project. The location of the project is on route US 59 near Panama, in LeFlore County, Oklahoma. Work called for embankment widening in the southbound direction. A segment of the completed embankment experienced some characteristic dispersive clay erosional patterns following an above normal rainfall period. Representative samples of the embankment material were taken from standard penetration test (SPT) split spoon samplers, thin walled tube samplers, and by hand auger. Laboratory analyses were made to determine soil classification, in place density, moisture content, and moisture-density relationships for the embankment material. To determine the dispersive characteristics, the tests used were the pinhole, double hydrometer, soluble salts in the pore water, and the crumb test. Statistical analyses were made for the different dispersion test results. All four laboratory tests indicated a highly dispersive clay material. Correlations were observed between compaction water content and density and dispersion. An analysis of the effects of the soluble salts in the pore water and clay dispersion was made. It is believed that the main mechanism in triggering this embankment erosion was rainwater flowing in cracks that formed as a result of earlier drying of the clay. Significant contributing factors were found in the plan design and during construction. Repair of the damaged embankment consisted of undercutting and filling of holes, gullies and tunnels, planting with select material and flattening the design slope.

INTRODUCTION

Certain natural soils have a tendency to disperse in the presence of relatively pure water. They are highly susceptible to erosion and piping. The principal difference between dispersive clays and ordinary erosion resistant clays is the nature of the cations in the pore water **(1)**. Dispersive clays contain sodium as the predominant cation in the pore water whereas nondispersive clays contain calcium and magnesium. The presence of the dominant sodium ions increases the thickness of the diffused double water layer surrounding the individual clay particles. This leads to a deflocculated structure in which the repulsive forces exceed the attractive forces so that the individual clay particles go into suspension in the presence of water. Dispersive clays generally have low to very low permeability rates **(2)**. As a result, the velocity of water moving through the pores is not sufficient to cause movement of the soil particles, even under very high heads. However, once a crack or opening occurs, the dispersed clay particles go into suspension and are easily carried away with the water moving through the opening. The tendency for dispersive erosion in a given soil depends upon such variables as the mineralogy and chemistry of the clay. Studies have shown that soils with montmorillonite as the predominant clay mineral tend to be more dispersive than those containing kaloinite and verniculite **(3)**. Further studies have revealed that dispersive clays have at least 12 percent of their particles, computed on a dry weight basis, finer than 0.005mm as determined by ASTM D422-72 (2). They have a plasticity index greater than 4 and tend to have a pH well on the acidic side **(4)**.

Surveys conducted by the Soil Conservation Service in Oklahoma have provided a general area distribution of dispersive clays in Oklahoma as shown in figure 1 **(4)**. The more modern soil surveys in Oklahoma which are published since 1975 contain significantly more engineering and chemical data and indicate the dispersive potential and location of mapped soil units more accurately.

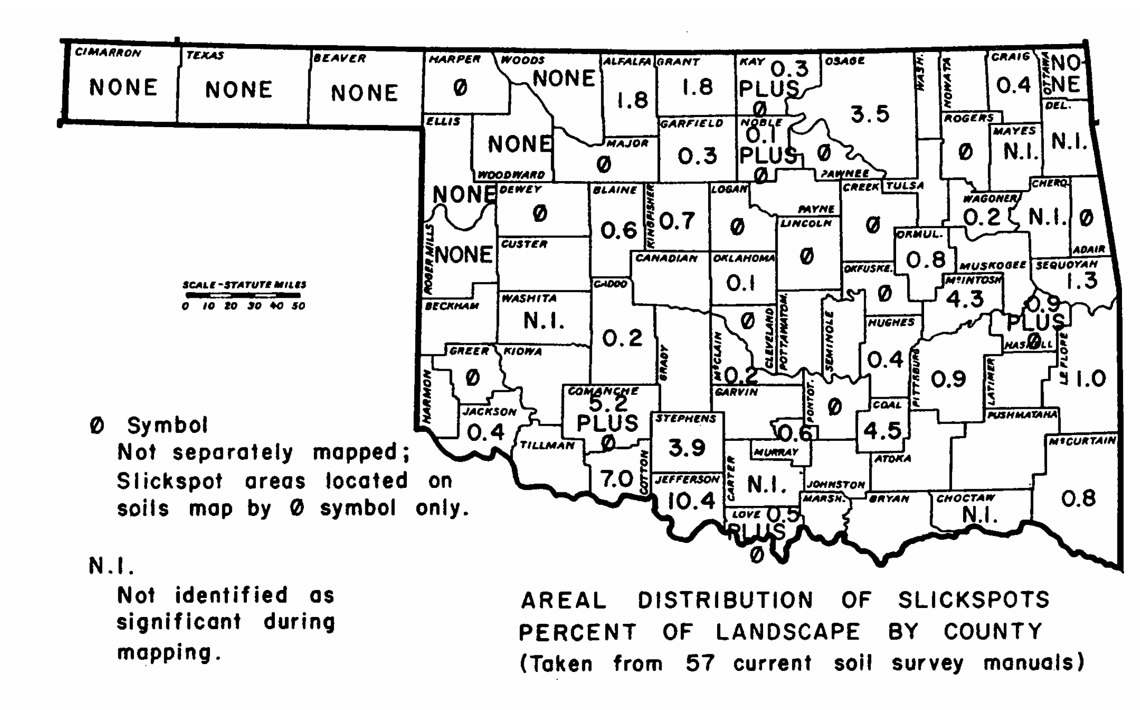


Figure 1. Area Distribution of Dispersive Clays Within Oklahoma

Dispersive clays are common in Oklahoma. They occur randomly within residual and alluvial soil deposits derived from shales of the Permian and Pennsylvanian geologic periods.

This report presents a case history of the geotechnical investigation of the conditions and factors influencing the severe dispersive clay erosion of a highway embankment. The project was in LeFlore County in eastern Oklahoma. It involved widening the existing two-lane section of US 59, between Panama and Poteau, to four lanes with a center median. In order to accomplish this, the existing embankment was widened as shown in figure 2. The southbound lanes were constructed almost entirely on new fill with a 6:1 roll off slope increasing to a 2:1 side slope down to the toe.

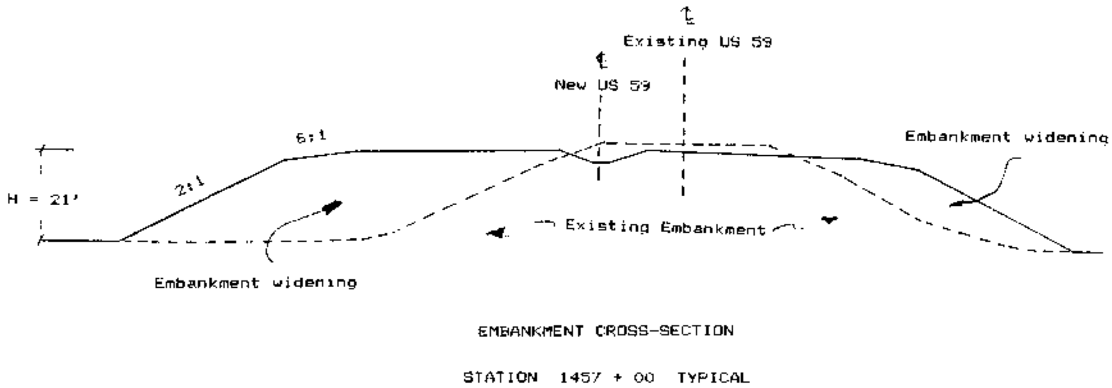


Figure 2. New Construction Typical for US 59

A soil type used in the embankment construction from one of the adjacent borrow pits was mapped by the USDA Soil Conservation Service as the Cupco Series. It was placed solely within a quarter mile extent of the embankment. Cupco soils at this site occur on broad flood plains of Brazil Creek. They consist of deep deposits of somewhat poorly drained silty clay loams that are strongly to very strongly acidic in the 'A' horizon ranging to neutral in the 'B' horizon **(5)**. The initial phase of the project began with the construction of the southbound embankment starting in September 1990. Earthwork was completed the following November. The final stages of this phase, involving the pavement surface and base courses and the side slope protective vegetation, were set aside from December 1990 through August 1991 until the southbound bridges were constructed. The embankment was left unprotected for approximately nine months.

Asphalt pavement construction began in late August and continued into September. The pavement design called for 3.5 inches of type 'B' asphalt concrete surface course underlain by 12.0 inches of a very open graded asphalt concrete base, type 'G'. The pavement design called for no edge drains. The embankment slope was treated shortly after the asphalt paving with vegetative mulch tracked in by a dozer because the 2:1 side slope was too steep to use a disk. As shown in figure 3, rainfall over the nine month period prior to paving was near normal. However, much heavier total monthly rainfalls occurred in September, October and November. A total of 12.0 in of heavy rain was recorded between October 24th and October 30th.

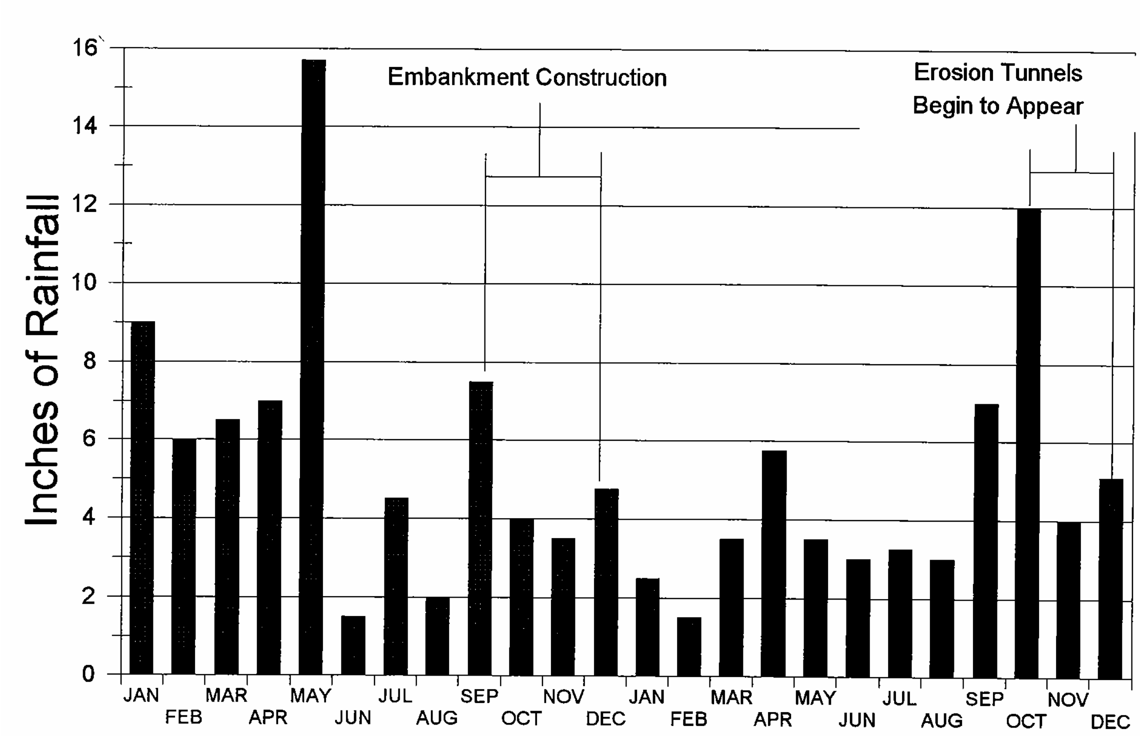


Figure 3. Monthly Rainfall Totals January 1990 to December 1991

Within the next two weeks project inspectors observed the formation of typical dispersive clay erosional features in the form of rills, gullies, tunnels, and jugs within the Cupco section of the embankment.

SITE INVESTIGATION

Personnel, from ODOT's Soils and Foundations Branch, were requested to investigate this problem in early November 1991. Initially a survey was made of the most prominent erosion feature, an open hole on the surface, commonly called a jug, which leads down to an underground tunnel. The survey identified 242 erosional holes within the extent of the 6:1 roll out slope within the Cupco embankment. The average hole (jug) depth was 4.1 feet with a range of 1.5 - 14.7 feet. Additionally the embankment was heavily rilled and had numerous gullies and tunnels that broke out on the 2:1 side slope. Figures 4 and 5 typify the embankment surface appearance.

Field observations noted a significant amount of water draining from the edge of the type 'G' asphalt concrete base. As shallow trenches were excavated perpendicular to the pavement edge, large quantities of oily water flowed from the type G base into the trenches. The process was repeated at various stations along the eroded sections of the embankment, each time having the same results. This implied that a significant quantity of water had become trapped in the open graded asphalt concrete base and in turn was stripping the asphalt binder from the aggregate.

Figure 4. Erosion Hole (Jug) Formed at Pavement Edge

Figure 5. Gullies and Rills Formed on Embankment Slope

A review of the USDA Soil Conservation Service soil survey for LeFlore County, published in 1981, showed that the entire embankment and adjacent borrow pit were in a large mapped extent of the Cupco soil series. The Cupco series are noted in the LeFlore County soil survey to have a high sodium content **(5)**. A pedological soil survey identifying all soil horizons was made by hand auger at an undisturbed location between the Cupco embankment and the borrow pit. Adjacent hand auger borings and exposures in the borrow pit confirmed the Cupco soil identity. The embankment soils appeared to be consistent and uniform since the soil samples obtained on the surface and through the embankment matched those of the Cupco soil series.

<u>LABORATORY TESTS</u>

Soil samples from the embankment were obtained from the standard penetration test (SPT) split spoon samples, thin-walled tube samplers, and by hand auger along the embankment. A total of 164 samples were tested and classified according to ASTM D 4318-94 and 2487-90 respectively and measured for moisture content according to ASTM 2216-80. Seventy- six samples were taken from the thin-walled tube samplers, and the in-place densities were measured from chunk density tests according to AASHTO T 233-86. A family of curves and line of optimums was developed using 21 Harvard miniature proctor curves from samples of the embankment material. A comparison of the in-place densities to the back-calculated line of optimums is presented in figure 6. This comparison shows the embankment soils were compacted considerably dry of the optimum moisture content.

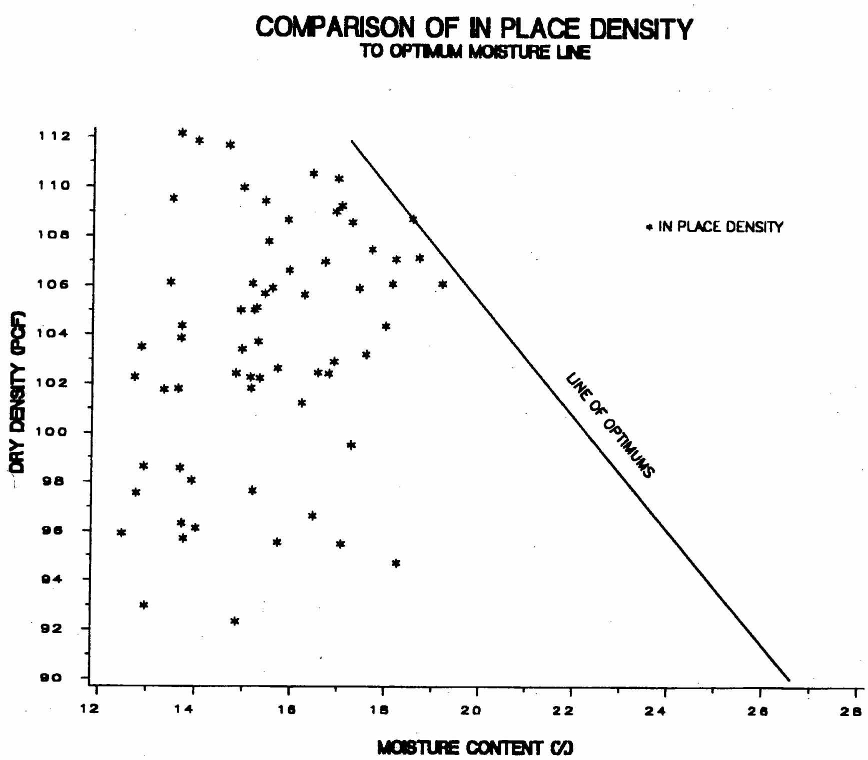


Figure 6. In-Place Densities for Southbound Embankment

Compaction dry of optimum tends to increase the chances of the formation of surface cracks. Any surface cracks still present during the placement of the type 'G' asphalt base served as paths for water infiltration, which, in turn, would initiate erosion provided the water was able to reach the cracks. After observing the amount of water trapped in the type 'G' base, there left little doubt that this was possible if not probable.

Standard tests used to classify soils for engineering purposes do not identify the dispersive characteristics of fine-grained soils. There are currently four laboratory tests that are commonly used to identify dispersive clays. These four tests are:

1. Pinhole Test (ASTM D 4647-87)
2. Chemical Analysis of Pore Water Extract
3. Double Hydrometer Test (ASTM 4221-90)
4. Crumb Test

To study the dispersive character of these embankment clays, all four of the recommended laboratory tests were used. No single test of the four is conclusive for all soils. It is always better to perform all four tests if possible. A minimum of three is recommended unless previous experience indicates otherwise.

The pinhole test was performed according to ASTM D 4647-87 Method A **(6)** on 20 embankment samples. In this test, a compacted specimen is extruded into a test mold as shown in figure 7. A 1.0 mm diameter pinhole is punched through the specimen and distilled water is run through the sample under heads of 2, 7, 15, and 40 inches. The specimen dispersion is rated based upon the turbidity of the effluent, the final hole diameter, and the flow rate of the distilled water.

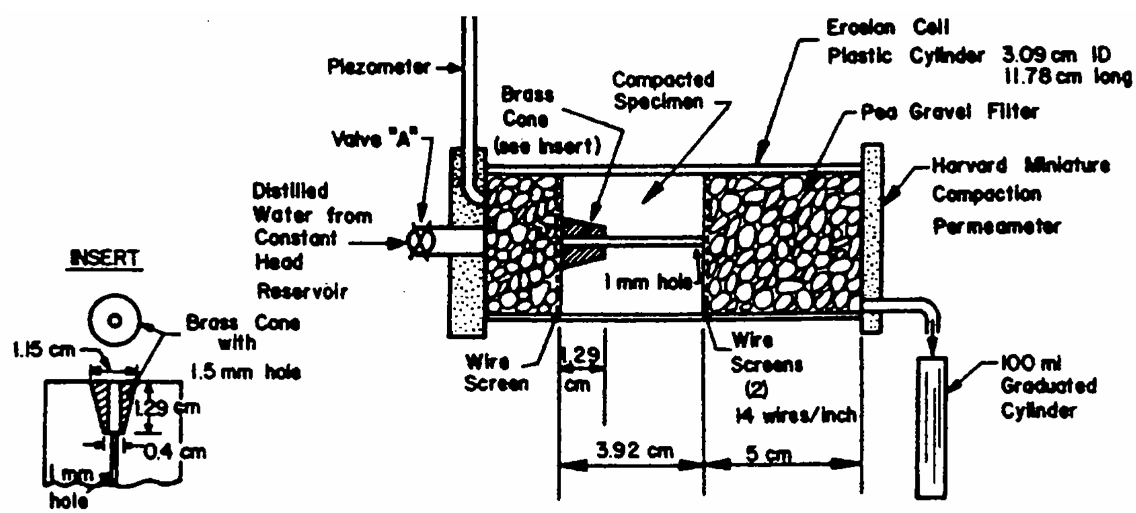


Figure 7. Pinhole Test Erosion Cell

The criteria for classification of dispersive soils based upon the pin hole test are listed in Table 1 Ratings of D-1 and D-2 are considered dispersive, ND-4 and ND-3 are considered moderately dispersive and ND-2 and ND-1 are non-dispersive. Dispersive clays erode rapidly under the 2 in. head while non-dispersive clays typically do not erode even under heads of 40 inches.

Table 1. Criteria for Evaluating Pinhole Test Results

Head	Test Duration (min)	Final Flow Rate (ml/s)	Cloudiness of Effluent	Final Hole Size (mm)	Classification
2"	5	1.0-1.4	Dark	≥ 2.0	D-1
2"	10	1.0-1.4	Mod. Dark	> 1.5	D-2
2"	10	0.8-1.0	Slightly Dark	≤ 1.5	ND-4
7"	5	1.4-2.7	Barely Visible	≥ 1.5	ND-3
15"	5	1.8-3.2	Barely Visible	≥ 1.5	ND-3
40"	5	>3	Clear	< 1.5	ND-2
40"	5	<3	Perfectly Clear	1.0	ND-1

Twenty pinhole tests were conducted with eighteen classified as D-2 and two classified as ND-4 indicating a high degree of dispersion.

The test for soluble salts in the soil pore water **(7)** is a standard test in which a soil sample is mixed with water to a consistency near the Atterburg liquid limit. A pore water sample is drawn out by vacuum using a filter and is examined using atomic absorption spectroscopy or flame photometry to determine the quantities of the four main metallic cations in solution (calcium, magnesium, sodium, and potassium) in milliequivalents per liter. The total dissolved salts (TDS) equals the total of these four cations. The percent sodium equals the quantity of sodium divided by the TDS. The Soil Conservation Service, in Lincoln Nebraska developed a relationship between pore water salts and soil dispersion, using the pinhole test **(8)**. This relationship is illustrated in figure 8.

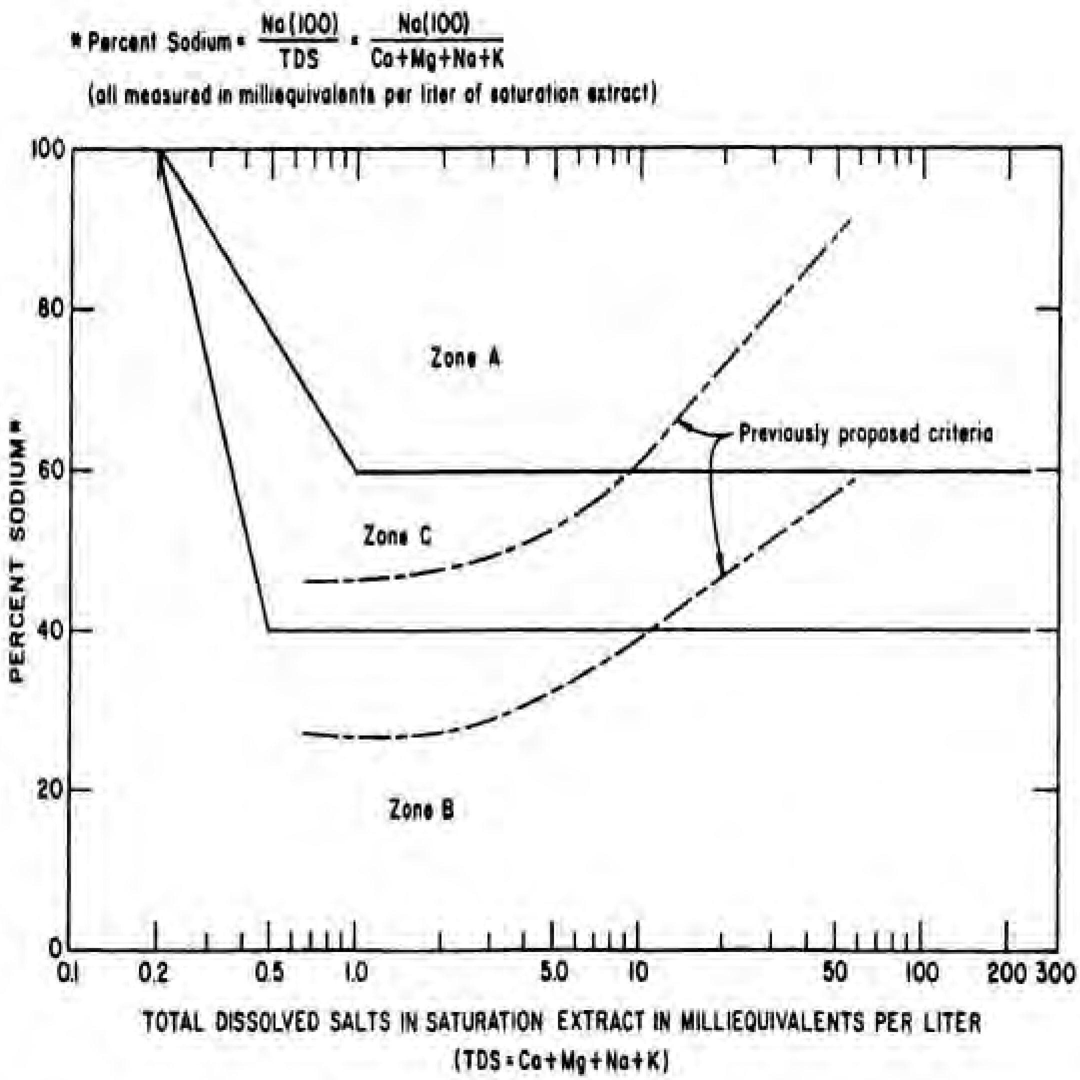


Figure 8. Relationship of Pore Water Salts for Identification of Dispersive Soils

Zone A is the high sodium area and almost all soils, which fall into this zone, are dispersive. Zone B is the low sodium area and most soils in this category are non-dispersive. Zone C is the intermediate area and soils in this zone may be dispersive or non-dispersive. Fifty pore water extracts were taken from the embankment soils and analyzed to determine the percent sodium. Analysis results indicated that approximately 75% of the samples were either dispersive or moderately dispersive as shown in figure 9.

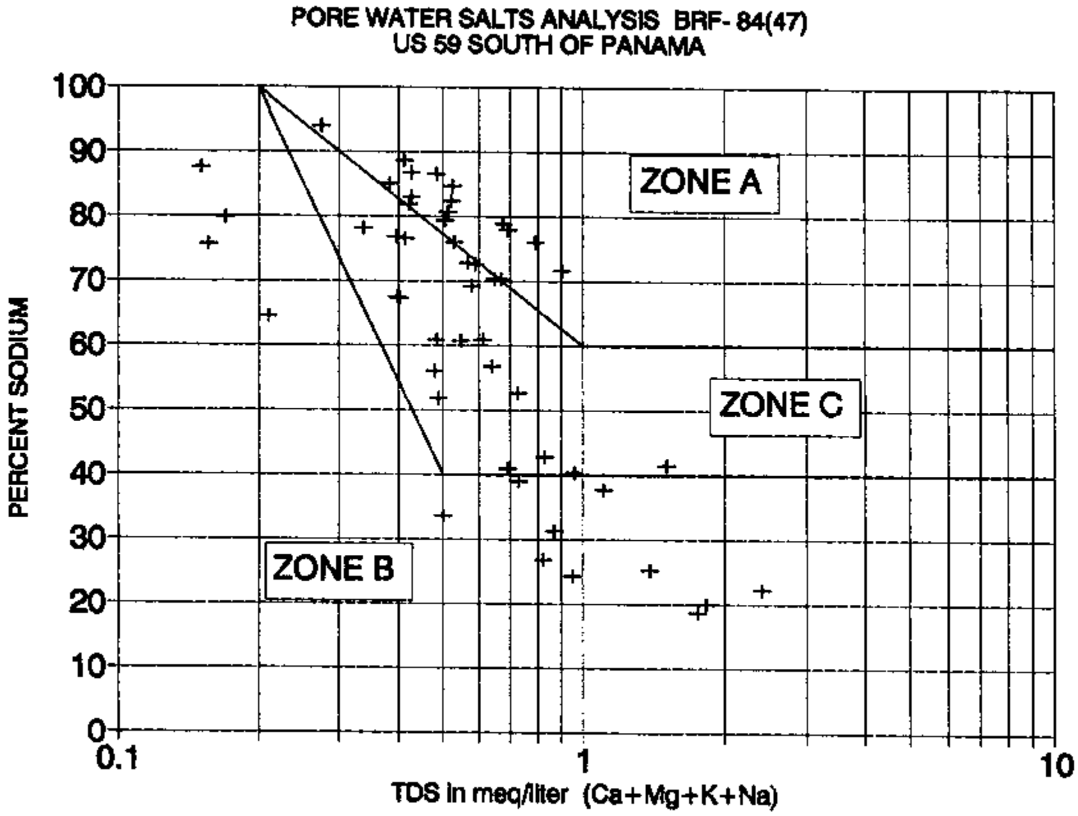


Figure 9. Pore Water Salts Analysis of Samples

The double hydrometer test or SCS laboratory dispersion test **(9)** was developed by G.M. Volk and has been widely used by the Soil Conservation Service since 1940. The test consists of conducting two hydrometer analyses to determine the soil particle size distribution. The particle size distribution is first measured using the standard hydrometer test (ASTM D422) in which the sample is dispersed in the hydrometer bath using strong mechanical agitation and a chemical dispersant such as sodium hexametaphosphate. This particle distribution is shown as curve 1 in figure 10.

A second hydrometer analysis is conducted without strong mechanical agitation or chemical dispersant. This particle size distribution is shown as curve 2 in figure 10. Curve 2,in figure 10, shows less colloidal particles than curve 1 and the difference is a measure of the tendency of the clay to disperse naturally. The percent dispersion of the soil is the ratio of clay particles passing the 0.005mm size in the two tests.

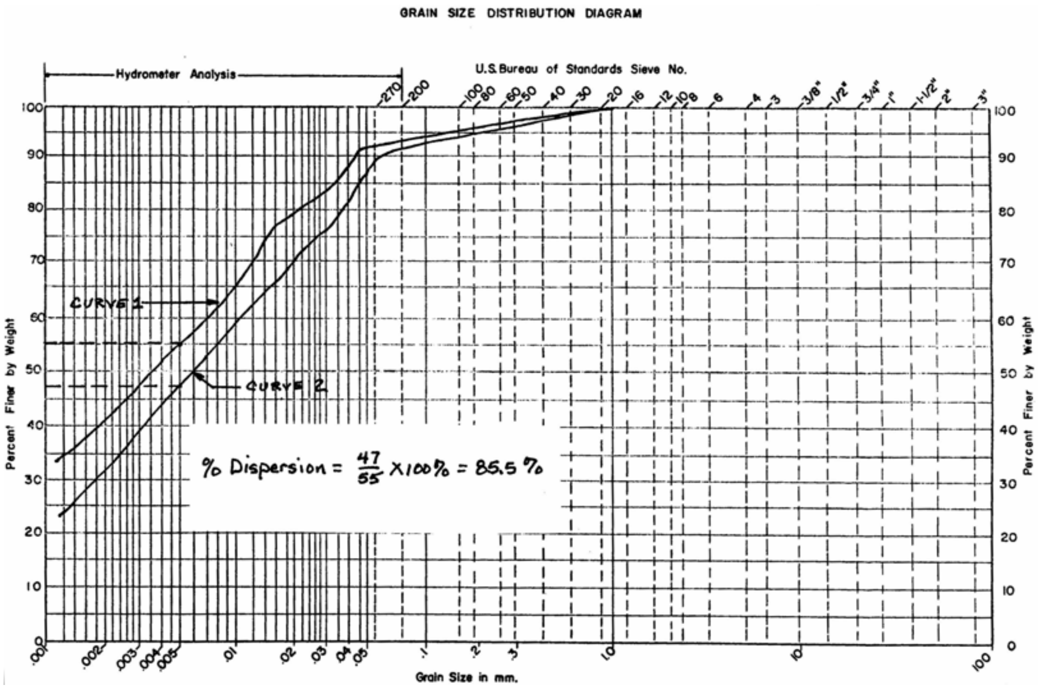


Figure 10. Double Hydrometer Analysis

Eighty- eight double hydrometer tests were conducted on the embankment soil samples. The results displayed in figure 11, revealed that 80 % of the samples had a percent dispersion of 70 % or greater.

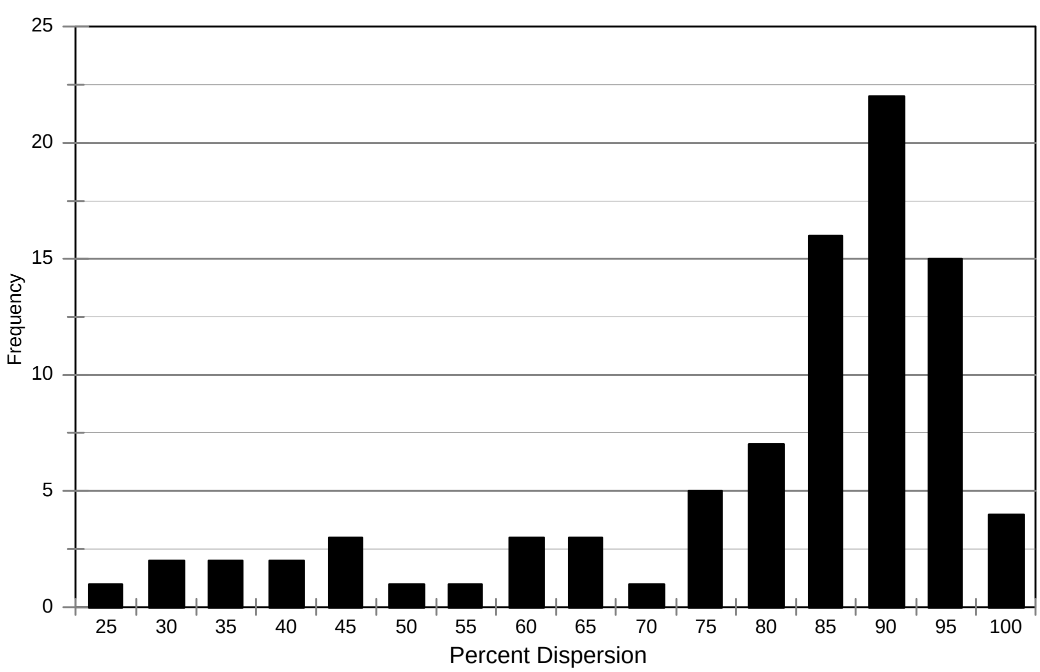


Figure 11. Double Hydrometer Test Results

The crumb test **(10)** is a fast, easily reproducible test developed by W. W. Emerson. The test consists of placing a small soil cube, approximately 1/4 in. to 3/8 in. in size, in a beaker containing 150 ml of distilled water. After 5 to 10 minutes, an observation is made on the tendency of a colloidal cloud to form from the disintegrating soil cube. The presence of this cloud is indicative that the soil is dispersive. The dispersivity of the soil is classified into four grades based upon the appearance of the colloidal cloud.

Grade 1. No reaction: No indication of cloudy water. The soil slakes and particles spread along the bottom of the beaker.

Grade 2. Slight reaction: slight indication of a cloud formation at the surface of the soil crumb.

Grade 3. Moderate reaction: Easily visible colloidal cloud forms around the soil crumb and spreads outward along the bottom of the beaker.

Grade 4. Strong reaction: Colloidal cloud covers nearly the whole bottom of the beaker. In extreme cases all of the water in the beaker turns cloudy.

Ninety- four crumb tests were conducted with the results indicating that 93.6% of the samples exhibited a moderate to strong reaction to the distilled water.

ANALYSIS AND DISCUSSION

The consistency of the embankment was very uniform based upon the site inspection, soil classification tests, and in-place density. Of the 164 samples tested, all were low plasticity clays with 163 being classified as CL and one a CL-ML. As noted, 12.0 in. of rain were recorded after the slope mulch treatment was placed on the embankment slopes. A review of precipitation records at the Poteau Water Works Station, approximately 7 miles south of the project, indicated an unusually wet cycle was occurring in the area beginning in September and agrees with records kept at the site. Preceding this wet September and October were nine months of near average rainfall. During this period the embankment was left unprotected. As previously shown in figure 6, the embankment soils were placed considerably dry of optimum. Compaction dry of optimum tends to increase the chances of the formation of surface cracks. There was ample time, during approximately nine months preceding the placement of the asphalt, for crack development. It has been shown that whenever heavy rainfall and runoff can attack exposed dispersive clays, it is the surface drying and settlement cracks that provide the avenue for dispersion to begin **(11)**. This fact was observed over the four month investigative period by noticing seemly unaffected embankment sections, with cracks, slowly degrading. Compounding the problem further was the permeability of the type 'G' asphalt concrete base and no provision for drainage. The base material was back calculated to have a coefficient of permeability of approximately 4 in/hr. After observing the amount of water flowing out of this asphalt base section, there is little doubt that water had been trapped beneath the asphalt surface course.

Other contributing factors were as follows:

1. The 2:1 embankment slope allows surface runoff to flow downslope faster than a flatter slope, hence helping to accelerate the erosion.

2. A drainage ditch was cut at the toe of the slope to allow runoff to drain to Brazil Creek. This ditch let water back up on to the slope. This further accelerated the dispersive erosion during heavy rains and flooding of the creeks. This event was observed twice during the investigation. On these occasions it was noted that the erosional exit tunnels lined up slightly below the high water marks on the 2:1 slope. The tunnel erosion rate changed during the course of the rise and fall of the water in the ditch.

3. The tracks of the bulldozer, used in placing the slope mulch treatment, are thought to have broken up the slope surfaces, therefore providing another access for rainfall to enter into the soil. The dispersion of the embankment slopes started occurring and accelerated before the planned 6 in. treatment of topsoil and sodding could be placed.

RECOMMENDATIONS

At the very beginning of the investigation it was suggested that erosion could continue to occur beneath the newly completed asphalt pavement. This created a unique situation in terms of the remedial actions that could be taken to prevent further erosion. The southbound lanes were completed and were being used to carry both northbound and southbound traffic while the northbound lanes were being constructed. Since US 59 serves as the main connection route for citizens in and around the city of Poteau to Interstate 40, removal of the asphalt pavement and treatment of the underlying embankment soil was disregarded as an option since it would require closing the highway completely. Using these constraints, a five stage remedial plan was developed.

Stage 1 called for filling all of the known holes with a sand- cement grout. This grout was to have a sand-cement ratio ranging from 2:1 to5:1. It was to be pumped using a conventional grout batch system while limiting the grouting pressure 0.75-1.0 psi. per foot of overburden depth. Estimated average quantities of grout were approximately 1/3 cubic yard per hole. Reliance on grouting to fill all of the holes along and under the pavement posed a problem since it was not possible to determine the extent of hole development under the pavement.

Stage 2 called for lime modification of the top 1.0 ft. of the roll off slope soil and the top lineal 10.0 ft. of the 2:1 slope. This would be accomplished in two phases. The top 6 in. of soil would be excavated and stock piled while the bottom 6 in. would be treated in place with 5 percent hydrated lime. Upon completion of the bottom lift treatment, the bottom lift would be scarified and the top 6in. would be returned and treated similarly. In lieu of the lime treatment of the top 1.0 ft. of the embankment, undercutting and replacing with nondispersive select material was recommended.

Stage 3 required filling in any holes along the 2:1 slope that were unable to be grouted with soil that was classified as nondispersive according to double hydrometer analysis (ASTM 4221-90) and the pinhole test (ASTM 4647-87).

Stage 4 required flattening the existing 2:1 slope to a 4:1 slope beginning at the 6:1 roll off slope extending to the toe of the embankment. Soil used in this stage was to be classified as nondispersive. This new slope construction was to be benched in to the existing slope according to Section 202 (c) of the Oklahoma Standard Construction Specifications as shown in figure 12.

Stage 5 called for planting the 4:1 slope and the 6:1 slope with Bermuda grass solid slab sod.

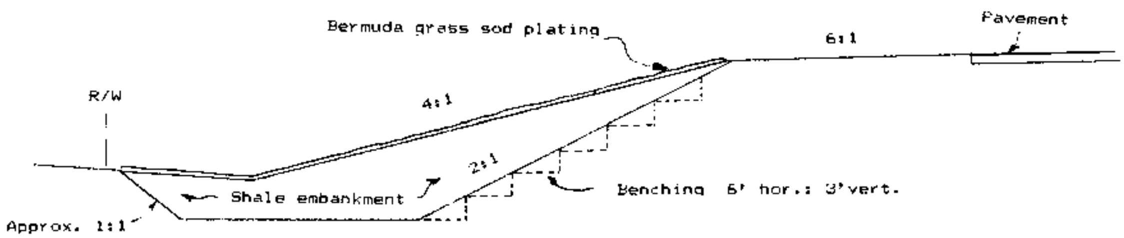


Figure 12. Repair Details for Eroded Sections of Embankment

CONCLUSIONS

The field and laboratory evidence conclusively identified the soils used for embankment construction as, high sodium, dispersive clays. The use of the four tests for dispersive soil identification is still considered an appropriate practice for positive identification.

The recommendations suggested were accepted by the construction engineers responsible for the project and implemented by the project contractor. The final stage was completed in early April 1992. The side slopes of the embankment have displayed no signs of erosion but there has been a continuing problem with depressions developing underneath the pavement where no treatment was applied. Inspection of these depressions revealed the presence of large voids under the pavement that had an average depth of 4.0 ft. This presents a significant maintenance problem as well as a liability problem if these depressions continue to appear. The project has been monitored for several years with no significant problems occurring.

The normal practice of the Oklahoma Department of Transportation had been, as in this case, to allow for unclassified borrow to be used as embankment material, in order to get low contract prices. This process requires only soil classification and moisture-density tests for quality control without any preliminary screening for dispersive soils.
The lesson in this case history is twofold:

a. In the development of preliminary soils reports, a greater effort has been made to examine the potential for dispersive soils appearing in the highway alignment and any potential borrow sources.

b. Where the use of dispersive soils appears to be unavoidable in certain areas of the state, design recommendations will be made for their incorporation in the project earthwork that will negate their effect.

The use of dispersive clays in the construction of earthen dams and highway embankments can lead to costly repairs or, in extreme cases, catastrophic failures. Previous case histories have shown that the use of dispersive clays can be reliable provided the soil is adequately protected through chemical treatment or protective coverings. A standard soils testing program should include testing for dispersivity using the four basic dispersion tests if there is the slightest indication that the project area contains dispersive clays.

REFERENCES

(1) Sherard, James L., Dunnigan, Lorn P., Decker, R.S., and Steele, Edgar F. "Pinhole Test for Identifying Dispersive Soils", Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, No GT1

(2) McElroy, Charles H. "The Use of Chemical Additives to Control the Erosive Behavior of Dispersed Clays", Engineering Aspects of Soil Erosion, Dispersive Clay and Loess , ASCE, Geotechnical Special Publication No. 10

(3) Holmgren, G.G.S., and Flannigan, C.P., "Factors Affecting Spontaneous Dispersion of Soil Materials as Evidenced by the Crumb Test", Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects , ASTM, STP 623

(4) Ryker, N.L. “Encountering Dispersive Clays on Soil Conservation Service Projects in Oklahoma”, Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects, ASTM, STP 623

(5) Soil Conservation Service, Soil Survey of LeFlore County Oklahoma, USDA.

(6) ASTM D 4647-87, "Standard Test Method for Identification and Classification of Dispersive Clay Soils by the Pinhole Test", Annual Book of ASTM Standards Section 4, ASTM Vol. 04.08

(7) Sherard, J.L., Dunnigan, Lorn P., and Decker, Rey, S., "Identification and Nature of Dispersive Soils", Journal of the Geotechnical Engineering Division, ASCE, Vol.102, No.GT4

(8) Flanagen, C.P. and Holmgren, G.G.S., "Field Methods for Determination of Soluble Salts and Percent Sodium from Extract for Identifying Dispersive Clay Soils", Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects, ASTM, STP 623

(9) ASTM D 4221-90, "Standard Test Method for Dispersive Characteristics of Clay Soil by Double Hydrometer" , Annual Book of Standards Section 4, ASTM Vol. 04.08

(10) Sherard, J.L., Dunnigan, Lorn P., and Decker, Rey S., "Identification and Nature of Dispersive Soils", Journal of the Geotechnical Engineering Division, ASCE, Vol.102, No.GT4

(11) Lewis, D.A. and Schmidt, N.O., "Erosion of Unsaturated Clay in a Pinhole Test", Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects, ASTM, STP 623

A Case Study of Methods used to Study a Sinkhole on Interstate 40, Pender County, NC

Thomas J. Douglas[1], Brad Worley[1], and Colin Mellor[2]

1. North Carolina Department of Transportation, Geotechnical Engineering Unit, 1589 Mail Service Center, Raleigh, NC, 27699-1589; tdouglas@dot.state.nc.us; bdworley@dot.state.nc.us

2. Geophex, Ltd., 605 Mercury Street, Raleigh, NC, 27603-2343; (919) 839-8515; cmellor@geophex.com

ABSTRACT

On August 3rd, 2001 a sinkhole developed at mile marker 410.5 in the eastbound lane of I-40 near Rocky Point, NC, 10 miles North of Wilmington, NC. A 25 ft diameter depression subsided a maximum of 14 inches, creating a serious hazard at interstate highway speeds. The sinkhole was remediated by undercutting, placing a layer of geotextile and backfilling with aggregate base course material to "bridge" the subsided area and reopen the interstate to traffic. An extensive grouting program was employed after traffic had resumed.

After this incident, NCDOT became concerned that more sinkholes may open up on this stretch of I-40. The middle Eocene Castle Hayne Limestone and the massive sands of the Cretaceous Peedee Formation underlie the surrounding area and the majority of southeastern North Carolina. Solution cavities are well documented within the Castle Hayne Limestone and sinkholes are a common occurrence in the area.

NCDOT retained a geophysical consulting firm to study a one mile section of the highway in an attempt to identify potential areas where sinkholes could be a problem in the future. Electrical resistivity, microgravity, and ground penetrating radar surveys were collected between mile 410 and 411 of I-40. Historic aerial photography, current aerial photography, and several methods of drilling were also used to characterize the problem.

INTRODUCTION

On Friday, August 3rd, 2001, a sinkhole developed on I-40 near Rocky Point, NC, 10 miles North of Wilmington, NC, between mile markers 410 & 411. The sinkhole was reported to the highway patrol at 5 AM in the morning and no injuries were reported due to the sinkhole. The asphalt pavement sagged and cracked but effectively bridged the sinkhole. The oval-shaped sunken area of pavement measured 28 by 20 feet and had a maximum depth of 14 in. (Plate 1). A majority of the sinkhole was in the right eastbound lane. Approximately 22,000 vehicles use this stretch of I-40 on a daily basis. Eastbound traffic was temporarily rerouted along secondary roads. Westbound traffic flow was unaffected. After a quick remediation by NCDOT, the eastbound lane was reopened to traffic within 32 hours.

Plate 1 20 X 28 ft depression. Note 15" sag, skid marks, and gashes in pavement.

Plate 2 Undercut for 42 ft X 40 ft "patch"

In an effort to lower the potential for dangerous traffic-threatening sinkholes in the future, NCDOT is attempting to learn as much as possible about the geology that led to the development of the sinkhole. This will hopefully lessen the impact of such failures and increase the safety of the motoring public. To this end, NCDOT decided to use drilling methods and non-destructive technologies to study the area around the sinkhole and develop a monitoring program to evaluate this area in the future.

Construction on this portion of Interstate 40 began in December 1982 and was completed in 1984. During construction in 1983, several sinkholes were documented in the NCDOT right-of-way. Other than the August 3rd, 2001 sinkhole, no sinkholes have been documented in NCDOT right-of-way between 1983 and 2003 in or near the vicinity of this section of I-40.

REMEDIATION

Initially, to reopen the interstate as fast as possible, NCDOT drilled a single boring and a geophysical consulting firm conducted a ground penetrating radar survey. With these results, NCDOT decided to undercut to a depth of 4 feet. The excavation measured 42 by 40 feet and covered the entire width of the eastbound lanes (Plate 2). In an attempt to "bridge" the sinkhole and reduce the effects of future subsidence, the removed material was replaced with a layer of geotextile, 4 feet of aggregate base coarse (ABC), and several inches of asphalt.

To help prevent further subsidence of the sinkhole, the Geotechnical Unit followed the initial patch with 4 weeks of grouting. A grout volume of 127.3 yd^3 was pumped into 22 holes in the vicinity of the sinkhole. Grout was pumped at approximate depths of 12 and 25 feet in an attempt to stop or lessen the subsidence. Additional subsidence of 1 to 3 inches occurred in the left eastbound lane. Almost all of the grout was pumped under zero pressure and was effective in stopping the subsidence. No other significant subsidence has occurred since this time.

STRATIGRAPHY

Coastal Plain Sedimentary Soils and Rock

Two formal stratigraphic units, the Peedee Formation and Castle Hayne Limestone (See Figure 1, southeastern portion of NC, Green = Peedee Formation and Light Orange = Castle Hayne Limestone), were observed in borings at the Interstate 40 (Pender County) sinkhole locality and in the nearby Martin Marietta quarry at Rocky Point. They are described as follows:

Peedee Formation

The Cretaceous Peedee Formation (Swift and Heron, 1967) typically consists of massive dark greenish gray, micaceous, glauconitic, argillaceous, fine-grained sand. In the vicinity of the I-40 sinkhole, the Rocky Point Member can be observed overlying the dark green sands of the Peedee. The Rocky Point is a sandy, pelecypod-mold grainstone (sandy limestone). The Peedee sands and overlying Rocky Point Member represent a single depositional cycle that suggests an overall shallowing upward highstand deposit (Harris, 2003).

Castle Hayne Limestone

Soils of the middle Eocene Castle Hayne Limestone (Miller, 1912) are found underlying embankment soil in all borings at the sinkhole locality. Zullo and Harris (1986, 1987) identified five depositional sequences within the Castle Hayne Limestone, three of which are recognized in the vicinity of the I-40 sinkhole (Harris, 2003). Sequence 1 (See Figure 2) consists primarily of fine- to medium-grained, unconsolidated and unlithified (non-indurated) quartz-rich bryozoan sand. The lowermost lithology of Sequence 1 is a light gray, dense molluscan, phosphate pebble conglomerate. This basal lithology of Sequence 1 never exceeds 1.1 feet in thickness and forms a sharp disconformable contact with the underlying Cretaceous Peedee Formation (Harris, 2003). This disconformity marks the Cretaceous-Tertiary (K-T) boundary in this locality.

Sequence 2 disconformably overlies Sequence 1 and has been observed to have a maximum thickness of 12.0 feet (Harris, 2003). Sequence 2 lithologies are very similar to those observed in Sequence 1 and consist primarily of tan-gray, fine- to medium-grained, cross-bedded bryozoan grainstone. In both Sequences 1 and 2 the bryozoan sand is observed to form “sand waves” (Plate 3) with a north-south orientation (Harris, 2003).

Sequence 3 disconformably overlies Sequence 2 and is the thickest and most extensive unit of the Castle Hayne Formation found in the area (Harris, 2003). Sequence 3 primarily consists of tan, poorly consolidated, well-washed, highly cross-bedded, bryozoan grainstone. Sequence 3 is overlain by recent surficial deposits in this area.

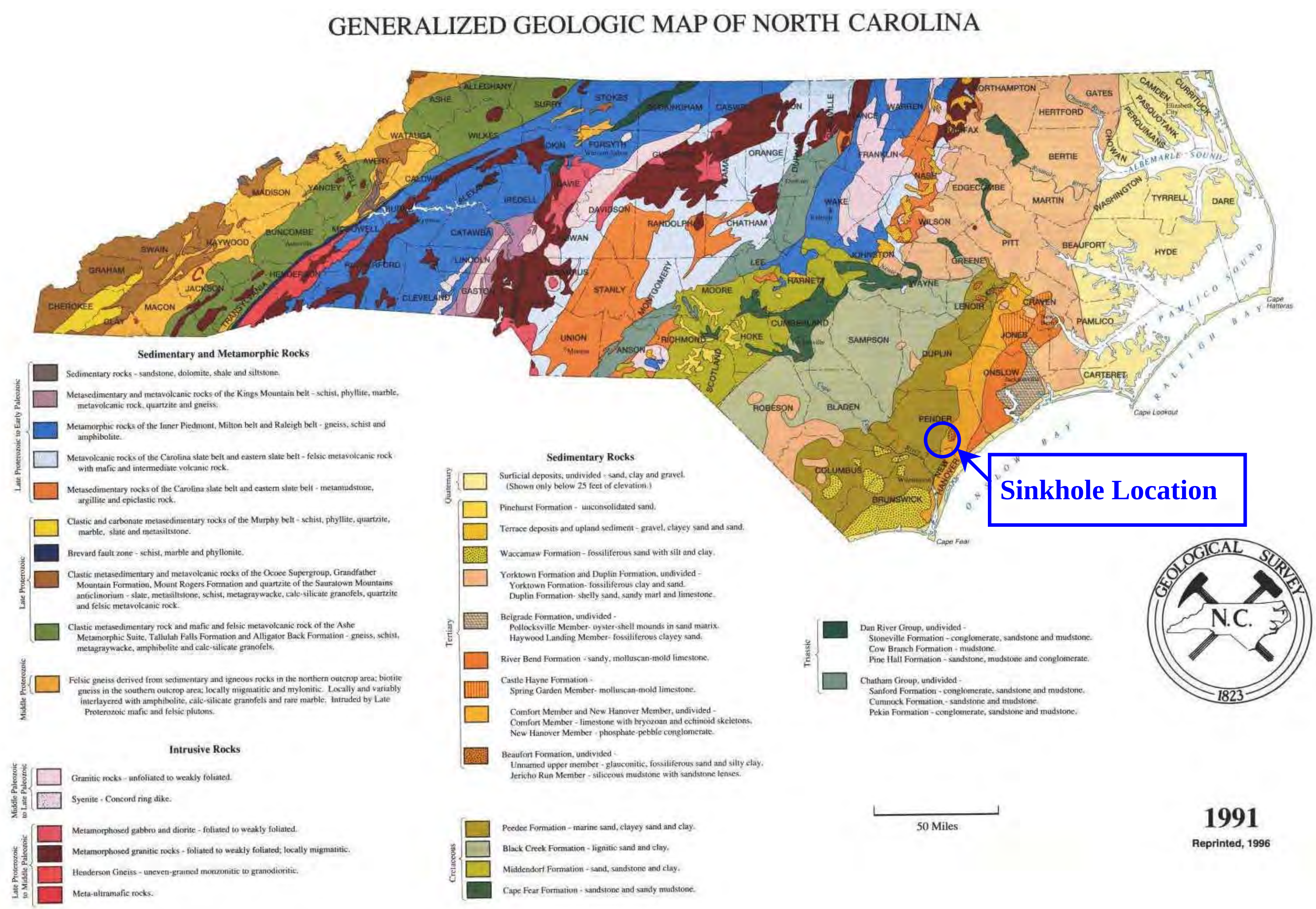


Figure 1 Generalized Geologic Map of North Carolina from, North Carolina Geological Survey, Division of Land Resources, Department of Environment, Health, and Natural Resources, 1991.

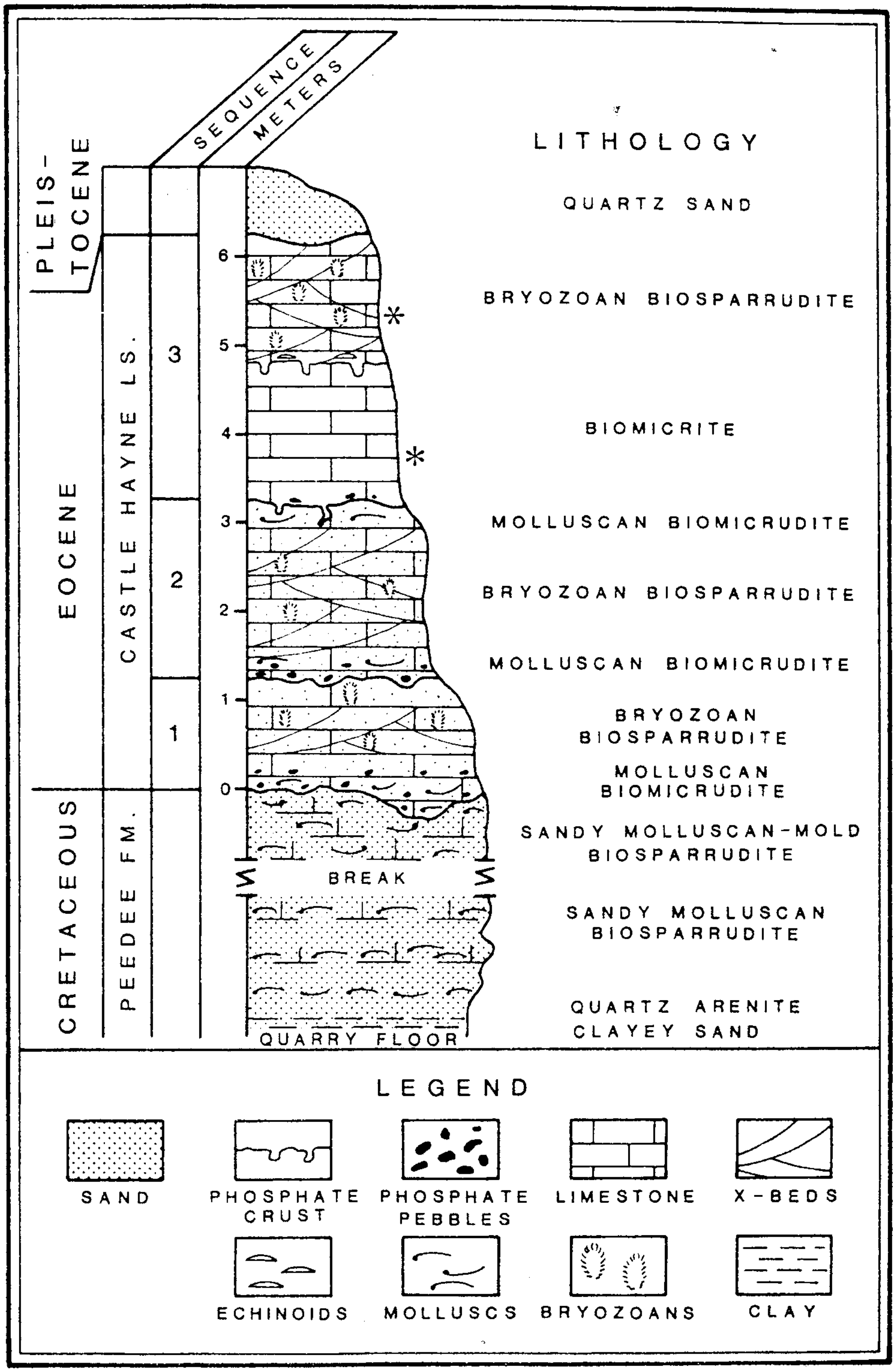


Figure 2 Stratigraphy column showing relationship between Sequences 1, 2, and 3 of the Castle Hayne Limestone and Peedee Formation. (Harris, 1986, 2003)

Material Observed in Sinkhole Borings

Roadway Embankment Fill

All borings performed at the I-40 sinkhole locality encountered roadway embankment fill material ranging from 8.5 to 13 ft in depth. Fill material has been in place since construction of this stretch of I-40 in the early 1980's. This fill material is medium dense, tan to dark brown, silty fine sand (AASHTO classification: A-2-4) and SPT drives yielded N-values of 30 to 100 blows per foot.

Castle Hayne Limestone

Ten borings have been completed while investigating the I-40 sinkhole locality. Borings 1-4 were advanced to ~30.0 feet. Borings 6, 7, and 8 were advanced to 60.0 to 80.0 feet and Borings 9 and 10 were advanced to ~121.0 feet. All borings encountered soils of the Castle Hayne Limestone. Underlying the roadway fill sands, down to approximately 20.0 feet, were gray-brown to gray, loose to dense, silty fine to medium sand (A-2-4) and gray, very soft to soft, sandy clay (A-6) interpreted here to be Sequence 3 of the Castle Hayne Limestone. Underlying Sequence 3, from approximately 20.0–35.0 feet, is a gray, medium stiff to very stiff, sandy silt (A-4). This material is interpreted here to be the uppermost lithology of Sequence 1 or Sequence 2. Underlying the A-4 soil is a tan-gray to gray, very loose, non-indurated, saturated, bryozoan sand (A-2-4). N-values within this material range from “rod-drop” to 5 blows per foot. These completely unconsolidated, non-indurated sands are interpreted here to be “sand waves” observed in Sequences 1 and 2 of the Castle Hayne Limestone (Plate 3).

Peedee Formation

Lithologies of the Peedee Formation were found in all borings advanced beyond 30.0 ft. Stratigraphically, the Rocky Point Member of the Peedee Formation was found underlying the lowermost Castle Hayne Limestone sequences. A complete section of the Rocky Point Member was observed only in Boring 10, located approximately 2000 feet south of the sinkhole, where it was 11 ft thick. In Borings 6, 7, 8, & 9, the Rocky Point was observed as a gray, very dense, coarse sand with fossil fragments (A-1) but the pelecypod-mold limestone was absent.

Underlying the Rocky Point Member, in all borings advanced beyond 30.0 feet, are the dark green, dense to very dense, fine sand (A-3). In several borings, very thin (0.25 feet) zones of moderately indurated material were documented, but not sampled. Borings 9 and 10 were terminated at approximately 121.0 feet in these dark green glauconitic sands. These are interpreted here to represent the uppermost depositional sequence of the Peedee Formation.

Plate 3 Rocky Point Quarry exposure of Bryozoan “sand wave” of the Castle Hayne Member on top of the “pelecypod-mold limestone” of the Peedee Formation

GEOPHYSICS

To better characterize the subsurface under I-40 near the sinkhole, a geophysical consulting firm, Geophex, Ltd., was retained to carry out several geophysical surveys. Ground penetrating radar (GPR), electrical resistivity, and microgravity surveys of up to one mile in length were conducted and the results are summarized below. The following instruments were used: MALA Geoscience RAMAC X3M GPR (100 & 250 MHz shielded antennas), Advanced Geosciences Sting/Swift (28 electrode) and SuperSting (56 electrodes) resistivity imaging systems, and Scintrex CG-3M gravimeters.

Ground Penetrating Radar

Four GPR profiles 550 feet in length and four GPR profiles 4250 feet in length were collected. The transition from embankment to natural ground is apparent in the surveys. Profiles show truncated layers near the surface in the location of the sinkhole and appear to show continuous reflectors at a depth of 40 feet. This would suggest that the dissolution area is above this depth (See line in Figure 3).

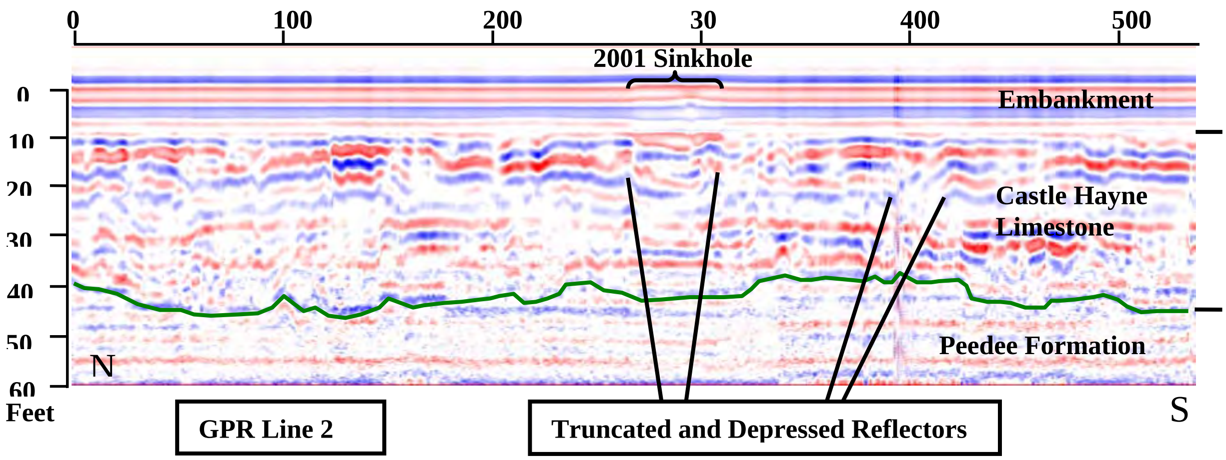


Figure 3 550 ft GPR profile taken with 250 MHz antenna.

Electrical Resistivity

Three resistivity profiles 550 feet in length and two resistivity profiles 4250 feet in length were collected using standard dipole-dipole geometry. Initial surveys identified the existing sinkhole and another similar anomaly 80 feet South (Figure 4). This anomaly was confirmed with Boring 6. Very soft material was encountered at a depth of 34-39 feet in Borings 6 and 8.

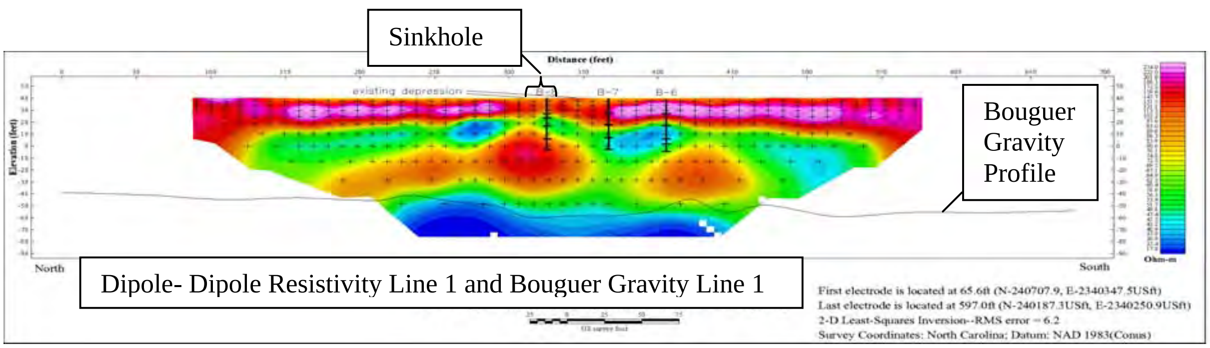


Figure 4 682 ft Bouguer Gravity Profile and Borings 6, 7, & 8 superimposed on top of 550 ft Electrical Resistivity Profile (standard dipole-dipole geometry).

Microgravity

Before grouting was completed, three Bouguer gravity profiles 682 feet in length were collected at 26.2 foot and 52.5 foot intervals in the east bound lanes. After grouting was completed, three additional profiles were collected over the entire width of the interstate at 30 foot intervals from mile 410 to 411. Gravity data show changes between the before grouting surveys and after grouting surveys, apparently showing the grouting to be effective (Figure 5). The gravity profiles also show several local drops in gravity, which could indicate areas where dissolution has occurred. The mile long gravity survey showed a broad gravity low, believed to correspond with an absence of the pelecypod-mold limestone, and possibly even representing a paleochannel (Figure 5).

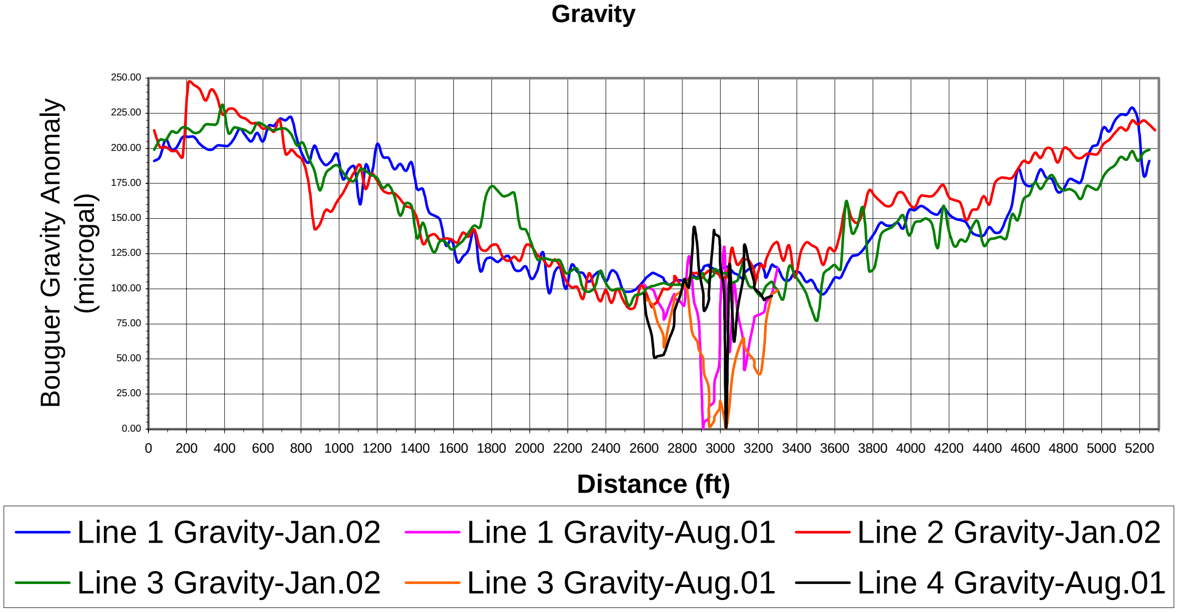


Figure 5 Bouguer gravity profiles. Three gravity lines one mile in length showing broad gravity low. Three gravity (black, orange, and purple) lines at bottom center show gravity before grouting operation.

CONCLUSIONS

Data from the geotechnical borings, gravity surveys, and electrical resistivity indicate the 10 foot thick pelecypod-mold limestone (Rocky Point Member) of the Peedee Formation is absent for over 3000 ft within the study area, with the August 3rd, 2001 sinkhole lying in the center of this 3000 ft. These factors lead to the assumption that a paleochannel exists and eroded the Rocky Point Member. Where the pelecypod-mold limestone is absent the Castle Hayne Limestone Formation is directly overlying Peedee Formation sands, creating a conduit for water flow, dissolution, and piping.

FUTURE MONITORING

NCDOT is interested in developing a proactive solution to reduce the risks of sinkhole development in eastern NC in the future. NCDOT is currently considering a monitoring program over a 4 mile length of interstate. This program will consist of aerial photography by helicopter on an annual basis and falling-weight deflectometer testing also. The aerial photography would then be used to generate DTM's (accurate to +/- 0.3 inches) to measure cumulative displacements along this 4 mile stretch to identify areas or zones of subsidence before it becomes a risk to the travelling public. Under this plan, automated groundwater measuring wells will also be installed to record daily water table fluctuations. Groundwater fluctuations caused by flooding from hurricanes and other heavy rainfall events or prolonged drought conditions will be documented and help us to understand and possibly prevent future sinkhole development.

REFERENCES

Miller, B.L., 1912, The Tertiary formation, *in* Clark, W.B., Miller, B.L., Stephenson, L.W., Johnson, B.L., and Parker, H.N., eds., The Coastal Plain of North Carolina: North Carolina Geologic and Economic Survey, v. 3, p. 171-258, 272-366.

Swift, D.J.P., and Heron, S.D., Jr., 1967, Tidal deposits in the Cretaceous of the Carolina Coastal Plain: Sedimentary Geology, v. 1, no. 3, p. 259-282.

North Carolina Department of Transportation Report, 1984, Sinkhole Investigation Near Martin-Marietta Quarry, 42 pg.

Zullo, V.A., and Harris, W.B., 1986, Introduction: Sequence Stratigraphy , Lithostratigraphy, and Biostratigraphy of the North Carolina Eocene Carbonates, SEPM Field Guidebooks, Southeastern United States Third Annual Midyear Meeting, Society of Economic Paleontologists and Mineralogists, p 253-332.

Zullo, V.A., and Harris, W.B., 1991, Eocene and Oligocene Stratigraphy of the Outer Coastal Plain, *in* Horton, J.W., Jr., and Zullo, V.A., eds., The Geology of the Carolinas, *Carolina Geological Society Fiftieth Anniversary Volume*, The University of Tennessee Press, p. 251-262.

Gardner, C.H., 1994, Small Generalized Geologic Map of North Carolina, North Carolina Geological Survey, North Carolina Department of Health and Natural Resources.

Mellor, C.A., 2001, Geophysical Investigation on I-40 Eastbound, Pender County, NC, Geophex Job No. 1132, Geophex, Ltd. report for North Carolina Department of Transportation.

Mellor, C.A., 2002, Gravity Survey along Interstate 40 From Mile 410 to 411, Pender County, NC, Geophex Job No. 1155, Geophex, Ltd. report for North Carolina Department of Transportation.

Mellor, C.A., 2002, Geophysical Investigation along I-40 EB 2-D Resistivity and Ground Penetrating Radar, Rocky Point, Pender County, NC, Geophex Job No. 1194, Geophex, Ltd. report for NCDOT.

Mellor, C.A., 2003, Geophysical Investigation along I-40 Eastbound 2-D Resistivity and Ground Penetrating Radar-Addendum, Rocky Point, Pender County, NC, Geophex, Ltd. report for NCDOT.

Harris, W.B., 2003, An overview of the marine Cretaceous and Tertiary deposits between Cape Fear and Cape Lookout, North Carolina, Field Trip Guidebook. Department of Earth Sciences, University of North Carolina at Wilmington.

Statistical Analysis of Unconfined Compressive Strength of Rock Types found in Oklahoma

Vincent G. Reidenbach, Ph.D., P.E. Oklahoma Department of Transportation
James B. Nevels, Jr., Ph.D., P.E. Oklahoma Department of Transportation
Curtis J. Hayes, Geologist Consulting Geologist

ABSTRACT

This paper presents a statistical analysis of intact unconfined compressive strength and unit weights of major rock types found in the State of Oklahoma. Variations in rock strength and unit weight data with depth are presented. The rock types studied include shales, sandstones, granite, gypsum, chert and limestones. This statistical analysis utilizes a large number of samples to determine data average, standard deviation and an estimate of statistical distribution of the data. These data show that unconfined compressive strength and unit weight are adequately represented by a normal distribution.

Unconfined compressive strength was determined by using current ASTM standard procedure D 2938. Samples of cores were cut into nominal 4 inch long by 2 inch diameter test specimens. A compression machine capable of applying a constant strain rate until sample failure was utilized for this study. Unconfined compressive strength was then determined based on ASTM D 2938 standard practice. Intact unit weight was determined by using AASHTO T 233 standard practice.

The rock cores were obtained using a CME 75 drill rig with a conventional NX core barrel 5 feet in length. Core samples were taken from all borings according to ASTM D 2113 standard practice with the preservation and transport of core samples following ASTM D 5079 standard practice. These cores were obtained from various locations in Oklahoma as a result of exploratory geologic borings for bridge foundations. Location and approximate extent of rock types in Oklahoma are reported. Each boring core was extended 35 feet into the rock mass.

INTRODUCTION

Unconfined compressive strength and unit weight are mechanical properties that are used in almost all geologic and engineering analysis of rock. These properties are determined through standard laboratory measurements and exhibit a natural variability that should be considered when performing an engineering analysis or geologic assessment. The average values are used as an input for determining RMR while the standard deviation will provide a level for confidence for the values. This variability in properties is a function of mineral size, mineral type, grain size, shape as well as other geologic phenomenon. Aday and Pusch (1) reported that weakness in rocks range in size from sub-microscopic and microscopic up to millimeters in length. Wu et al. (2) studied Granite and conducted statistical analysis that demonstrated unconfined compressive strength as well as other engineering properties can be represented by a normal distribution. Rock cores of five different types of rocks representing the range from hard to soft were obtained from several locations in Oklahoma for testing. Unconfined compressive strengths and unit weights were determined for all intact samples. A statistical analysis of the results is presented in this paper.

FIELD CORING PROCEDURE

All rock samples used in this study were obtained from cores of rock masses that were cored for exploratory geologic and engineering analyses for bridge foundations at various locations in Oklahoma. These coring procedures were done following ASTM D 2113 standard practice. A CME 75 drill rig with a conventional NX type core barrel 5 feet in length was used to obtain all cores. A 5 foot length of cores was sampled from the rock mass, removed from the core barrel and field logged. Figures 1 and 2 show this rig and drill crew while performing some of the coring and logging procedures.

After logging, all cores were wrapped and boxed for preservation and transport following ASTM D 5079 standard practice. Figure 3 shows a 5 foot core after being wrapped and boxed. Figure 4 is a map of nonpetroleum mineral resources of Oklahoma and shows the general location of different rocks types in Oklahoma (3).

LABORATORY TESTING PROCEDURE and DATA ANALYSIS

In the laboratory, a detailed core log was prepared of each core run. Testing samples were marked and cut from the intact sections of each 5 foot core run. A tile cutting saw was used to cut each test specimen into nominal 4 inch long cylinders with perpendicular ends, with a nominal diameter of 2 inches. Figure 5 shows some of the samples after being prepared for testing.

Wet unit weight of the samples was then determined by using AASHTO T 233 standard practice. Unconfined compressive strength was determined based on ASTM D

2938. Three diameter measurements were averaged and used to determine the average cross-sectional area. A compression machine capable of applying a constant strain rate until sample failure was used to determine peak failure load for each sample. Unconfined compressive strength was computed by dividing the peak load by the average area for each sample. Figure 6 shows the compression machine used to determine peak load.

An Excel spreadsheet was used to statistically analyze these sample data. This includes wet unit weight and unconfined compressive strength, which were used to determine the average values and standard deviations. Plots were then constructed to represent the variation in these data. Histogram plots were also constructed to represent the distribution of these data, see Figures 7 - 18.

GEOLOGIC CHARACTERISTICS and ANALYSIS RESULTS

Shale and Sandstones

These cores were obtained from a bridge foundation investigation in Pottawatomie County in central Oklahoma. The Garber-Wellington geologic unit consists of reddish shales and considerable amounts of sandstone and form the “red beds”. The shales along with the sandstones are mostly soft to very soft. The sandstones are poorly cemented and massive (up to 40 feet thick) and reddish in color. The unit outcrops in a north-south band 12 to 24 miles wide through central Oklahoma. The unit usually underlies gently rolling prairie plains in the upper portion of the unit, where the sandstones are prominent and covered with black-jack trees. This north-south area is geographically known as the Oklahoma “blackjack belt”. Figures 7, 8, 9 and 10 are histogram plots of the data showing the frequency distribution of the data.

Granite

These cores were obtained from a county bridge foundation investigation in south central Oklahoma in western Johnston County. The bridge site was located on an outcrop of Tishomingo granite. This rock is coarse grained, pinkish, homogenous, very hard, massive-bedded, with very low jointing. This very coarse-grained granite weathers much more easily than monument grade granite. The depth of the Tishomingo granite has been estimated to be several thousand feet thick. This granite is commercially mined and used as granite aggregate   Figures 11 and 12 are histogram plots of the data showing the frequency distribution of the data.

Gypsum

These cores were obtained from a road widening project crossing the Gypsum Hills in north-western Oklahoma. The Blaine geologic unit, located in western Major County of Oklahoma, consists of three prominent gypsum beds that are separated by shales. The gypsum beds are the Medicine lodge gypsum (15-31 feet) thick, the Nescatunga gypsum (10-22 feet) thick, and the Shimer gypsum (11-15 feet) thick. This unit outcrops in a 1 mile to 6 miles wide northwest-southeast irregular band across western Oklahoma. The thick gypsum beds form ledges that extend the entire length of the outcrop and form the "Gypsum Hills" of northwestern Oklahoma. These Gypsums are barren of vegetation and contain numerous sinkholes, caverns and other karst features. Figures 13 and 14 are histogram plots of the data showing the frequency distribution of the data.

Chert

These cores were obtained from a bridge foundation investigation in eastern Oklahoma. The Reed Springs geologic unit is located in Adair County in east central Oklahoma. This unit is a continuous sequence of very hard, dark gray to black cherty limestones. The Reed springs is about (30 to 35 feet) thick and contains no other rock types. This unit outcrops in eastern Oklahoma and provides rolling to hilly terrain with many vertical cliffs and narrow stream valleys. Figures 15 and 16 are histogram plots of the data showing the frequency distribution of the data.

Limestone

These cores were obtained from a bridge foundation investigation in north-eastern Oklahoma. The Boone geologic unit is located in Delaware County in northeast Oklahoma. This unit is a massive unit that contains limestone, limy chert and shale. The total thickness of this unit is about (200 to 300 feet) and contains different amounts of limestone, limy chert and shale. The Boone unit outcrops in northeastern Oklahoma and the outcrop area is rolling to hilly with many vertical cliffs. Figures 17 and 18 are histogram plots of the data showing the frequency distribution of the data.

DISCUSSION

The results of this statistical analysis show that unconfined compressive strength and unit weight can be represented adequately by a data mean and standard deviation. The analyzed rock type varied from very soft shales to very hard cherts. These data were fairly well represented by a normal distribution. Table 1 shows the number of test samples, averages, and standard deviations of the data. The Granite, Gypsum, Chert, and Limestone data all had a large sample size and all were well represented by a normal distribution. For these data the standard deviation value was approximately 25% of the data average value. The Shale and Sandstone data had a much smaller sample size. A normal distribution did not represent these data as well as the data with higher sample size. Also, the Shale and Sandstone samples were very soft with low strength. For these data the standard deviation value was approximately 75 % of the data average value.

CONCLUSIONS

(1). Unconfined compressive strength and unit weight of rocks can be adequately represented statistically by a normal distribution.

(2)**.** Data averages and standard deviations represent these data in a statistically good manner and provide an easily determined property for use in other types of analysis.

REFERENCES

(1) Adey, R.A. and Pusch, R., “Scale dependency in rock strength”, Engineering Geology 53 (1999), pp.251-258.

(2) Wu, C., Hao, H., and Zhou, Y., “Statistical Properties of Bukit Timah Granite in Singapore,” Journal of Testing and Evaluation, JTEVA, Vol. 28, January 2000, pp. 36-43.

(3). Mankin, C. J. and Johnson, K. S., “Oklahoma Geology Notes”, Oklahoma Geological Survey, Norman, Oklahoma.

*Note: psi = pounds per square inch
pcf = pounds per cubic foot

Figure 1. Photo of CME 75 rig used for coring operations.

Figure 2. Photo of a 5 foot section core being logged by the driller.

Figure 3. A section of core is being wrapped and boxed for transport.

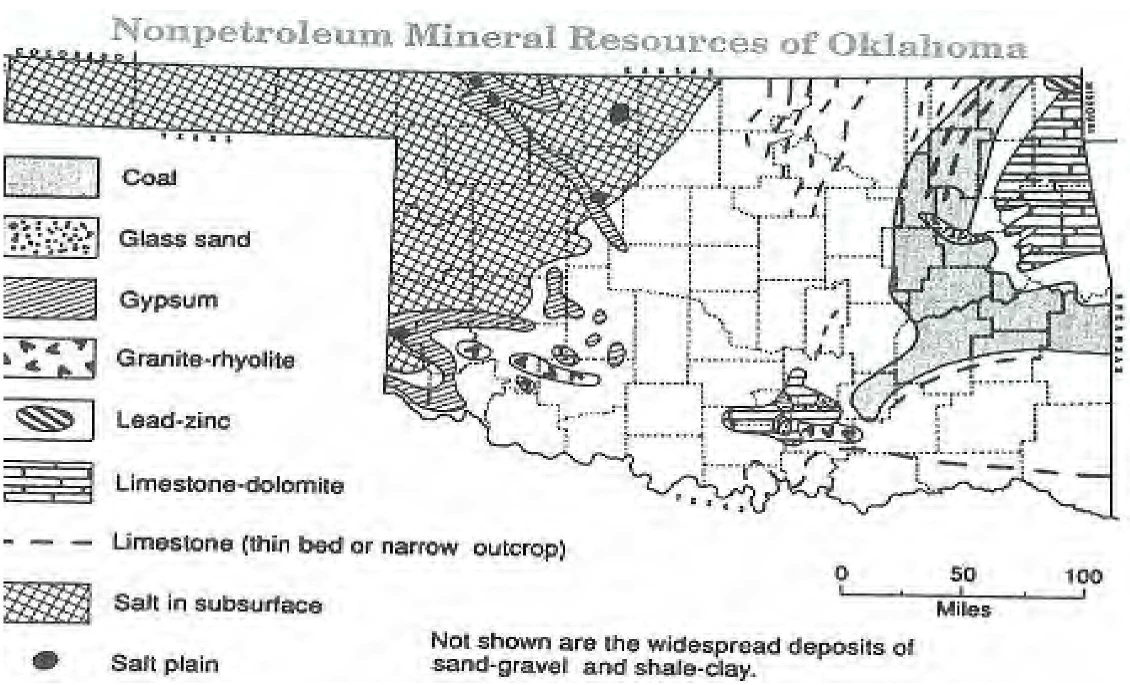


Figure 4. Map of nonpetroleum mineral resources of Oklahoma.

Figure 5. Prepared samples for testing.

Figure 6. Compression machine used for unconfined compression testing.

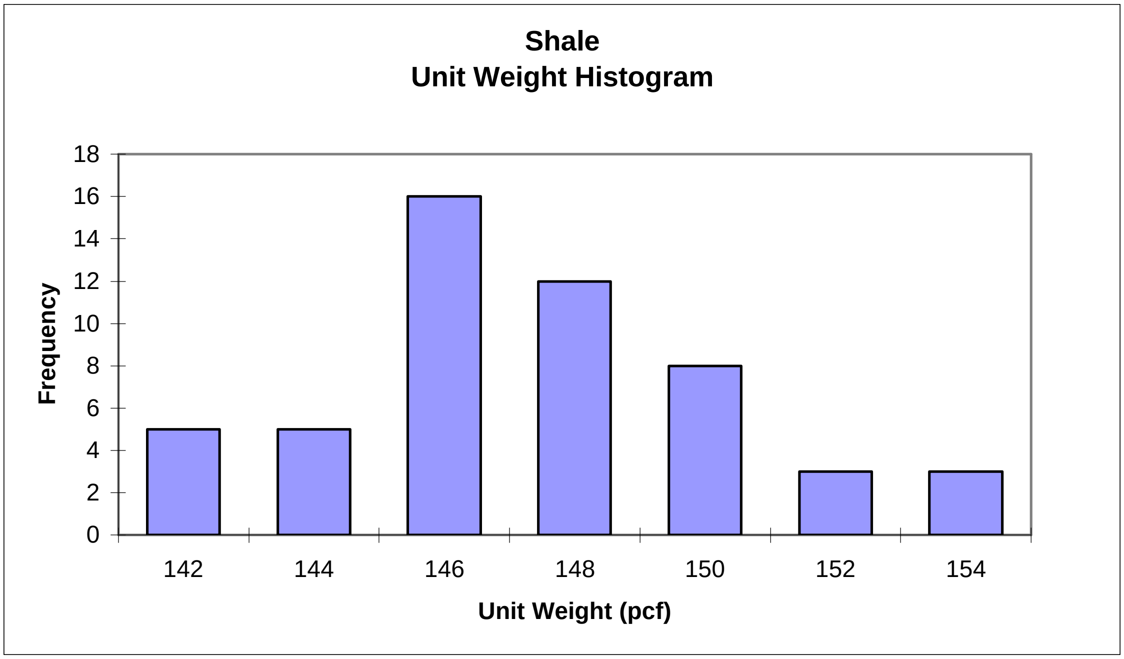


Figure 7. Shale Unit Weight Histogram.

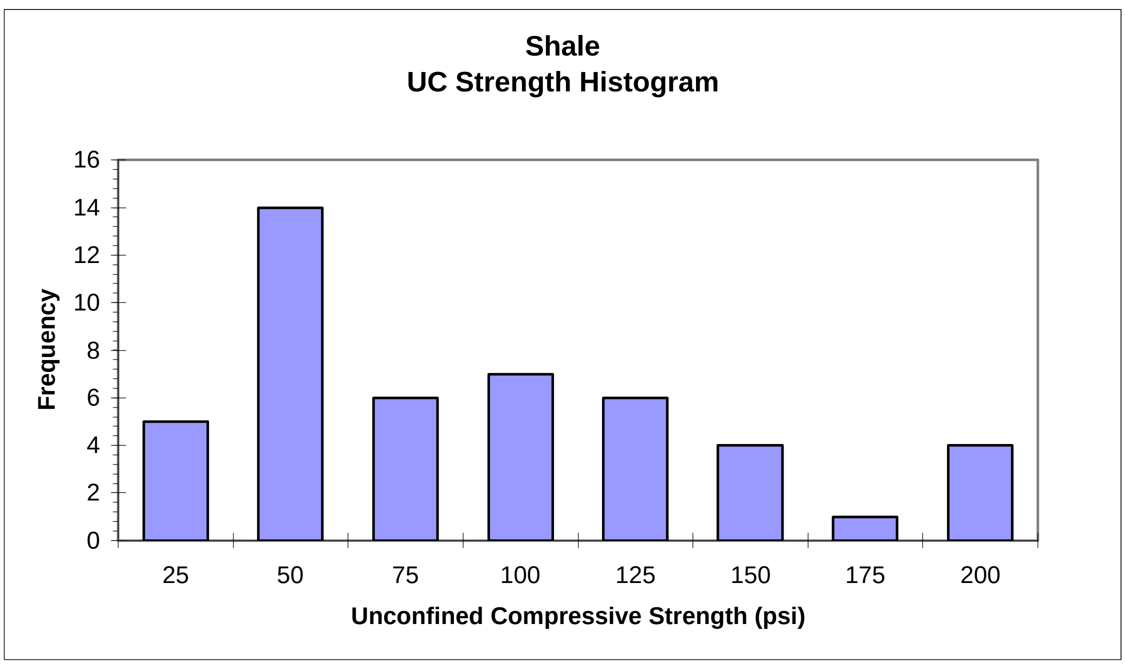


Figure 8. Shale Unconfined Compressive Strength Histogram.

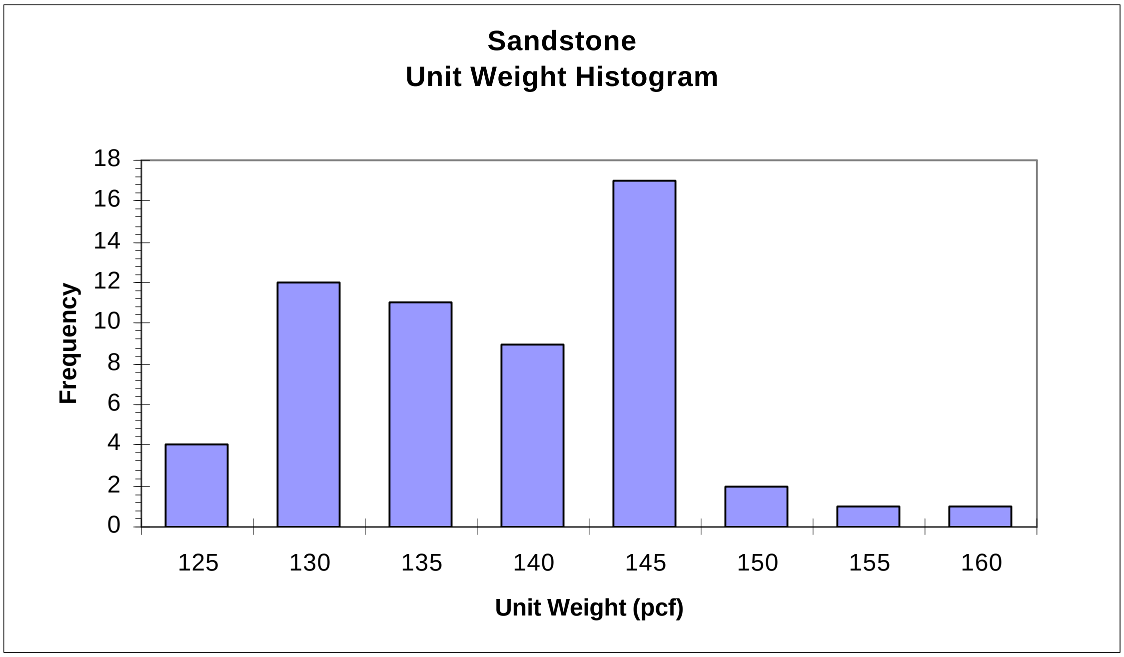


Figure 9. Sandstone Unit Weight Histogram.

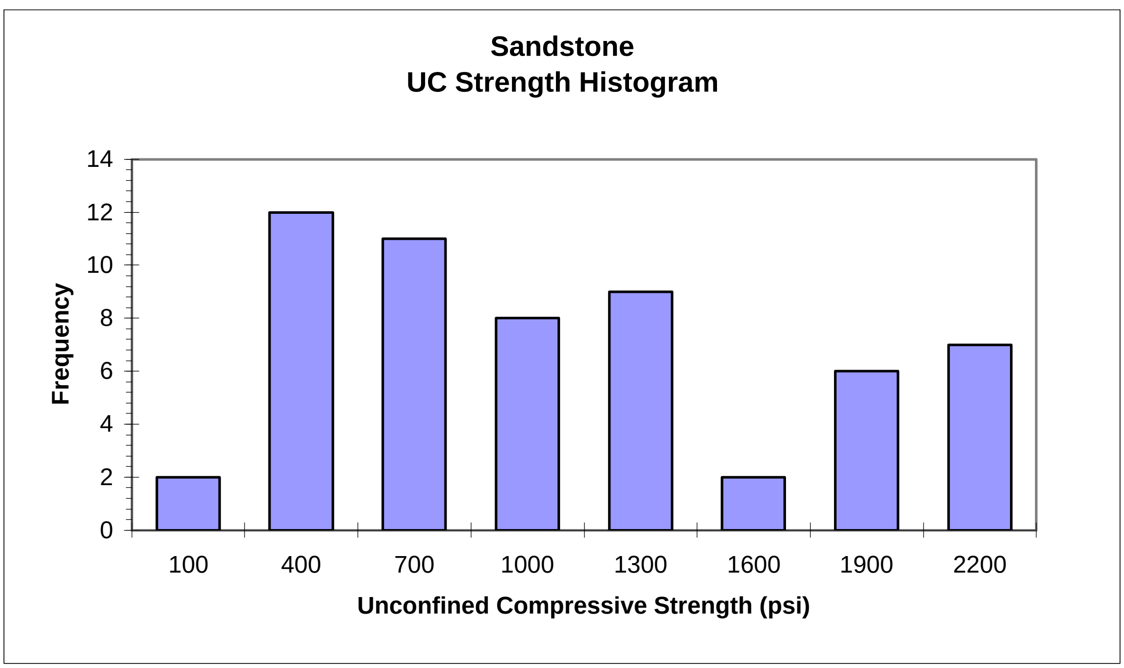


Figure 10. Sandstone Unconfined Compressive Strength Histogram.

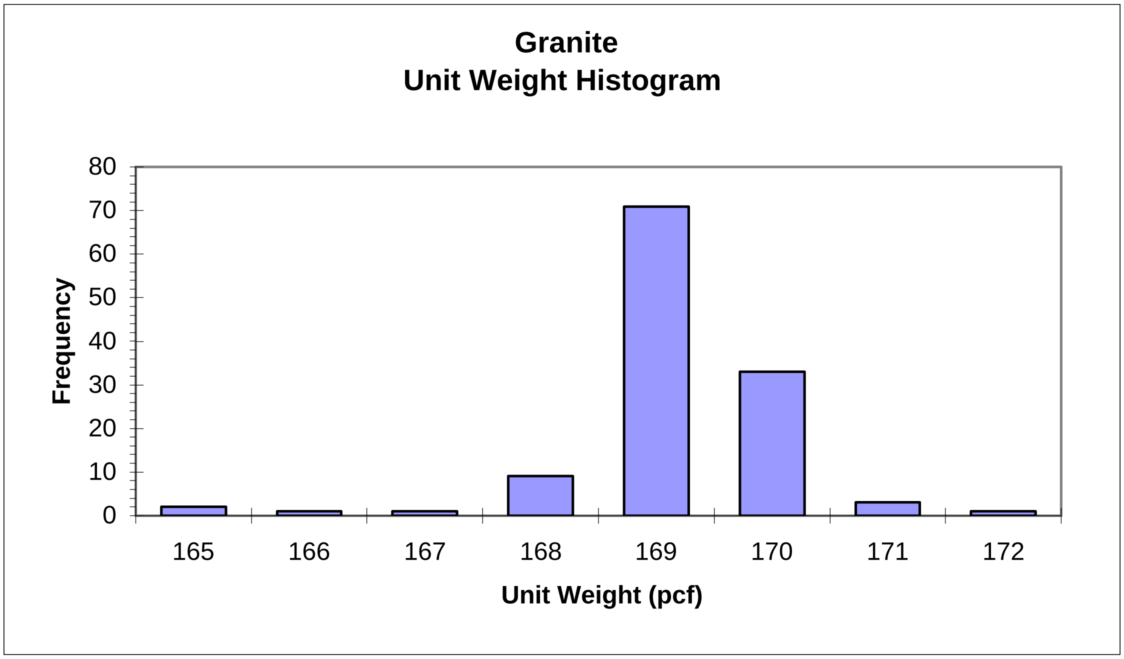


Figure 11. Granite Unit Weight Histogram.

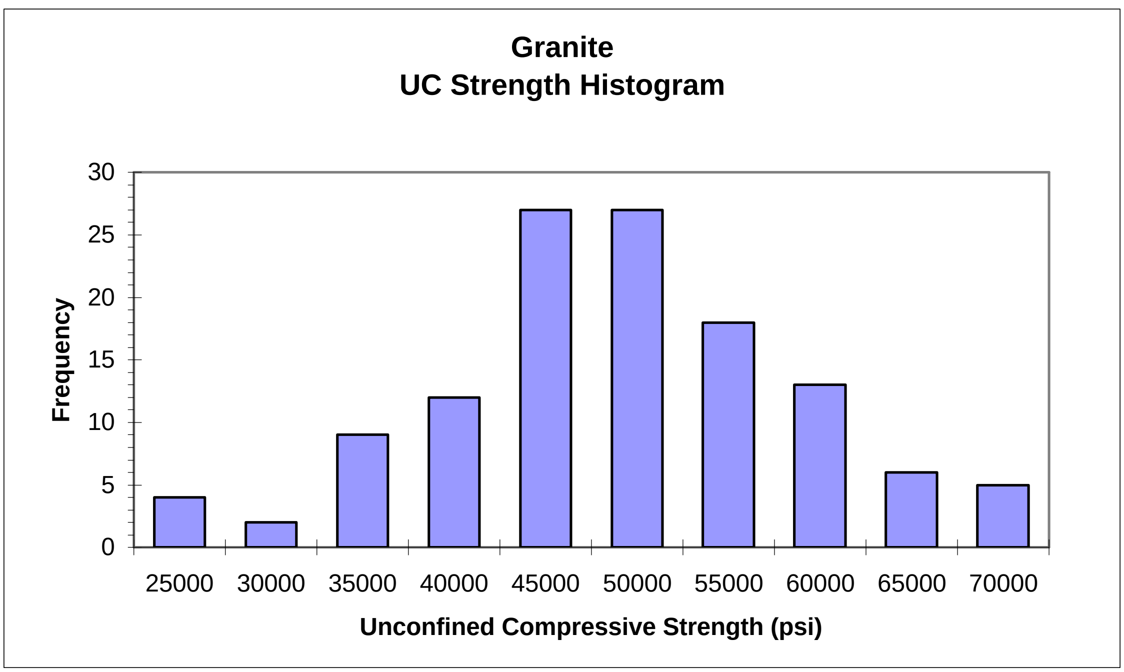


Figure 12. Granite Unconfined Compressive Strength Histogram.

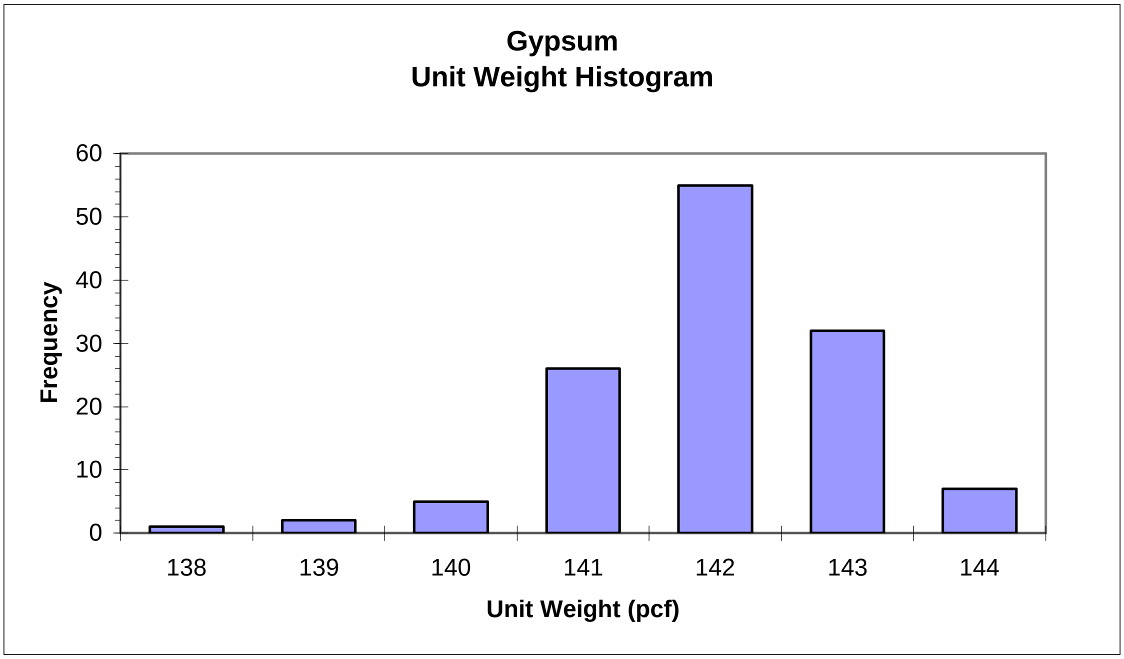


Figure 13. Gypsum Unit Weight Histogram.

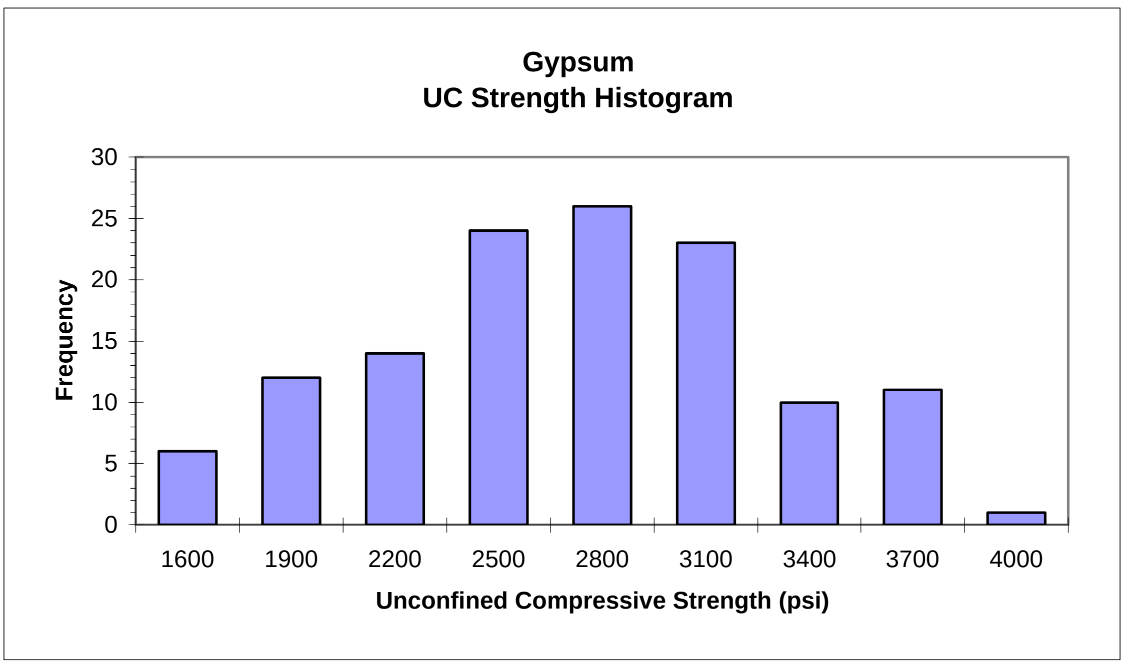


Figure 14. Gypsum Unconfined Compressive Strength Histogram.

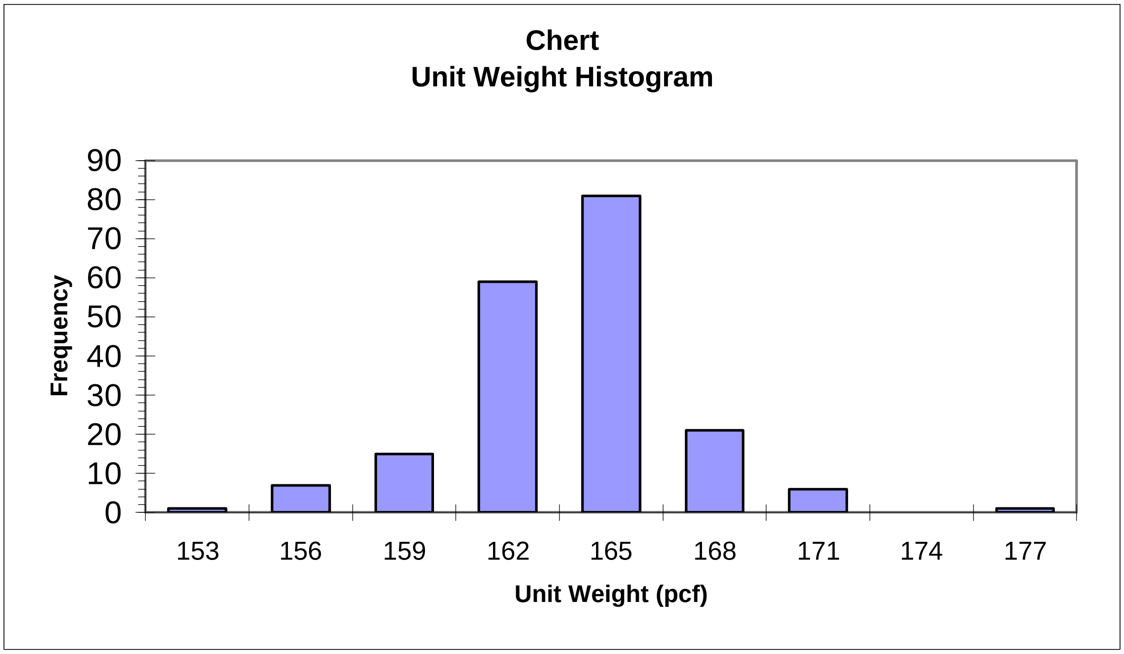


Figure 15. Chert Unit Weight Histogram.

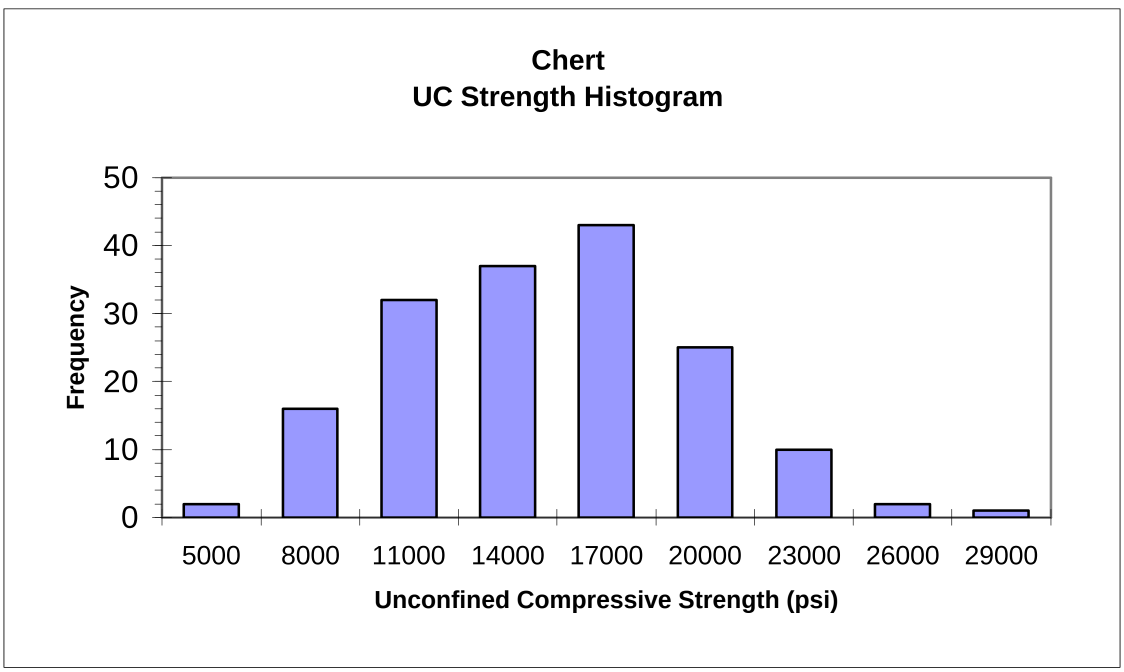


Figure 16. Chert Unconfined Compressive Strength Histogram.

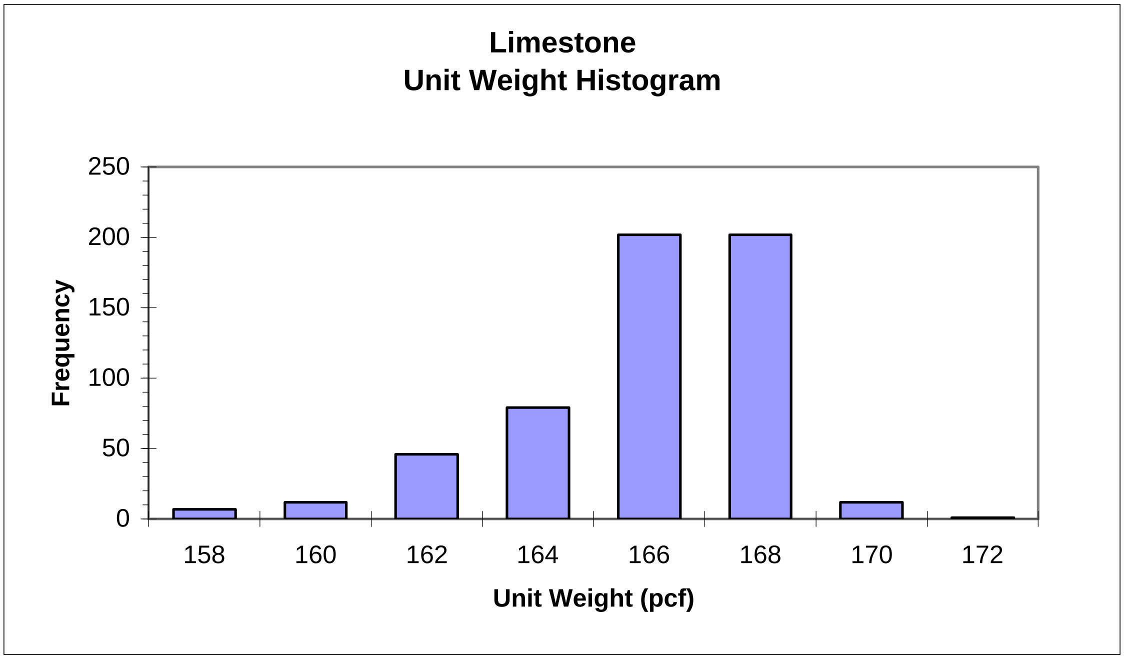


Figure 17. Limestone Unit Weight Histogram.

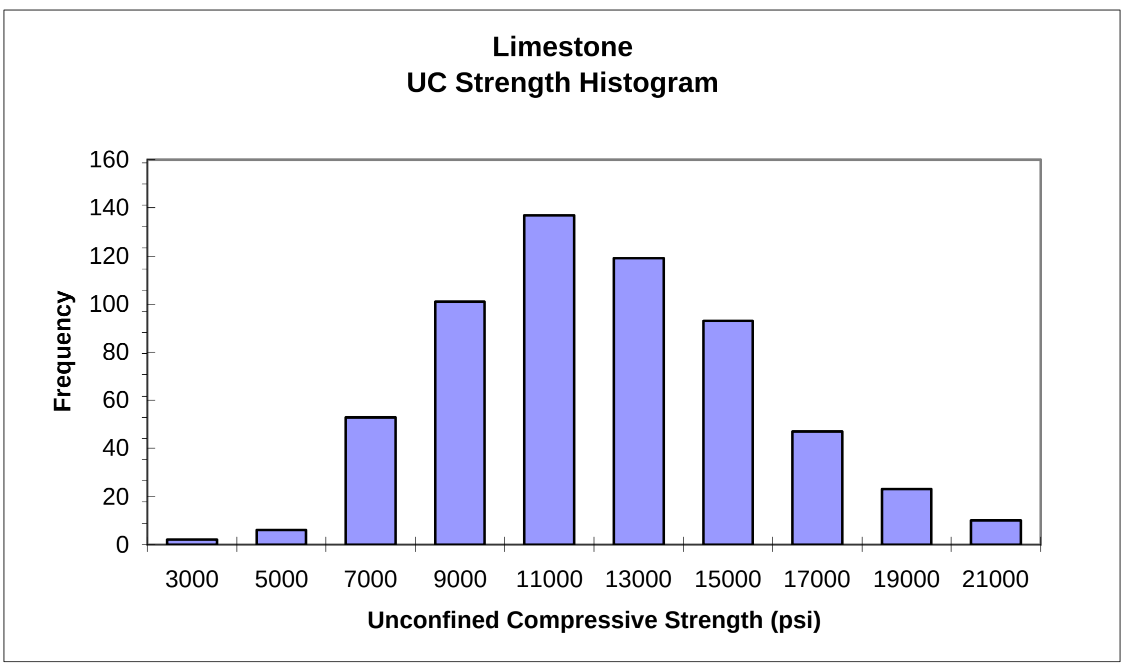


Figure 18. Limestone Unconfined Compressive Strength Histogram.

Table 1. Averages and Standard Deviations of data.

		Unconfined Compressive Strength			Unit Weight	
	Number of Samples	**Average (psi)**	**Standard Deviation (psi)**		**Average (pcf)**	**Standard Deviation (pcf)**
Shale	52	101	88		146.39	3.11
Sandstone	59	1005	703		140.94	7.65
Granite	122	46568	10029		168.68	0.86
Gypsum	128	2586	581		141.46	1.02
Chert	193	13822	4597		162.26	3.34
Limestone	594	11271	3481		164.96	2.25

Maryland's Experience with Large Scale Grouting for Roadway Stabilization in Karst Terrain of I-70

Larry R.Bolt. P.G., Engineering Geologist, Engineering Geology Division, Maryland State Highway Administration, Brooklandville, MD, 21022

James L. Stuby, M.Sc., Project Geophysicist, Earth Resources Technology, Inc., Jessup, MD, 20794

Dr. Peter H. Li, P.G., Principal, Earth Resources Technology, Inc., Jessup, MD, 20794

A.David Martin, C.P.G., Chief, Engineering Geology Division, Maryland State Highway Administration, Brooklandville, MD, 21022

ABSTRACT

I-70 near Frederick, Maryland, has a history of sinkholes and dolines developing in soils overlying folded and fractured limestone. For its reconstruction, geophysical methods were utilized to select zones for stabilizing by pressure grouting along the soil-rock contact.

Grout holes were drilled on 12 ft. centers through the soil and epikarst into 5 ft. of limestone. Grouting started at the top of rock and continued to the surface. Grout was produced by an on-site plant to achieve minimum time between drilling and injection.

Only roadway areas were grouted, and this was accomplished early, prior to main construction. Approximately 2200 borings, consuming over 10,000 cubic yards of grout were completed. Daily drilling and grout quantity data were plotted by hand to insure quality control, to design secondary borings, and to alter the limits of the grout zones. The depth-to-rock and grout volume data were later analyzed and compared to the geophysical survey using 3D software. Evaluation is still in progress, but based on past performance of smaller projects, the method appears to achieve the result of roadway stabilization at an efficient cost.

INTRODUCTION

The city of Frederick is a rapidly growing urban area in central Maryland. Interstates 70 and 270 that converge in Frederick are high volume traffic corridors leading to Baltimore, Washington D.C. and westward to I-81 and the central U.S. The I-70 alignment is roughly that of US 40, which was patterned after the 1806 Baltimore National Pike. Reconstruction of I-70 adjacent to the city of Frederick is necessary due to the increased traffic demands from regional growth and commuting pressure. Unfortunately, the area is underlain by a karst system whose activity has been accelerated by ground water changes brought on by urbanization and quarrying operations. Reconstruction of the I-70 and MD 85 interchange with ramps through Frederick is proceeding currently in Phase 2 A. Karst remediation is being addressed in Phases 2A, 2B and 2C. A proactive approach of pressure grouting and conventional filling and capping of sinkholes was completed prior to the major construction activity of Phase 2A. Major surface water capture, ponding and redirection along with additional grouting are planned primarily for Phase 2B. Phase 2A and Phase 2B are probably the largest grouting projects ever undertaken for highway construction. Approximately 10,000 cubic yards of grout were injected into over 2200 borings in Phase 2A.

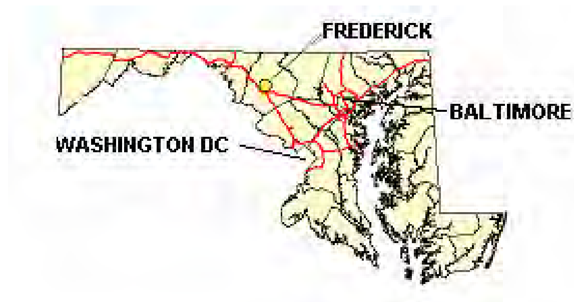


Figure 1. I-70 Project Location Map

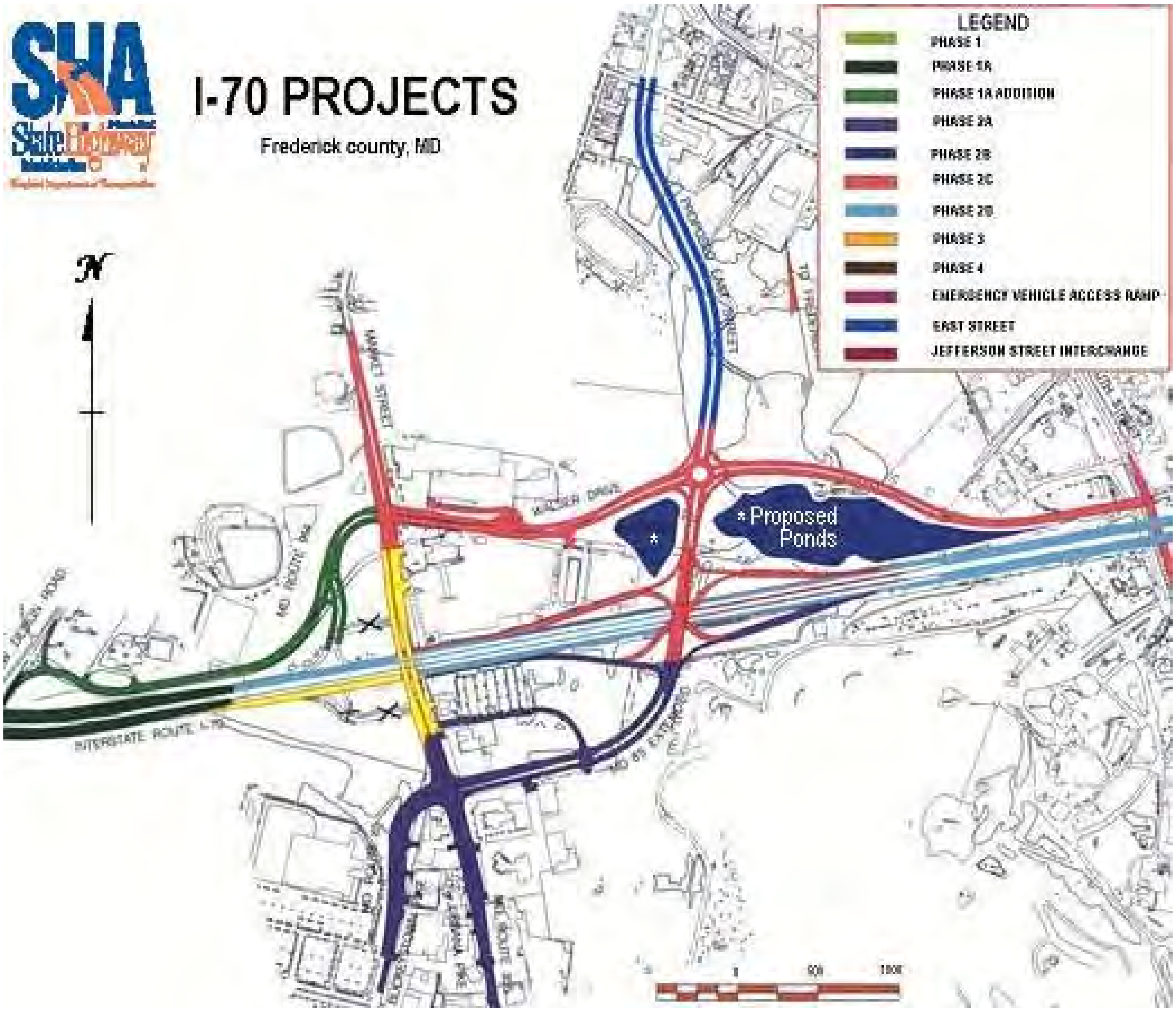


Figure 2. I-70 Project Phases

HISTORY

The alignment of I 70 roughly follows that of US 40 which was patterned after the 1806 Baltimore National Pike. In the area of the city of Frederick, MD, the interstate was built in the mid 1950s, and passes to the south of the city. At that time, the current standards for storm water management were not employed, and the risks of sinkholes were not seriously considered. As part of the growth, an adjacent quarry reached 300’ in depth in 2001. The quarry is dewatered at the rate of over 1,000,000 gallons per day.

In 1985, the Maryland State Highway Administration (SHA) initiated planning studies for interchange improvements and widening of I-70 through the Frederick area. Particular attention was given to the issues of storm water management and the risks of sinkholes. The studies determined that in this area, no runoff left the area as surface water. Numerous sinkholes were found. The planning studies were shelved from 1985 to 1995. During that period, the formation of sinkholes throughout the Frederick valley accelerated.

In 1994, a motorist's fatality, caused by a sinkhole in another part of the state, resulted in a heightened awareness of the risks posed by sinkholes and the SHA adopted a more aggressive policy to prevent further sinkhole related accidents.

The policy included the following:

1. An inventory of roadways listed by mile posts in the carbonate areas
2. High quality aerial photographs of all roadways in carbonate areas
3. Sinkhole awareness training for maintenance forces
4. Careful control of drainage for new projects in carbonate areas
5. An open end contract for subsurface grouting of sinks
6. An open end contract for geophysical surveys
7. An agreement with the Maryland Geological Survey for up to date geological mapping.

Construction projects include items for the lining of all drainage ditches and ponds. Existing sinkholes are excavated and backfilled when possible. Sinkholes, and incipient sinks, that are too deep for normal excavation methods and equipment to address are treated by pressure grouting through a grid of grout holes

Although the Frederick area has always been the location of sinkholes, the frequency of their occurrence has significantly increased during the last decade. In 1995 a pavement depression developed in I-70 that was severe enough to require milling to maintain acceptable ride quality. A boring study indicated that while voids were not present, there were soft, saturated soils in deep cutters between the limestone pinnacles. These soils were apparently migrating out through the solution channels in the rock and the roadway was settling.

In the fall of 1995, 12 days after a fire hydrant was struck by an automobile, a 30' by 60' sink 18' deep opened on a city street adjacent to the on and off ramps of eastbound I-70. It was noted that the residual soils were unusually sandy and subject to rapid subsurface erosion. Following this event, in February of 1996, a 6' diameter sink opened on the side of a nearby embankment for I-70 that undermined the guardrail. This embankment area supported the area of depressed pavement described above. The same sandy soils were clearly evident. Knowing that the sandy soils could move fast enough to cause catastrophic pavement failure, SHA geologists recommended that emergency action in the form of pressure grouting be initiated immediately. The grouting was completed during the late winter and early spring of 1996. The grouting was successful in arresting the settlement, and the milled pavement surfaces were repaved during the normal construction period of that year. Senior management asked that a comprehensive study be made to determine if there were additional areas in the two mile stretch of highway that needed attention. A geophysical contract was awarded to Technos Inc.of Miami, Florida to run conductivity and micro gravity. A paper on this study is included in the proceedings of the 49th HGS.

Between 1996 and 2003, eight sinkholes along I-70 were treated by grouting. In 2003, the end of a regional drought with a record amount of precipitation resulted in dozens of sinkholes in a short period. Karst activity is so prevalent in the area that senior management agreed that a proactive grouting program was justified for the I-70 improvement projects.

GEOLOGY

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

The Frederick Valley, and the project area, is underlain by carbonate units of the Upper Cambrian Frederick Formation and the Upper Cambrian to Lower Ordovician Grove Formation. In the northern and western areas of the valley, but not in the project area, Triassic redbeds of the New Oxford Formation overlie the carbonate units.

The Frederick formation was divided into three members by David Brezinski of the Maryland Geologic Survey in the most recent study beginning in 2001. The Basal Unnamed Member is lithologically variable, but is predominately calcareous mudstone and dark gray limestone with interbeds of black calcareous shale. Thick beds of polymitic breccias thin drastically from west to east, indicating a steep depositional slope. The thickness is estimated at 7500 feet in the western valley to near 1000 feet just 7 miles to the east.

The Adamstown member is characterized by dark gray, thinly bedded limestone with shaley partings and thick polymitic breccias that may represent submarine slides at the distal edge of large submarine fans. The assumed thickness in the project area is approximately 1200 feet.

The Lime Kiln member is lithologically variable, but is characterized by thin bedded, dark gray limestone that grades upward into thicker bedded, medium gray grainstone and packstone. Most intervals have abundant sand grains. The upper 200 feet is marked by algal thrombolitic limely mudstone and stromatolitic beds. Within the project area, the thickness is estimated at 750 feet.

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

The carbonate rocks form a double plunging synclinorium that strikes north-northeast (average N 25-30E) and is overturned to the west. Triassic redbeds of the New Oxford Formation cover the carbonates to the north and northwest. The project area lies on the western limb of this structure and bedding dips gently to the southeast.

KARST FEATURES

The two most common karst features of the Frederick Valley are dolines and sinkholes. The dolines are topographically expressed as closed depressions that are variable in shape, but generally less than 300 feet in diameter and less than 10 feet in depth. Limestone in these areas is being slowly dissolutioned, but sinkholes are known to occur after large rain events. The active sinkholes are generally cover collapse type and are occurring at an increasing rate. Small pipes, less than 3 feet in diameter are common. However, the threat to highway safety is from larger openings that are up to 40 feet in diameter with throats over thirty feet deep. They also occur suddenly and without warning.

The active sinkholes develop from groundwater movement at depth that unravels material in the bottom of deep cutters, usually with no indication on the surface prior to catastrophic collapse. Sinkholes also occur from surface water, either flowing or standing, that finds openings into near surface voids. The development of this type of sinkhole shows a tendency to occur along fracture traces and in colluvial soils that appear to be remnants of paleo-drainage systems.

The best model, based on drilling and geophysical surveys, indicates a rock-soil surface that has developed an epikarst zone that is highly variable in its nature. Some areas demonstrate a moderately dissolutioned rock surface while other areas have deep cutters with erratic pinnacles and floaters. The extent of the epikarst development appears to be related to lithologic variation, contrasts in bedding thickness and tectonic preparation caused by bedding plane faulting and fracturing. Dr. Barry F. Beck (P.E. LaMoreaux & Associates) has studied the epikarst development in this area and presented papers at previous HGS symposiums. A perhaps unique feature of the karst development in the Frederick Valley is the abundance of sand that is weathered from the carbonate beds. The incohesive sand generally fills the cutters to great depth and can contribute to catastrophic failure.

Detailed mapping, which includes karst development, is currently being completed by David Brenzinski of the Maryland Geological Survey as part of mapping the Frederick Quadrangle. Brenzinski has found that karst development can be related specifically to the lower member of the Grove Formation and the Lime Kiln member of the Frederick Formation. Using a global positioning system, the existing sinkholes and areas of susceptibility will be plotted on a special map to evaluate the likelihood of future development.

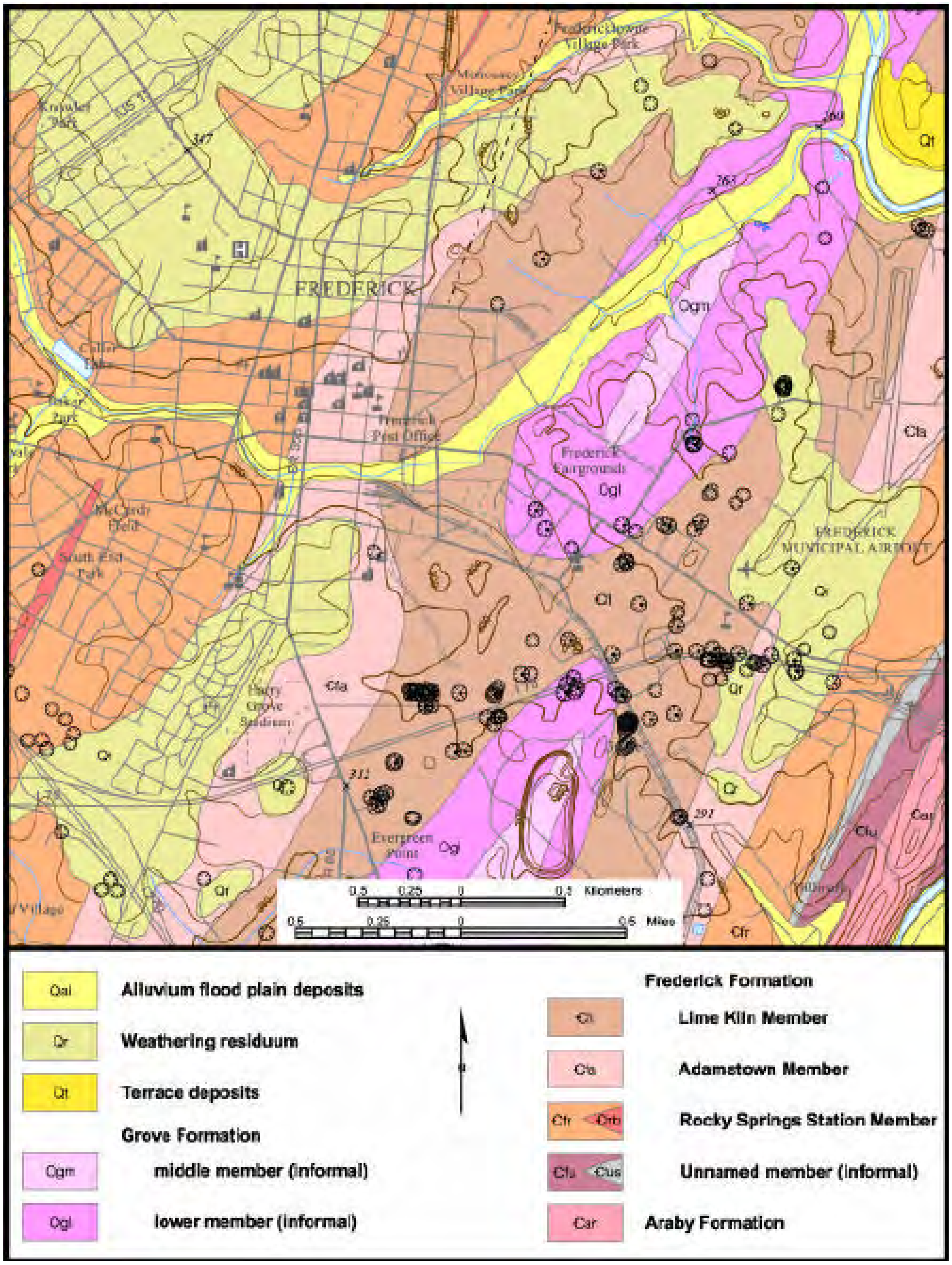


Figure 3, Geologic map with sinkhole locations (circles with hachures) for a part of the Frederick 7.5- minute quadrangle (Brezinski, in progress).

Figure 4. Typical cover collapse sinkhole in cut along. I-70, Frederick Valley

Figure 5. Catastrophic collapse of 30 ft. deep sinkhole. Note: No prior surface indication or water activity.

GEOPHYSICS

The purpose of the geophysical investigations carried out at the Phase 2A area along I-70 was to map areas of potential subsidence below the MD 85 right-of-ways and associated ramps in order for SHA to proceed with their grouting program for remediation. Earth Resources Technology, Inc. (ERT) was subcontracted to do the work. The primary method used to map potentially subsiding areas was resistivity tomography. Several other methods were used to collect supplemental data. Electromagnetic data were collected with the Geonics EM31 over all resistivity profiles to detect utilities and confirm shallow anomalies. Seismic refraction was attempted in one area in order to determine the bedrock profile and compare it with the resistivity data, but the data were prohibitively noisy due to surrounding highways. A magnetometer survey was carried out in the area of the quarry berms in order to detect subsurface metallic objects.

Resistivity tomography data are acquired in the form of two-dimensional profiles that display differences in the resistivity of subsurface materials such as the bedrock and the overburden. A minimum of three resistivity profiles was acquired along each ramp, and six were acquired along the MD Route 85 right-of-way. In addition, two profiles were acquired along the south side of I-70 within the project area. One profile was acquired over the access road.

A set of equipment manufactured by Advanced Geosciences, Inc. (AGI) was used for the resistivity tomography survey. AGI's Sting R1 Earth Resistivity Meter and the accompanying Swift relay box were used to control a set of up to 84 programmable electrodes (smart electrodes) arranged along up to six long cables. The dipole-dipole method was used in this investigation, in order to provide good horizontal resolution of features. The electrode spacing was either 5 or 10 feet, the *a*-spacing ranged 1 to 6 times the electrode spacing,and the *n*-spacing ranged from 1 to6.

Processing of the resistivity data was accomplished using a standard format recommended by the manufacturer of the equipment. Readings acquired in the field with errors greater than 10% were eliminated. The program RES2DINV was used to process the data by iterative inversion (inverse modeling) to produce a cross section of subsurface resistivity. First, the program was used to manually eliminate erratic data points from the data set. Second, topography was added to the data. Third, the data were iteratively inverted. Up to five inversions may be carried out on each data set, successively lowering the root mean square (RMS) error with each iteration. The program stops the inversion process when the relative difference between RMS errors in successive iterations becomes less than 5%. RMS errors on the iterations used to create model sections averaged 17.0%, and ranged from 6.1% to 31.2%. The program Surfer was then used to contour the resistivity model data using the method of triangulation with linear interpolation.

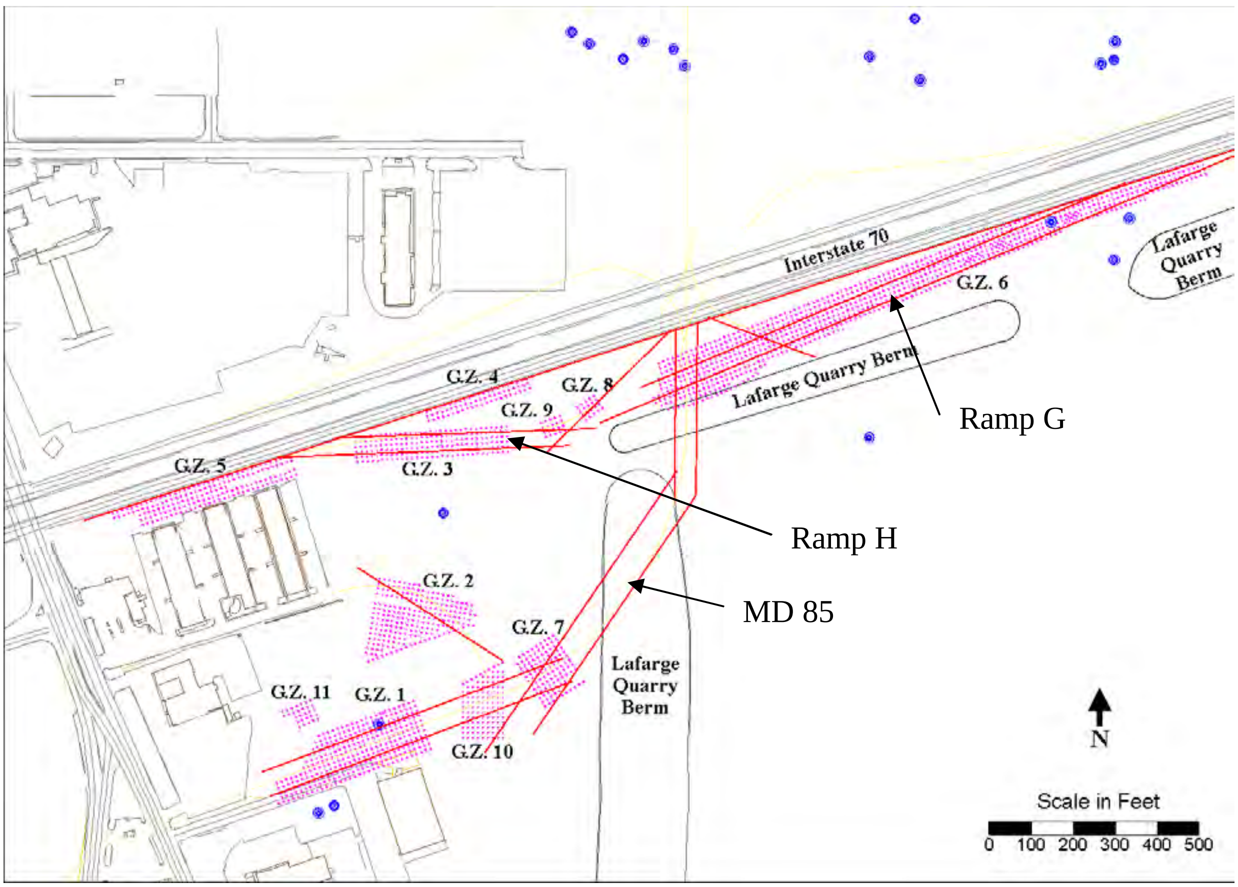


Figure 6: Phase 2A area showing I-70, centerlines of proposed right-of-ways (yellow), resistivity profile locations (red), grout holes (magenta dots) and grout zone numbers (e.g., "G.Z. 1"), and sinkholes at the time of the project (blue dots).

The model sections (profiles) were used to create a map of resistivity anomalies. This map was in turn used to outline zones for remediation by grouting prior to construction of the new roadway. A total of ten zones were proposed by ERT , and one was later added by SHA. The project area (prior to construction) is shown in Figure 5, along with the locations of resistivity profiles (red lines), the locations of grout holes (magenta dots), and existing sinkholes at the time of the geophysical survey (blue dots).

After grouting was completed, borehole logs and grout take records were provided to ERT, who then compiled this data and contoured the results. Orthographic perspective drawings of grout volume take and bedrock elevation are shown for Grout Zone 1 and Grout Zone 3 in Figure 7. To obtain the bedrock elevation contours, 5 feet was subtracted from the total depth of each hole to obtain the depth to rock. The approximate elevation of the top of the hole was obtained from a high-resolution topographic contour map provided to ERT by Whitman, Requardt, & Associates for use in the original geophysical investigation (the grout holes were not surveyed). Depth to rock was subtracted from ground elevation to obtain bedrock elevation. “Floaters,” or sections of solid rock that are less than 5 feet in thickness within a borehole, are not accounted for in any of the bedrock elevation maps. Nor are voids. The contour maps represent the upper surface of bedrock that is 5 feet or more in thickness only.

Figures 8 and 9 compare the bedrock contour maps of grout zones 1 and 3 with resistivity profiles crossing those zones. The resistivity profiles are described fully in the Phase 2A report by ERT (2000). Only segments of the profiles crossing the zones are shown in the figures. “Slices” through the bedrock contour maps were generated digitally and plotted as cross sections of distance versus elevation. These cross sections were superimposed over the resistivity profile segments for direct comparison. Curving black lines on the resistivity profiles show the bedrock elevation. Again, floaters and voids are not accounted for.

The results of comparison of bedrock elevations with resistivity profiles are interesting. Rarely is this much “ground truth” provided to geophysicists. There is some correspondence between resistivity contours and bedrock elevation contours, although the correspondence varies. There is good correspondence on the profiles presented in Figures 8 and 9, for example, where bedrock lows correspond to resistivity lows near the surface, and the bedrock profile is roughly parallel with the 200 to 500 Ohm-meter (Om) contours.

Dunscomb and Rehwoldt (1999) published a paper stating that generally the bedrock surface occurs between the values of 150 and 600 Om on resistivity profiles. The values corresponding to the bedrock surface as defined in these maps show a wider range than this. In isolated areas (on profiles not published here) the bedrock contour passes through model resistivity values as high as ~8000 Om or as low as ~32 Om. Generally, however, the bedrock surface passes between about 2000 Om and 64 Om. It may be that the contrast in ranges can be accounted for by the fact that Dunscomb and Rehwoldt had better control on the position of boreholes relative to their resistivity profiles.

Many factors influence the modeled resistivity values seen on the profiles. First, the data acquired by resistivity equipment can be affected by ground conditions to the sides of the line of electrodes.

These conditions are called “out of plane effects.” For example, a low-resistivity cutter could be parallel to a high-resistivity pinnacle directly under the line of electrodes, and cause the resistivity values on the resulting profile to be lower than they would have been if the cutter had not been there. Also, the water content of the soil or rock, the degree of fracturing or weathering in the rock, or changes in the lithology (such as quartz grain content) of the rock surface can all contribute to the observed resistivity. Similarly, errors during acquisition and differences in processing can affect final modeled resistivity values such that the results obtained by ERT may differ from those of other companies acquiring similar data.

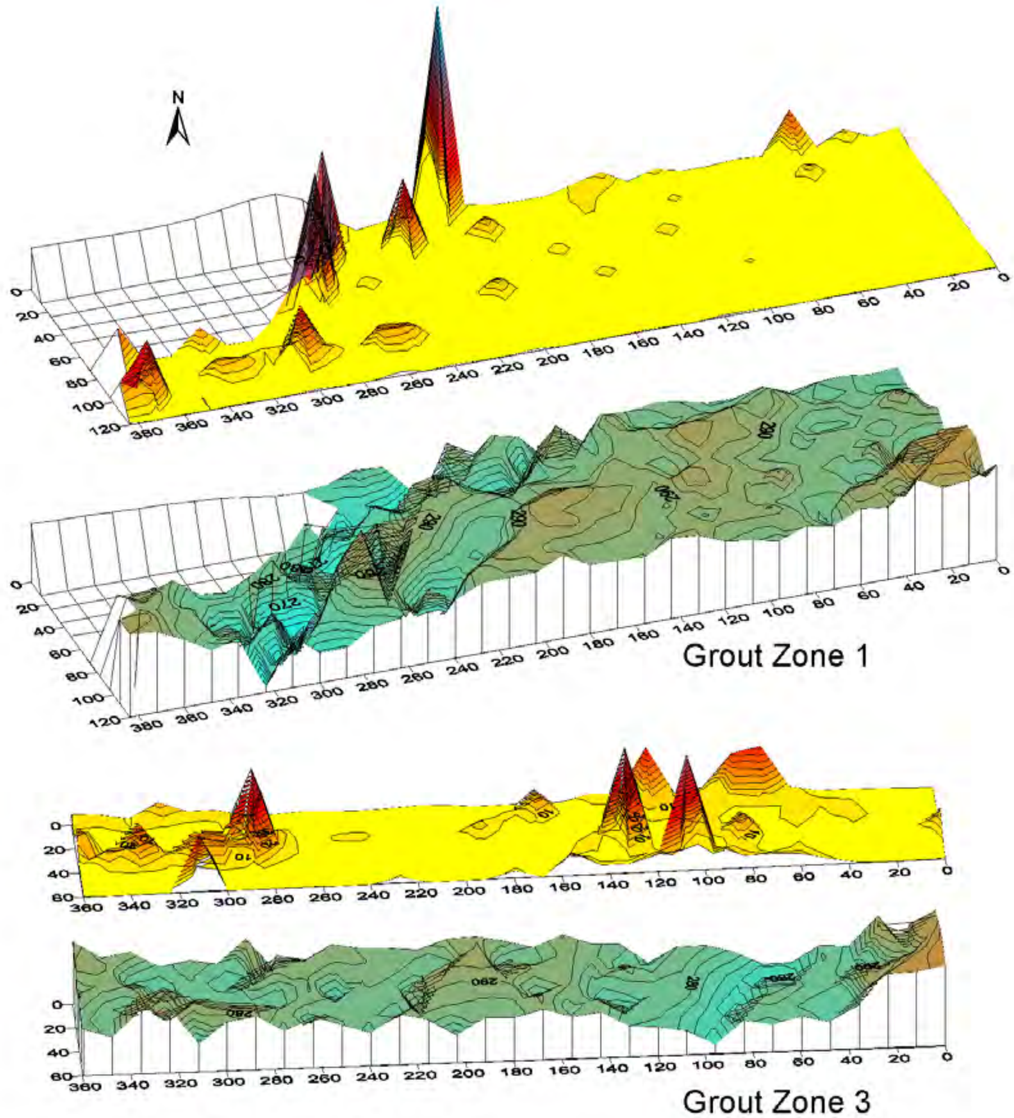


Figure 7: Grout Volume shown above Bedrock Elevation perspective drawings of Grout Zone 1 and Grout Zone 3, south of I-70, Frederick, Maryland. Orthographic perspective, 30 deg. tilt, with North at the top. Scale: 1 inch = 80 feet, except the vertical axis of the grout maps which is in cubic yards of grout injected. No vertical exaggeration in bedrock drawings. Note how largest grout takes roughly correspond to bedrock lows, or cutters.

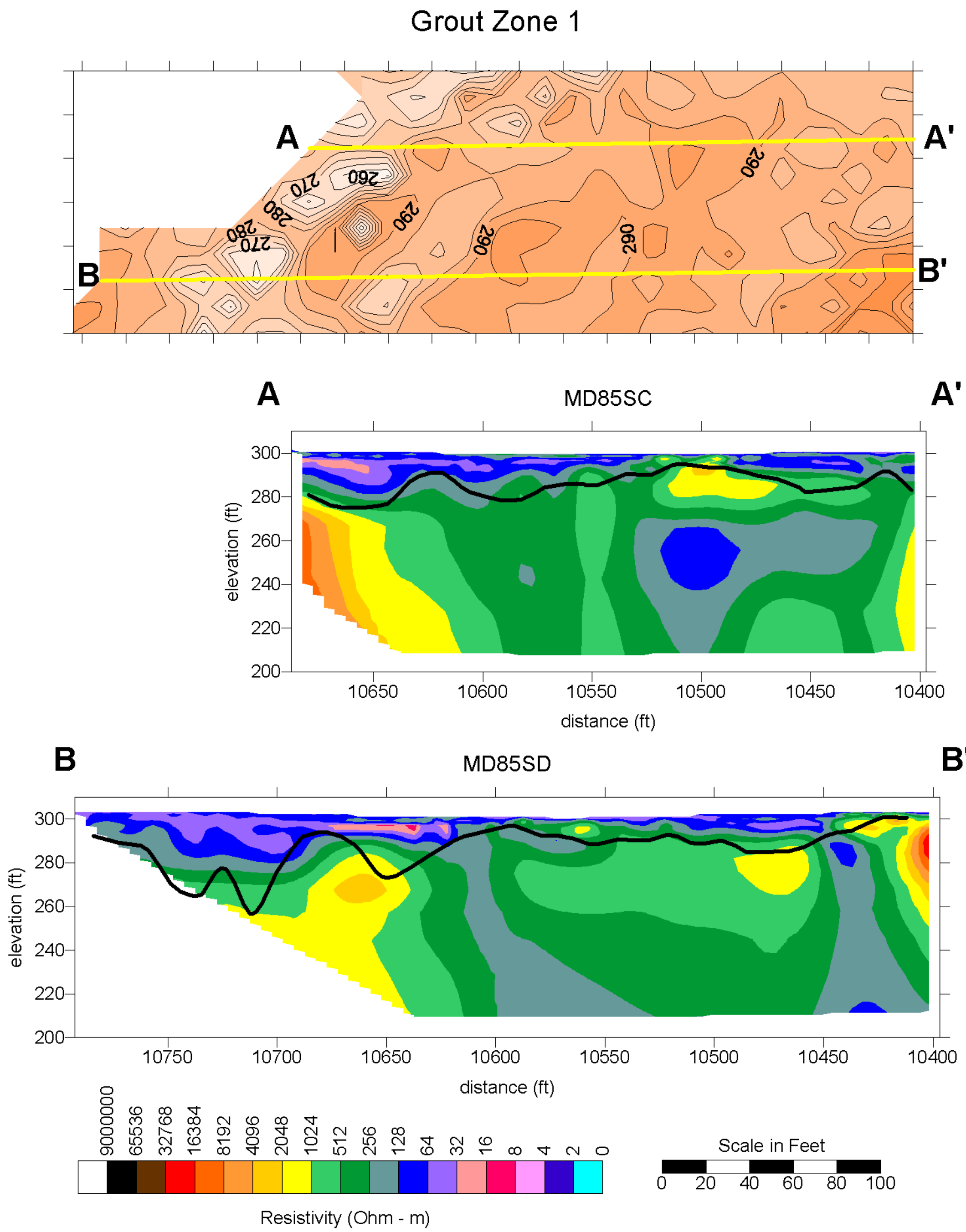


Figure 8: Comparison of Bedrock Topography with Resistivity Tomography Model Sections, Grout Zone 1, south of I-70, Frederick, Maryland. Black lines on resistivity profiles are bedrock surface representing 5 feet of penetration by drilling. Profiles shown with elevation on left axis, distance on bottom axis, and name on top. Scale: 1 inch = 80 feet

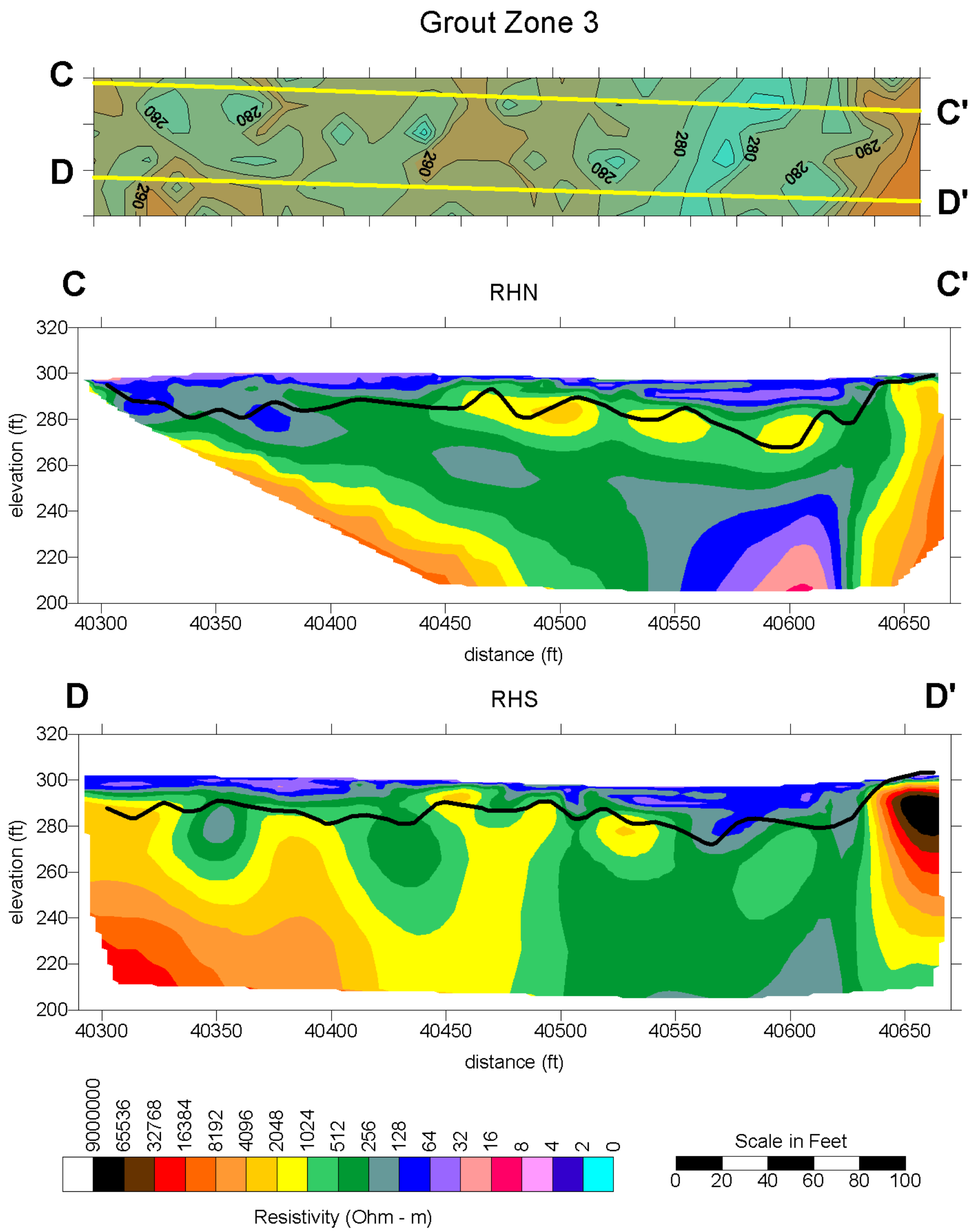


Figure 9: Comparison of Bedrock Topography with Resistivity Tomography Model Sections, Grout Zone 3, south of I-70, Frederick, Maryland. Black lines on resistivity profiles are bedrock surface representing 5 feet of penetration by drilling. Profiles shown with elevation on left axis, distance on bottom axis, and name on top. Scale: 1 inch = 80 feet

GROUTING

Ten grouting zones were developed as part of Phase 2A. The work was done in the fall of 2002 and completed prior to the start of main highway construction. Ten zones were defined based primarily on interpretation of the results of the resistivity tomography surveys. One additional zone was added as part of an agreement with an adjacent landowner. Grouting was done on a 12 foot grid system within each zone. The borings were accomplished by truck mounted drills and in special cases by small crawler type rock drills where overhead clearance beneath power lines was an issue. The borings were advanced through the residual soils and an additional 5 feet into fresh rock. Fresh rock was defined as hard unweathered limestone as judged by gray cuttings. The borings averaged approximately 30 feet depth with maximum depths to 90 feet. Maryland was experiencing a drought at the time and the project is within the Zone of Influence of an adjacent active quarry. Water in the borings was not a problem, but the soils were generally moist, particularly in the areas of well developed or deep epikarst. In no case did the grouting take place below the water table.

Grout was mixed by an on site plant. Distribution was by pumping through a 5 inch pipeline and by a conventional concrete delivery truck. The grout was composed of Portland cement, number10 limestone screenings, fly ash, and water. This was a lean mix that was specified to have 100 psi at 28 days and cylinders were made on a daily basis. The fly ash acted as a lubricant and allowed pumping to borings at 500 feet distance. No special additives were necessary. Grout quantities were calculated by calibration of the pumping cylinders and counting the number of extrusions. Radio contact was maintained between the pump operator and the grouter at the boring site to measure and record grout takes for each boring.

Grout was injected into the borings using a 3.25 inch diameter mandrel or pipe suspended from a tracked crane. The mandrel was lowered to the bottom of the boring and grout injected. A pressure of 300 psi was specified at the bottom of the boring and was gradually reduced as the mandrel was withdrawn to the surface. A hydraulic pressure gauge was installed in the grout line approximately 100 feet back of the mandrel connection. This setup did not function well because the gauge frequently failed, was difficult to observe by the crane operator and the pressure reading was thought not to represent the true down hole pressure at the end of the mandrel. The grouting was usually done immediately behind the drill to minimize the possibility of caving. Two drills and 2 grouting cranes worked a standard hour shift. The work required 90 days to completion and resulted in approximately 10,000 cubic yards of grout injected into 2200 borings in ten zones.

Grout takes ranged from small amounts that represent bore hole filling to over 300 cubic yards. Communication between borings was common and the borings that took large amounts generally were surrounded by borings that required no grout or small amounts of grout. This was apparently due to grout filling both true soil voids and dissolution channels in the rock that were greater in length than the 12 foot boring spacing.

Existing open sinkholes were conventionally filled and covered with a 2 foot thick re-enforced concrete slab if they were located beneath the pavement or ditch line. The extent of the slab extended to 5 feet beyond the apparent edge of the sinkhole.

A number of terms have been specifically used to describe the different methods and types of grouting. None seem to exactly describe what was done in Frederick. We used the term "cap grouting", but that may not be acceptable to all grout specialists. The intent was to produce a layer or cap of grout between the rock and soil and seal this zone to prevent or slow karst development. We believe that this was accomplished in most of the area by cap grouting, but there are certainly many examples of void and cavity filling and also some pressure grouting cases. In general, pressure grouting was restricted to limited areas of soft clays or material above its optimum moisture content. As noted previously, the cutters were commonly filled with sandy clay. This material is virtually incompressible when dry or slightly moist. No check areas were excavated to confirm this due to the depth of the grouting, but observations from other examples in the Frederick Valley confirm this possibility.

DISCUSSION

Grouting was used in this project for several reasons. Maryland has had success with grouting for over eight years in cases where grouting was the only option due to high traffic volume and closing the roadway to excavate was an issue. In the case of Frederick, grouting was the method of choice because the area to be remediated was large, the bedrock was deep in many places and the need to complete the work and not interfere with the main construction schedule was an important issue

Experience from this project will be used to better prepare for the next phases of the project north of I-70 where the karst development is deeper and more advanced. Grouting is the method of choice there and projected grout quantities will be in the range of 20,000 cubic yards. In Maryland, special provisions are written for each contract and are not standard. Problems encountered in Phase 2A will be addressed by new special provisions. There were no major problems encountered, but several issues will need to be addressed.

The decision to drill 5 feet into bedrock will be changed because there were cases of floaters that were thicker than 5 foot. In Phase 2A, the problem was addressed by secondary borings. An additional pay item for drill footage beyond 5 feet in suspected cases would be more cost effective and time saving.

Perhaps the most difficult problem was the matter of grout pressure control. A specified pressure of 300 psi at the soil line was desirable, but could not be verified. The gauge set up was not adequate or effective. A method of calculating bottom pressure can perhaps be accomplished by considering the weight of the grout in the pipe line and mandrel and develop a chart that the crane operator could read in the cab. This problem has yet to be solved.

Boring and grout data was plotted at the end of each day by the contractor and by the SHA geologist. The amount of data from two grouting crews is substantial and the time available from the staff was limited by requirements from other projects. Boring logs were converted to geologic logs and along with grout takes were hand plotted on cross sections to help guide future grouting. A software program has been purchased and immediate input in the field on a lap top will be utilized on future projects.

Grout quality control was an issue. Test cylinders of grout were taken daily for 7 and 28 day breaks to evaluate the required 100 psi strength. Obviously this is not the most effective control procedure. A situation did occur when one set of cylinders failed and the fact was not known until the contractor had left the job. This required additional borings to evaluate the grout integrity.

CONCLUSION

The selection of grouting as the proactive approach for I-70 was based on the high success rate of its use on maintenance projects during the past eight years. Twenty projects have been completed and only one small area of one site has required re-grouting. The use of geophysics to manage the grouting program is effective when a karst model and geology are utilized in an overall approach. This project illustrates the need for a method to track the distribution of the grout and for more accurate estimation of the required quantities. A detailed cost comparison with other methods has not been done, but from generalized studies, grouting compares favorably with commonly available methods.

REFERENCES

Beck, Barry F., 2001, Sinkhole Risk Assessment for the Proposed Interchange and the I-70 Improvement near Frederick, Maryland. Report submitted to Maryland State Highway Administration from P.E.LaMoreaux and Associates, Inc.

Boyer, Bruce W., 1997, Sinkholes, Soils, Fractures, and Drainage: Interstate 70 Near Frederick Maryland. Environmental & Engineering Geosience, vol.3, no. 4, pp. 469 – 485.

Brezinski, David K., 2001, Interim Report on the Geology of the Frederick Valley and its Relationship to Karst Development. Maryland Geological Survey. In progress.

Duigon, M.T., and Dine, J. T., 1987. Water Resources of Frederick County, Maryland. Maryland Geological Survey Bulletin 33, 106 p.

Dunscomb, Mark H., and Rehwoldt, Eric, 1999, Two-dimensional resistivity profiling; geophysical weapon of choice in karst terrain for engineering applications. In Beck, Petit, and Herring (eds.), Hydrogeology and Engineering Geology of Sinkholes and Karst, Balkema, Rotterdam.

Jonas, A. I. and Stose. G. W., 1938. Geologic Map of Frederick County, and Adjacent Parts of Washington and Carroll Counties. Maryland Geological Survey Geologic Map, scale 1: 62,500.

Li, Peter H. and Stuby, James L. 2000. Report of Geophysical Investigations: Interstate 70 and MD 85 Interchange, South Side, Frederick County, Maryland. Final Report submitted to Whitman, Requardt & Associates for Maryland State Highway Administration, Baltomore, MD.

Meyer, Gerald., 1958. The Ground-Water Resources in the Water-Resources of Carroll and Frederick Counties: Maryland Department of Geology, Mines and Water Resources Bulletin 22 p. 1 – 228.

Newton, J. G., 1987, Development of Sinkholes Resulting from Man's Activities in the Eastern United States: U.S.Geologic Survey, Circular 986, 54 p.

Nutter, L. J. and Otton, E. G. 1969. Ground-Water Occurrence in the Maryland Piedmont: Maryland Geological Survey, Report of Investigation no.10, 56 p.

Reinhardt, J., 1974. Stratigraphy, Sedimentology and Cambro-Ordovician Paleogeogrephy Of the Frederick Valley, Maryland. Maryland Geological Survey, Report of Investigations No. 23, 73 p.

Southworth, S., 1996. The Martic Fault In Maryland and it's Tectonic Setting in the Central Appalachians. Pp.205 – 221, In Brezinski, D. K. and Reger, J. P. eds., Studies in Maryland Geology – Commemoration of the Centennial of the Maryland Geological Survey, Maryland Geological Survey Special Publications 3, 286 p.

Technos, Inc. 1997 Assessment of Karst Conditions Along I-70 near the Town of Frederick, Maryland. Final report Submitted to Rummel Klepper & Kahl for Maryland State Highway Administration, Baltimore, Maryland. Paper presented at 49^{th} HGS, 1998.

Slope Design for Improvements to New Mexico State Highway 48, near Ruidoso, New Mexico

By Nancy C. Dessenberger, P.E., and Francis E. Harrison, P.E., Golder Associates Inc., Lakewood, Colorado, and Robert A. Meyers, P.E., New Mexico State Highway and Transportation Department

Abstract

Proposed widening of the road template for New Mexico Highway 48 requires enlargement of nine existing rock cuts and construction of eight Reinforced Steep Slope (RSS) fills over approximately 4 miles of highway through mountainous terrain. The geology of the area included gently dipping Cretaceous and Tertiary sediments intruded by Tertiary diorite bodies which are highly irregular in form, extent, and occurrence. The irregularity of the geology presents a particular challenge for design of cuts.

For rock cut design, both kinematic failure potential and global stability of the rock mass were factors in the selection of recommended cut slope angles. Hoek-Brown criteria, based on the estimated Rock Mass Ratings (RMR) for the dominant rock types, was used to estimate rock strength parameters for the analysis of global stability.

Rock fall analysis was completed for the rock cuts to support design of rock fall catchment. The CRSP model was used to determine the estimated rock fall retention, and compared with recommendations indicated by the FHWA/ODOT Rockfall Catchment Area Design Guide.

Most of the RSS fills, with face angles of 0.75H:1V, are to be constructed on existing fill slopes as steep as 1.5H:1V. The existing fill as a foundation for the RSS presents unusual conditions, particularly in terms of global stability and settlement.

Introduction

New Mexico State Highway 48 (NM 48) traverses steep terrain in southern New Mexico near Ruidoso. This paper addresses proposed improvements to the alignment between mileposts 9.03 and 13.19, between Capitan and Alto, New Mexico. Proposed improvements include widening, nominally on the existing alignment, with enlarged rock cuts and Reinforced Steep Slopes (RSS).

Golder Associates Inc. was assigned responsibility for geotechnical design of rock cuts and associated rock fall mitigation measures, and design of Reinforced Steep Slopes (RSS), as a subconsultant of Geotest, Inc. in Albuquerque, under an on-call services contract with the New Mexico State Highway and Transportation Department (NMSHTD). The work was led by the authors, Dessenberger and Harrison, under the technical direction Bob Meyers of NMSHTD, who is also a co-author. Golder's work was administered contractually by Mr. Charles Miller, of Geotest, Inc. Other work on the project referenced herein was performed by URS (direction of drilling investigation), CRUX Subsurface (geotechnical drilling), AMEC (geophysics), and NMSHTD (lab testing and geophysics) (see References).

This paper is presented as a case study of slope design issues and resolutions, with the intent to share methods and solutions to other practitioners. Rock slope and rock fall issues are discussed in the first major section, followed by RSS stability and design issues in the second.

Geologic Setting

The project area lies within sedimentary deposits of Cretaceous and Tertiary age that have been intruded by dikes and sills, probably of Tertiary age (Ash and Davis 1964). Most of the subject alignment lies within rocks of the Cretaceous Mesa Verde Group, although the north end of the project, north of the Rio Bonito, is characterized by rocks of the Tertiary Cub Mountain Formation. Within the project area, both of these formations are extensively intruded by Tertiary diorite dikes and sills.

The Mesa Verde Group includes interbedded sequences of marine and non-marine shales, claystones, siltstones, and sandstones. The most prominent sandstones in outcrop along the existing alignment are yellowish-gray to buff, calcareous, massive to thin-bedded, and fine to medium grained. The shales and claystones are gray to olive, and occasionally carbonaceous. Siltstones are thin-bedded and gray to yellowish-brown. In the project area, the dip of these units is generally toward the west (Ash and Davis 1964).

The Cub Mountain Formation includes continental red-beds. In the road cuts near the north end of the project, the formation displays massive purple sandstone, and various colored from white to pink-gray to purple siltstones and shales.

The project site is in an area of low seismic risk. Peak Ground Acceleration at the site is on the order of 0.05g for a 10 percent probability of exceedence in 50 years (Frankel et. at. 1997). Therefore, seismic shaking was not evaluated as a critical design condition in this study.

Colluvial soils of variable thickness overlie the natural slopes in the project area. The colluvium is composed of clay, silt, sand, cobbles, and boulders derived from the local rock formations, and varies in composition depending on local sources. Fill associated with the existing highway is also present at most locations proposed for RSS. In many locations, the RSS footprint bears entirely on existing fill.

ROCK SLOPE DESIGN

A field reconnaissance was conducted by representatives of NMSHTD, Golder, and URS on January 15 and 16, 2003. The primary purpose of the field reconnaissance was to define areas for more detailed study during the rock slope mapping and the subsurface drilling investigation. For purposes of description, a total of nine cuts were identified, each of which was treated as a separate study location.

The existing rock cuts in the project area have mixed conditions in terms of lithology, rock mass properties, and geologic structure. Most of the cuts expose sediments with variable intact strength, fracture intensity, and weathering, intersected by intrusive rocks which are also of varied strength and degree of weathering. Some cuts are dominated by sediments, and others by intrusive rocks. Sedimentary sequences generally dip to the west at less than 20 degrees; prominent joints in both the sediments and the intrusive rocks dip steeply to the west-northwest and northeast, although there is considerable scatter in structure orientations throughout the project area.

Colluvium is present at the crest of the slope in some of the existing cuts. No indications of water in the slopes or evidence of seepage were observed in any of the cuts during any of the field studies.

Geotechnical Characterization

Characterization efforts included geotechnical drilling, lab testing, seismic refraction surveys, and rock structure mapping. The drilling investigation included acquisition of structural data through use of videotape logging with a down-hole optical televiewer (proprietary name COBL) at three cut locations. The COBL logging provided oriented structure data in spreadsheet format for the features and discontinuities intersected by the boring. It also provided videotaped and photographic records of the logged holes. The structure data was particularly useful, as it provided a subsurface profile of fracture orientations at locations near the proposed cut faces, in a format readily imported for use in the DIPS stereographic plotting program.

Based on the drilling program, the character of the rock masses behind the existing cut slopes is similar to that exposed in the cuts. This is significant for design in that better quality rock is not expected to be exposed by new cuts. The only exception to this was found in one of the cuts, where the drilling program encountered competent sandstone near the proposed cut line, rather than the weathered intrusive rock characterizing the existing cut.

Laboratory testing included unconfined compressive strength testing of selected cores from all drill holes, and slake durability testing of two shale samples. Point load tests were also conducted on rock samples during drilling of most coreholes. The results of the point load tests corresponded well with field observations (both of cores and exposures in cut faces), and were judged to be more representative than the laboratory unconfined tests. Seismic surveys were also used to supplement assessment of anticipated rock conditions.

Existing rock cuts were mapped to collect structural data, define rock types and distribution. Figure 1 presents an example of rock types and features mapped on photos of the rock cuts. Mapping windows within the cuts were selected to provide information representative of rock structure and/or rock mass conditions in the range of conditions observed in the cuts. This is a somewhat subjective means of data collection, but appropriate because exposure varied considerably across the site, and there were limited locations that featured both suitable access and measurable structure surfaces.

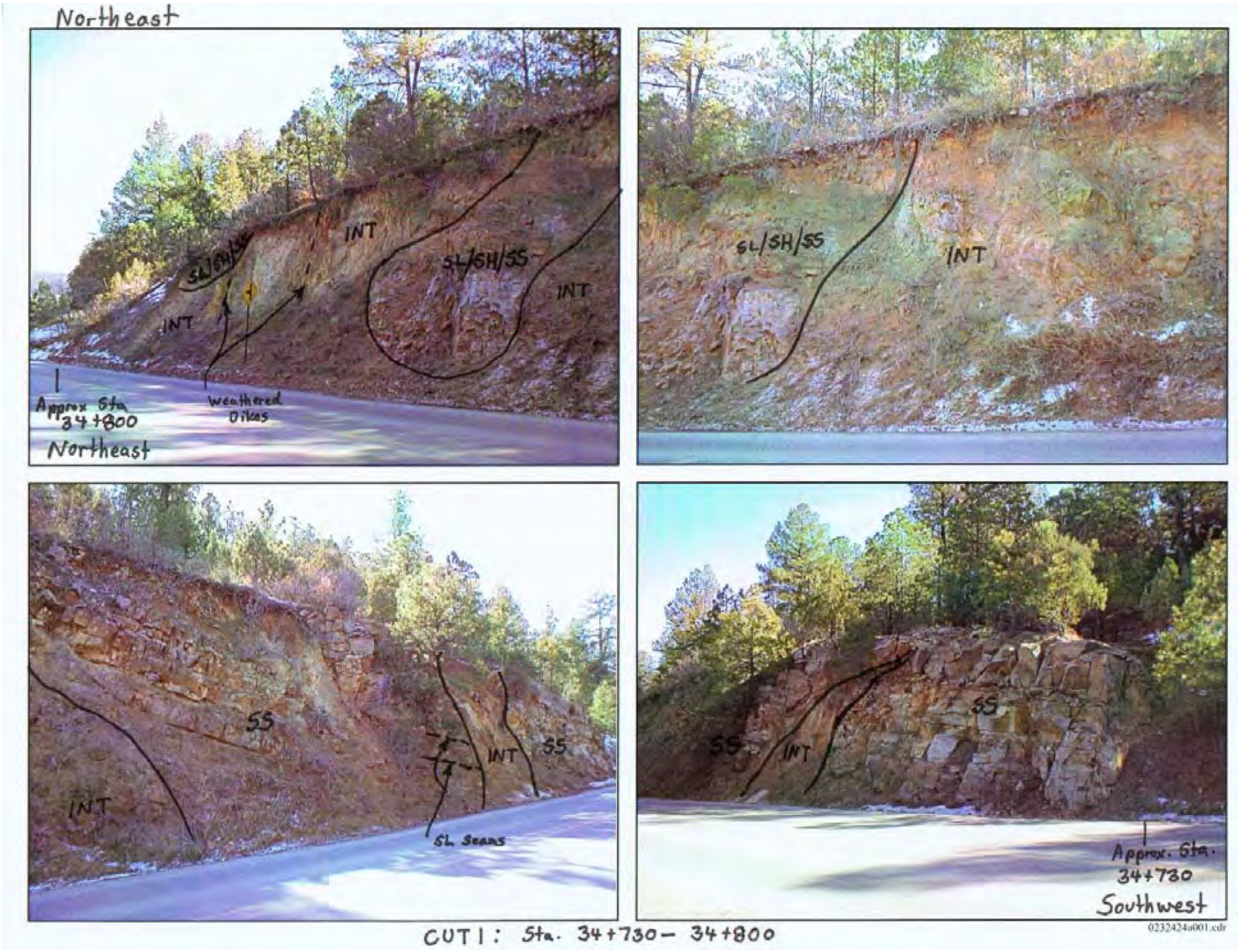


Figure 1: Rock Cut Photo Mapping

The data collection format included the following:

- cut number;
- structure type (joint, foliation, shear/fault, bedding, vein, dike);
- dip;
- dip direction or strike;
- persistence;
- termination;
- aperture/width;
- nature of infilling;
- rock type;
- strength of rock;
- qualitative description of the surface roughness;

- qualitative description of the surface shape; and
- evidence of water flow.

Over 400 data points were collected. A contoured stereographic plot is included as Figure 2, indicating the concentrations of major discontinuity sets, as well as the considerable scatter in the data. Structural data from the COBL logging is presented as Figure 3. The reader should note that the structure detected by COBL is dominated by bedding planes. In a vertical borehole, the data will be biased toward those features closest to perpendicular with the borehole axis (bedding planes in this case). The COBL data was particularly useful to examine the range of bedding plane orientations in those areas where variable-dip bedding was observed in the existing cut.

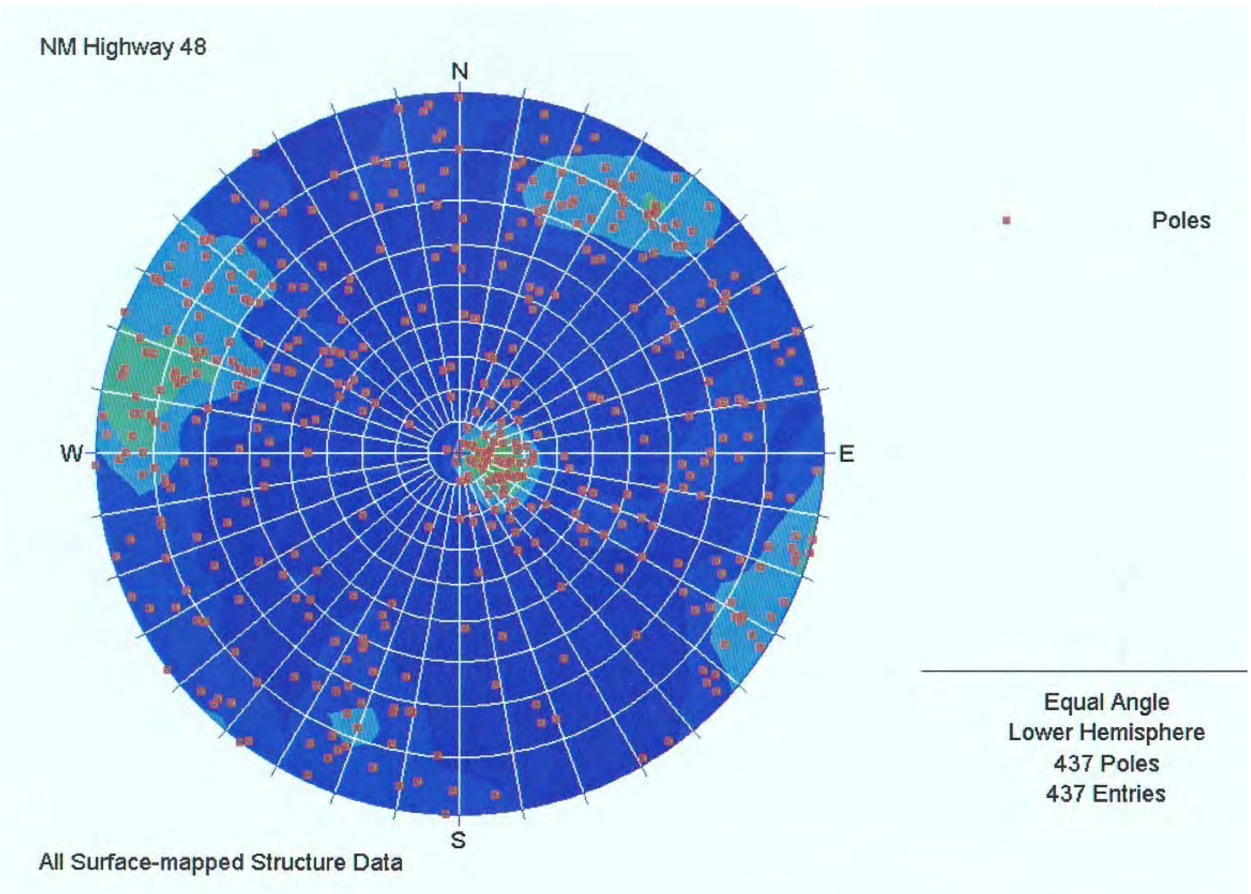


Figure 2: Stereographic Contour Plot of Site-wide, Surface-mapped, Structural Data (Poles)

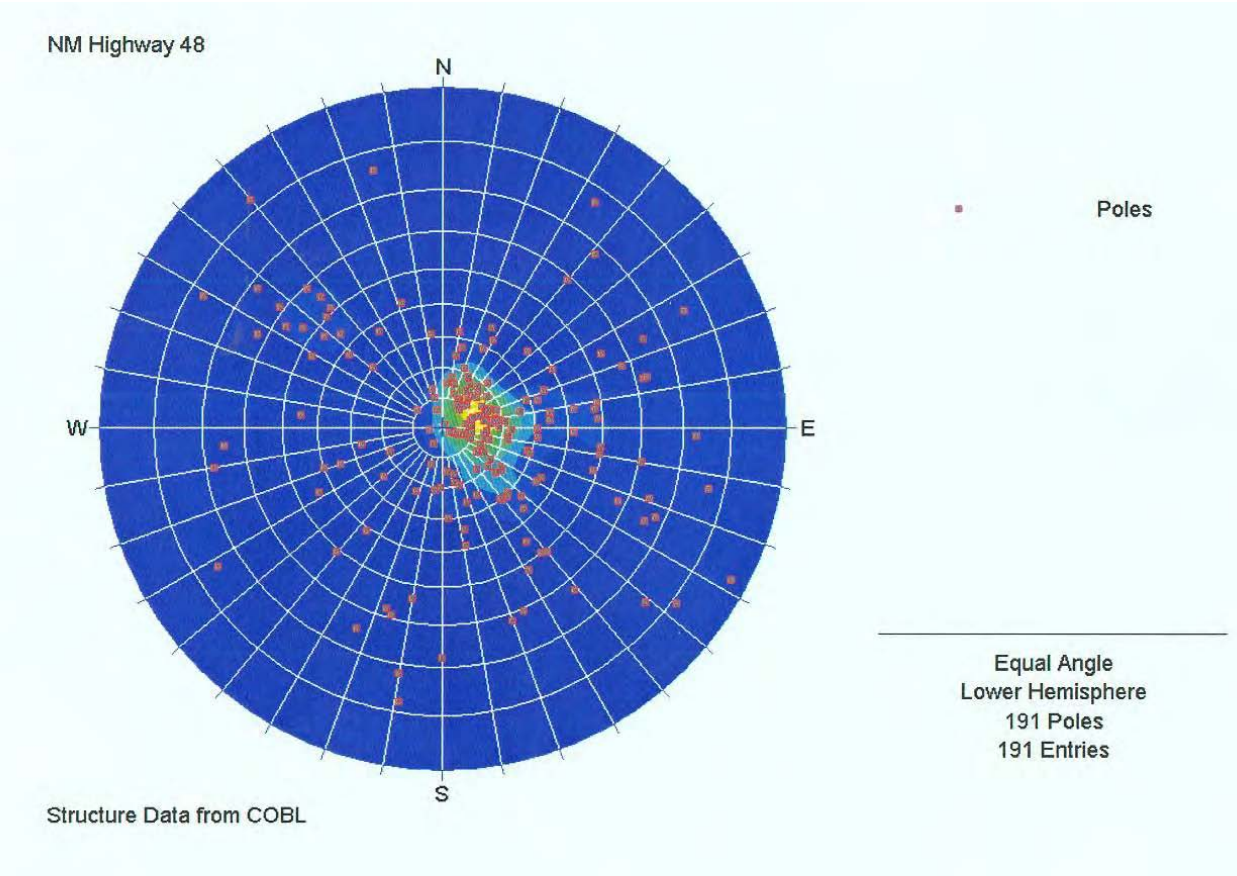


Figure 3: Stereographic Plot of COBL Structural Data (Planes)

Stability Analyses

Cut slopes on this project were analyzed for kinematic stability as well rock mass stability. For this paper, the writers assume the reader is familiar with these types of analyses.

Kinematic Analyses

For the kinematic analysis, joint and bedding friction was conservatively assumed as 20 degrees in the sedimentary rocks, due to the presence of shale in most cuts. For the intrusive rocks, discontinuity shear strength was represented by a friction angle of 36 degrees.

Interpretation of the structure data for the purposes of kinematic analyses focused on identifying systematic discontinuity sets as well as unique, non-systematic structures that could influence stability. This involved a combination of stereonet analysis with DIPS software (Rocscience 2002) and field observations. Cut face angles used in the kinematic analyses are assumed to be approximately parallel the existing road alignment.

Data along each cut were plotted initially as poles, then contoured to identify prominent pole concentrations. These concentrations were considered to represent sets of parallel discontinuities that are ubiquitous in the rock mass, i.e., systematic discontinuity sets. Such discontinuity sets can be represented by "design" orientations based on average dip and dip direction of the structures within the sets. Systematic discontinuity sets present in the existing cuts include bedding in the sedimentary rocks, and moderately to steeply-dipping joints in both the sedimentary and intrusive rocks.

As noted previously, there is considerable scatter in joint orientations. "Outliers" to the averages will likely occur in the proposed cuts, along with non-systematic structures that are not accounted for in the kinematic analyses. While these structures could contribute to structurally-controlled failures, our field observations suggest that these failures would not involve frequent events or large volumes of material. Therefore, we do not believe it is appropriate to consider these structures as limitations on cut designs for this project.

An example of a Markland plot illustrating kinematic analysis for the cuts is presented as Figure 4.

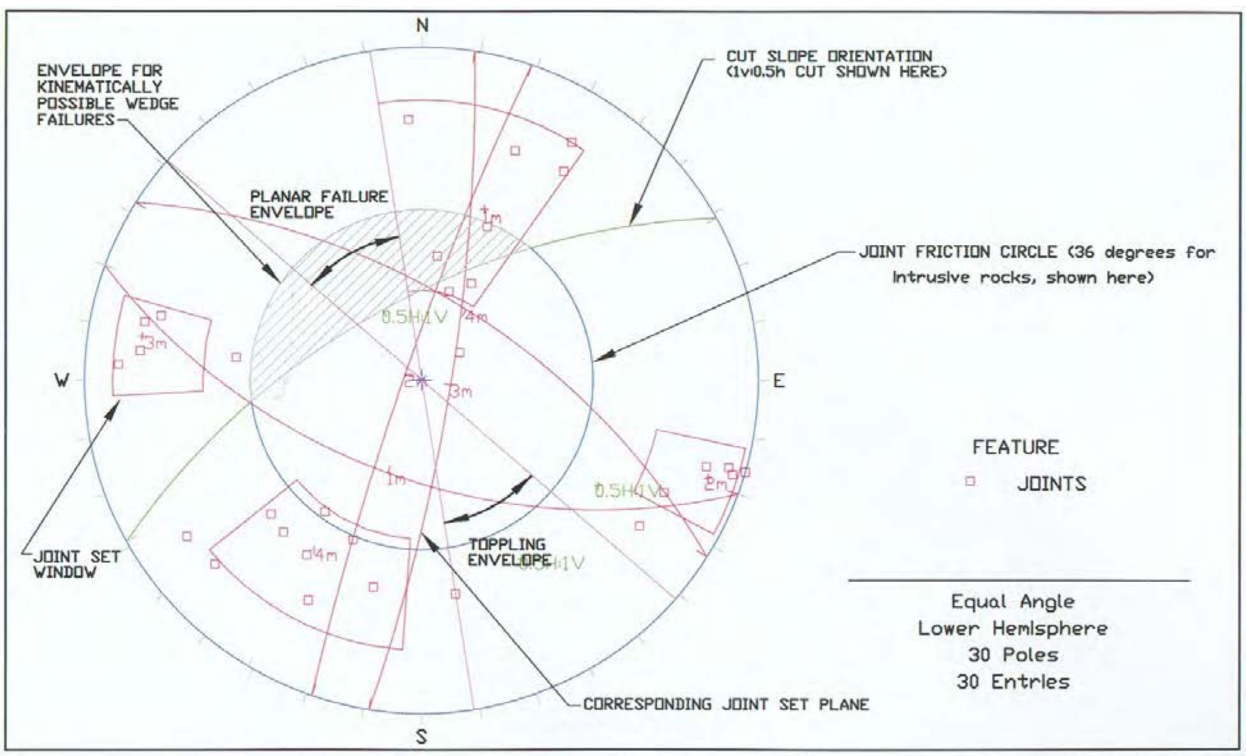


Figure 4: Stereographic Plot of Markland Kinematic Analysis

Rock Mass Stability

Rock Mass stability analyses were carried out using the SLIDE software (Rocscience 2002). Circular failure surfaces were assumed, using Spencer's method.

A general characterization of Rock Mass Rating (RMR) for the main rock types was developed from field observations and geotechnical core logging data. This characterization was applied to similar rock types regardless of location, with a few exceptions where specific rock types or characteristics were confined to certain cuts. Table 1 presents the parameters used to estimate RMR for the various rock types in the cuts.

Table 1

Rock Type	**Strength of Rock Material**	**RQD Rating**	**Joint Spacing**	**Joint Condition**	**Ground Water**	**Rock Mass Rating**	**Class**
Cut 1 Sandstone	7	14	10	6	10	47	III
Cut 3 Shattered Sandstone	7	3	7	10	10	37	V
General Sandstone	7	13	10	10	10	43	IV
Shale	2	8	5	10	10	35	V
Siltstone	4	6	7	10	10	37	V
Diorite (weathered, variable)	0 - 2	3	20	12	10	45-47	IV
Cut 4 Competent Diorite	7	17	17	20	10	71	II

Although kinematic analysis was used to define appropriate cut slope angles, observation of rock mass conditions in the existing cuts suggested that in some cases rock mass strength might be the dominating factor in cut slope stability. Therefore, analysis of stability based on rock mass strength was a key part of the evaluation. Hoek-Brown criterion (Hoek and Brown 1988; Hoek, et. al. 1995; Hoek and Karzulovic 2000) was used to estimate rock mass strength for all materials except colluvium. The Hoek-Brown approach is generally applied to brittle, fractured rock masses, and in contrast to the Mohr-Coulomb criterion, it defines a non-linear relationship between shear and normal stress, with a shape determined by three basic parameters, namely:

- Unconfined compressive strength (UCS) of intact rock pieces;
- The constants m_i and s for intact rock; and
- A parameter that links laboratory testing parameters (m_i, s) with the field-scale rock mass parameters. In earlier versions of this criteria, this involved the use of Bieniawski's (1976) RMR. More recently, a parameter called the Geological

Strength Index (GSI) has been introduced for this purpose. The need for the GSI grew out of the fact that it is difficult to estimate RMR for very poor quality rock masses. For more competent rock masses (RMR>25), the GSI value is equal to the RMR, i.e., use of RMR to estimate rock mass shear strength is still appropriate if greater than 25.

Rock mass shear strength parameters *m* and *s* were calculated from the RMR values summarized above and the estimated rock strength, using the RocLab (Rocscience 2002) software program. In addition to RMR input the program requires selection of parameters for rock type and degree of blasting disturbances. For the purpose of this analysis, poor blasting conditions were conservatively assumed.

Strength properties for colluvium were defined based on Mohr-Coulomb parameters, which define the material in terms of internal friction angle (ϕ) and cohesion (c). Parameters for ϕ and c for the colluvium were selected based on field observations, and are consistent with properties for a gravelly, silty, sand soil.

Recommended cut slope design geometry was developed based on the analyses of both kinematic and rock mass stability. Rocommended cut slope angles ranged from 2V:1H in competent sandstone to 1V:1H in weathered rock, and 1V:1.25H in colluvium. At two of the cuts, combination cuts were used to accommodate layered rock types. For example a cut with competent sandstone overlying shale was designed as 1.33V:1H in the upper sandstone portion, with the underlying shale to be cut at 1V:1.5H.

Rock Fall Analyses

Locally adverse structure and variable rock quality present a rock fall hazard that needs to be accommodated in cut designs. This hazard can be mitigated by providing adequate catchment along the base of the cuts. Catchment requirements were evaluated in light of guidelines presented in FHWA (2001), and rock fall modeling with the computer program CRSP (CDOT 2000). The importance of this analysis is underscored by the scatter and outliers in the kinematic analyses, indicating that rockfall events are likely with implementation of the recommended slope angles.

Rockfall analyses were conducted based on catchment ditch geometry as indicated on Figure 5, which includes a 4.8 meter wide catch ditch. In some cases, this geometry resulted in estimated rockfall retention less than 90 percent. For these cases, a “Jersey Barrier” catchment wall was added at the pavement edge.

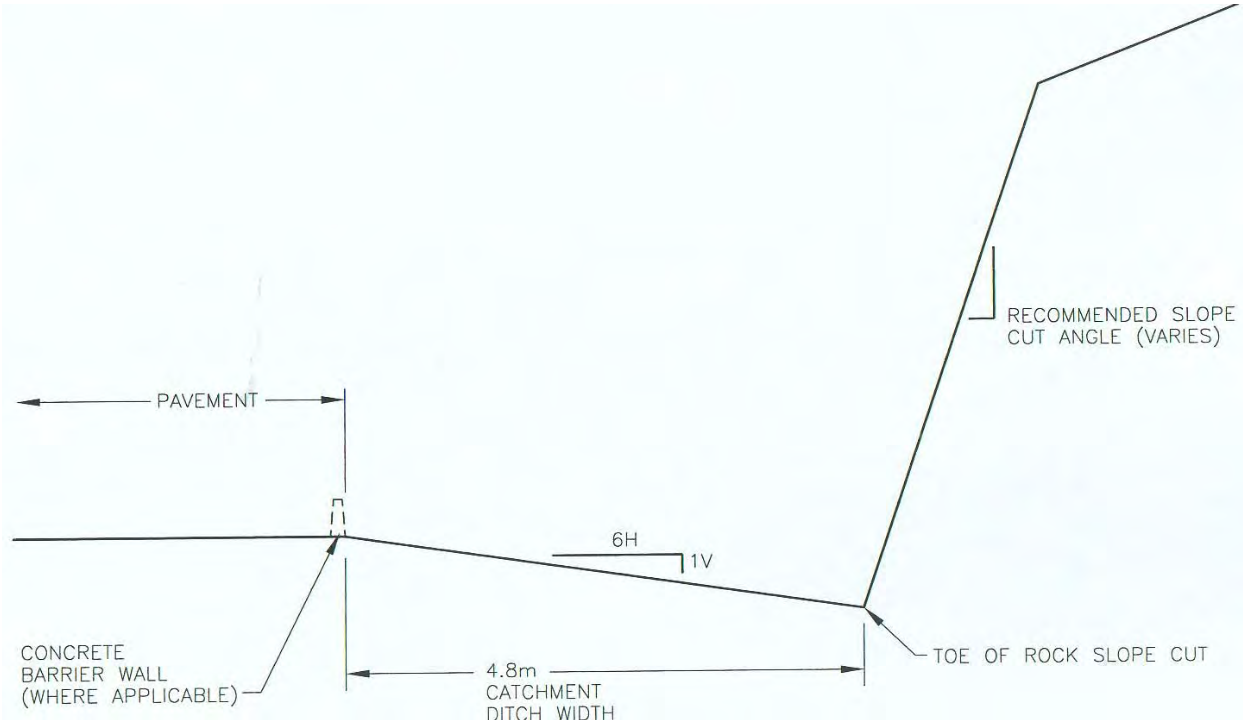


Figure 5: Rock Fall Catchment Ditch

The Rockfall Catchment Area Design Guide (FHWA 2001) was used to estimate required catchment ditch widths. The Design Guide presents a series of charts to determine the required width for a given combination of cut slope angle, slope height, ditch bottom configuration, and desired rockfall retention.

According to the Design Guide, most sections require ditch widths greater than 4.8m to provide a rockfall retention of 90 percent or better. The Design Guide results are considered to be conservative for the conditions examined for NM 48. Use of the CRSP model provides less conservative results, but is based on more project-specific conditions, and is thus considered to be adequate for design of NM 48.

Slope geometry is defined in CRSP by a series of segments or cells. Slope properties within each cell are represented in CRSP by the following coefficients:

- R_t - tangential coefficient; a measure of the degree to which the component of rock velocity parallel to the slope is slowed by impact
- R_n - normal coefficient; a measure of the degree to which the change in velocity normal to the slope caused by impact
- S - roughness coefficient

These coefficients are best determined by site-specific calibration, but in the absence of this type of data, CDOT (2000) provides guidelines for parameter values.

In rock cuts of 1V:1H, the rock slope was modeled as a softer shale or weathered surface. For cuts steeper than 1V:1H, the rock slope was assumed to have a harder surface typical of the competent sandstones.

For most of the cuts, rock fall was assumed to originate from points between road level and the crest of the cut. This assumption is consistent with our field observations, and based on the assertion that rock fall has a uniform opportunity to initiate at any point on the slope. An exception to this, at least in the existing cuts, occurs in areas where colluvium at the crest has been undermined, and presents the potential for release of rocks. This circumstance was not considered to be the design condition for the purposes of this study, and it has been assumed that these conditions will be effectively eliminated in the new cuts by rounding of the crests and scaling of boulders in the colluvium where present.

However, an exception was also analyzed for one of the cuts. Here the CRSP analysis conservatively considered only rock fall from the upper 4 m of the cut. This reflects the field observation that steeper bedding near the top of the existing cut is encouraging kinematic rock falls not seen in the other existing cuts.

Rock sizes of 0.6 m were assumed, as reflected by field observation of approximate maximum block sizes in the existing cuts. Rock shape was assumed to be spherical, which is somewhat conservative since actual rock shapes will be irregular. The assumed initial horizontal and vertical rock fall trajectory velocities were between 0.3 and 0.6 m/s. While the upper range of velocity is considered quite high, it was used to trigger consistent rockfall runout in the CRSP model for some slopes.

For each simulation, 1,000 rocks were numerically “rolled” down the cut slope, and the number of rocks passing a specified analysis point were tracked. With the analysis point set at the end of the ditch adjacent to the pavement shoulder, ditch widths of 4.8 meters were analyzed. Results are in the form of percent rock fall retained in the ditch, i.e., the percentage of rocks that do not pass the analysis point at the pavement edge of the ditch. Where the CRSP analysis indicated that a ditch width of 4.8 meters was insufficient to provide the 90 percent retention, a “Jersey Barrier” wall was added at the edge of the pavement. In all cases examined, use of the “Jersey Barrier” provided greater than 95 percent rockfall retention.

In the cases where barriers are used to provide rock fall protection, the barriers must be capable of withstanding the energy of rock impacts. For the cases where barriers are recommended, CRSP calculated the following energies and velocities at the barrier face. Table 2 gives examples of the rock energy data calculated by CRSP, which could be used to design or check the adequacy of appropriate barriers.

Table 2

Location	Maximum Velocity (m/s)	Average Velocity (m/s)	Maximum Energy (Joules)	Average Energy (Joules)
1.33V:1H cut slope up to 15m high	6.59	3.63	9552	3392
1V:1H cut slope up to 12m high	6.06	3.19	7478	2684

Rock Slope Design Summary

Rock slope design recommendations were summarized and tabulated for use by NMSHTD in the construction documents. Included were rough estimates of percent excavation requiring blasting. Required rock blasting volumes were estimated based on the results of the seismic surveys, borehole findings, and conditions observed in the existing cuts. NMSHTD also uses a D-8 rippability criteria to define pay quantities for blasting versus mass excavation. Controlled blasting was assumed for all rock cuts steeper than 1:1. As part of the design project recommended specifications for cushion blasting to be used as part of NMSHTD's road construction specification package were developed.

Compound slopes were recommended for two of the cuts. The exact locations and elevations for the slope changes were not designated. These will be a function of the rock boundaries within the new cut slope. This could be better defined by further drilling and surveying, but given the rock mass variability inherent over most of the project, it may be more practical to make these determinations in coordination with slope construction.

Reinforcement was not recommended for any of the cuts. In most cases, this relates to rock mass conditions that do not appear to be amenable to the effective use of reinforcement. While significant kinematic failures are not expected, there are locations where the potential for kinematic failures exist and "spot" reinforcement in the form of tensioned bolts or untensioned dowels may be appropriate on an as-needed basis.

RSS DESIGN

A significant concern for the RSS sections on the project was the analysis, design, and performance of these sections considering that most RSS sections are supported on steep colluvial slopes or fill embankments. This section of the paper provides a review of the characterization and design process, with emphasis on addressing this RSS foundation issue.

Geotechnical Characterization

A field investigation that included auger and core drilling, seismic refraction surveys, and surface reconnaissance was completed for this project. In addition, investigations carried out for the cut slope design portion of the project also provide useful data.

RSS Design Basis

The RSS structure consists of three basic elements: the soil backfill material, the reinforcing elements, and the slope facing. The interaction of the first two components is what gives the RSS its internal strength and allows the slope face to be constructed at a steeper angle than would be practical with unreinforced fill. The RSS construction for NM48 is proposed to utilize synthetic geogrid reinforcement. It is anticipated that the soils used in the RSS will be from local borrow sources, and will consist of well-drained, low plasticity, predominantly granular soils. The third component (the slope facing) provides surficial stability and addresses aesthetic needs.

Design of the RSS requires developing a combination of geometric, soil, and reinforcement properties which allow the structure to meet the designated performance criteria. Three failure modes must be considered:

1. External Stability, where the failure surface passes beneath and behind the RSS, not intersecting the structure itself. This type of stability is most dependent upon the surrounding earth material properties and is not a function of the RSS internal strength. It is, however, affected by the geometry of the RSS, generally assumed for analysis purposes to be infinitely strong in relation to the surrounding soils.
2. Internal Stability, where the failure occurs within the structure itself. This type of stability is dependent upon the type and spacing of reinforcement combined with the soil properties.
3. Compound Stability, where the failure surface passes through both the RSS structure and the adjacent backslope or foundation. This type of stability is a more complex analysis, as it must include assumptions for the strength of the reinforced structure.

Standard procedures for RSS design are included in FHWA publication NHI-00-043, *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines*. This procedure was used in the design of the RSS slopes for NM48, along with the Tensar Earth Technologies Inc. design procedures referenced by NHI-00-043. The following discussion of project design is consistent with the steps outlined in Chapter 7 of NHI-00-043. Stability analyses were performed using the limited-equilibrium stability analysis software SLIDE (V. 5.0).

Stability Model

A generalized cross section of the RSS slopes is presented as Figure 6. At most of the RSS slopes, the toe of the RSS will be founded on the existing highway fill. In a few cases the RSS may toe on natural hillslope colluvium. In a few locations, shallow bedrock may be present below colluvium. In all cases, it is anticipated that the backslope (or retained soil behind the RSS) will be in existing fill, which is typically clayey sand, or a silty clay in some areas.

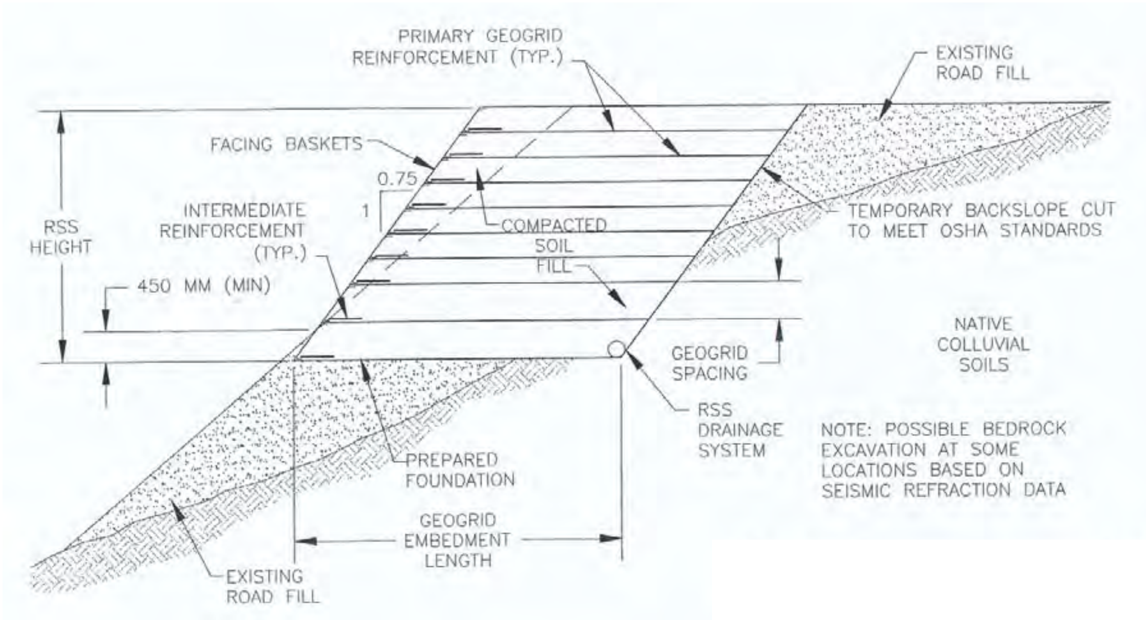


Figure 6: Schematic Cross-Section of RSS Slope

Table 3 presents the soil strength parameters selected for the fill and colluvium. Values for the colluvium and proposed fill are based on published values (Hunt 1984) for the observed material types and densities. Soil parameters for the existing road fill are based on a back-analysis of the existing fill slopes. An assumed safety factor of 1.3 was used in the back analysis, based on the observed adequate performance of these slopes. The back-calculated strength factors were then used to model the existing fill in the design analyses. Since the proposed slope configurations are designed to a minimum calculated safety factor of 1.3, this methodology yields a design that meets or exceeds (where FS > 1.3) the stability condition of the existing road fill slopes.

Table 3 - Estimated Material Properties For Strength Analysis

Material	Moist Density (kN/m3)	Friction Angle (phi)	Cohesion (kPa)	Comments
Existing Fill (SC)	21	34	3.4	Expected to characterize most of existing fill. Strength values back-calculated based on performance of existing fill.
Gravel Colluvium (GM)	20.6	40	0	Expected to characterize most foundation conditions below fill.
Clay (CL) Fill and Colluvium	21	28	0	Occurrence of clay detected at several locations.
Proposed Fill (see Section 5.3)	21	28	0	New fill assumed to have same properties as existing clay fill.

The reader should understand that this approach to selection of parameters for global stability does not unconditionally guarantee or assure a factor of safety of 1.3 for global or foundation stability. This approach instead produces global stability that is the same or better than the stability of the current embankment or colluvial slopes. The practitioner is encouraged to consider 2 factors in adopting this approach to other projects:

- Is the performance of the currently existing slopes or embankments satisfactory (i.e., is the current factor of safety likely to be adequate?
- Are the back calculated strength parameters reasonable?

If these questions can be answered affirmatively, the writers believe that this approach is a reasonable and practical method to address this issue for RSS structures in steep terrain. Additional consideration must be given to deflections or settlements, as discussed further herein especially when the system is supported on fill, colluvium, or other unconsolidated materials.

Observation of the materials recovered from the borings and surface observations do not indicate that seepage occurs within the existing fills or underlying materials. However, certainly saturation of the lower portion of the fill and portions of the foundation could occur, even if only as a transient condition. Drains are proposed at the heel of the RSS fills, and thus elevated piezometric levels are not considered within the new fill for purposes of stability analysis.

Reinforcement properties were assumed per FHWA SA-96-071 and 072. These include definition of ultimate strength and appropriate partial factors of safety.

Design Analyses

Procedures for analysis and design were followed based on FHWA publication NHI-00-043. These procedures dictate an initial analysis or unreinforced stability.

The stability analyses are performed using conventional computer analysis programs. SLIDE (version 5.0) was used in this study. The range of failure conditions examined includes both circular and wedge failures, failures through the toe, the slope face, and the foundation below the RSS. The size and configuration of the soil zone to be reinforced is approximated by the critical failure mode which just meets the required factor of safety. If critical failure surfaces exit below the toe of the planned slope, improvement of foundation and bearing capacity must be addressed in the design, or longer reinforced zones may be considered.

Internal Stability

On many projects, the design of reinforcement for internal stability is performed by the reinforcement manufacturer, using the specific properties and dimensions of his propriety products. However, requirements for the design of reinforcement and internal stability, as indicated by the results of the global stability analysis, are provided as a guide for the final design layout by the reinforcement provider. Details on the design of primary reinforcement are presented in Table 4, which includes a reinforcement schedule to aid the final design layout. This table is for use in estimating project costs and as a guide in final design layout of the RSS. For RSS heights between those evaluated here, it is recommended that the geogrid information for the next larger RSS be used. These reinforcement schedules assume the use of a wire face basket bent to the slope angle specified on the plans of 1 vertical to 0.75 horizontal.

The maximum spacing of the primary internal geogrid reinforcement used for this project is recommended to be 1 meter nominal, which deviates slightly from the spacing suggested in NHI-00-043. The use of a nominal 1 meter spacing, in our experience is constructible, cost effective, and the design calculations support the use of 1 meter spacing. In addition to the primary reinforcement, a secondary or intermediate reinforcement will be required between the individual wire baskets.

Table 4 - RSS Reinforcement Schedules

Reinforcement Schedule	RSS Height (m)	Vertical Range for Given Geogrid Spacing, Measured from RSS Crest (m)	Minimum Longterm Allowable Tensile Strength (kn/m)	Geogrid Embedment Length (m)	Geogrid spacing* (m)
1	9*	4.0 – 9.0	21.8	8.61	0.36
	9	2.5 – 4.0	21.8	8.61	0.72
	9	0 – 2.5	21.8	8.61	1.08
2	7*	4.0 – 7.0	21.8	6.81	0.36
	7	2.5 – 4.0	21.8	6.81	0.72
	7	0 – 2.5	21.8	6.81	1.08
3	5.5	4.0 – 5.5	21.8	5.46	0.36
	5.5	2.5 – 4.0	21.8	5.46	0.72
	5.5	0 – 2.5	21.8	5.46	1.08
4	4	2.5 – 4.0	21.8	4.11	0.72
	4	0 – 2.5	21.8	4.11	1.08
5	2.5	0 – 2.5	21.8	2.80	1.08

*Alternate layout for 9 and 7 meter high RSS may be as follows:

Reinforcement Schedule	RSS Height (m)	Vertical Range for Given Geogrid Spacing, Measured from RSS Crest (m)	Minimum Longterm Allowable Tensile Strength (kn/m)	Geogrid Embedment Length (m)	Geogrid spacing* (m)
1 or 2	7 to 9	5.4 – 9.0	29.2	8.61	0.36
1 or 2	7 to 9	4.0 – 5.4	29.2	8.61	0.72

Notes:

1) In addition to the primary reinforcement, secondary or intermediate reinforcement will be needed between the individual wire baskets. The intermediate reinforcement is assumed to be a biaxial geogrid with 1.2 meters of minimum embedment into the RSS backfill.

2) Design assumes use of wire face baskets bent to 1V:0.75H slope angle, galvanized, with W4 gauge and 4 inch center to center apperative spacing.

External Stability

Eight cases were modeled to conservatively represent the range of anticipated conditions for the project using the more conservative foundation assumptions from the range of estimated conditions and maximum slope heights. The assumed conditions include a distributed load of 250 psf to model standard AASHTO traffic loading. Computed factors of safety varied from 1.3 to 1.6.

Deflection and Settlement

In the writer's experience, deflection or settlement associated with RSS or MSE structures, especially in steep terrain, is sometimes not given proper attention in the design process.

As discussed above, the RSS fills will be founded on either existing sloping fills or colluvium, in most cases. The RSS fills will impose new stresses to these compressible soils, and some resulting settlement is inevitable. Quantitative consolidation data for these soils is not available, and indeed would most likely not produce a practical estimate of settlement if it were.

We conducted a rough one dimensional total settlement estimate using correlations to soil modulus for various soil types; this estimate does not suggest excessive total settlements for a fill embankment. However, this simple estimate does not reflect two important aspects of the situation at hand:

1. Settlement caused by a surcharge loading (such as an RSS fill) placed above a relatively steep slope of compressible material is not accurately modeled by a one

dimensional settlement analysis; a significant portion of the vertical settlement that occurs will be due to lateral spreading or deflection of the material, and

2. Poor settlement performance of RSS or MSE structures is most often, in our experience, not due to total settlement, but to differential settlement. Differential settlement, where vertical settlement is greater under the toe of the fill than at the heel, produces outward rotation of the reinforced mass, and can result in a tension crack at the back of the reinforced mass, which may lie within the paved roadway.

However, if the settlements to occur on this project happen relatively rapidly, during or immediately following construction (i.e., prior to paving), neither of these issues should severely impact project performance or pavement integrity. The general granular nature of the materials present (existing fill and colluvium), coupled with reasonable surface and subsurface drainage provisions, suggest that the risk of long term settlement is low.

Furthermore, our experience confirms that RSS fills are most often much less vulnerable to the effects of differential settlement (as identified above) than vertical faced MSE fills. The geometry of an RSS fill is such that resultant forces act with much less eccentricity, so rotation and resulting tension cracks are much less likely. The rotation and tension crack phenomenon is fairly common (though not catastrophic) for vertical MSE walls built over compressible material at steep slope angles; it is rare for RSS fills. However, it can still happen.

Such cracking from differential settlement is typically not catastrophic, and can usually be repaired or patched. To reduce the possibility of such maintenance measures, foundation improvement can reduce settlement. Alternately, the upper one or two reinforcement layers can be extended further beneath the new pavement surface, although this approach may complicate traffic maintenance during construction. Our judgment is that the risk is small, and the consequences not severe, so the additional cost is probably not justified. Therefore, these mitigative measures were not incorporated into the design approach.

<u>Seismic</u>

Seismic stability was not evaluated as the project is not located in an area of known significant seismic activity. Liquefaction is not considered a potential failure mode at NM48, due to generally well-drained foundation soils and lack of static groundwater in the foundation areas.

<u>Drainage</u>

The RSS design includes a drain detail at the heel of the RSS fill, and composite drainage panels will be placed on the backslope behind the RSS fill to collect subsurface seepage. No evidence of significant groundwater or seepage was found during the field investigations. The proposed drains to be incorporated into the RSS are expected to adequately handle incidental transient groundwater occurrence.

SUMMARY AND ACKNOWLEDGEMENTS

The writers trust that this case study will be useful to other practitioners meeting similar challenges in similar settings. We wish to acknowledge the cooperation, support, and contribution of the following individuals and organizations:

- The New Mexico State Highway and Transportation Department
- Mr. Charles Miller, P.E., of Geotest, Inc.
- URS Corporation
- AMEC

REFERENCES

AMEC. 2003. Refraction Seismic Evaluation, NMSHD Geotech On Call Task No. 9, NM 48 north of Ruidoso, New Mexico, Work Assignment WA3688A9. Report to Geotechnical Design Section, New Mexico State Highway and Transportation Department, dated February 11, 2003.

Ash, Sidney R., and Leon V. Davis (ed). 1964. Guidebook of the Ruidoso Country. New Mexico Geological Society. Fifteenth Field Conference, October 16, 17, and 18, 1964.

Bieniawski, Z.T. 1974. Estimating the Strength of Rock Materials: S. African Institute of Mining and Metallurgy, V. 74, p. 312-320.

Bieniawski, Z.T. 1976. Rock Mass Classifications in Rock Engineering. Proceedings on the Symposium on Exploration for Rock Engineering, Rotterdam. Vol. 1, p. 97-106.

Brock, E., and Franklin, J.A. 1972. The Point Load Strength Test. International J. Rock Mechanics and Mining Sciences, Vol. 9, p. 669-697.

Colorado Department of Transportation. 2000. Colorado Rockfall Simulation Program, Version 4.0. Colorado Department of Transportation, Colorado School of Mines, and Colorado Geological Survey. CDOT-SYMB-CGS-99-1. March 2000.

Department of the Navy. 1982. Soil Mechanics Design Manual 7.1. NAVFAC DM-7.1. Department of the Navy, Alexandria, Virginia, May, 1982.

Federal Highway Administration. 1998. Rock Slopes, Training Course in Geotechnical and Foundation Engineering. FHWA Publication No. HI-99-007. October 1998.

FHWA. 2001. Rockfall Catchment Area Design Guide: FHWA Publication FHWA-OR-RD-02-04.

Federal Highway Administration. 2001. Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines. FHWA Publication No. FHWA-NHI-00-043. March 2001.

Frankel, Arthur; Muller, Charles, Barnhard, Theodore; Perkins, David; Leyendecker, E.V.; Dickman, Nancy; Hanson, Stanley; and Hopper, Margaret. 1997. National 1996 Seismic Hazard Maps: U.S. Geological Survey, Open File Report 97-131 (via http://eqint.cr.usgs.gov).

Golder Associates Inc. 2003. Geotechnical Report Work Assignment WA3895A2, Contract Number C04385, Geotechnical Services for Design of Rock Slopes for New Mexico Highway 48. Prepared for New Mexico State Highway and Transportation Department by Golder Associates Inc., dated June 9, 2003.

Golder Associates Inc. 2003. Geotechnical Report Work Assignment WA3895A2, Contract Number C04385, Geotechnical Services for Design of Rock Slopes for New Mexico Highway 48, Fill Slope Analysis and RSS Slope Design. Prepared for New Mexico State Highway and Transportation Department by Golder Associates Inc., dated July 22, 2003.

Hoek, E., and Brown, E.T. 1988. The Hoek-Brown Failure Criterion - A 1988 Update. 15th Canadian Rock Mechanics Symposium, Toronto, Canada.

Hoek, E., Kaiser, P.K., and Bawden, W.F. 1995. Support of Underground Excavations in Hard Rock: Rotterdam, Balkema.

Hoek, E., and Karzulovic, A. 2000. Rock Mass Properties for Surface Mines *in* Slope Stability in Surface Mining: Society for Mining, Metallurgy, and Exploration, Littleton, Colorado.

Hunt, Roy E. 1984. Geotechnical Engineering Investigation Manual. McGraw-Hill Book Company, New York, 1984.

New Mexico State Highway and Transportation Department. 1998. Construction Plans, TPM-BR-0048(16), Lincoln County, Control No. 1245. Approved December 2, 1998.

New Mexico State Highway and Transportation Department. 2003. Interdeparmental Correspondence, Seismic Refraction Survey, TPM-BR-0048(16), CN 1245, N.M. 48, Angus. Dated February 24, 2003.

Ritchie, A.M. 1963. Evaluation of Rockfall and Its Control: Highway Research Record, No. 17, pp. 13-28.

Rocscience Inc. 2002. Slide, 2D Limit Equilibrium Slope Stability for Soil and Rock Slopes, Version 4.0. Rocscience Inc. Copyright 1989 – 2002.

Rocscience Inc. 2002. Dips, Plotting, Analysis and Presentation of Structural Data Using Spherical Project Techniques, Version 5.0. Rockscience Inc. Copyright 1989 – 2002.

Rocscience Inc. 2002. RockLab, Version 1.0. Rocscience Inc. Copyright 2002.

Tensar Earth Technologies, Inc. 2003. Faxed materials regarding product data and design guidelines, included in Appendix B of this report.

URS. 2003. Reference to be provided in final document.

URS. 2003. Memorandum "NM 48 Rock Slope Geotechnical Drilling," prepared by URS Corporation for New Mexico State Highway and Transportation Department, dated April 1, 2003.

Wyllie, D.C. 1992. *Foundations on Rock*. Chapman and Hall, London.

CASE HISTORIES OF TIEBACK AND SOIL NAIL WALLS FOR ROADWAYS

By: Harry W. Schnabel P.E.[1] and Kevin Cargill,[2] P.E.

The use of permanent tiebacks and soil nailing has become an increasingly acknowledged method of constructing permanent walls for roads and highways. Tieback and soil nail walls allow the widening or straightening of roads with reduced encroachment or disturbance to adjacent properties. They may be the only practical construction method where walls must be constructed into steeply sloping hillsides. Tieback walls are also used to stabilize landslides in unstable topography.

This paper describes two recent projects where soil nailing and tiebacks were successfully used to construct roadway walls. The first project is located on the south side of Route 100B in Moretown, Vermont. A soil nail wall was constructed into the side of a hill in two tiers to allow the alignment of the roadway to be straightened. The steep slope of the hill made the excavation and backfilling required for a conventional gravity or mechanically stabilized earth (MSE) wall too costly. The soil nail wall consisted of drilled and grouted permanent soil nails and a structural shotcrete facing. Insulation and wood planks were attached to the face of the shotcrete to finish the wall.

The second project involved the construction of a series of tied-back walls on Route 321 near the Great Smokey Mountains in Gatlinburg, Tennessee. The walls were constructed using tied-back soldier piles and timber lagging. A cast-in-place facing was then poured against the face of the wall. One of the walls on this project had to be reconstructed when excavation in front of the original wall activated an existing landslide. The Gatlinburg project utilized contracting methods that allowed the specialty contractor staff to best utilize their expertise in providing value engineering to redesign the walls.

Overview

Permanent tieback and soil nail walls have been in use in the United States since the late 1970's. In some instances they offer considerable advantages over conventional gravity and reinforced earth walls. Experienced specialty contractors who have developed unique and specialized equipment and construction procedures usually construct these walls. Permanent tieback and soil nail walls are routinely used where:

- Walls are built into steep hillsides where the excavation for a gravity or reinforced earth wall is cost prohibitive or impossible (Figure 1a).
- There are structures, such as buildings or bridge abutments, behind the wall that preclude the excavation required for a gravity wall (Figure 1b).
- There is an unstable land or rock mass behind the wall that must be stabilized (Figure 1c).

This paper presents a case analysis of two such projects where at least one of these conditions existed. The first project discussed is a permanent soil nail wall constructed on the south side

[1] Vice President and Northeast Regional Manager of Schnabel Foundation Company
[2] Southeast Regional Manager of Schnabel Foundation Company

of Route 100B in Moretown, Vermont. The second project involved the construction of a series of tied-back walls on Route 321 near the Great Smokey Mountains in Gatlinburg, Tennessee.

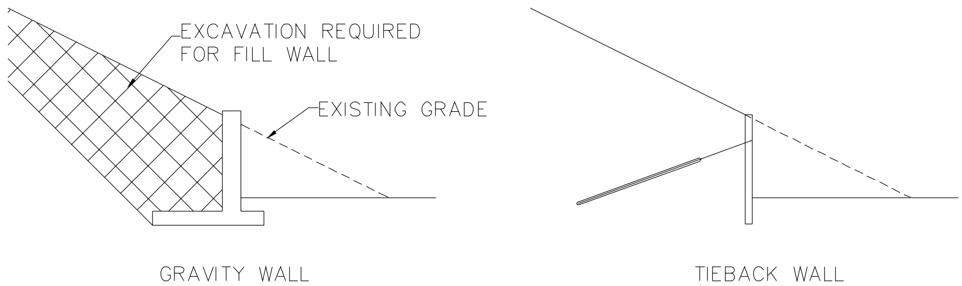


Figure 1a
TIEBACK WALL IN STEEP SLOPE

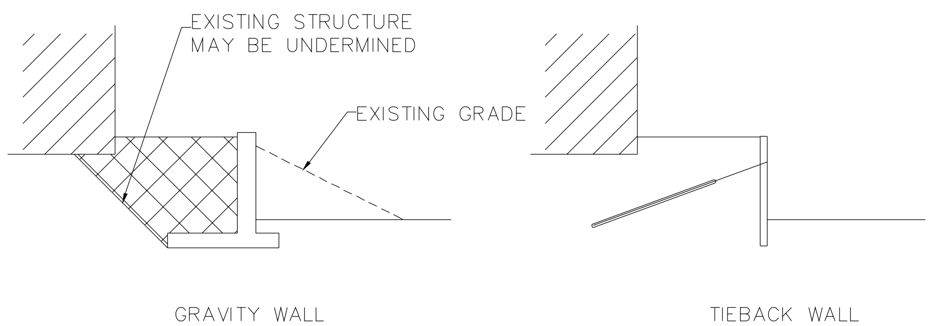


Figure 1b
TIEBACK WALL WITH STRUCTURE BEHIND

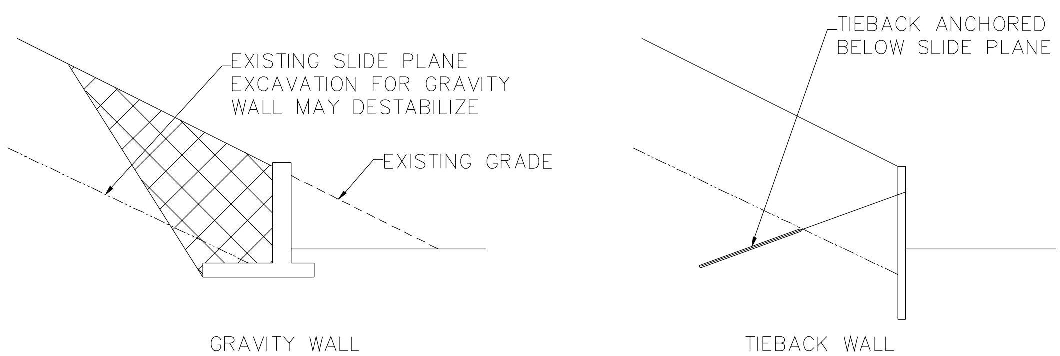


Figure 1c
TIEBACK WALL IN UNSTABLE LAND MASS

Permanent Soil Nail Wall – Route 100B Moretown, Vermont

The permanent soil nail wall in Moretown, Vermont was constructed into the side of a steep hill to allow the alignment of Route 100B to be straightened. The wall was around 134 meters long with a maximum height of 16.8 meters. The total exposed face of the wall was around 1,500 square meters. The wall was constructed in two tiers and battered at 1H:6V to make it more aesthetically pleasing. A cross section through the wall is included as Figure 2.

Soil conditions at the site consisted of a fairly homogeneous medium dense silty sand. One of the principal requirements of a successful soil nail project is soil that has sufficient cohesion or cementation to allow it to stand vertical long enough for the soil nails to be drilled and the shotcrete sprayed on the surface. The installation of soil nails and shotcrete is usually

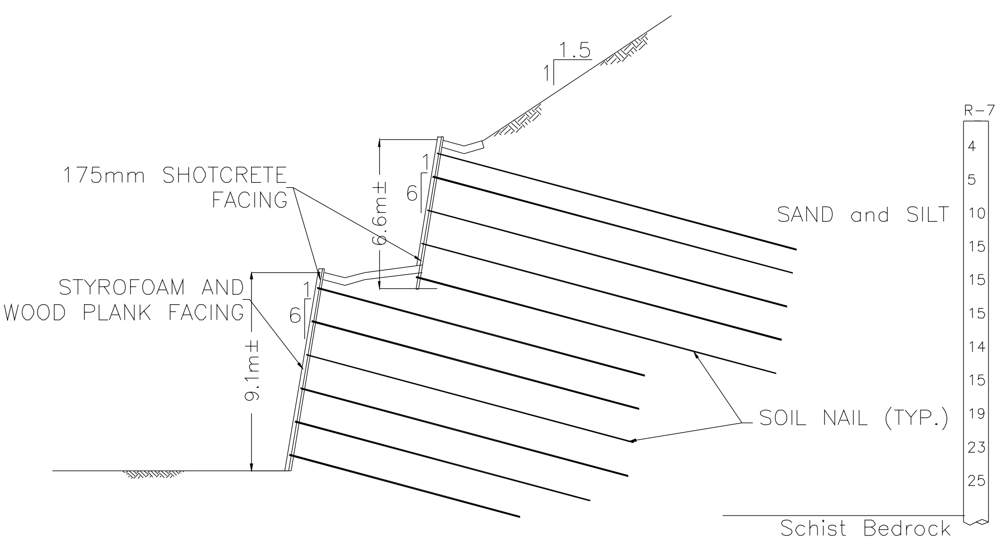


Figure 2

SECTION THROUGH SOIL NAIL WALL
MORETOWN, VERMONT

accomplished on the same day as the face of the cut is exposed thus the soil must typically stand vertical for 6 to 24 hours. The Vermont Agency of Transportation (VTrans) excavated test pits prior to the bid and invited selected specialty contractors to view them to assess the ability of the soil to stand vertical. This helped ensure that the soil nailing would be successful. In addition, the VTrans made periodic inspections of the test pits to see if there was any sloughing of the soil with time. Their observations were made available to the bidders.

The wall was constructed using conventional procedures for constructing soil nail walls. First a vertical cut was made in the side of the hill at the back face of the wall. The soil at this site stood up well for the cut heights required, which varied from 1.2 to 1.5 meters, and confirmed the observations made in the pre-bid test pits. Drainage board was attached to the face of the cut to become part of the permanent drainage system behind the wall.

The soil nails were installed using specialized equipment capable of drilling from the narrow benches that were cut into the side of the hill. The majority of the holes for the soil nails were drilled open hole using a sectional auger. Casing had to be used at some locations where the holes would not stand open. Around 700 soil nails varying in length from 6.1 to 16.8 meters were installed. The nominal spacing of the nails was 1.5 meters horizontally and 1.5 meters vertically. The soil nail tendons consisted of Grade 60 #8 and #10 all-thread bars. The tendons were encapsulated in a shop-grouted corrugated PVC tube to resist corrosion. The annulus between the soil nail and drill hole was tremie grouted using a water and cement grout.

A testing program was conducted on the job to verify the adhesion value of 16.78 kN/M that was used in the design. Verification of the adhesion value is important to substantiate that the required factor of safety is being achieved. If the adhesion values are lower than expected the nail lengths may have to be lengthened to achieve the required factor of safety. The ability to modify the design during construction is another important benefit of soil nail walls.

Verification nails were installed at twelve locations. Seven of the verification nails were tested to pullout failure. These verification nails failed at between 2.75 and 4.0 times the design capacity. The remaining five verification nails were tested to between 2.5 and 4.5 the design capacity and did not experience failure. The verification tests confirmed the adequacy of the adhesion value used in the design calculations. In addition to the verification tests, proof tests to 150% of the design value were conducted on thirty-three additional nails.

Reinforced structural shotcrete was installed at each lift as wall construction progressed. A bearing plate and nut were used at the end of each nail to connect the nails to the wall and reduce the possibility of punching failure. Once the wall was complete to subgrade the General Contractor installed the insulation and wood plank facing to the face of the shotcrete.

The wall was constructed with relatively little difficulty. Survey monitoring of the wall and readings taken in two inclinometers installed behind the face of the wall showed no measurable movement. The completed wall is shown in Figure 3.

This project illustrates one situation where soil nailing is often used. Cutting into a hillside where it is not possible to slope back to construct a conventional gravity wall requires the use of smaller benches for equipment. Equipment used for soil nailing is generally smaller than that used to install sheeting or soldier piles and thus is frequently the best method for constructing retaining walls in the side of a hill.

Figure 3 - Route 100 B Soil Nail Wall

Permanent Tieback Wall – Route 321 Gatlinburg, Tennessee

The permanent tieback walls in Gatlinburg, Tennessee were constructed into the sides of Route 321 to allow the existing roadway to be widened from two lanes to four with a new bicycle path. A total of thirteen tieback walls were constructed on the project. Several of the walls were located adjacent to land within the Smokey Mountain National Park. The tieback walls varied in length from 70 to 300 meters long with heights up to 12 m. The total exposed face of the walls was around 18,500 square meters. The tieback walls were designed and constructed under the supervision of the Tennessee Department of Transportation (TDOT).

Subsurface conditions at the site consist of sandy silt over highly fractured siltstone. The siltstone varied in depth from 0 to 2 meters below the ground surface. Since the majority of the walls were being built through the highly fractured siltstone, extensive mapping of the rock joints was conducted to identify potential failure planes in the rock mass. These joint sets were then used to evaluate the pressures that the rock would exert on the tieback walls. Two types of rock mass failures were considered in the design of the tieback walls. Both 2-dimensional and 3-dimensional wedge analyses were performed to address these two failure modes. A 2-dimensional analysis was used to determine design loads resulting from rock wedges formed by a single joint set. A 2-dimensional wedge analysis produces accurate loads for joints dipping towards the wall that strike within 5^o, and conservative loads for joints that strike between 5^o and 20^o. In areas where a single joint did not dip towards the proposed wall, a 3-dimensional analysis was used to determine the potential loads produced from the rock wedges formed by the intersection of two joint sets. In order to be valid, the two critical joint sets must form a line of intersection that plunges downward towards the wall. These two critical joints create a wedge of rock that can slide towards the wall. This wedge was analyzed to determine the loads that it produces on the wall. A typical joint set is illustrated in Figure 4.

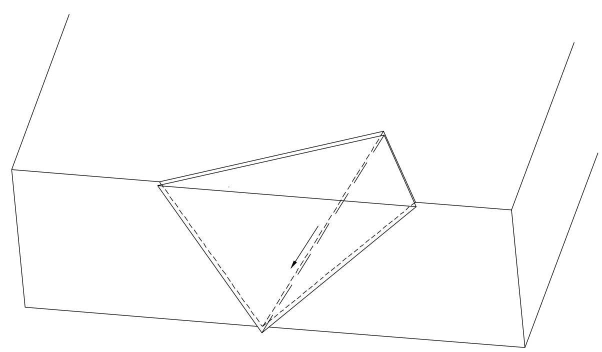


Figure 4

TYPICAL WEDGE SHAPED FAILURE SURFACE

Once the design was approved by TDOT, construction on the walls began. All work was performed while maintaining two lanes of traffic on the existing roadway. Concrete barriers were used to separate the work area from the traffic flow. The walls were constructed by first installing the soldier piles, spaced at 3 meters on center, on a small bench located at the top of the proposed wall. The soldier piles were installed using a down-hole hammer suspended from a crane located along the edge of the existing roadway. The hammer and crane were configured so that they could reach up to the top of the wall (Figure 5). After drilling of the hole was completed a steel soldier pile, which varied in size from an HP10X42 to a W10X49, was inserted into the hole and backfilled with lean concrete.

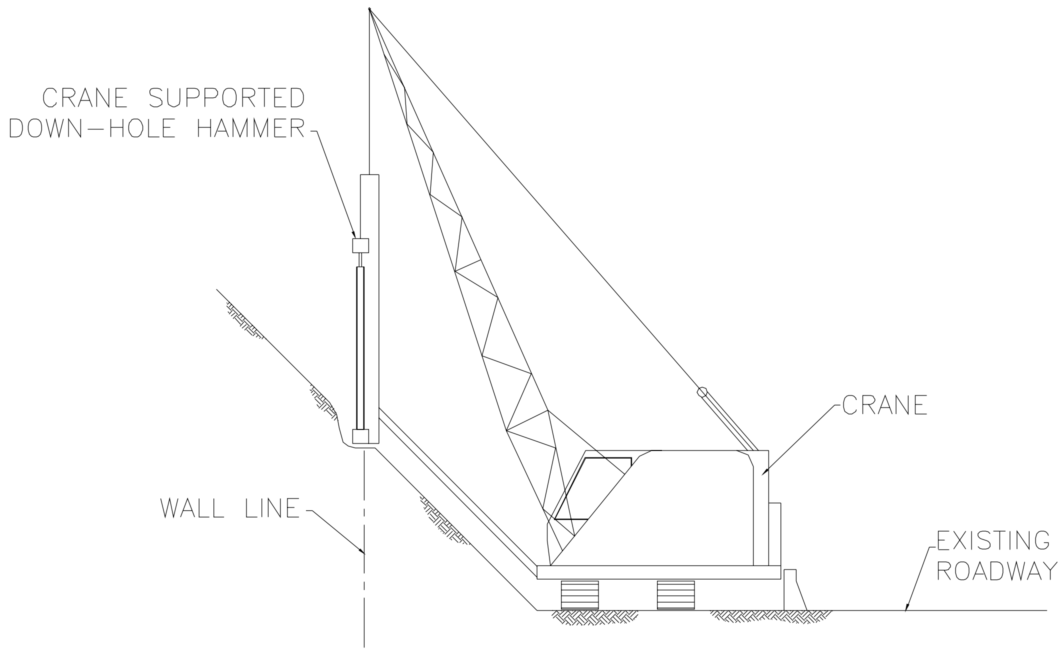


Figure 5
INSTALLATION OF SOLDIER PILES IN GATLINBURG

After the soldier piles were installed, the material in front of the wall was removed and tiebacks were installed as excavation in front of the wall proceeded. Timber lagging was used to support the soil and weathered rock and wire mesh was used to support the competent rock between soldier piles. The tiebacks were installed using a variety of equipment depending on access conditions. The majority of the holes for the tiebacks were drilled open hole using a down-hole hammer mounted on a set of crane supported leads. Tracked drills were used for the remaining tiebacks. Around 1200 tiebacks varying in length from 10 to 23 meters were installed. Tieback anchors were nominally spaced at 2 to 4.5 meters vertically at each soldier pile. The tieback tendons consisted of Grade 270 7-wire strands. The tendons were encapsulated in a grouted corrugated HDPE tube to resist corrosion. The annulus between the encapsulated tieback and drill hole was tremie grouted using a water and cement grout. The tiebacks were tested to design capacities that varied between 250 and 900 KN.

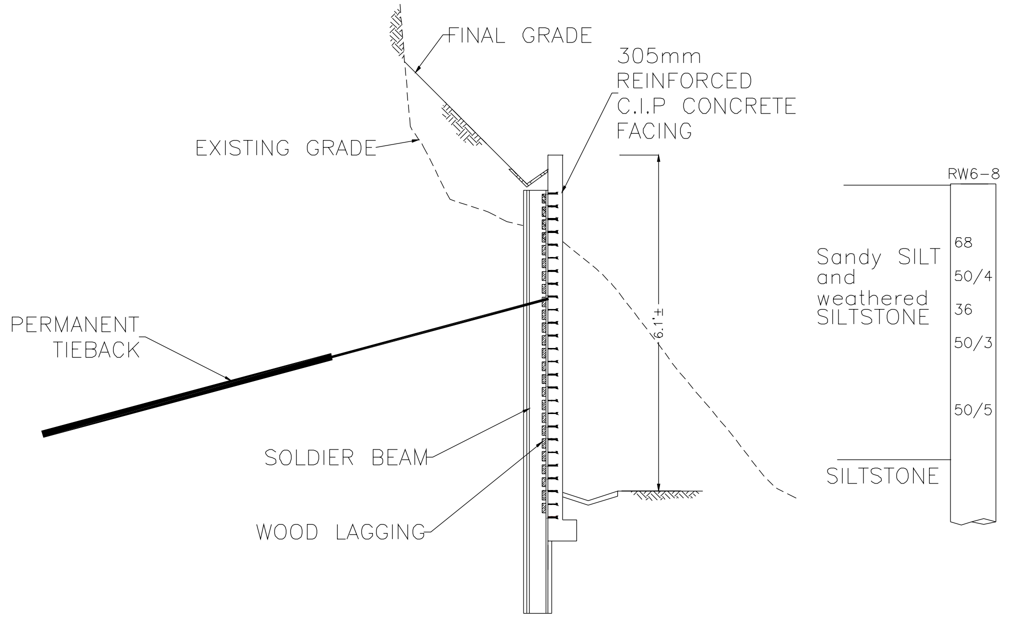


Figure 6
SECTION THROUGH TIEBACK WALL
GATLINBURG, TENNESSEE

A reinforced cast-in-place concrete face was then constructed on the front of the wall. The wall facing was tied to the soldier piles using headed studs welded to the face of the pile. A typical section through the wall is included as Figure 6.

When an ancient landslide failure surface was encountered during the construction of one of the tieback walls, a redesign was required. Two intersecting joints at the toe of the slope formed a large wedge that had a plunge of approximately 30º and was oriented perpendicular to the alignment of the wall. The sliding rock mass had an approximate width of 55 meters at the face of the wall. The wedge extended approximately 65 meters behind the wall and had an approximate height of 44 meters. Once the increased support requirement was recognized, a soil buttress was placed in front of the wall and this portion of the wall was re-assessed. Larger soldier beams were placed and longer tiebacks were then installed through the sliding mass into competent rock. One hundred and two (102) tiebacks having a design capacity between 735 and 935 KN were utilized. Thirty-four of the tiebacks were installed through a concrete waler located at the top of the wall and the remainder of the tiebacks were installed through the new soldier beams. The soil buttress in front of the wall was then removed as the tiebacks were installed. The wall was then faced with concrete to match the other tieback walls. The wall, prior to the application of the concrete facing, is shown in Figure 7.

The Gatlinburg walls illustrate some of the advantages of using tieback walls. As in the Moretown job, excavations were required into the sides of hills that would have made the use of conventional gravity walls impractical. By utilizing the tieback walls, right of way easements were minimized and the natural beauty of the surrounding environment was not compromised.

Figure 7 – Gatlinburg Tieback Wall

SUMMARY AND CONCLUSIONS

To summarize, soil nail and tieback walls are an increasingly popular method of providing permanent earth retention along roadways. These walls should not be used indiscriminately in all situations however. Instead experienced specialty contractors should be consulted early in the design process to assist the engineer in evaluating whether site conditions are suitable. In addition, contracting methods should ensure that only experienced specialty contractors are allowed to do the work. The two walls discussed in this paper are all performing well and were cost effective.

REFERENCES

Byrnes P.E., Dr. John R.; J. L. Walkinshaw P.E., et al. (1982). "*Soil Nailing Summary Report*" FHWA International Scanning Tour for Geotechnology,

Rock Slopes: Design, Excavation, and Stabilization FHWA-TS-89-045. DOT, FHWA. 1989.

Schlosser, Francois, and Myles, B. (1991). "*Recommendations Clouterre 1991* " French National Research Project Clouterre.

Schnabel P.E., Harry W., (2000). "*Soil Nailing – A Contractors Perspective*". Presented at the ADSC Earth Retention Seminar in Peabody, MA.

Schnabel P.E., Harry W., (2003). "*Construction of Soil Nail Walls in the Northeast United States*". Presented at the MIT Soil and Rock America Conference in Cambridge, MA.

Effective Interpretation of Borehole Inclinometer Profiles: What is Really Slope Movement and What is Probably Something Else

By Nancy C. Dessenberger and Francis E. Harrison, Golder Associates Inc., Lakewood, Colorado

Abstract

Borehole inclinometers are a valuable tool for monitoring ground movements at depth, tracking movement rates, and defining the location shear failure planes. Tracking the locations and rate of change of the measurements allows one to track landslide movements in both space and time.

Processed inclinometer measurements are generally displayed as a borehole profile. The profile is essentially a reflection of the casing shape, as it is deformed by various forces in the subsurface. Ground movements due to phenomena such as landslide creep are but one of the forces acting on the casing. Other forces include settlement of the casing and irregular deformation of the casing due to installation challenges or variable borehole conditions. Added to this can be errors and inaccuracies due to drift in the instrument calibration, cable stretch, temperature variation, instrument damage, and changing the probe used. It is the job of the interpreting geologist/engineer to "filter out" factors not relative to ground movements, so that an accurate interpretation of landslide behavior can be made.

This presentation explores the forces and factors which affect inclinometer profiles, examines actual measured profiles to discuss how these may appear on the profiles, and provides suggestions for effective interpretation.

Introduction

Borehole inclinometers are a valuable tool for monitoring ground movements at depth, tracking movement rates, and defining the location shear failure planes. The most common application of these tools is to determine the locations of failure planes and monitor the rate of movement in landslides. Inclinometers have also been used to monitor related types of ground deformation such as lateral spreading and 3-dimensional consolidation. The inclinometer measurement system typically consists of a specialty borehole casing which has vertical grooves for its entire length, internally flush joints, and is installed in a drilled borehole. The inclinometer is measured utilizing an inclinometer probe, which is lowered into the casing, with readings taken at regular intervals as the tool is pulled back up the casing.

Inclinometer probe tools have been developed which include specialty functions such as horizontal borehole probes, probes for measuring spiraling of the casing grooves (mostly applicable in deep inclinometers), and inclinometer sensors which remain in the borehole to allow continuous recording. For purposes of this discussion, we considering the use of a standard type of inclinometer probe, equivalent to the Model 6000 produced by Slope Indicator.

Similar equipment is produced by other manufacturers such as Geokon, RST, and others, and the general concepts of the following discussion also apply to those systems.

How Inclinometers Function

An inclinometer probe works by sensing its own angle of tilt relative to the gravitational pull of the earth. The probe unit contains accelerometers (gravity-sensing transducers), which sense the tilt angle relative to vertical. Typically, the probe is lowered to the bottom of the borehole via its electrical data-transmission cable, and as the tool is pulled up the specialized inclinometer borehole casing, readings are taken at regular intervals. The accelerometers produce a voltage output which varies with tilt angle and is recorded by the readout device.

In the biaxial probes most commonly in use, two accelerometers are offset at 90 degrees to allow measurements in two perpendicular planes. The primary measurement plane is coincident with the guide wheels of the probe, which ride in the grooves of the casing. Thus is it important to orient one set of grooves in the direction of expected displacement. The secondary measurement plane, at 90 degrees to the primary plane, is measured by the offset accelerometer to detect deformations perpendicular to the primary movement direction. This allows a three-dimensional interpretation of ground movements. Typically, two sets of measurements are taken in a reading event; both with the guide wheels in the downslope-oriented grooves, but reversed 180 degrees for the second pass. Reducing the data based on the differences between the 180-degree readings minimizes systematic instrument errors and aberrations due to casing irregularities.

Data Reduction

Field records consist of voltage output of the accelerometers. Each voltage reading, at each measurement level in the casing, is translated into a tilt angle (θ) measurement. Calculating the length between readings (typically measurements are taken at 2-foot increments) multiplied by $\sin \theta$ yields the incremental deflection of the casing. Adding the incremental deflections over the length of the casing yields the cumulative deflection. A plot of the cumulative deflection over the length of the casing, or borehole deflection profile, is what is most commonly used for interpretation of ground movements.

Interpretation of Deflection Profiles

Figure 1 presents cumulative and incremental deflection profiles for a location which has experienced a “classic” shear plane offset at a depth of about 110 feet. The plot of cumulative deflection (left side) reflects an approximate “picture” of the deformed casing, albeit at a greatly exaggerated horizontal scale.

The right side of Figure 1 presents the same data as a plot of incremental deflections. This plot does not reflect the actual casing shape. Instead, it shows the amount of deflection that has occurred between successive readings. Thus, for the same data set shown in Figure 1, we now see a deflection only at the shear surface location, with essentially zero deflection shown for the remainder of the borehole. The incremental plot is useful for isolating the magnitude of deflection at a specific location along the casing, without the accumulated effects of other displacements.

Figure 2 shows the same borehole with additional data sets collected over a period of about one year. From this plot, the progressive movement along the shear plane over time is apparent. Relative magnitude of movements over time, including seasonal variations, are apparent on the plot of cumulative deflection. To determine more exact magnitude of movement at the shear plane location, the incremental plot can be examined.

Figures 1 and 2 show examples of movement isolated along a distinct shear surface, such as along a bedding plane failure. Many types of ground movement are not this clear. Figure 3 shows an example of deep-seated creep movement, where soil near the ground surface is moving downslope at a faster rate than deeper layers. Here the cumulative profile shows a progressive increase over the zone of movement. The incremental plot shows that the deflections between measurements are quite small and relatively constant over the length of the casing.

Figure 4 shows combined effects of possible shear plane development at a depth of about 52 or 54 feet, with downslope creep indicated above the shear plane. This type of movement can occur in slide deposits where a weakened soil mass creeps downslope along a saturated and/or sheared basal zone.

Patterns Which are not Actual Ground Movement

Figure 4 also exhibits some features which produce distinct indications of displacement, but are not likely due to significant downslope ground movements. The large deflections at the top of the borehole, above and near the ground surface (zero depth), are due to surface effects, from surficial freeze-thaw, changing moisture, erosion, and human effects. The weight of the inclinometer probe and cable assembly is enough to produce significant apparent deflections near the ground surface. Typically, the upper portion of the profile is neglected in interpretation. If shallow-seated deflections are to be measured with an inclinometer, special measures need to be taken to immobilize the casing to well above the ground surface.

Figure 4 also shows how variation in successive readings can give a false sense of movement. Examining the readings from 4/17/2001 and 6/4/2001, it would appear that a significant downslope movement trend was developing. However, subsequent measurements indicate the casing “deforming back uphill” to near its former profile; not a likely circumstance. These variations in the measurements are most likely due to instrument sensitivity and repeatability. While these effects can be minimized by using a well-adjusted probe and operator care, they are hard to completely eliminate when only small deflections are detected. Diligence by the measurement operator can significantly reduce these effects, with attention to detail on consistent placement of the cable bracket in the same location, allowing the probe to equalize in temperature at the bottom of the hole before beginning readings, and being exact with measurement points when using the cable markings. Most important, care must be taken against prematurely interpreting small magnitude deflections.

Casing settlement is a phenomenon that often causes confusion in interpretation of deflection profiles. The plot in Figure 5 shows several displacement indications, both real and false. The surface deflection of the casing and shifting of the borehole alignment due to repeatability problems are both apparent in the upper portion of the profile. However, the most dramatic deflections are seen in the interval between about 80 and 130 feet. Several prominent "zigzags" occur in this interval, in both the primary (A) and secondary (B) measurement axes. These features are most likely the result of casing settlement. This particular borehole is a relatively extreme example of this phenomenon. The drilling was difficult at this depth in the boring, with caving hole conditions, stuck tools, and other factors that probably left a highly irregular and damaged borehole wall. Completion of the installation was probably affected by these conditions, and it is likely that grouting of the annulus was incomplete and of poor quality. The B-axis plot suggests that most of the settlement may have occurred in the spring following installation, rather than immediately. (This fact is masked on the A-axis plot by poor repeatability and lower-magnitude deflections.) A key clue in identifying settlement phenomena is the fact that the upper portion of the deflection profile returns to near the zero axis above the disturbance.

However, the plot in Figure 5 also indicates likely real shear plane displacements. The portion of the plot below 130 feet shows progressive offsets along a shear plane at about 152 feet. This lower portion of the hole appears very stable in terms of settlement "noise" on the plot. Probably the lower portion of the casing is within better-quality rock which allowed a stable borehole and took the grout well. However, given the repeatability issues with measurements in this casing, and the relatively small magnitude of the deflections indicated at 152 feet, it may be premature to draw conclusions on the nature and magnitude of the actual displacements.

Sometimes actual ground movement can be misinterpreted. For example, inclinometer casing is often installed through fills or embankments to monitor performance. If the fill induces differential settlement (such as might by attributable to uneven consolidation of underlying material), the fill mass may tend to tilt, and the inclinometer probe will then measure this tilt. The cumulative deflection profile will then typically display a "broken back" shape, with the hinge point approximately at the interface between fill and natural ground. In this case, the probe is truly measuring ground movement, but it is not likely to be shear. The probe is measuring tilt or rotation of the fill due to differential settlement.

Other errors can be introduced into the data, and thus result in inconclusive or confusing profiles. Examples include spiraling of the casing during installation, "drift" of the instrument calibration, indicator cable stretch, or damage to the probe unit. Analysis of these phenomena is difficult at best and is not considered herein. Some of these errors can be minimized by diligent care in handling and using the inclinometer probe, installing the casing, measurement procedures, and regular calibration checks.

Conclusion

Inclinometers are a wonderful tool that allows us to "see" what is happening with ground movements deep below the surface. There are many details and nuances that can be discerned from inclinometer deformation profiles. However, careful thought about what the instrument is actually detecting and how it reflects actual ground movements is essential to avoiding "false alarms", or leaving important information undetected. The following are key points to remember in interpreting deflection profiles:

- Deformation of the casing inside the borehole is what is actually being measured, which may or may not accurately reflect movements of the ground.
- Keep in mind the circumstances of borehole completion, installation, and grouting.
- Keep in mind the different ways that errors can be introduced through equipment and personnel changes.
- Keep in mind the scale of the apparent displacements in light of the instrument limitations and repeatability of measurements.

Finally, the interpreter must have a clear understanding of the geologic environment to properly interpret how the casing shape and deformation profile reflects what is actually happening in the ground.

References

Dunnicliff, John. 1993. Geotechnical Instrumentation for Monitoring Field Performance. New York: John Wiley & Sons, Inc.

Slope Indicator Company. 2000. *Manual for* Digital Inclinometer Probe 50302599. Bothell, Washington: Slope Indicator Company.

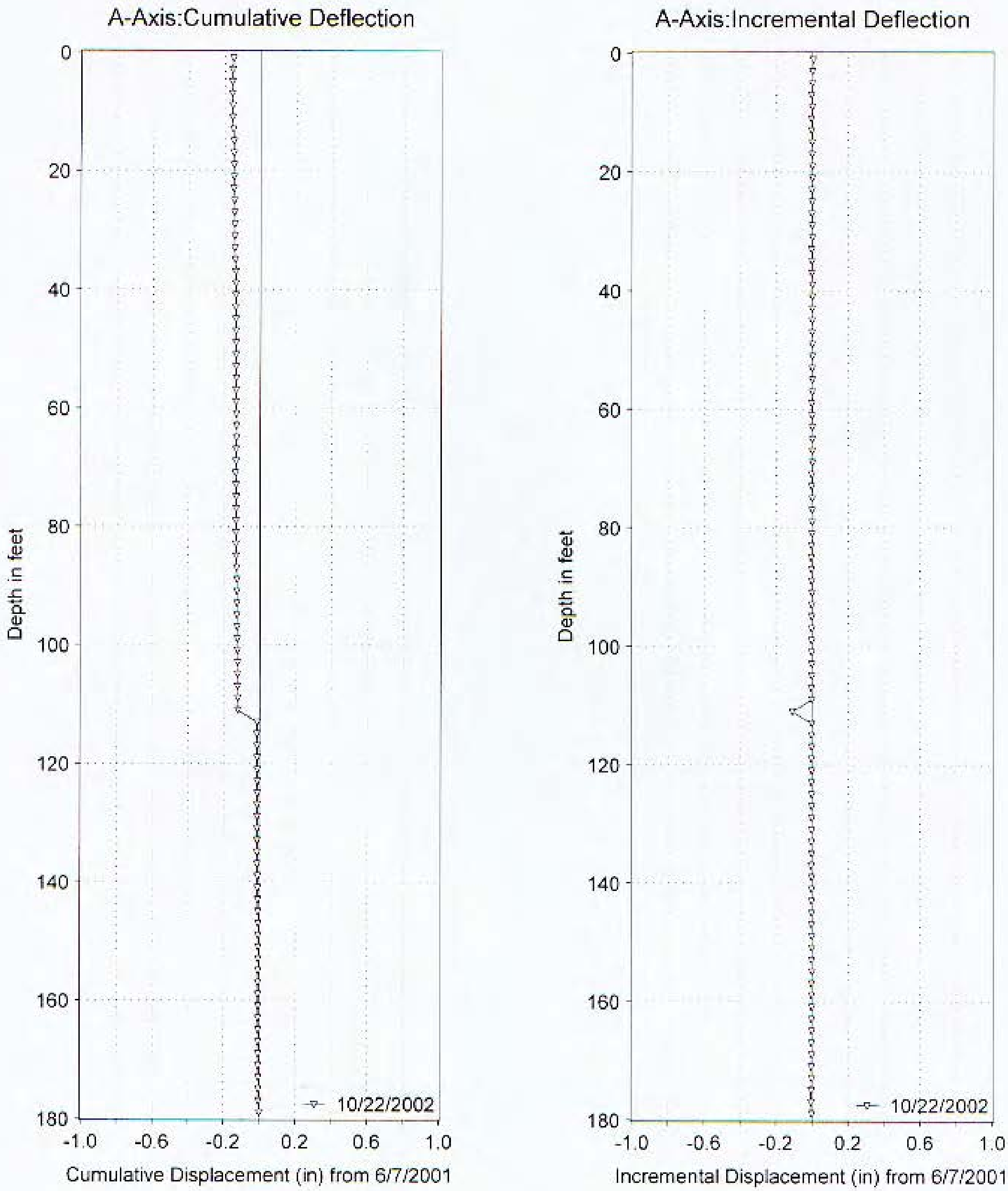

A-Axis:Cumulative Deflection
Depth in feet
0
20
40
60
80
100
120
140
160
180
-1.0
-0.6
-0.2
0.2
0.6
1.0
10/22/2002
Cumulative Displacement (in) from 6/7/2001
A-Axis:Incremental Deflection
Incremental Displacement (in) from 6/7/2001

FIGURE 1

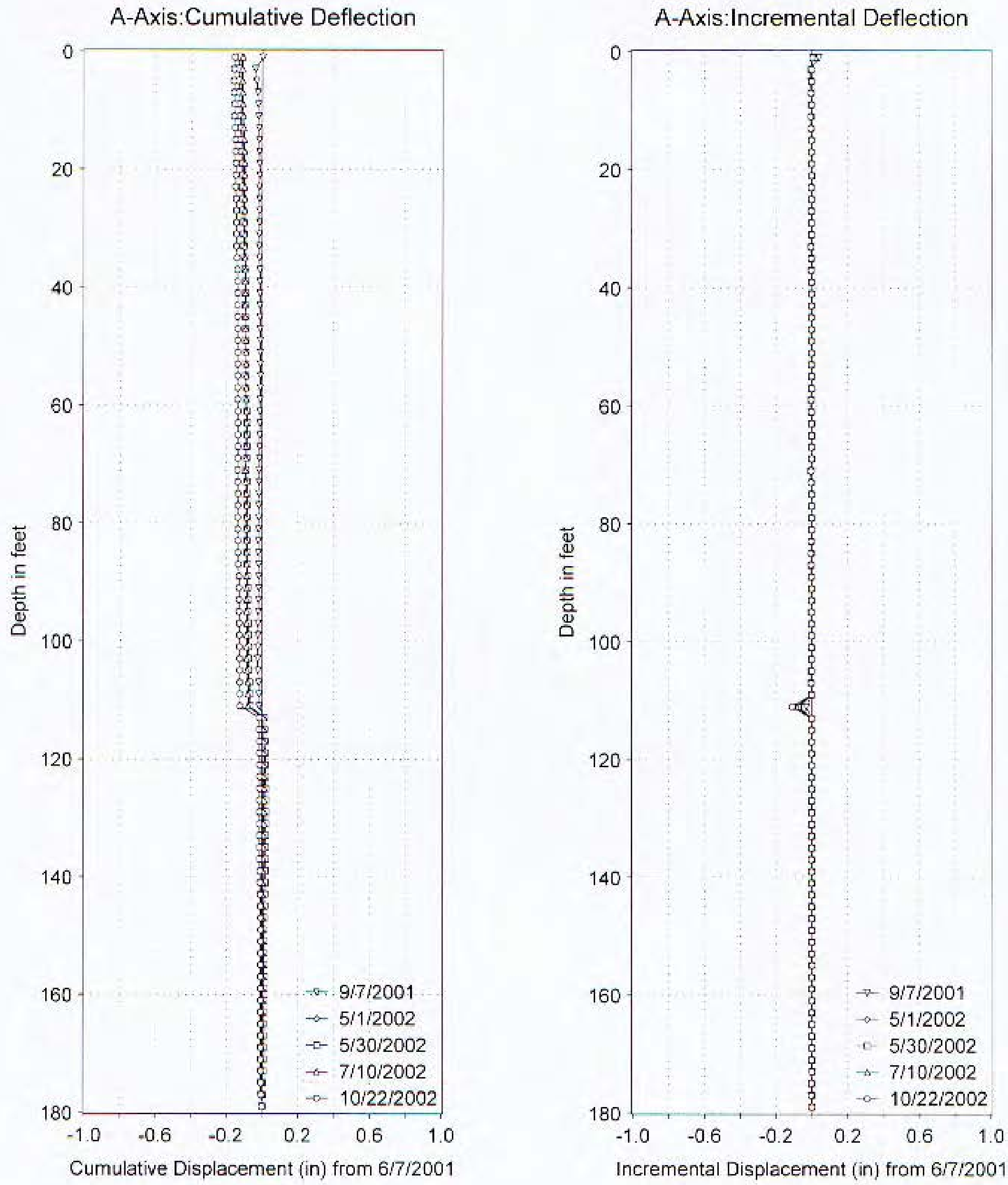

A-Axis:Cumulative Deflection
Depth in feet
0
20
40
60
80
100
120
140
160
180
-1.0
-0.6
-0.2
0.2
0.6
1.0
Cumulative Displacement (in) from 6/7/2001
9/7/2001
5/1/2002
5/30/2002
7/10/2002
10/22/2002
A-Axis:Incremental Deflection
Incremental Displacement (in) from 6/7/2001

FIGURE 2

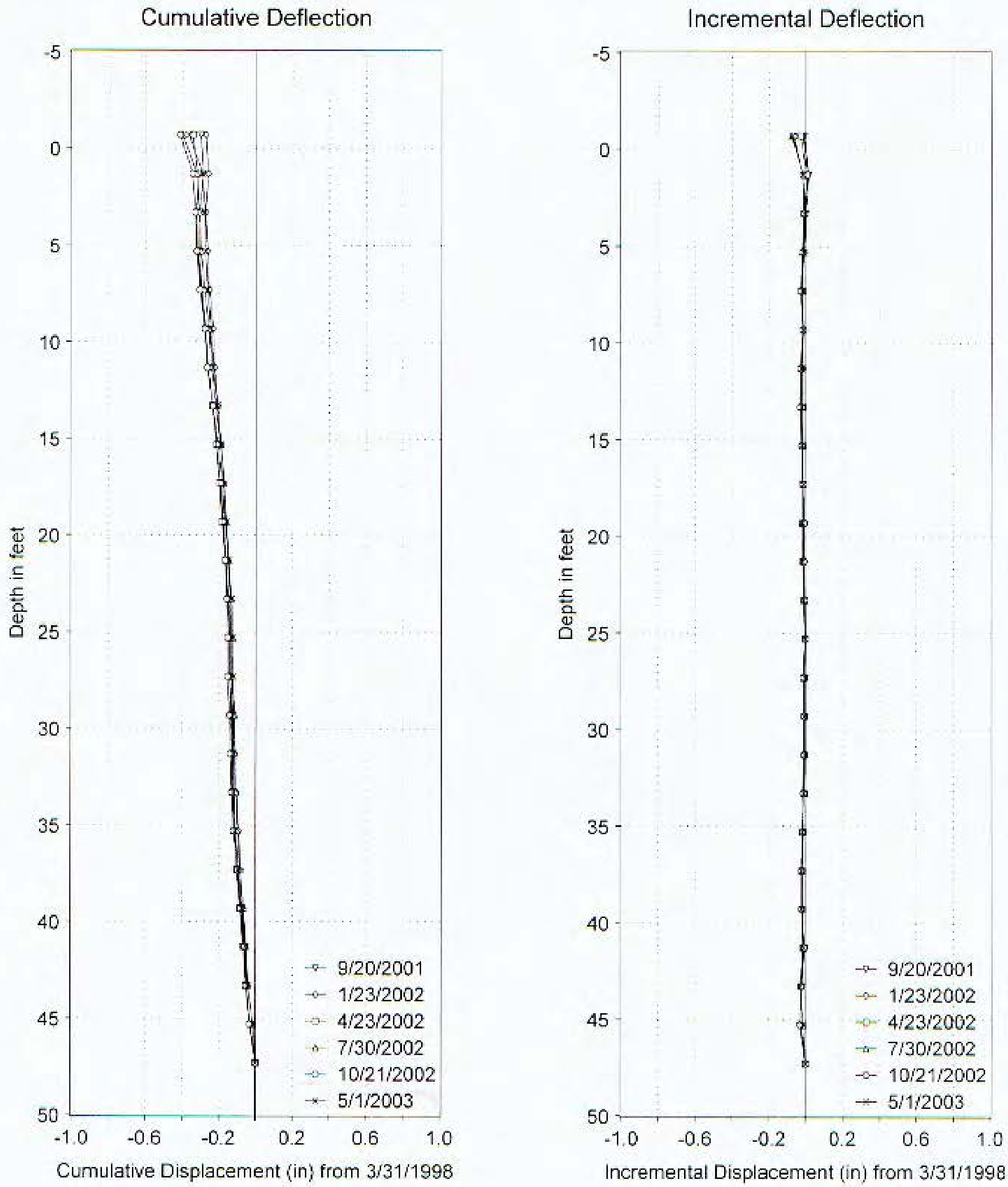
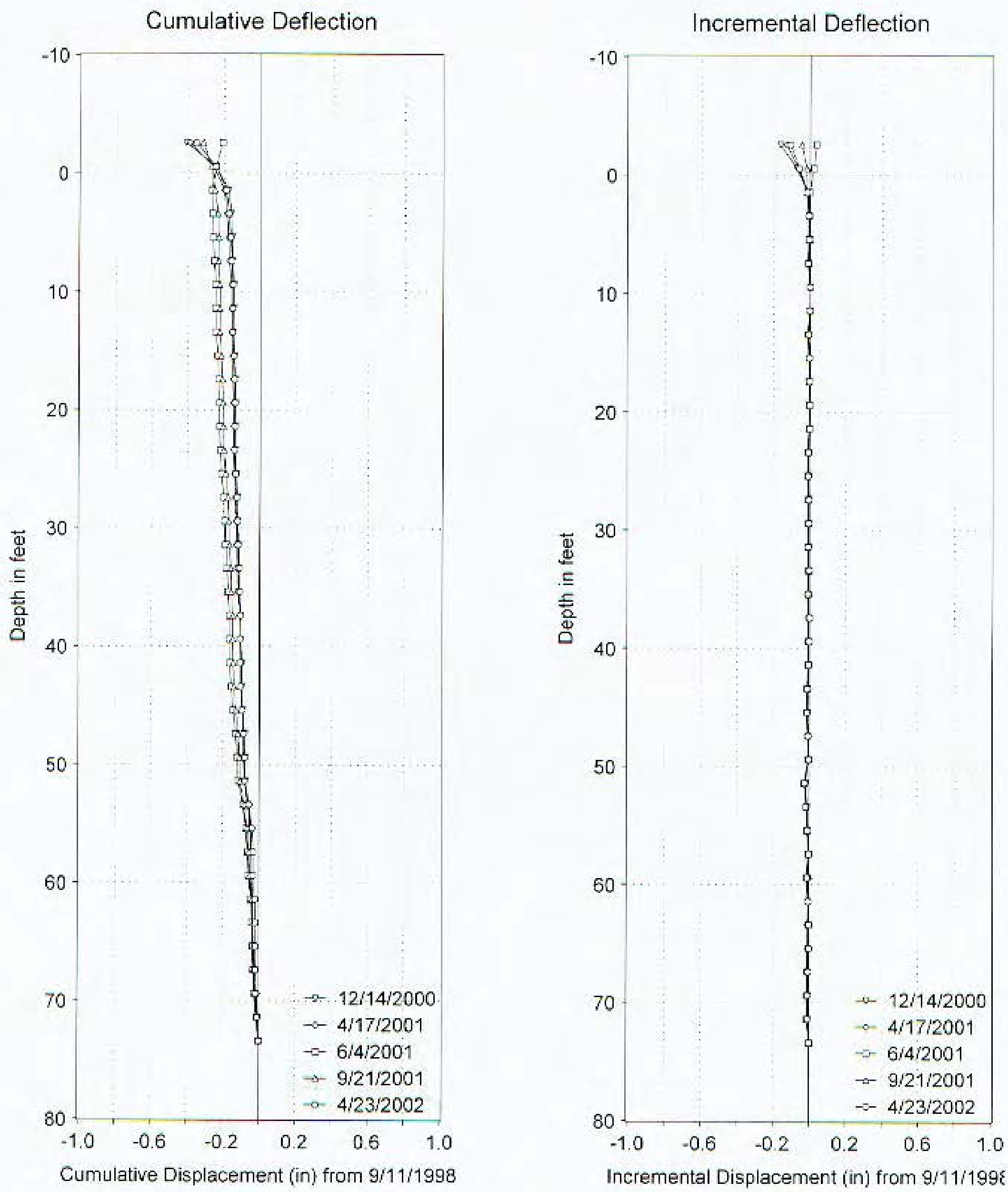
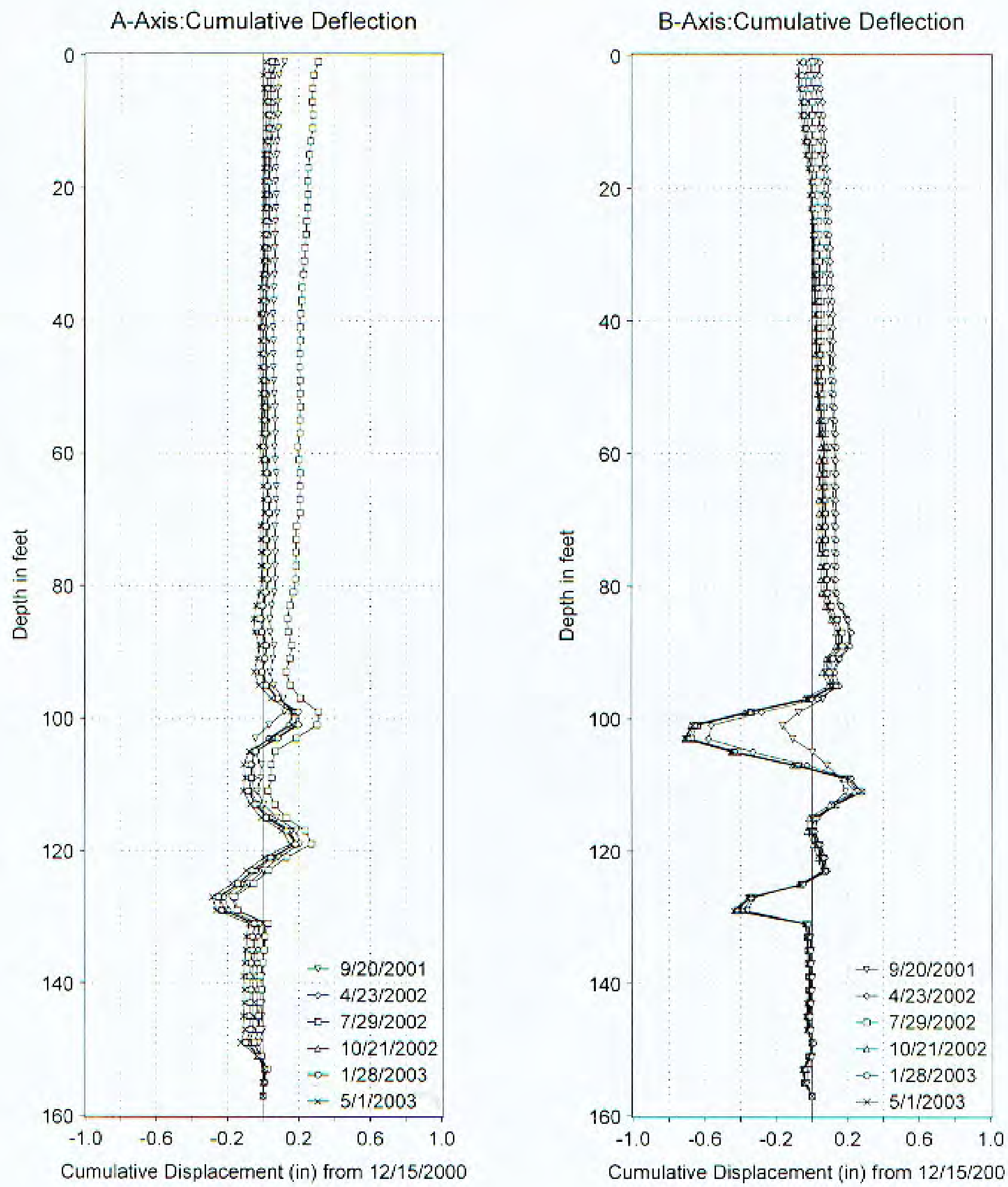

Cumulative Deflection
Depth in feet
Cumulative Displacement (in) from 3/31/1998
9/20/2001
1/23/2002
4/23/2002
7/30/2002
10/21/2002
5/1/2003
Incremental Deflection
Depth in feet
Incremental Displacement (in) from 3/31/1998
9/20/2001
1/23/2002
4/23/2002
7/30/2002
10/21/2002
5/1/2003

FIGURE 3



Cumulative Deflection
Depth in feet
Cumulative Displacement (in) from 9/11/1998
12/14/2000
4/17/2001
6/4/2001
9/21/2001
4/23/2002
Incremental Deflection
Incremental Displacement (in) from 9/11/1998

FIGURE 4



A-Axis:Cumulative Deflection
Depth in feet
Cumulative Displacement (in) from 12/15/2000
9/20/2001
4/23/2002
7/29/2002
10/21/2002
1/28/2003
5/1/2003
B-Axis:Cumulative Deflection
Depth in feet
Cumulative Displacement (in) from 12/15/200
9/20/2001
4/23/2002
7/29/2002
10/21/2002
1/28/2003
5/1/2003

FIGURE 5

Contingent Mechanisms of Stability and Collapse
"Old Man of The Mountains" Human Profile
Franconia Notch, New Hampshire
U.S.A.

Brian K. Fowler, CPG/NHPG
North American Reserve
67 Water Street, Suite 207
Laconia, NH 03246-3300

ABSTRACT

On May 3, 2003, the Old Man of The Mountains natural rock Profile collapsed, resulting in the unfortunate loss of the official emblem of the State of New Hampshire. A systematic reconnaissance of its stability had been performed in 1976 by the NH Highway Department as part of the environmental impact statement for Interstate 93. This reconnaissance estimated the Profile's in-place stability and its capacity to withstand blasting vibration from below. The work showed: (1) that the dead weight of the Profile's blocks acting at their combined center of gravity created a delicate stability; (2) that the Profile from the Nose up was more stable than from the Upper Lip and Chin down; (3) that the Profile was subject to toppling collapse if natural processes or dynamic stress disturbed it; and (4) that blasting could take place beneath it if no vibration in excess of those in the ambient natural environment was allowed to reach the rock mass. Careful blast monitoring during construction of I-93 between 1985 and 1986 showed this vibration objective was achieved. Most recently, analysis of the mechanism and cause of the Profile's collapse shows it was a progressive toppling failure initiated by a sudden loss of intact compressive strength in the granite immediately beneath the center of gravity of the cantilevered Chin, the granite's intact strength having been naturally compromised over time by kaolinization decomposition and freeze-thaw degradation.


FIGURE 1. Old Man of The Mountains from Franconia Notch. Viewpoint from Profile Lake, about 600 meters north and 500 meters below (1976; B.K. Fowler).

Introduction

On May 3, 2003, the Old Man of The Mountains natural rock profile (the “Profile”; see Figure 1 & Table 1), collapsed and fell approximately 250 m onto the talus slope along, and approximately 300 meters above, Interstate 93 in Franconia Notch, 120 km north of the State Capital of Concord. The collapse resulted in the loss of an important landmark and the official emblem of the State of New Hampshire. This natural event brought to a close an often sublime, nearly 200-year relationship between the people of New England and the “Old Man”, a relationship characterized by remarkable human effort to understand how it formed, the mechanism of its stability, ways to secure and preserve it, and how to embrace the humanistic and philosophical significance of its natural, but utterly “human” profile (e.g. Hawthorne, ca. 1840). This paper summarizes the results of the various “geotechnical activities” that have taken place on the Profile over the past 198 years with particular attention to those of the past twenty-seven (Schile, 1975; Fowler, 1982; Fowler, 1997). Table 2 provides a convenient historical summary (from Fowler, 1997).

As shown in Table 2, for more than 100 years and up to the mid-1970’s, activities concerning the Profile were undertaken mainly by private individuals relying on limited, mostly personal resources or by very limited governmental initiatives. In the mid-1970’s, this changed when better but indirect funding became available through public works projects located near the Profile (e.g. I-93). But, over the years, no substantial funding directly dedicated to the study of the Profile’s stability or long-term security was ever made available, principally because it was feared that field implementation of preservative schemes developed from such studies might themselves endanger the apparently delicate security of the important landmark. As a result, no formal or scientifically rigorous geomechanical study of this complex rock mass was ever made. As is so often true with “case histories” like this, the observations, recommendations, and conclusions reported, both then and now, are constrained by the nature of the work permitted by the time and money available.

Table 1

Dimensional Information
Rock Mass of The Actual Profile
Old Man of The Mountains

Height	13.7m	± 45 ft
Width	9.1m	± 30 ft
Thickness	19.8m	± 65 ft
Volume	2468m^3	81,156 ft^3
Weight (Mass)*	6471 tonnes	7190 tons

* 2643 kg/m^3 or 165 lbs./ft^3

Table 2

Chronology Of Human Involvements & Geotechnical Activities
Old Man Of The Mountains

DATE	PEOPLE & EVENTS
1805	First recorded sighting by surveyors scouting road locations in Franconia Notch.
1828	Gen. Martin Field publishes first widely-distributed article on the remarkable Profile and its sublime implications.
+/- 1840	Nathanial Hawthorne publishes his famous short story, "The Great Stone Face".
1853	The elegant Profile House is built near the present location of the Cannon Mountain Tramway. Its owner caters to patrons seeking inspiration from The Profile, and he declares its preservation to be vitally important.
1872	The Appalachian Mountain Club and a Boston newspaper of the day collaborate on a comprehensive article about the Old Man, including its apparently delicate structure.
1906	Rev. Guy Roberts of Whitefield begins a one-man, 10-year campaign to convince the local town fathers (the Notch belonged to the Town of Franconia then) to take some measures to secure and preserve the Profile.
1915	Rev. Roberts arranges a field meeting on the Profile with E.H. Geddes, an expert granite quarryman from Quincy, MA, and several local officials. Geddes agrees to furnish his expertise to “secure the rock mass” to preserve it; the local officials approve, but work is funded independently by Roberts and Geddes.
1916	The first short, 25 mm (1”) tie-rods are installed with hand-drilling techniques by Geddes on top of the Forehead slab to prevent its pieces from sliding or rolling off and upsetting the "center-of-gravity relationships” (all these devices remained in place until the collapse in 2003).
1928	Franconia Notch State Park is established by the NH Legislature.
1937	Geddes revisits the Profile to check his earlier work. He decides to install several additional tie-rods, seal-over several cracks where water seeps between the slabs, and add several poured-in-place cement blocks to provide baselines for detection of incipient movement between the slabs.
1945	NH Legislature makes the Old Man Of The Mountains the official State Emblem.
1954	State Geologist Ralph Meyers, Prof. Donald Chapman (UNH), and Director of NH Parks Austin Macauley make an official visit for the Legislature and report the Profile is very unstable, in spite of the good work done on top of the Forehead slab. They recommend detailed study of the blocks beneath to determine the Profile's true state of stability, its security, and thus its longevity.

Table 2-Continued

Chronology Of Human Involvements & Geotechnical Activities
Old Man Of The Mountains

1958	After much discussion (stimulated by fear the Profile might be so delicate that doing anything might knock it down) a series of long, 76 mm (3") turnbuckles are installed, this time with mechanical drilling equipment, between the two largest pieces of the Forehead slab to keep the front portion, with its perched crest block, from sliding off the Profile. Strain gauges are mounted on the turnbuckles to begin the first geotechnical monitoring on the Profile, but no reinforcement investigation or related work is undertaken on the critical blocks beneath, despite the 1954 report. Neils Neilsen and his staff at the Bridge Maintenance Div., NHDOT, begin their annual inspection & maintenance program that continued each year until the summer of 2002.
1965	General reconnaissance & natural-background seismic investigations of the Profile are made by NHDOT consultants and the U. S. Geological Survey, respectively, for I-93 planning. They find wind to be a frequent source of substantial vibration (particle velocities up to 12 cm/second) but conclude construction can proceed beneath the Profile, "if carried out very carefully", with blasting vibrations kept as far as possible below these ambient natural levels.
1975	Dr. Richard Schile, Thayer School of Engineering-Dartmouth College, and his students undertake the first systematic study of the blocks beneath the Forehead. This work is hampered by the inability to obtain accurately reproducible dimensions on the blocks comprising the rock mass while suspended from climbing ropes, and their mechanical calculations are not completed.
1976	Roger Martin and Brian Fowler (then civil engineer & engineering geologist, respectively, for the NHDOT) solve many of these rock-climbing problems and, with photogrammetric help through the I-93 EIS, finish the field work started by the Schile team and complete the first rudimentary structural-mechanical analysis of the Profile's support mechanism and state of stability. The study confirms construction can take place beneath the Profile if vibration reaching the rock mass is restricted by careful blasting design to the smallest fraction possible of the ambient natural levels observed in 1965.
1980	Franconia Area Heritage Council publishes <u>Saving The Great Stone Face</u> reviewing the history of efforts to preserve the Profile to that time (Hancock, 1980).
1982	Results of the 1976 study are published (Fowler, 1982).
1985-86	Construction of the I-93 Parkway in the Notch and beneath the Profile is undertaken and completed with no damage observed and no blasting vibration of more than 10% of ambient natural levels recorded at the Profile's rock mass.
2003	Profile collapses May 3rd.

Mechanics of The Profile's Stability

In 1976, at the request of the Federal Highway Administration, the NH Highway Department (now the NHDOT) directed the author (then an employee) to conduct a systematic reconnaissance of the structural geology and basic rock mechanics of the Profile. The work was conducted in conjunction with the preparation of the draft environmental impact statement (EIS) for Interstate 93 and its various alternatives through Franconia Notch, between Lincoln and Franconia, NH. It was prompted by general concern about the possible effects that the several design and construction alternatives for the highway might have on the well-known landmark, the overall stability of which had been a matter of considerable concern during the planning process. The purpose of the reconnaissance was to measure and document, for the first time, the actual dimensions and structural relationships among the blocks comprising the Profile; to make a basic assessment of its state of stability; to study ways in which dynamic stress, such as that from construction blasting, might affect the rock mass; and to make recommendations for the security of the Profile during the proposed construction below. The 10 weeks of field and office work in the summer of 1976 were carried out by geologists and engineers at the NHDOT who were also (conveniently for the project) experienced rock climbers.

Most earlier work on the Profile concentrated on the stability of the partially-separated, obviously unstable Forehead block and the smaller blocks on its surface (Table 2: 1916 & 1958). However, nothing had been done regarding the blocks underneath which were critical to the stability and security of the Profile. Work by Schile (1975) indicated for the first time that the state of static stress in the rock mass as a unit, and in these lower blocks in particular, might be delicate and that detailed dimensional and spatial information was needed for more complete analysis. The work described here started where Schile's work left off and resulted in a rudimentary but, as events would show, fairly accurate description of the mechanism and state of stability of the Profile.

Geologic Structure of the Profile

The Profile and each of the uniquely-shaped blocks comprising it were formed by fortuitous weathering and selective breakage along five discrete sets of structural features (joints, fractures, etc.) in its rock mass. Figures 2-5 illustrate and identify these features. Set 1 included all of the joints in a subhorizontal plane which cut through the rock mass and which were selectively fractured on their easterly edges to form the Profile view (Figure 2). Set 2 included the subvertical joints along which breakage had occurred to create the cliff face south of the Profile, while Set 3 included the joints along which the cliff was formed north of the Profile.

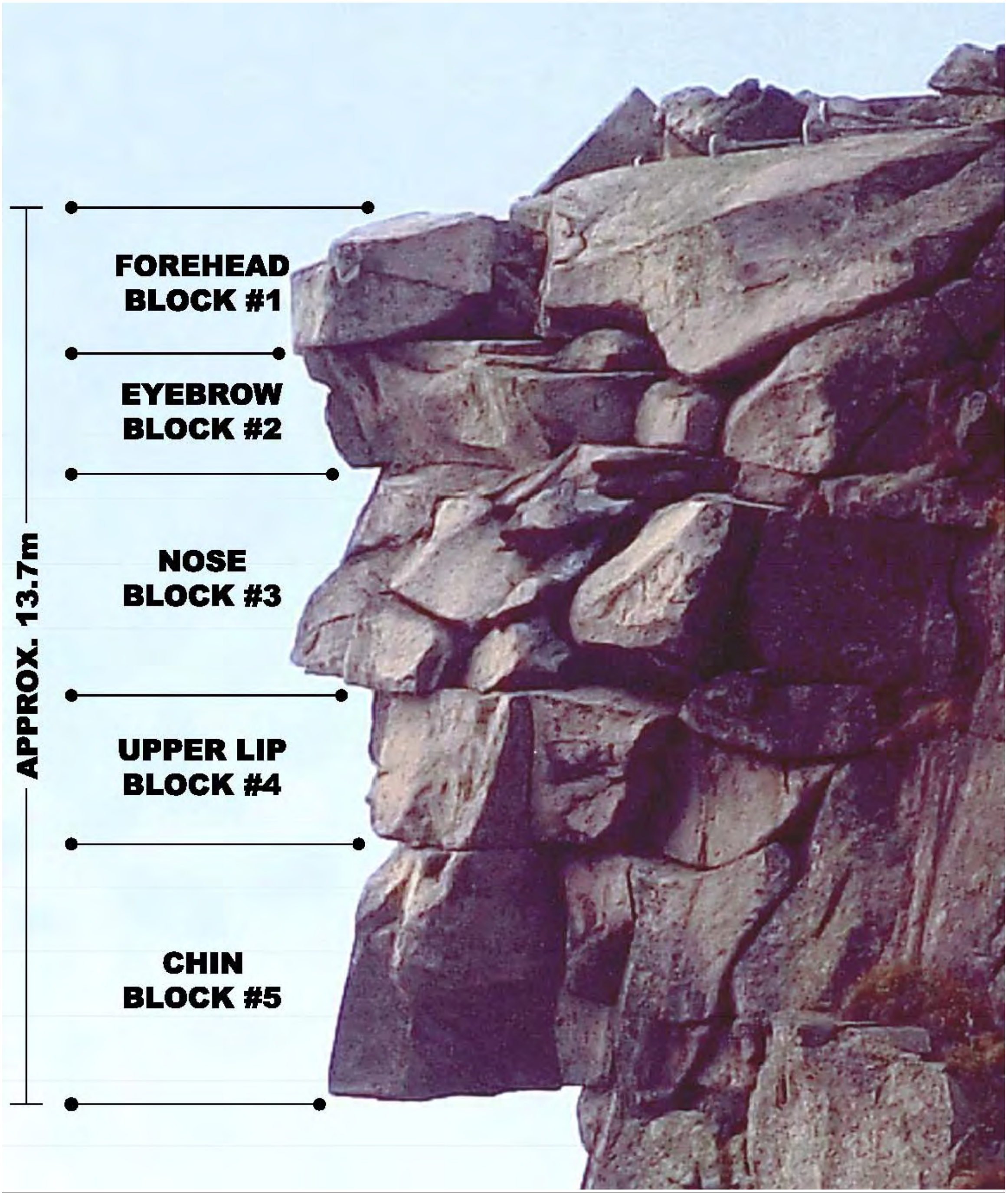


FIGURE 2. Telescopic view from Figure 1 viewpoint with Profile's component block combinations identified (2001; Associated Press – Jim Cole).

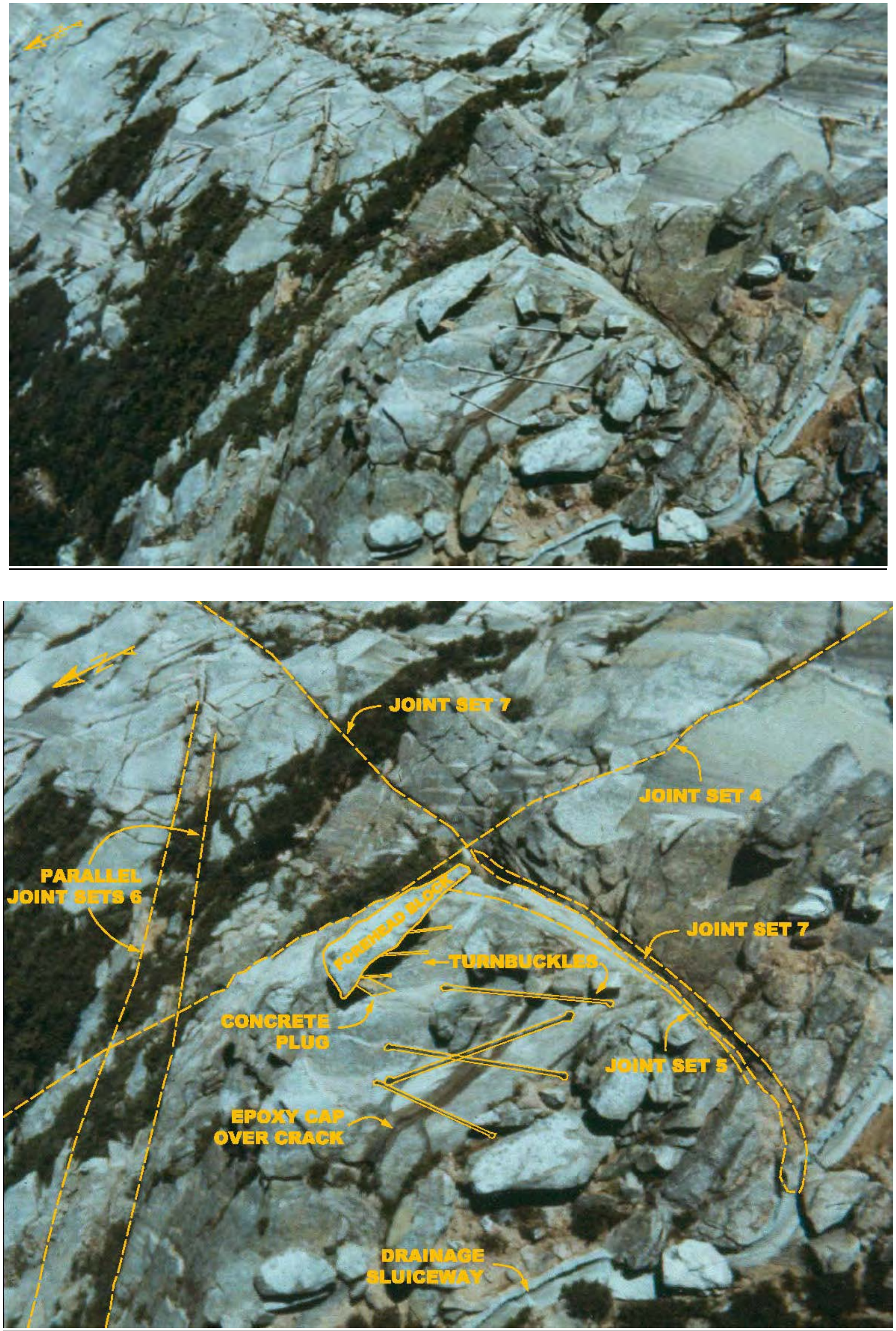


FIGURE 3. Profile's rock mass from about 30 meters directly overhead (top) and magnified view with some important structural geologic and man-made features identified (1976; Dick Hamilton).

Set 4, which included only one joint, was located such that it represented the likely cut-off joint on the easterly side of the blocks making up the Profile. Set 5, again containing only one joint, represented the south face of the Profile, and Set 6 represented the north face of the Profile's rock mass. Selective breakage along these last three sets (4-6) was responsible for the distinctly triangular shape of the Chin and Upper Lip blocks (Figure 4) which, as will be discussed later, were so important in the support mechanism of the Profile. Set 7, which was not directly part of the Profile, included (and still includes) a narrow fault-bounded shear zone that represented the structural geologic reason the Profile was allowed to develop on the cliff.

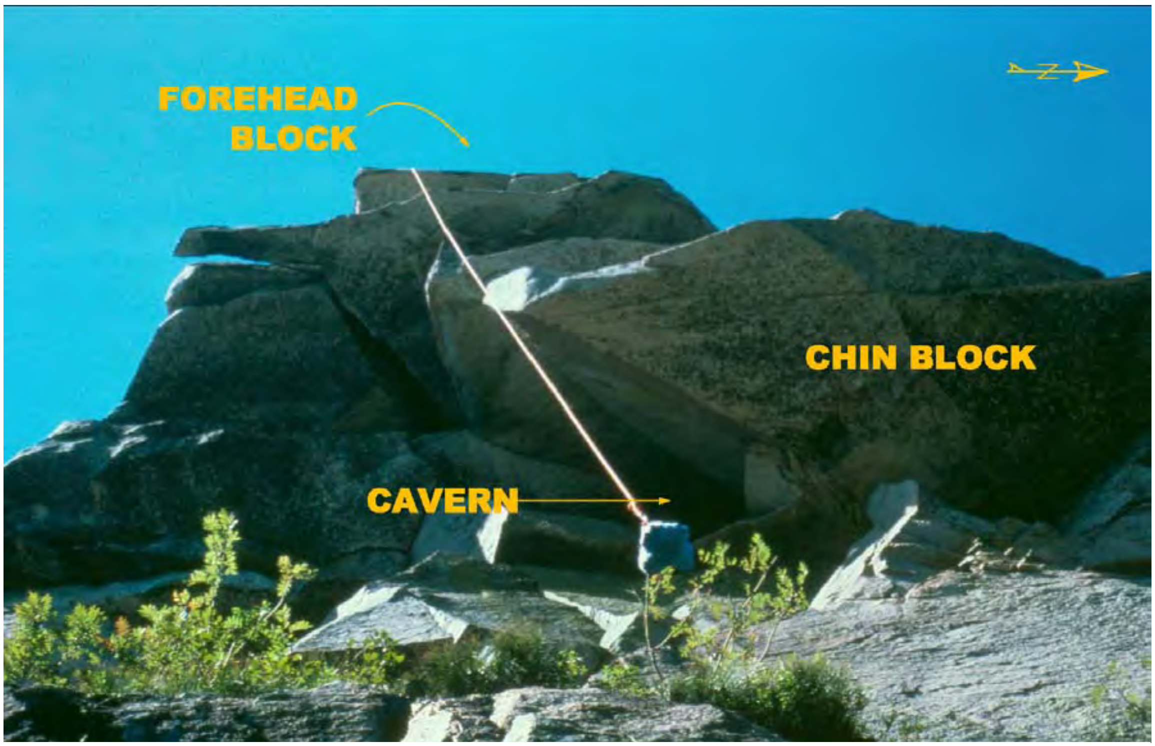


FIGURE 4. Profile's rock mass from about 25 meters directly beneath (1976; B.K. Fowler). Note location of the cavern and the roughly triangular-shaped "Chin" and "Upper Lip" blocks cantilevered in place at rearward tip of their combined mass.

The significance of Set 7 in the formation of the Profile was most interesting (Figures 3 & 5). The faulted shear zone at the junction of the south face of the Profile and the cliff face and a similarly oriented zone 15 meters to the south (Figure 6) are major structural features in the rock mass of the cliff. These two features appear to have been formed in the granite intrusion at some point in its early cooling history, but after the formation of transverse cooling joints within the intrusion. Shearing movement within Set 7 appears to have been a combination of linear and twisting displacement which resulted in a net rotation that, as can be seen in Figures 3 & 5, changed the orientation of the subvertical face-making joints north and south of the Profile from N20E to N35E (Sets 2 & 3).

This change was fortunate for the Profile's development because the resulting subhorizontal dip of joint Set 1 was then slightly into the plane of the cliff north of this zone instead of nearly parallel to the plane of the cliff south of the zone. If this serendipitous reorientation had not occurred, the dip direction of joint Set 1 would have been sufficiently parallel to the cliff face so the Profile's rock mass would have fallen away long before the Profile could have formed as the combined cliff-forming processes of weathering and freeze-thaw cycling proceeded.

This formation hypothesis was well-supported by a structural analysis that estimated the optimum strength direction in the bulk rock mass. The technique, a derivative of the friction-circle concept (Goodman, 1976), is illustrated in Figure 5 where circles have been drawn around the main pole to the average plane of each structural-feature set. The radii of these circles are equal to the estimated angle of internal friction for the granite comprising the Profile (35 degrees). According to the technique, the shared area common to the greatest number of circles represents the orientation at which the greatest strength is mobilized within the bulk rock mass. In the case of the Profile, this direction was N23E, 45NW. Thus, because the most important joints (Set 1) dipped back into the cliff in the general direction of this optimum strength orientation and because the Profile's mass configuration had not collapsed, it appeared likely its center of gravity was located somewhere just behind the junction of the lower cliff and the Profile, somewhere beneath the supported portion of the Chin, as illustrated in Figure 7.

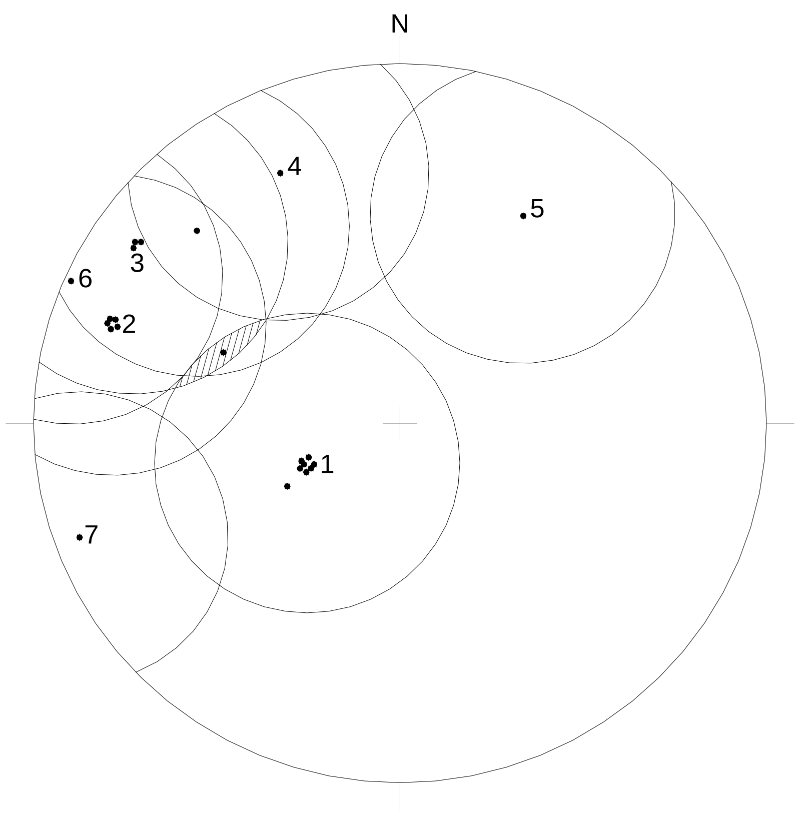


FIGURE 5. Pole diagram of structural features (lower hemisphere): Set 1, subhorizontal plane through blocks; Set 2, defines cliff face south of Profile; Set 3, defines cliff face north of Profile; Set 4, cut-off joint east of Blocks 2-5; Set 5, south face of Profile; Set 6, north face of Profile; Set 7, shear zone south of Profile.

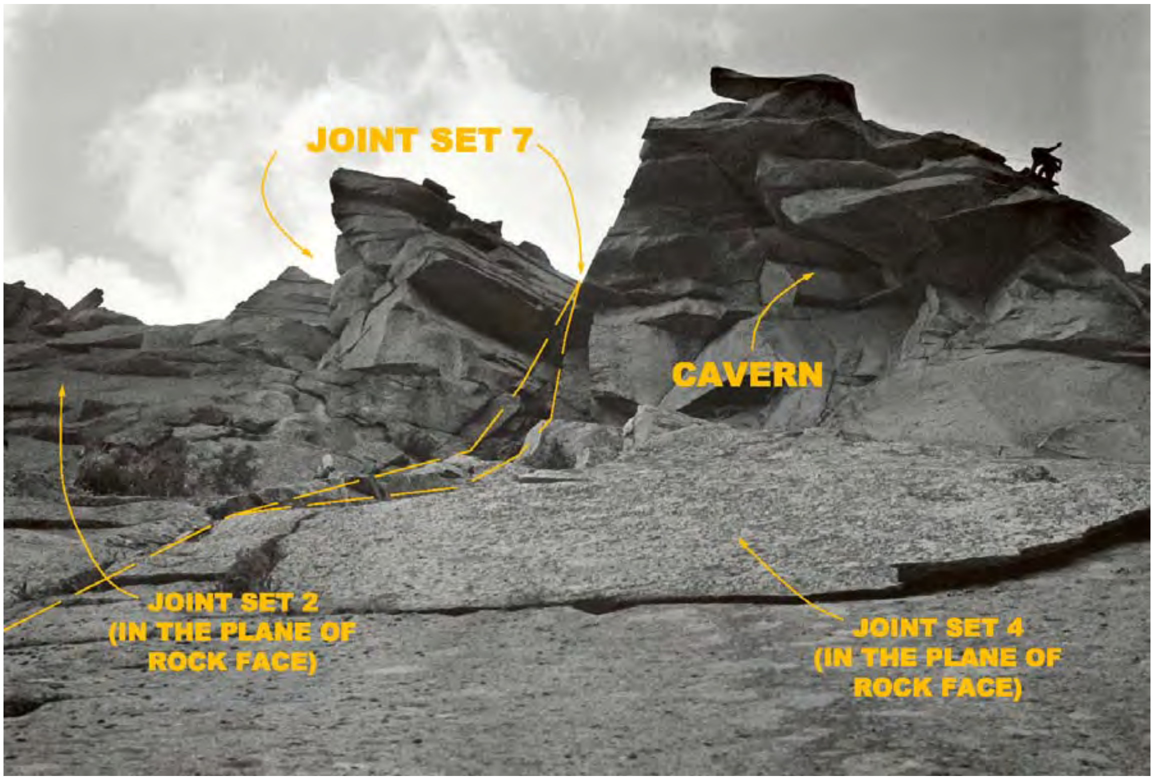


FIGURE 6. Profile's rock mass on the right (north) and adjacent rock mass on the left (south) showing the location and influence of the two parallel Set 7 structures on block breakage patterns on the cliff (1965; Stan Young).

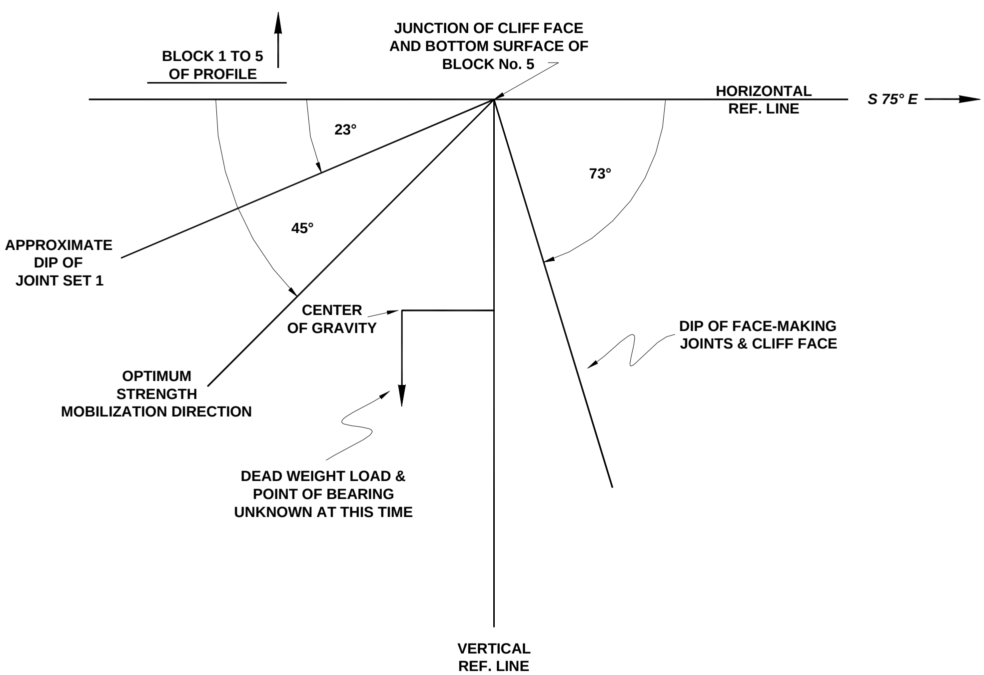
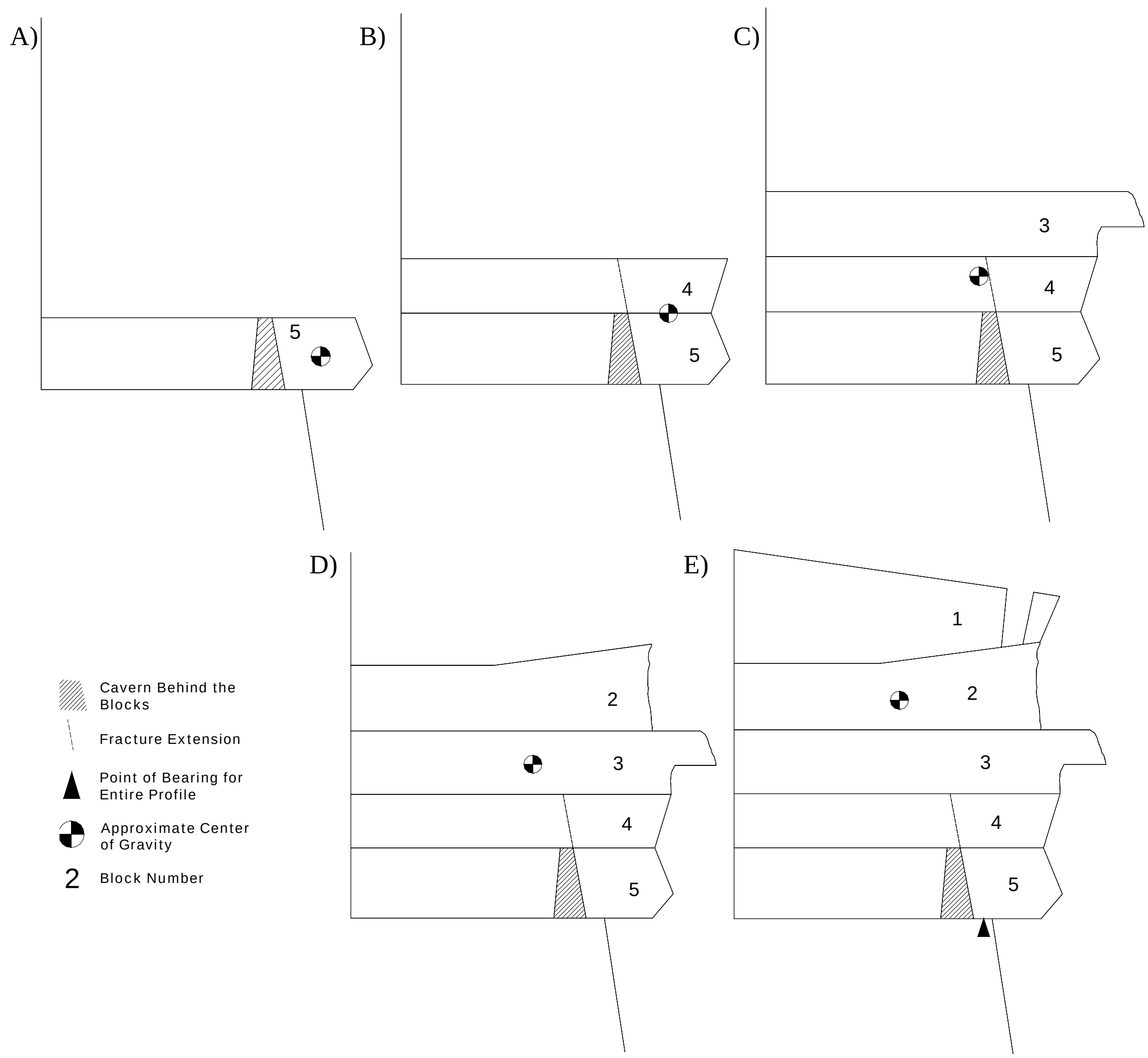


FIGURE 7. Structural-mechanical relationships immediately beneath the Chin (Block 5)

What this interpretation suggested was that the natural dead weight forces resulting from the location of the Profile's mass at its delicate, partially-cantilevered position on the cliff were solely responsible for its very delicate stability. The security of this mechanism was clearly delicate because of the narrow difference between the estimated optimum strength direction and the dip of joint Set 1 and the precariously-cantilevered configuration of the blocks in the rock mass, especially Blocks 4 and 5. It was consequently concluded that only a minor redistribution of the stresses developed by the configuration might precipitate the collapse of the rock mass. With all the foregoing in mind, three principal conclusions were drawn at the end of the reconnaissance and basic analysis.

First, based on the dimensional and spatial data and the rudimentary mechanical analysis, the mechanism of the Profile's in-place stability was postulated as illustrated in Figure 8 (diagrammatic view from a southern perspective).



FIGURES 8. Progressive stability mechanics ('A' through 'E') that led to the delicate cantilevering of Blocks 4 & 5.

Second, from the analysis and field observations, the intersections of the joints in the sub-vertical plane at the rear of the Profile's overall rock mass suggested the likely cut-off joints for Blocks 1-3 were located sufficiently into the mountainside behind the center of gravity of the Profile so the stability of their comparatively broad flat blocks was better than that of the more blocky, triangular and extensively overhung Blocks 4 & 5 below. These two blocks were instead cut-off from the rest of the rock mass by the cavern and fracture extension behind them (Figures 4, 6, & 8) and thus appeared close to toppling collapse. If this were to occur, given their direct connection with the likely foundation of the Profile, the subsequent toppling of Blocks 1-3 would quickly follow.

Third, because the Profile had survived substantial vibration during the earlier drilling and was currently surviving significant ambient vibration from wind (Table 2: 1958 & 1965), it was concluded it could withstand construction blasting below, as long as related vibration reaching the rock mass was restricted to the smallest, technically-possible fraction of the previously observed ambient natural vibrations.

Construction Blasting and Progressive Natural Changes In The Rock Mass

Ten years later in 1985-1986, following a lengthy process of decision-making, a 2-lane "parkway" section of I-93 was constructed through Franconia Notch and beneath the Profile. The author (then a consultant) was retained by the contractor responsible for the construction immediately beneath the Profile to review the design of each blast before detonation to ensure minimal blasting-related vibration reached the Profile or its immediate vicinity, as specified by the earlier reconnaissance and the NHDOT contract. The work included field installation and maintenance of a sensitive, continuously-recording seismograph on the cliff between the Profile's rock mass and the blasting below to record all types of vibration that reached the rock mass during the construction period and to determine if this specification was continually met.

During the monitoring, each of the blasts detonated was detected by the sensitive instrument. Each detection recorded was very close to the detection limit of the instrument (0.25 cm/second), demonstrating that no blast created a vibration at or near the Profile that was observed to cause damage or that was more than 10% of the ambient natural vibration the Profile was otherwise enduring during the construction period. In addition, and because blasting on earlier projects located further away and to the south of the Profile had been designed to meet the same requirement, the monitoring showed that none of the blasting along the corridor created potentially damaging vibration at or near the Profile's rock mass.

The continuous monitoring did detect the other types of ambient natural vibrations with readings ranging from 7.24 to 12.19 cm/second, more than an order of magnitude greater than any related to the construction. The distinctive signatures of these non-construction vibrations were easily identified on the dated and timed records by their non-blast-time occurrence and their substantial difference in intensity and frequency from vibrations typical of blasting. Back-checking and observations during instrument maintenance confirmed, as previously observed (Table 2: 1965), that these substantial vibrations resulted from wind gusts, thunderstorms, and aircraft over-flights and showed that the Profile was routinely and naturally subjected to dynamic stresses significantly larger and more potentially damaging than those related to the construction.

Also during the maintenance visits, many incipient changes in the condition of the surfaces and fractures in the rock mass near the base of the Chin were noted by comparing photographs with those from the earlier reconnaissance. These observations showed natural kaolinization decomposition was steadily deteriorating the granite and vigorous freeze-thaw cycling was frequently "quarrying-off" small blocks from nearby parts of the cliff. Speculation centered then and since (Fowler, 1997; Davis & Fowler, 1998) on how long these natural processes could continue before the Profile's stability was compromised. It was simply noted that these processes were those that formed the Profile, that they were for the most part uncontrollable by any then-feasible means on the lower portions of the rock mass, and that they would thus continue into the indefinite future with a generally detrimental impact on the security of the Profile.

Mechanics of The Profile's Collapse

Twenty-seven years later, the Profile collapsed on May 3, 2003. Analysis of the mechanism and cause began immediately by careful comparison of photographs. The best of these comparisons are presented in Figure 9. These views, before and after collapse, respectively, permit fairly precise determination of what portions of the rock mass collapsed. The blocks marked "A" through "D" in both photographs show clearly that the Chin and Upper Lip block combination fell away from the joints separating them from blocks "A" and "B", and the failure line on Figure 9 shows the approximate back line of the blocks and portions of the rest of the blocks that collapsed.

Figure 10 is a view of the top of the residual rock mass showing the rear portion of the Forehead slab into which the large stapled turnbuckles had been installed (Table 2: 1958). The backward-curled pattern of deformation of the forward staples that were stripped out of the front portion of the "Forehead" as it moved away indicates the rock mass toppled forward rather than sliding downward. Had sliding occurred, these staples would have been curled in the opposite direction and the line of breakage behind the Profile would not have been left as cleanly defined as shown.

The mechanism and sequence of the collapse can be reliably conceptualized by referring back to Figure 8E, a sketch of the Profile's pre-collapse rock mass viewed from the south. Based on all that was known of its pre-collapse structure and the particular stability of its various parts, it appears the collapse likely occurred in two nearly instantaneous stages.

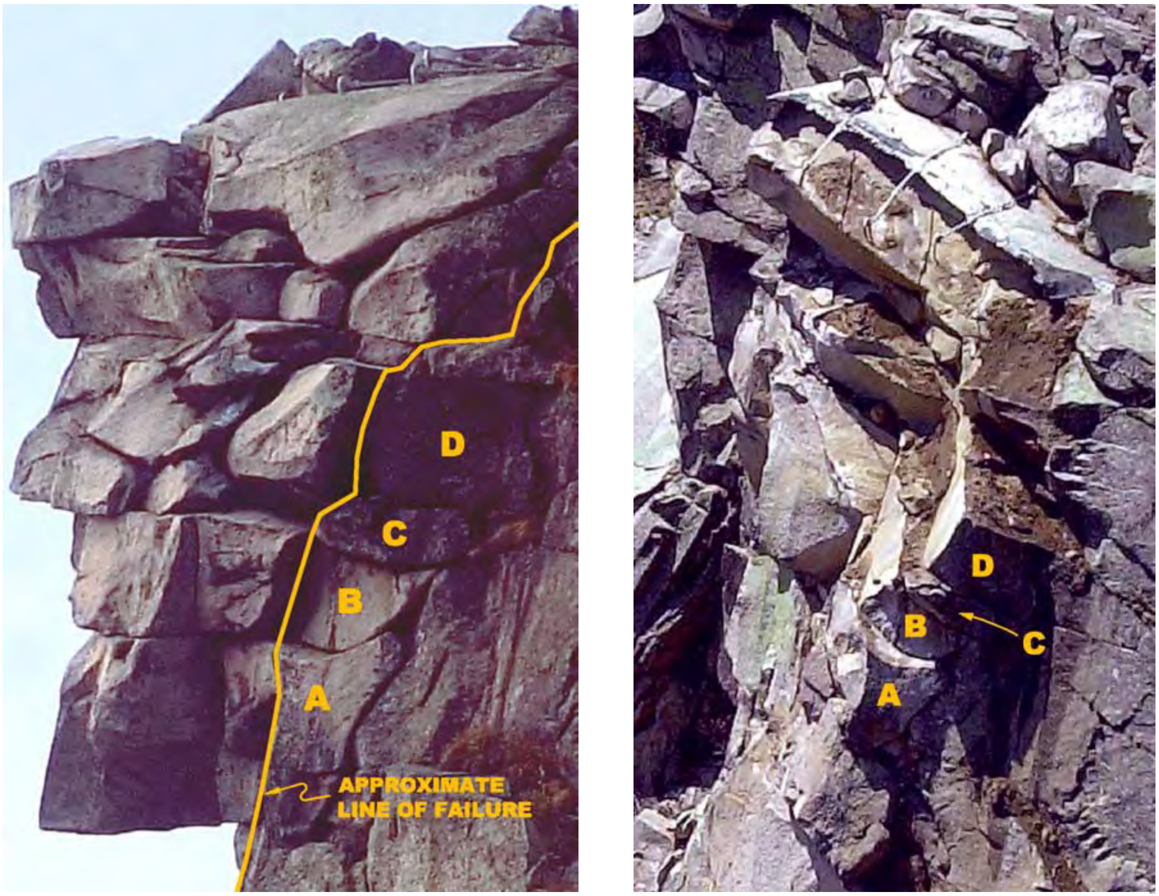


FIGURE 9. Profile's rock mass before (left) and after collapse (right) showing extent of toppling failure. Note Forehead turnbuckles in both photographs. (2001, Associated Press-Jim Cole; 2003, Union Leader Corp., respectively)

As discussed earlier, much of the roughly triangular Chin and Upper Lip block combination overhung the cliff below and was held in position by the weight of the 3 blocks above cantilevering the rearward triangular point of their combined mass onto the cliff at a point of bearing in the narrow granite bench below, just in front of the cavern (Figure 8E). Based on the deterioration observations of 1985-86 and the fact that only incipient construction-related vibration reached the rock mass during I-93 construction, it appears that the naturally progressive processes of kaolinization decomposition and vigorous freeze-thaw degradation compromised the intact strength of the granite in the narrow bench and gradually weakened it sufficiently so it could no longer remain intact. When this foundation suddenly disintegrated, the Chin was no longer supported and toppled forward and downward, taking the Upper Lip with it as that block broke off along the joint bounding it to the rear (Figure 8E, dashed line). Once the cantilevering force of the rest of the rock mass was no longer resisted, it quickly followed by also toppling forward.

FIGURE 10. Close-up, top of the Profile's residual rock mass (2003; Union Leader Corp.).

Conclusions

In the wake of the Profile's collapse, it is evident that the results of the rudimentary stability analysis of the mid-1970's turned out to be essentially correct. Its suggestions that the Profile's mechanism and circumstances of stability were delicate, that the Chin and Upper Lip held the key to its security, and that it might collapse in a toppling failure all proved true, providing those involved with that work with professionally gratifying, but not very satisfying, validation of the general assumptions, methods, heuristics, and estimates that had to be used to complete the work.

It is clear from the foregoing that the cause of the Profile's collapse was exclusively natural, there being no evidence of any kind of human-induced deterioration of its stability on the cliffside or of human influence in the initiation of the rock mass collapse. The collapse, while unfortunate for us in our time, is a normal consequence of the cliff-forming and mass-wasting processes that have been operating on the cliff since the departure of the last glacial ice from the Notch about 12,000 years ago. In short, the processes that formed it are those that caused its collapse.

Finally, the work of the last 27 years demonstrates that sometimes poorly-funded, and thus rudimentary, studies like these can yield important results that should not be discounted because of their relative lack of rigor or sophistication of method. As the collapse of the Profile shows, those concerned that its constitution might not be able to survive attempts to save it were probably right. But, even if it could not be saved, at least these basic studies documented how the remarkably "human" feature formed, how it endured, and how it finally expired. It all comes together now as a compelling example of "geology in action".

Acknowledgments

The following individuals (several deceased) lent valuable support, assistance, and encouragement to this project at various times over the past 27 years: Theodore Comstock, former Division Administrator, Federal Highway Administration; John Flanders, former Associate Commissioner, New Hampshire Department of Transportation; George Gilman, former Commissioner, New Hampshire Department of Resources and Economic Development; George Hamilton, former Director of New Hampshire Parks; Harry Reid, former Manager, and Mikey Libby, former Ski Patrol Chief, Franconia State Park; and Neils Neilsen and his staff at the Bridge Maintenance Division of the NHDOT.

Special thanks are due Roger Martin for expert technical rock climbing assistance without which much of the 1976 field work would not have been completed and to Roger Moody who provided valuable insight during the basic mechanical analysis of the rock mass. Special thanks also go to Dick Hamilton who provided invaluable technical assistance with the unique photographic activities associated with the 1976 field work and, posthumously, to Stan Young for valuable insights into the 1958 and 1965 activities on the Profile. And finally, special thanks go to Jeffrey Cloutier, Craig Cyr, and Lesley Fowler-Nesbitt each of whom made valuable suggestions to improve the manuscript and, in the case of Cyr, prepared for publication the complex of diagrams and unique photographs used in this paper.

References Cited

Davis, P.T. and Fowler, B.K., 1998, Large rockfall on Cannon Cliff near the Old Man of The Mountains, Franconia Notch, White Mountains, New Hampshire: Geological Society of America, Abstracts with Programs, v. 30, no. 7, p. A-299.

Fowler. B.K., 1982, Rock mass stability studies, Old Man of The Mountains, Franconia Notch, NH: University of Massachusetts, Conference Proceedings – Geotechnology in Massachusetts, Farquhar, O.C., ed., p. 141-148.

Fowler, B. K. , 1997, Geology of Franconia Notch & The Old Man of The Mountains: Bulletin of the Mount Washington Observatory, v. 38, no. 4, p. 10-15.

Goodman, R.E., 1976, Analysis of a sliding block on a plane—the friction circle concept: Methods of Geological Engineering, West Publishing Co., Los Angeles, p. 88-90 & p. 237-244.

Hancock, F.A.J., 1980, Saving The Great Stone Face: Franconia Area Heritage Council, R. Ayres-Givens, ed., Phoenix Publishing Co., 183p.

Hawthorne, N., ca 1840, “The Great Stone Face”, in The Great Stone Face & Other Tales Of The White Mountains, Houghton Mifflin Co., 1935, p. 1-31.

Schile, R.G., 1975, Stability studies on the Old Man of the Mountains, Franconia Notch: Thayer School of Engineering, Dartmouth College. Unpublished report.

Stability Analysis of the Rock Ridge Dam, Below Interstate 80/94, Thornton Quarry, Chicago, Illinois

Yunhua Lu and Terry R. West
Dept. of Earth & Atmospheric Sciences
Purdue University, West Lafayette, IN 47907-2051

Abstract

The Thornton Quarry on the south side of Chicago, IL is an extensive, open pit mine in a Silurian-age, reef complex of the Racine Dolomite. Interstate 80/94 extends east-west across the quarry forming a north lobe and larger south lobe, the main portion of the quarry. The highway is founded on a rock ridge, 2700 feet long, rising 325 feet high on the north side and 405 feet high on the south. The ridge is 150 feet wide at road elevation and steps outward to 600 feet wide at the base of the north lobe.

Evaluations are underway to utilize the north lobe as a surface reservoir to alleviate flooding on Thornton Creek within the Little Calumet drainage basin. As part of TARP (Tunnel and Reservoir Plan) water would be stored in the north lobe and ultimately released to the sewage treatment facility. The ridge will act as a rock dam with up to 300 feet of head in the north lobe and must resist unbalanced hydrostatic loads during all phases of reservoir filling and emptying.

Three major discontinuities occur in the rock ridge, two joint sets plus bedding. Along the east-west trending ridge, bedding dips gently northward on the east increasing somewhat to the west. Joint sets are nearly vertical yielding blocks of different configuration that are subject to sliding. Various block configurations were analyzed for stability under different hydrostatic conditions. Relationships between water levels in the reservoir and safety factor for block sliding were investigated. Friction angles related to stratigraphic details were also considered. The authors acknowledge the U.S. Army Corps of Engineers for data provided in this analysis.

INTRODUCTION

Flooding and surface water contamination are major problems in metropolitan Chicago owing to the extensive hard surfaces in urban areas. Vegetative cover and infiltration are greatly reduced while runoff is significantly increased. Following heavy rainfall events sewage treatment plants cannot accommodate the increased water volume. For a 1” rainfall in the city, more than 5 billion gallons of runoff are produced. In 1972, the Chicago Water District approved an ambitious project; a series of tunnels to collect the water until it could be reclaimed by the treatment plants. Today the total cost for the 109 mile tunnel system is approaching $4 billion. Previously, overflow water from combined sewers was released to surface waterways without treatment, causing flooding and contamination and a major contribution to urban pollution (TARP, 1999, West, 1995).

In the Tunnel and Reservoir Plan (TARP), underground tunnels ranging from 9 to 33 feet in diameter, have been driven 150 feet to 360 feet below the ground surface and used to transfer sewer overflow to sewage treatment plants (West, 1995). Figure 1 is a diagram of the deep tunnel project. Because the first flush volume exceeds the capacity of the deep tunnel during heavy rainfall, it is proposed that the remaining flow (the second flush) be stored in surface reservoirs for future treatment. When completed, the surface reservoirs will increase the capacity of the

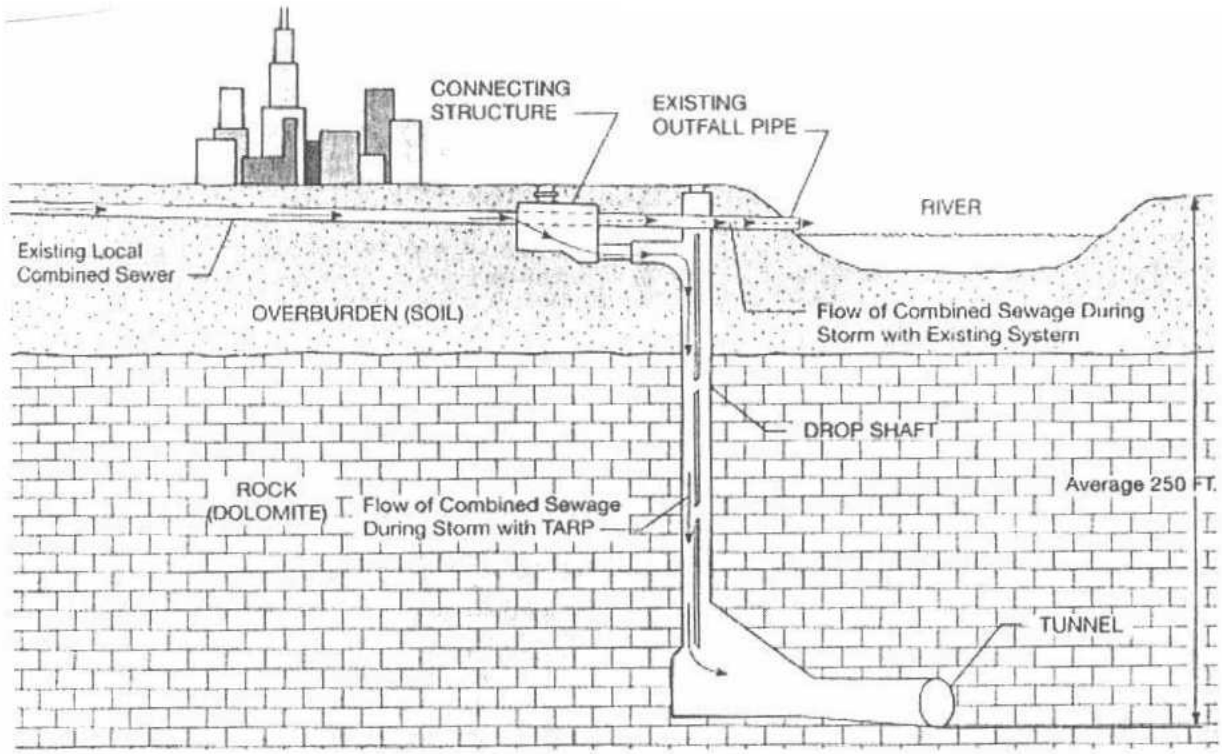


Figure 1. Diagram for Chicago Deep Tunnel Project (West, 1995).

TARP system to 15.6 billion gallons. All tunnel construction should be completed by 2004; today more than 93 miles of tunnels are finished and in service, whereas 8 miles are under construction (TARP, 1999).

Of the nearly $4 billion total cost of TARP about 75% was funded by US EPA. The project consists of the following: Phase I, nearly completed, 109 miles of underground tunnels, drop shafts and pump station to collect the first flush of storm water. Phase II will involve the development of large surface reservoirs to store overflow from the tunnel system (the second flush) and hold it for subsequent treatment, thereby preventing overflow and flooding of surface waterways (US Army Corps. of Engineers, 2002). 75% of this phase will be funded by the Army Corps of Engineers.

CUP-THORNTON RESERVOIR PROJECT

The Chicagoland Underflow Project (CUP) includes the Thornton reservoir project located in southern Cook County, IL on the far south side. Near Interstate 80/294, a major expressway, it involves the north lobe of Thornton Quarry, which is an active aggregate mine operated by Material Services Corporation.

Figure 2 is the site map showing the location and the details of the north and south lobes (STS, 2001). The north lobe, which will comprise the permanent CUP-Thornton storage reservoir, is bounded by private property on the north, Vincennes Road on the northeast and the Chicago and Eastern Illinois Railroad on the west. Interstate 80/294 (Tristate Tollway), separates the main lobe from the north lobe and forms the south boundary of the reservoir.

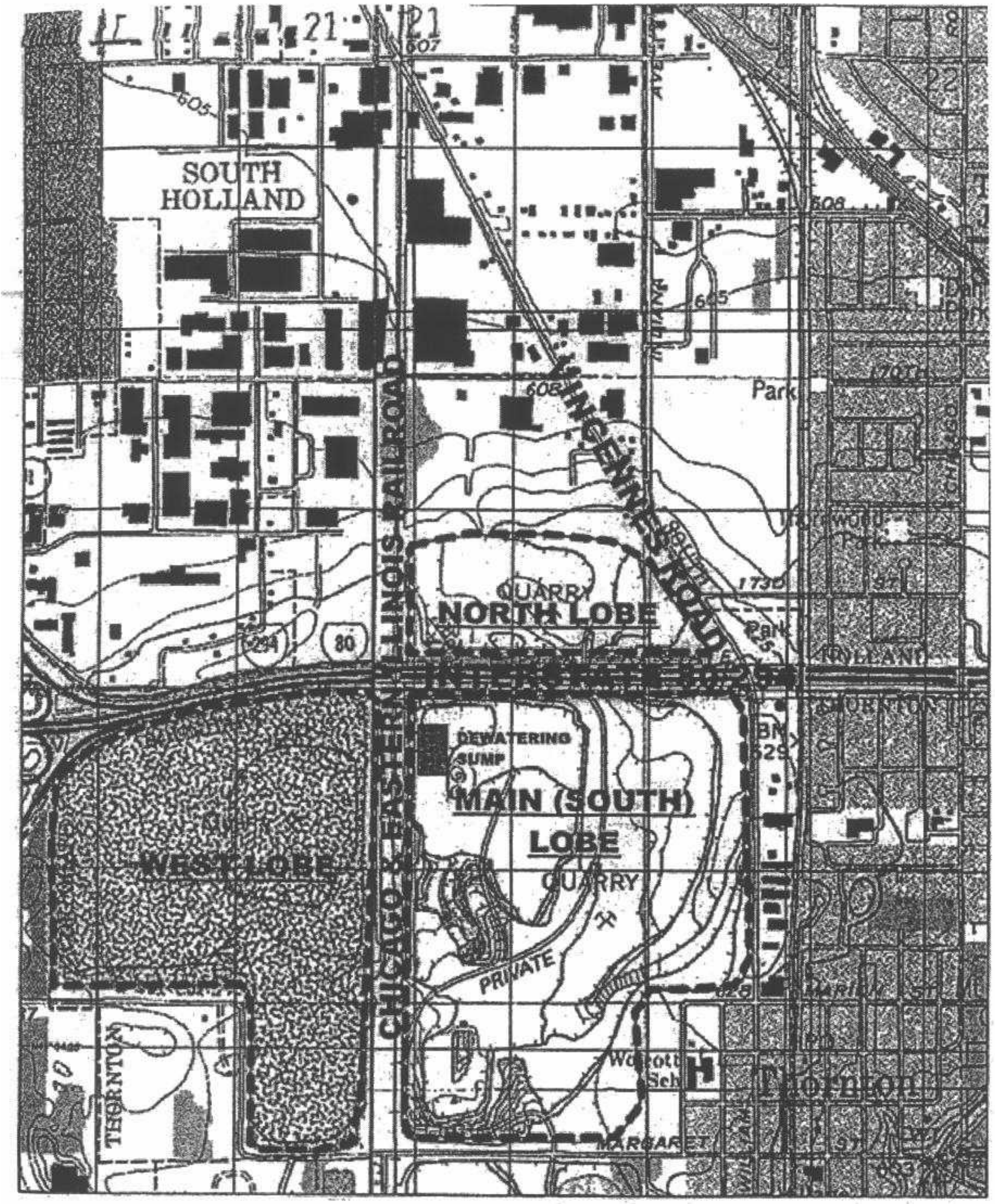
The roadway is constructed on a 2700-foot long (east-west) wall of the rock ridge, rising more than 300 feet above the quarry floor. The rock ridge (or rock pillar) widens with depth due to quarry development and contains a notch which must be filled in to contain the reservoir. Three levels comprise the rock ridge (see Figure 3), the first extends from the ground surface to elevation -100 CCD (Chicago City Datum). and is about 340 feet wide (north-south). Level 2 extends from -100 CCD to -210 CCD and measures approximately 470 feet. Level 3, extending from -210 CCD to the bottom of the quarry floor, -298 CCD, is nearly 600 feet thick. Similar levels exist on the south side of the rock ridge, however, from about -295 CCD, an additional level extends to the quarry floor at a depth of -380 CCD (Figure 3)..

The reservoir would provide additional storage for the existing Calumet “Deep Tunnel” System and capture and store combined sewer overflows for subsequent treatment. Such flows would otherwise be discharged as untreated water into the Calumet-Sag Channel and the Little Calumet River and the Grand Calumet Rivers, causing flooding and contamination.

The Thornton Reservoir project was authorized by the Water Resources Development Act of 1988 and modified in 1999. Reservoir volume is 24,200 acre-feet, or about 7.9 billion gallons. Construction costs are estimated as $110 million and the project is scheduled for completion by 2014 (STS, 2000).

An important design issue involves the stability and performance of the south rock wall of the reservoir, which separates the north lobe from the main lobe. This rock ridge also supports Interstate 80/294 (Tristate Tollway) and must resist unbalanced hydrostatic forces during all phases of reservoir operation, including reservoir drawdown.

The current report, based on a master’s thesis by the first author (Lu, 2003), focuses on stability analysis of the rock ridge by applying a deterministic approach. Patrick Engineering Inc. (PEI, 1999) conducted a preliminary stability analysis on the rock ridge based on their geologic



scale: 1:12,800

Figure 2. Site Location, Thornton Reservoir (STS, 2001)

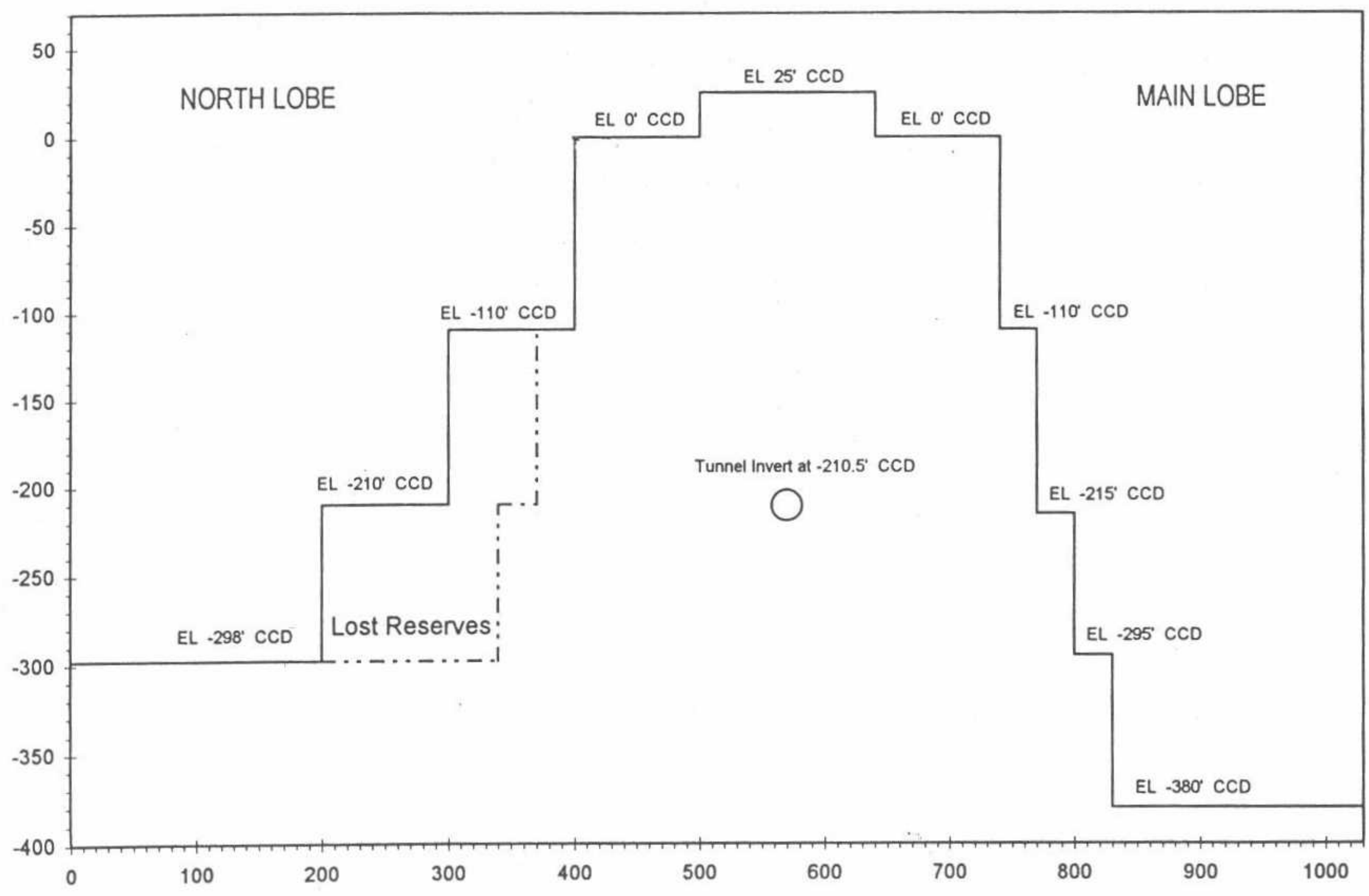


Figure 3. Cross Section of Rock Ridge Looking Eastward.

field mapping data. The work here is an extension of that study. Initially, a probability analysis was also planned for the thesis but that research has not been accomplished. Probability analysis similar to that performed by H.J. Park at Purdue University had been anticipated (Park, 1999; Park and West, 2001).

Geometry of the failure blocks used here differs somewhat from the geometry developed by PEI, 1999. Additional considerations are presented which included further calculations of hydraulic head in the reservoir and rock ridge. The possible impact of regional horizontal stresses on the rock ridge is also considered. Blocks are formed by vertical release planes so that frictional resistance is provided only by the planar failure surface at the base of the block (See Figure 4 after Hoek and Bray, 1981).

GEOLOGICAL AND GEOTECHNICAL SETTING

The Thornton quarry is located primarily in the Racine Formation, Silurian-age dolomite. Because of shallow soil cover, subsurface construction typically involves bedrock. A rock tunnel has been driven along the rock ridge at a depth of about 225 feet (see Figure 3). Bedding in the tunnel is essentially horizontal (personal observation, West, 2003). As indicated previously the quarry was developed in the Racine Formation, a reef dolomite structure extending downward to the Romeo Formation. Figure 5, a stratigraphic section developed by Patrick Engineering (PEI, 1999) shows the contact between the Racine and the Romeo formations at elevation -300 ft., CCD.

McGovney's comprehensive study (1978) proposed that the Racine Formation originated as an ancient marine limestone reef with five distinct reef rock facies made up of reef core, flank and inter-reef. These deposits, biologically produced by corals and algae, were changed to dolomite during diageneses. They are chemically purer than adjacent rocks and provide excellent quality construction aggregates.

The Racine Formation increases in thickness from northeast to southwest, consistent with the center of the reef structure located to the southwest (Willman, 1973, Pray, 1976, McGovney, 1978). Dip of the bedding planes diminishes with distance from the core. Also bedding dips decrease with an increase in quarry depth. The Racine formation is described as a fine to medium grained, dark gray dolomite.

RESERVOIR DETAILS

The north lobe consists of about 61 acres. The south reservoir wall, nearly vertical as a consequence of quarry operations, rises about 300 feet above the quarry floor. The existing ground elevation along the rock ridge on the I-80/294 highway pavement is about +25 ft (CCD.

Hydrogeology studies by Roadcap, et al., (1993) indicate that the Silurian dolomite is a leaky, confined aquifer. Quarrying necessitates that water levels in the north lobe be drawn down to the quarry floor elevation. Packer tests in boreholes indicate that the hydraulic conductivity of the Racine Formation ranges from $2.5x10^{-6}$ to $2.5x10^{-5}$ cm/sec. As quarry depth increases, the hydraulic conductivity also decreases slowly (STS, 2000).

Low values for Rock Quality Designation (RQD) occur near the top of bedrock, but increase rapidly with depth. At depths of 50 feet, the RQD reaches 90% (STS, 2000). Unit weight of the Racine Dolomite ranges from 145 pcf to 173 pcf (average 167.5), specific gravities

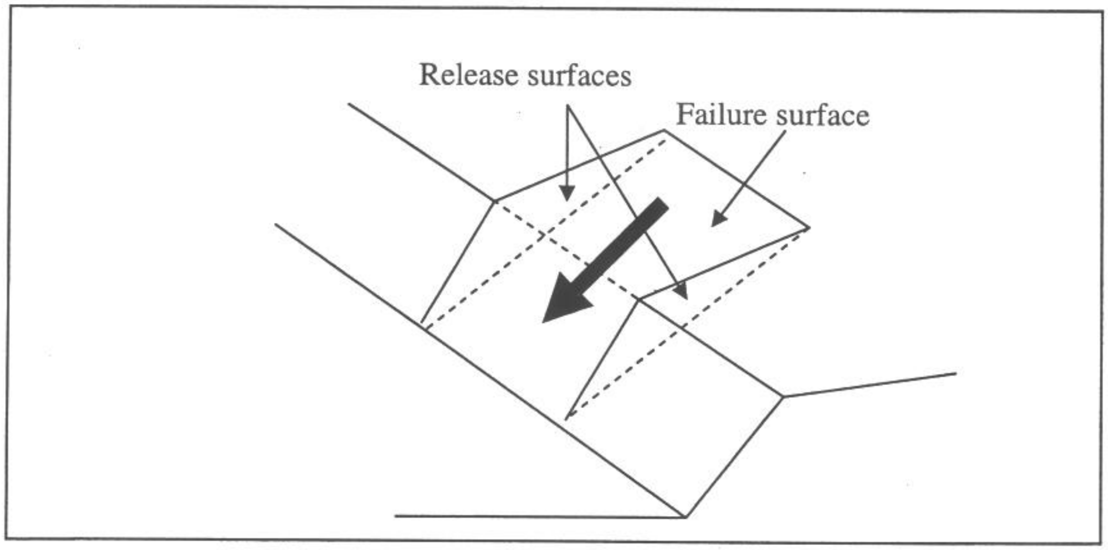


Figure 4. Release Surfaces for Plane Failure (Hoek and Bray, 1981).

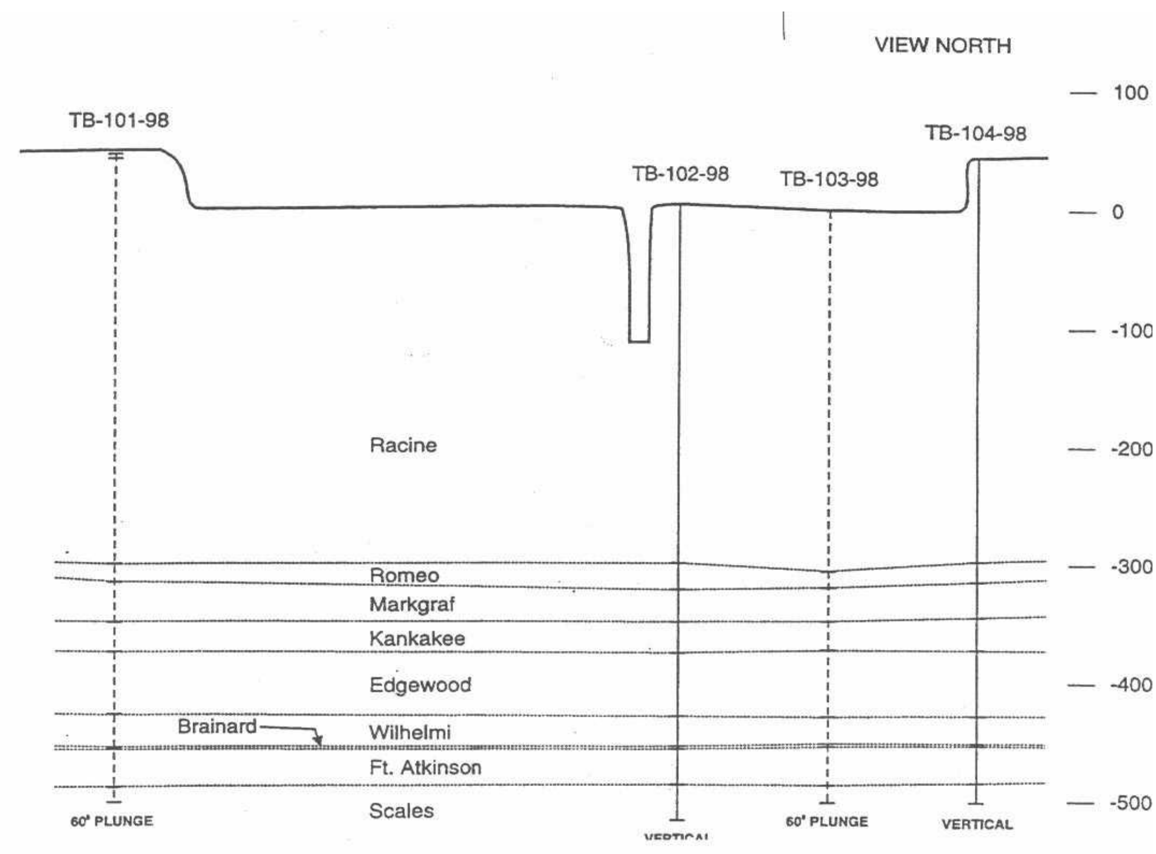


Figure 5. Stratigraphic Section of Rock Ridge (PEI, 1999).

range from 2.76 to 2.88, (average of 2.83) and porosity ranges from 2% to 19%, (average 8%) (STS, 2000).

STRUCTURAL DISCONTINUITIES

Thornton Quarry lies within a bedded dolomite reef deposit that is steeply dipping on the reef flanks and nearly horizontal in the core. The major structural discontinuities consist of bedding planes plus two sets of vertical to subvertical joints (PEI, 1999; STS, 2001). Data sets were evaluated using basic stereographic techniques (Goodman, 1989; Watts and West, 1985; Norrish and Wiley, 1996; Priest, 1993).

Bedding and Bedding Plane Partings (Discontinuity Set 1)

The general orientation of the bedding shows an east-west strike with a dip of 10-15° north. However, some beds are horizontal. As depth increases, the dip also decreases approaching the horizontal. On the east side of the rock ridge, the dip of the bedding is essentially horizontal, but on the west, the dip increases up to 25°. Clay coatings and clay layers occur along bedding as a consequence of sedimentary deposition; layers can be up to 6 inches thick. Some solution zones also occur along bedding planes, particularly in the upper 50 to 100 feet of the bedrock. Solution zones consist of broken or granulated, weak rock with open pores between particles. Generally, these zones are ½ to 6 inches wide. Some solution effects extend for the full height of the rock ridge. Clay layers and solutioned rocks decrease shear strength which encourages sliding failure of rock wedges or of the entire pillar mass. This is a major concern for stability of the rock ridge (PEI. 1999).

Discontinuity 2 (Joint Set)

Strike N50W to N80W, Predominantly N60W, Commonly Vertical

This prominent joint set typically is continuous for hundreds of feet and quite planar. Some major joints extend the full width of the rock ridge or pillar. Typical joint spacing is about 27 feet and joints are tight and clean. Mapping by PEI (1990) shows spacing of 80-100 ft. along the rock ridge, and prominent open joints exist with apertures ranging from 2 to 8 inches. Most open joints are filled with dark clay and/or with a dense, sandy soil (PEI, 1999).

Discontinuity 3 (Joint Set)

Strike N45E to N50E-Vertical (± 10°)

This joint set provides another prominent discontinuity. Joints are continuous and usually tight with a typical spacing of about 48 feet. These joints are more persistent and closely spaced on the west side of the pillar than on the east side (PEI, 1999). Joint openings vary from 2 to 4 inches and most are filled with a mixture of dark clay and rock debris.

CRITICAL WEDGES

The previously described bedding plane and two joint sets comprise the important discontinuities affecting rock ridge stability and ground water flow. They isolate rock blocks and wedges into units of different size. During operation of the reservoir, some of these wedges may become unstable. Pore water pressure along the joints and bedding planes could encourage sliding of rock wedges (PEI, 1999) yielding unstable blocks. Grouting of the rock dam has been proposed in order to reduce seepage from the reservoir and to provide increased slope stability (PEI, 1999).

Three factors affect rock ridge stability: geometry of critical wedges, shear strength parameters and magnitude/distribution of pore water pressure. PEI (1999) recommended that wedges be analyzed at 50-foot height intervals along the rock ridge. Bedding planes are nearly horizontal, or have a N10° dip. Discontinuity 2 has a joint spacing of about 27 feet and a vertical dip. Discontinuity 3 has a spacing of about 48 feet and also dips vertically.

The approximate shear strength values recommended for analysis by Patrick Engineering (PEI, 1999) are provided in Table 1. As shown, bedding plane partings with clay fillings have low friction values and inhibit drainage (Barton and Choubey, 1977). Their friction value is estimated to range between 10 and 15°.

Table 1. Shear strength parameters for discontinuities (PEI, 1999)

Discontinuity Designation	Cohesion	Friction
1. Bedding		
Clay-filled	0	10-15°
Massive/rough	0	25-50°
Vertical joints		
2. NW-SE (N60W)	0	20-25°
3. NE-SW (N50E)	0	30-35°

Seepage occurs along bedding partings and vertical joint sets. Permeability along joint systems will provide the majority of flow into and out of the reservoir. Currently, dewatering maintains the water level at the quarry floor (PEI, 1999). Observations along the rock ridge indicate that some discharge currently occurs along discontinuities near the quarry floor. Uplift pore pressure along joints and bedding planes will decrease the stability of the rock ridge. Under free drainage conditions the pore pressure distribution would develop a triangular stress diagram.

HORIZONTAL STRESS CONSIDERATIONS

The rock pillar, measuring 300 feet high, 340 feet wide and 2700 feet long, supporting Interstate 80/294, will act as a dam during reservoir operations. Gravity and hydraulic forces will act on rock wedges outlined by the three prominent discontinuities. Based on PEI's report (1999), special concern for stress and deformation of the rock mass at the end of quarrying is in order. As excavation (mining) continues in the quarry, the horizontal, in-situ stress within the rock mass may be transferred to the rock ridge. If this horizontal force is mobilized, displacements could occur along the existing rock mass discontinuities and disrupt the rock ridge

(PEI, 1999). Based on calculations performed in the current study, the factor of safety decreases dramatically when horizontal stress is included in the analysis of rock ridge stability. Also presented here is an alternative concept that the horizontal stress may simply be relieved by closing fractures within the rock ridge itself and yielding no further instability.

ANALYSIS OF ROCK RIDGE STABILITY

Rock ridge stability analysis was performed using a deterministic procedure. Three types of failure configurations were considered (PEI, 1999). Factor of safety was determined based on water level, friction angle of bedding planes and surface area of the failure plane. Three failure configurations were considered: 1. Blocks along horizontal bedding planes, 2. Horizontal wedges along bedding planes, 3. Bedding planes dipping into the reservoir.

Failure Configuration One – Blocks Sliding Downstream on Horizontal Bedding Planes

For failure configuration one, the failure plane allows blocks to slide downstream into the main lobe. A slice of unit thickness is considered. Release surfaces are provided by two dominant vertical joints and resistance to sliding along these joints is considered to be negligible. For this configuration $FS = \dfrac{(W - U)\tan f}{V}$, where W = weight of block, U = uplift force along basal bedding plane, and V = hydrostatic force at rear of block. Figure 6 shows this detail. Pore pressure distribution regarding rock slope stability has been discussed in detail by West, 1966.

Representative blocks from the rock ridge were analyzed. Dimensions of these blocks are provided in Figure 7 (PEI, 1999). Eleven different blocks were considered, V_1 through V_5 are horizontal and V_6 to V_{11} are inclined.

Failure Configuration Two – Horizontal Wedge Moving Downstream

For failure configuration two, the failure plane allows wedges to move horizontally downstream into the main lobe. The smallest wedge has an upstream length equal to 51.7 feet. Figure 8 shows the geometry of the failure block. Larger wedges occur if the release joints are farther apart along these parallel joint sets. Larger wedges are more stable as explained later in this discussion.

If regional horizontal forces are excluded, the factor of safety of the bedding plane for failure configuration two is obtained by: $FS = \dfrac{(W - U)\tan f}{V + P_a + P_b}$

In this case P_a is the pore pressure in one of the diagonal joints and P_b is that pressure in the other joint. Figure 9 shows this detail.

Failure Configuration Three – Bedding Plane Dipping into the Reservoir

The bedding plane considered here dips N10°. Figure 10 shows the geometry of the failure plane: h_c is the width of the block; a is the length of joint N50E; b is the length of joint N60W; c is the length of the block along the upstream side of rock ridge; h_1 is the height of the block on the downstream side and h_2 is the height of the block on the upstream side.

Figure 11 shows the view of the failure block that extends from level 1 down to level 3.

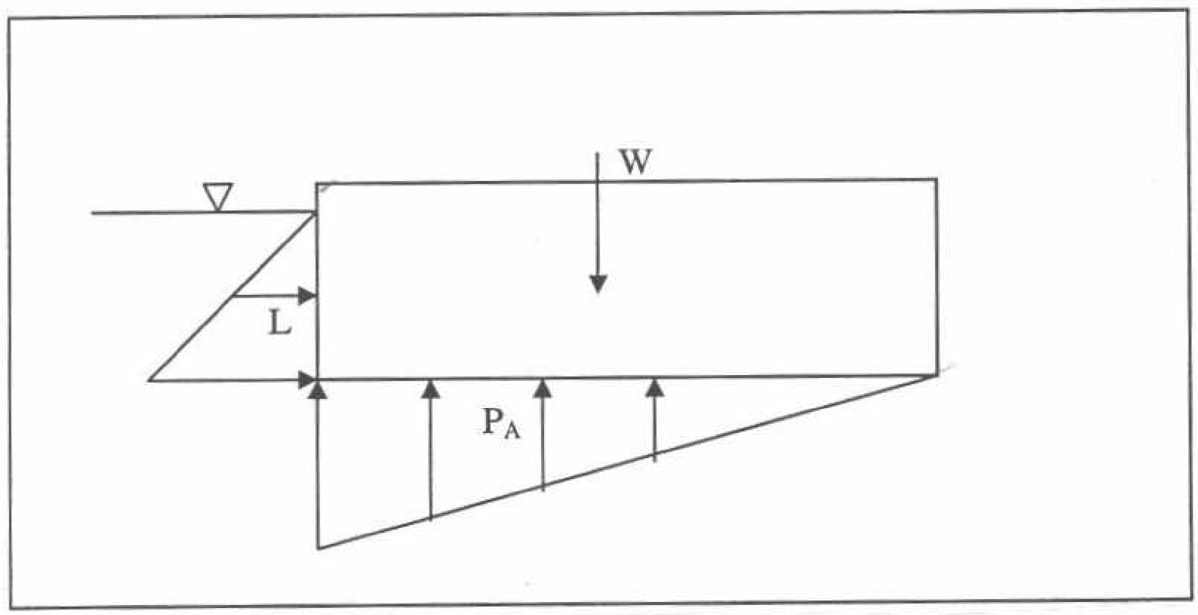


Figure 6. Horizontal Block, Unit Thickness Under Pore Pressure Conditions.

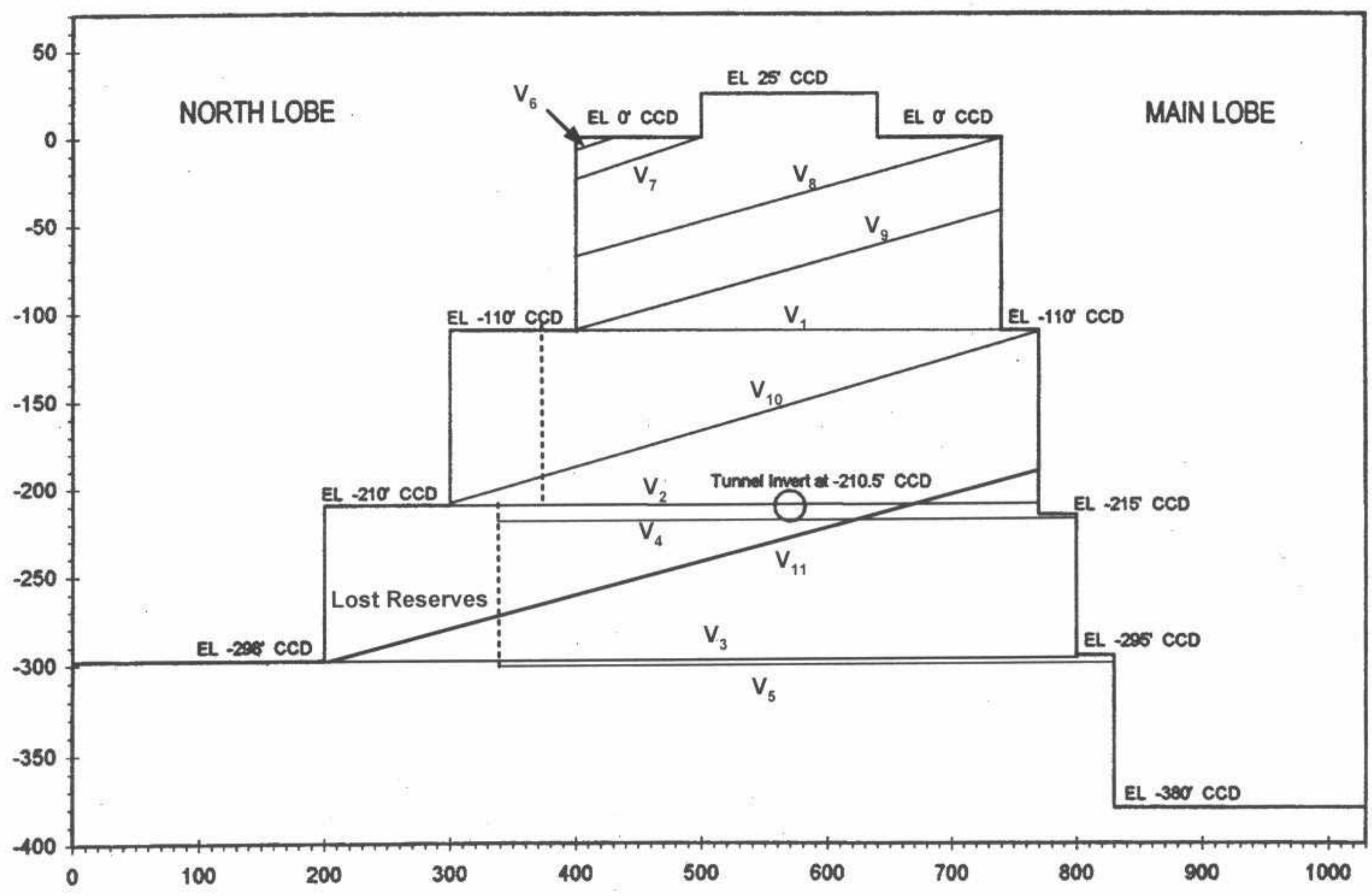


Figure 7. Cross Section of Rock Ridge Showing Potential Fracture Surfaces.

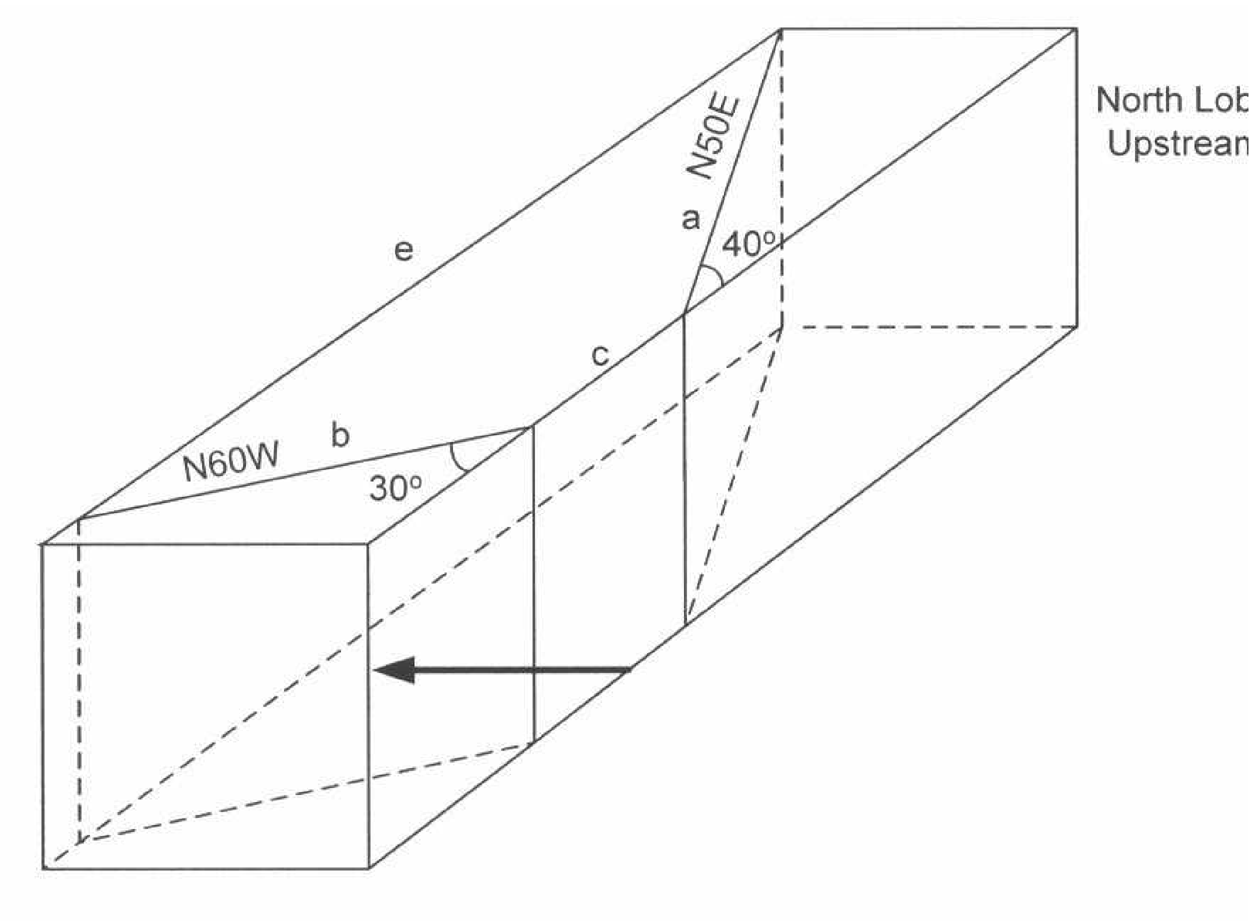


Figure 8. Geometry for Failure Configuration Two.

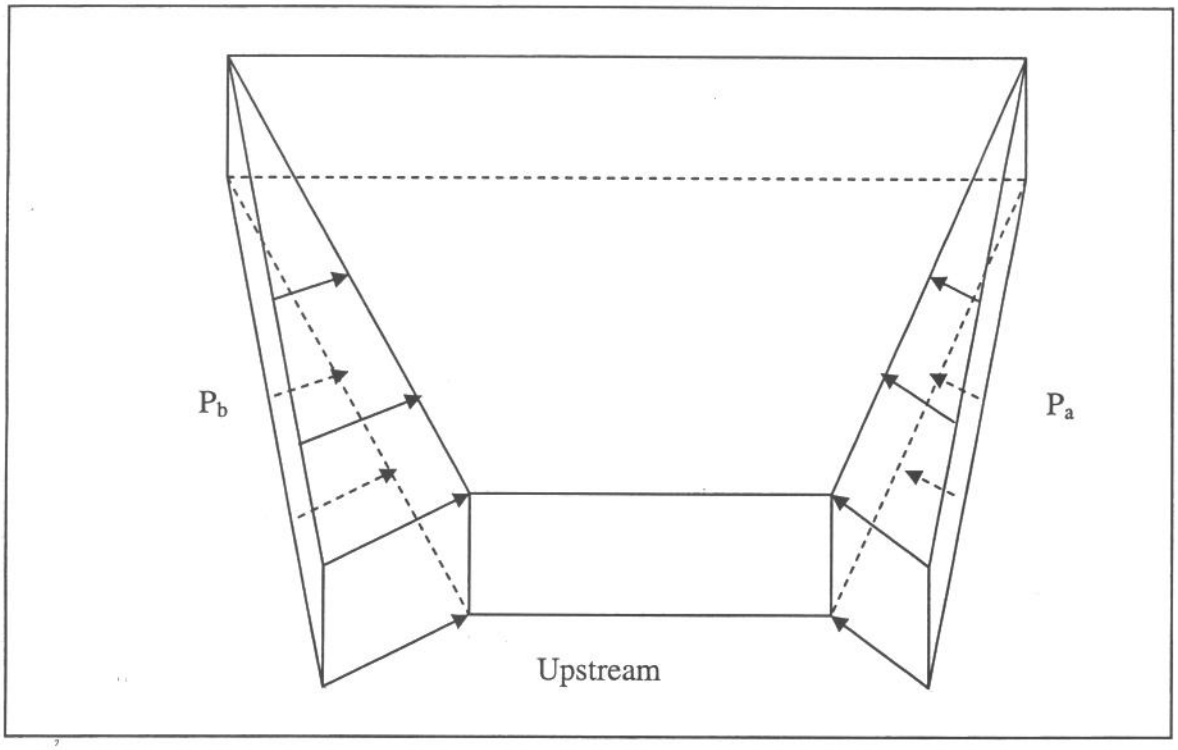


Figure 9. Pore Pressure on Joints, Configuration Two.

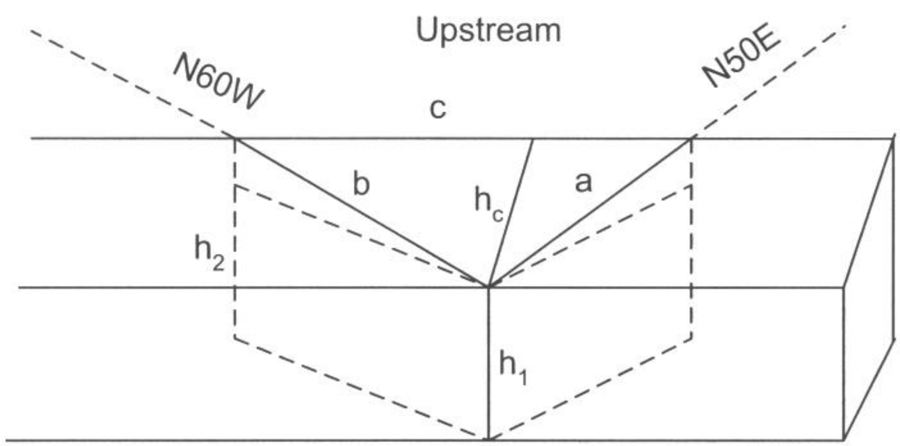


Figure 10. Wedge Geometry of Inclined Bed Failure, Configuration Three, 10 Degree Dip.

ANALYSIS OF RESULTS

Based on the previous discussion, the three configurations of failure were analyzed under different conditions. Results of this analysis are as provided below.

The first failure configuration involves a horizontal bedding plane moving downstream into the main lobe. Analysis is considered in three dimensions using unit lengths. Four stability conditions were considered based on recommendations from the PEI study (1999). These are: 1) end of construction with a dry reservoir, 2) 95% effective grout curtain and tunnel with full reservoir, 3) full reservoir with steady seepage, 4) partial drawdown with steady seepage and 75%, 50%, or 25% full reservoir. It is fairly certain that construction of a grout curtain along the center line of the rock ridge will be required to reduce seepage effects.

The first consideration involves the end of construction with a dry reservoir.

Based on the factor of safety equation for failure configuration one, failure will not occur along a horizontal bedding plane under dry conditions. The simplified equation becomes $FS = \frac{\tan f}{\tan ?}$ with θ = 0 and FS goes to infinity. The configurations for block failure of the rock ridge are shown in Figure 6. Eleven different configurations are included. V_1 through V_3 involve horizontal displacements at levels 1, 2 and 3. V_4 and V_5 are for levels 2 and 3 after further mining is accomplished in the north lobe. Approximately 100 feet horizontally of rock would be removed along the lowest level. Configurations V_6 through V_{11} involve blocks with a base line boundary sloping 10° to the north.

Calculations were made for the three basic failure types, horizontal block, horizontal wedge and bedding plane dipping northward. All eleven configurations (V_1 through V_{11}) were considered. In a series of calculations, the friction angle ϕ was increased from 10° to 35° using 5° intervals. In addition all four of the stability conditions mentioned above were evaluated.

The lowest factor of safety, as expected, occurred when a friction angle ϕ = 10° was employed. For the horizontal unit block, FS approaches one under full reservoir conditions. For the smallest horizontal wedge, FS for the ϕ = 10° case drops below 1.0. For the bedding plane dipping 10° into the north lobe, FS for ϕ = 10°, reduces to 0.8 for some situations. It seems unlikely, however, that a clay filled bedding plane would be continuous over a long distance without pinching out to yield a higher friction angle.

A final calculation performed involves the horizontal stress described previously. When this stress is considered, many combinations of wedge configurations and friction angles yield factors of safety values less than 1. However, manifestation of the horizontal stress onto the rock ridge seems to be a remote possibility. An alternate calculation regarding the horizontal stress is considered below.

After mining is completed, assume that the horizontal stress on the rock ridge is mobilized (PEI, 1999). Considering block V_5, the failure surface occurs at elevation -298 CCD or 333 feet below the ground surface. At this depth the regional horizontal stress ranges between 1000 and 1500 psi (PEI, 1999). Using an average value of 1250 psi, this yields the details shown in Figure 12.

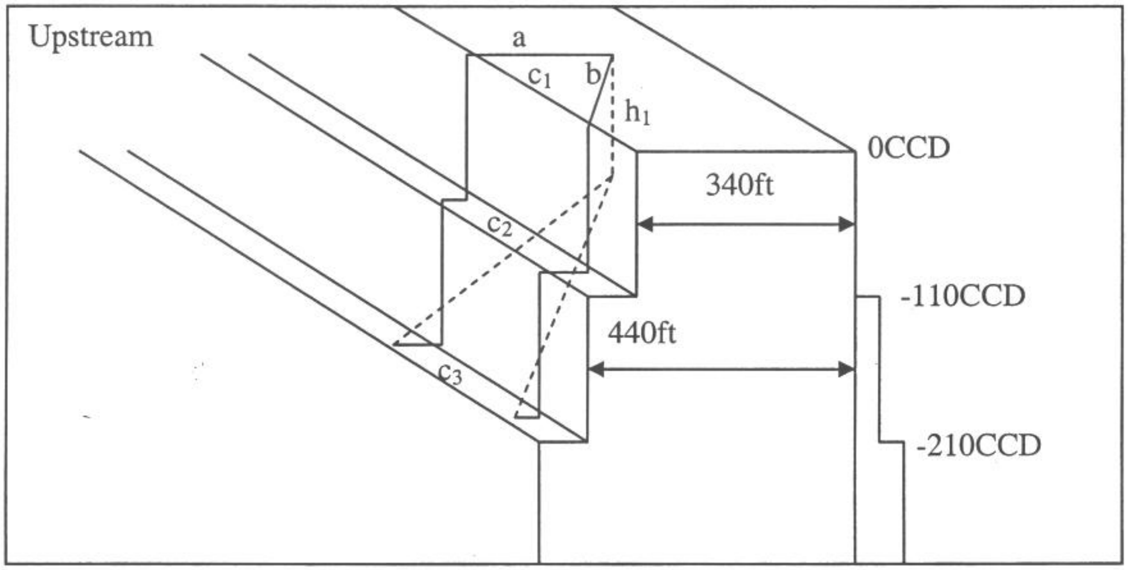


Figure 11. Sloping Failure Wedge, Configuration Three, Block V10.

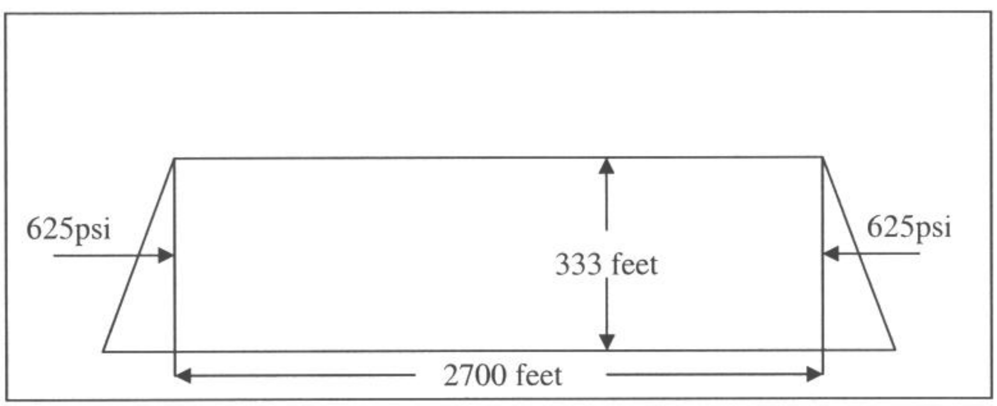


Figure 12. Horizontal Pressure Effect on Rock Ridge Stability.

To calculate the strain use, $\varepsilon = \frac{s}{E}$. E for massive dolomite = $8x10^6$ psi which is appropriate for the Racine Dolomite. However, for the rock mass, E_{RM} is reduced by 50% to yield 0.5E or $4x10^6$ psi. Therefore, $\varepsilon = \frac{625 psi}{4x10^6 psi} = 1.56 \times 10^{-4}$. $\Delta L = \varepsilon L = 1.56 \times 10^{-4} \times 2700 \times 12 = 5.1$ inches of displacement which would relieve the horizontal stress.

DISCUSSION

Comparison with Results by PEI

Comparing the analysis in this report to that of PEI (1999), the current results are slightly more conservative. For example, consider the third failure configuration involving block V_6 with long-term full reservoir and steady seepage. Current results indicate a slightly lower FS than that calculated by PEI. This is shown in Figure 13. The relationship between the friction angle and FS is nearly linear.

Effect of Friction Angle on Horizontal and Inclined Bedding Plane Failures

The influence of bedding plane friction angle on factor of safety for a horizontal bedding plane is shown in Figure 14. This involves the factor of safety for block V_5 regarding long-term steady seepage. It indicates that factor of safety increases linearly with increasing friction angle.

Effect of the Failure Plane Size

In the second failure configuration, wedges sliding along horizontal bedding planes are considered. As the wedge dimensions increase, the factor of safety also increases. Figure 15 shows the effect of length versus factor of safety for wedge (V_1) at bedding friction angle of 12°. The relationship is nearly linear. The smallest blocks have the lowest FS because pore pressures on the release planes become more significant relative to the decreased size and weight of the block.

Dip of Bedding Greater Than 10 Degrees

A final consideration involves the western portion of the rock ridge. Here bedding planes may dip at angles up to 25°. If the friction angle for the bedding plane is only 10°, failure under pore pressure conditions would be eminent. A FS less than 0.25 would occur. However, under dry conditions the FS = $\frac{\tan f}{\tan ?} = \frac{\tan 10}{\tan 25} = 0.38$ indicating failure under current conditions. However, block failure has not been observed under today's, dry conditions, prior to reservoir filling. Therefore, the extreme occurrence of continuous bedding planes with ϕ =10° within steeply dipping reef deposits is not prevalent on the western end of the rock ridge. In addition, the high RQD values obtained for the dolomite (over 90% at depths greater than 50 feet) suggests that clayey zones are not prevalent along the bedding. For a bedding dip of 25°, ϕ =35° is required to yield a FS greater than 1.0 under most pore pressure conditions.

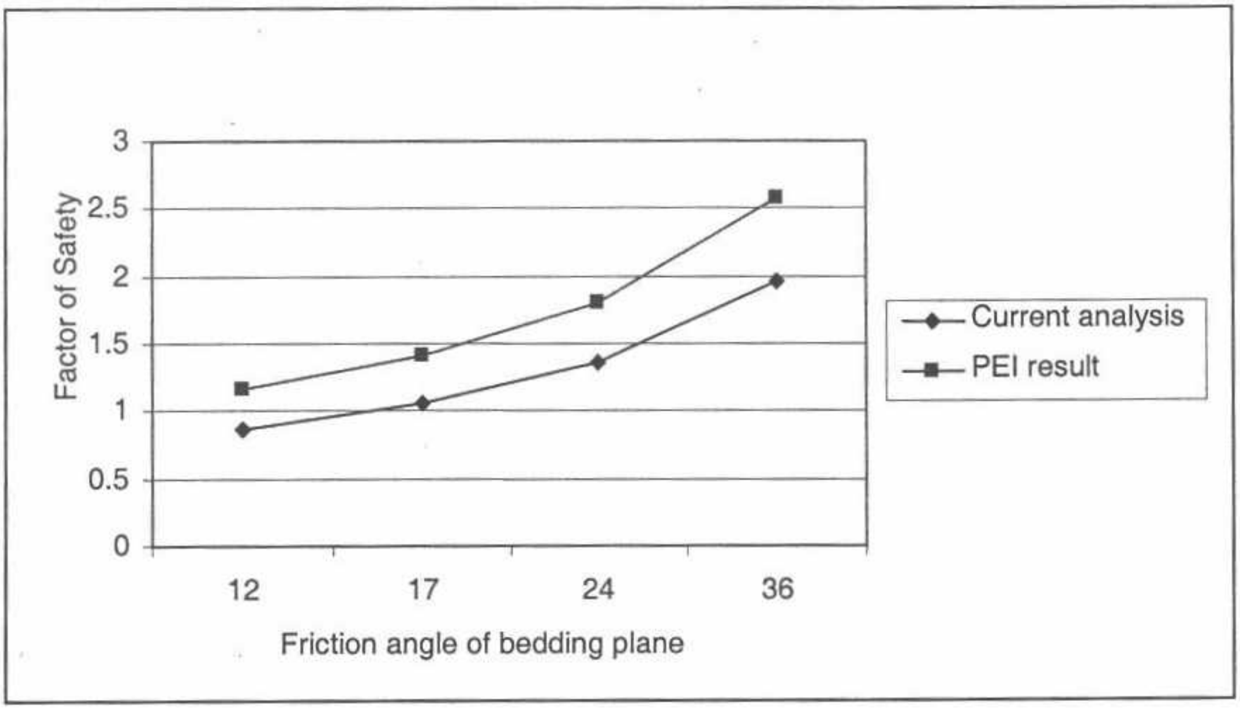


Figure 13. Comparison of Results with PEI, Inclined Beds, 10 Degrees, Block V6.

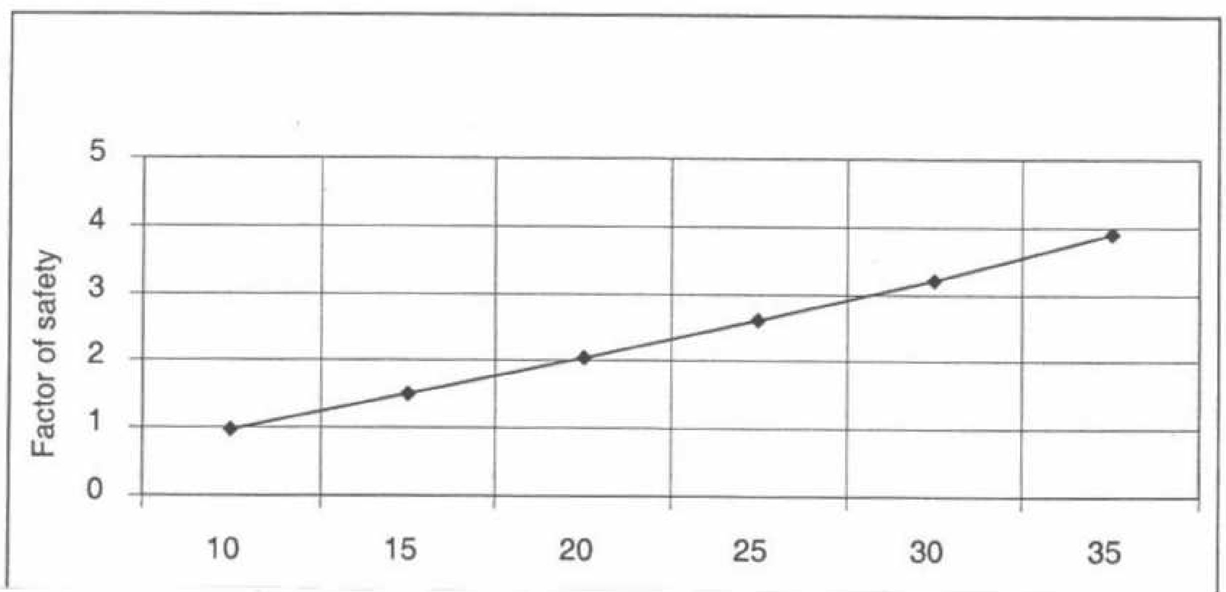


Figure 14. Friction Angle Effects on FS, Horizontal Bedding, Block V5.

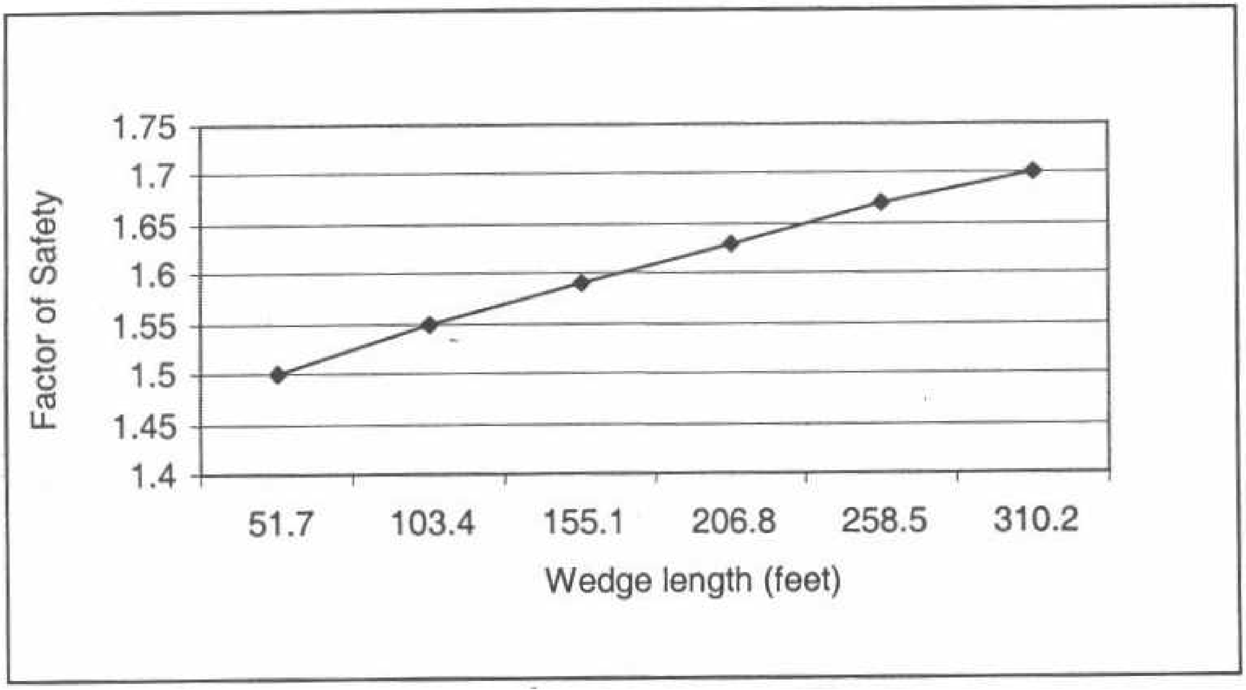


Figure 15. Wedge Dimension Effects on FS, Horizontal Bedding, V1, ϕ=12°.

REFERENCES CITED

Barton, N.R. and Choubey, V., 1977, The shear strength of rock joints in theory and practice, Rock Mechanics, 10, 1-54.

Goodman, R.E., 1989, Introduction to Rock Mechanics, 2nd Edition, John Wiley & Sons, New York, N.Y.

Hoek, E. and Bray, J.W., 1981, Rock Slope Engineering, 3rd Ed. Institute of Mining and Metallurgy, London.

Lu, Yunhua, 2003, Stability Analysis of the Rock Ridge Dam in the CUP-Thornton Quarry Reservoir, Thornton, Illinois. M.S. thesis, Purdue University, West Lafayette, IN

McGovney, James E., 1978, Deposition, Porosity Evolution, and Diagenesis of the Thornton Reef (Silurian), Northeastern Illinois, Ph.D. Thesis, University of Wisconsin-Madison.

Norrish, N.I. and Wyllie, D.C., 1996, Rock Slope Stability Analysis, Landslides; Investigation and Mitigation, Special Report 247, National Science Council.

Patrick Engineering Inc. (PEI), 1999, Report of Rock Dam Stability Investigation for the CUP-Thornton Reservoir Site, Thornton, IL.

Park, H.J., 1999, Risk Analysis of Rock Slope Stability and Stochastic Properties of Discontinuity Parameters in Western North Carolina. Ph.D. thesis, Purdue Univesity, West Lafayette, IN.

Park, H.J. and West, T.R., 2001, Development of a Probabilistic Approach for Rock Wedge Failure, Engineering Geology, vol. 59, 233-251.

Pray, Lloyd C., 1976, The Thornton Reef (Silurian), Northeastern Illinois, A 1976 Revisitation, paper contained in Guidebook for a Field Trip on the Thornton Reef (Silurian), Northeastern Illinois, North-Central Section of the Geological Society of America.

Priest, S.D., 1993, Discontinuity Analysis for Rock Engineering, Chapman and Hall.

Roadcap, Gorge S., Cravens, Stuart J., and Smith, Edward C., 1993, Meeting the Growing Demand for Water: An Evaluation of the Shallow Ground-Water Resources in Will and Southern Cook Counties, Illinois, Research Report 123, Illinois State Water Survey.

STS Consultants, Ltd., 2000, Revised Proposal for Work Order to Contract No. DACW23-99-D-0001, CUP-Thornton Reservoir Geologic Investigation, STS Proposal No, 13132-PP.

STS Consultants, Ltd., 2001, Geologic Mapping, CUP-Thornton Reservoir, Thornton, IL.

Tunnel and Reservoir Plan (TARP), 1999; http://www.mwrdgc.dst.il.us/plants.tarp.htm

U.S. Army Corps of Engineers, 2002, Thornton Reservoir Project website; http://www.lrc.usace.army.mil/projects/thornton.htm

Watts, C.F. and West, T.R., 1985, Electronic Notebook Analysis of Rock Slope Stability of Cedar Bluff, Virginia. Bulletin of the Association of Engineering Geologists, 22(1), 67-85.

West, T.R., 1995, Geology Applied to Engineering. Prentice Hall, Inc., Englewood Cliffs, N.J.

West, T.R., 1996, The Effects of Positive Pore Pressure on Sliding and Toppling of Rock Blocks with Some Considerations of Intact Rock Effects. Environmental & Engineering Geoscience, vol. 2, 339-353.

Willman, H.B., 1973, Rock Stratigraphy of the Silurian System in Northeastern and Northwestern Illinois, Illinois State Geological Survey Circular 478.

List of Figures