

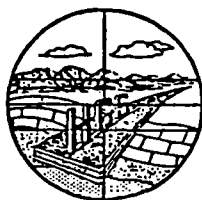
**PROCEEDINGS OF THE 38TH ANNUAL
HIGHWAY GEOLOGY SYMPOSIUM**

**"HIGHWAY CONSTRUCTION IN
UNSTABLE TOPOGRAPHY"**



**Pittsburgh, Pennsylvania
May 11, 12 & 13, 1987**

**CO-SPONSORED BY
Pennsylvania Department of Transportation
and
Engineers' Society of Western Pennsylvania**



**Proceedings of the 38th Annual
Highway Geology Symposium
and Exhibition**

**"HIGHWAY CONSTRUCTION IN
UNSTABLE TOPOGRAPHY"**

**In
Pittsburgh, Pennsylvania
May 11, 12 & 13, 1987**

co-sponsored by



Pennsylvania Department of Transportation

and



Engineers' Society of Western Pennsylvania

38th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

The 38th Highway Geology Symposium opened with welcoming remarks from Henry M. Nutbrown, District Engineer, Engineering District 11-0, Pennsylvania Department of Transportation; Donald L. Keller, Chief Soils Engineer, Pennsylvania Department of Transportation; and Chairman of Local Arrangements, Charles T. Janik, Pennsylvania Department of Transportation.

The first session was started by a presentation on the **Geology of Pennsylvania** by Donald Haskins, Director and State Geologist.

The Symposium included one and one half days of technical sessions and a one day field trip.

The field trip, lead by John DeRoss, included highway projects involved with consolidating trash dumps, constructed bridges over coal mines, construction on sliding ground, protecting bridges from slides, instrumenting walls and embankments, stabilizing rock slopes, protecting roadways from falling rocks, and excavating hillsides in ancient landslides. The trip concluded with a drive through the construction of the 400 million dollar Interstate 279 Expressway.

HIGHWAY GEOLOGY SYMPOSIUM

History, Organization, and Function

Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium was organized and held its first meeting on February 16, 1959, in Richmond, Virginia. Since then, 36 consecutive annual meetings have been held in 23 different states. Between 1950 and 1962, the meetings were held east of the Mississippi River, with Virginia, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as the host states.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona. Since then, it has rotated, for the most part, back and forth from east to west. Following meetings in Texas and Missouri in 1963 and 1964, the Symposium moved to Lexington, Kentucky in 1965; Ames, Iowa in 1966; Lafayette, Indiana in 1967; back to West Virginia at Morgantown in 1968; and then to Urbana, Illinois in 1969. Lawrence, Kansas was the site of the 1970 meeting; Norman, Oklahoma in 1971; and Old Point Comfort, Virginia the site in 1972.

The Wyoming Highway Department hosted the 1973 meeting in Sheridan. From there it moved to Raleigh, North Carolina in 1974; back west to Coeur d'Alene, Idaho in 1975; Orlando, Florida in 1976; Rapid City, South Dakota in 1977; and then back to Maryland in 1978, this time in Annapolis. Portland, Oregon was the site of the 1979 meeting; Austin, Texas in 1980; and Gatlinburg, Tennessee in 1981. The 1982 meeting was held in Vail, Colorado, and in Stone Mountain, Georgia in 1983. The 35th meeting in 1984 was held in San Jose, California, and the 36th HGS was in Clarksville, Indiana. This marked a return to Indiana for HGS after 18 years.

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 engineering geologists and geotechnical engineers from state and federal agencies, colleges and universities, as well as private service companies and consulting firms throughout the country. Steering committee members are elected for three-year terms, with their elections and re-elections being determined principally by their interests and participation in and contributions 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. Some of these committees are: By-Laws, Public Relations, Awards Selection, and Publications. The lack of rigid requirements, routine, and the relatively relaxed overall functioning of the organization is what attracts many of the participants.

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

The symposia are generally for two and one-half days, with a day-and-a half for technical papers and a full day for the field trip. The symposium usually begins 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.

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. In Wyoming, the group viewed landslides in the Big Horn Mountains; Florida's trip included a tour of Cape Canaveral and the NASA space installation; the Idaho and South Dakota trips dealt principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generating site; in Maryland the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; the Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and the Tennessee trip provided stops at several repaired landslides in Appalachia. The Colorado field trip consisted of stops at geological and geotechnical problem areas along interstate 70 in Vail Pass and Glenwood Canyon, while the Georgia trip in 1983 concentrated on highway design and construction problems in the Atlanta urban environment. The 1984 field trip had stops in the San Francisco Bay area which illustrated the interaction of fault activity, urban landslides, and coastal erosion with the planning, construction, and maintenance of transportation systems. In 1985 the one day trip illustrated new highway construction procedures in the greater Louisville area.

In 1986, the field trip in Montana provided valuable information concerning geology associated with road construction in mountainous conditions.

The 1987 field trip in Pittsburgh emphasized highway construction in a northeastern urban area with challenging geological conditions and informative geotechnical solutions.

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 these proceedings are out of print, but copies of most of the last fifteen proceedings may be obtained from the Treasurer of the Symposium, David Bingham, of the North Carolina Department of Transportation in Raleigh, 27611. Costs generally range from \$5.00 to \$15.00, plus postage.

***MEDALLION WINNERS
HIGHWAY GEOLOGY 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
William Sherman	— 1980
Virgil Burgat	— 1981
Henry Mathis	— 1982
David Royster	— 1982
Terry West	— 1983
David Bingham	— 1984

* In 1969, the Symposium instituted an awards program, and with the support of Mobile Drilling Company of Indianapolis, Indiana, designed a plaque to be presented to individuals who have made significant contributions to the Highway Geology Symposium over a period of years. The award, a 3.5" medallion mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

STEERING COMMITTEE MEMBERS

1986

	Term Expires		Term Expires
Henry Mathis - Chairman Manager, Geotechnical Branch Division of Materials Kentucky Department of Transportation Frankfurt, KY 40601 Phone - (502) 564-2374	1986	C. William Lovell Professor of Civil Engineering Purdue University Grissom Hall West Lafayette, IN 47907 Phone - (317) 494-5034	1988
Vernon L. Bump - Vice-Chairman Foundation Engineer Department of Transportation Division of Engineering Pierre, SD 57501 Phone - (605) 773-3401	1987	Harry Ludowise Federal Highway Administration 610 East Fifth Street Vancouver, WA 98661 Phone - (206) 696-7738	1986
Willard McCasland - Secretary Materials Division Oklahoma Department of Transportation 200 N.E. 21st Street Oklahoma City, OK 73105 Phone - (405) 521-2677	1988	Marvin McCauley California Department of Transportation 5900 Folsom Blvd. Sacramento, CA 95819 Phone - (916) 739-2480	1988
W. D. Bingham - Treasurer State Highway Geologist Department of Transportation Division of Highways Raleigh, NC 27611 Phone - (919) 733-6911	1987	David Mitchell Chief, Geotechnical Bureau Georgia Department of Transportation Forest Park, GA 30050 Phone - (404) 363-7546	1987
Joe E. Armstrong Chief Geologist Montana Department of Highways 2701 Prospect Avenue Helena, Montana 59620 Phone - (406) 444-6280	1986	Harry Moore Geological Engineering Supervisor I Tennessee Department of Transportation Geotechnical Section P. O. Box 58 Knoxville, TN 37901 Phone - (615) 933-6776	1988
John B. Gilmore Colorado Highway Department 4340 East Louisiana Denver, CO 80222 Phone - (303) 757-9275	1986	William F. Sherman Chief Geologist Wyoming Highway Department P. O. Box 1708 Cheyenne, WY 82002-9019 Phone - (307) 777-7450	1986
Joseph A. Gutierrez Mgr. Mine Planning and Development Midwest Division Vulcan Materials Company P. O. Box 4195 Winston-Salem, NC 27105 Phone - (919) 767-4600	1988	Mitchell D. Smith Engineer Research and Development Division Oklahoma Department of Transportation 200 N. E. 21st Street Oklahoma City, OK 73105 Phone - (405) 521-2671	1986
Jeffrey L. Hynes Colorado Geological Survey 1313 Sherman Street, Room 715 Denver, CO 80203 Phone - (303) 866-3520	1988	Berke Thompson Assistant Director Materials Control, Soil and Testing Division West Virginia Department of Highways 312 Michigan Avenue Charleston, WV 25311 Phone - (304) 348-3664	1988
Charles T. Janik - Chairman of Local Arrangements Soils Engineer Pennsylvania Department of Transportation 1118 State Street Harrisburg, PA 17120 Phone - (717) 787-4209		Terry R. West Associate Professor Department of Geosciences & Civil Engineering Purdue University West Lafayette, IN 47907 Phone - (317) 494-3296	1988

Term Expires**Term Expires**

W. A. Wisner
Geologist
Florida Department of Transportation
Office of Materials and Research
P. O. Box 1029
Gainesville, FL 32602
Phone - (904) 372-5304

1987

T. Leslie Youd
Brigham Young University
368 Clyde Building
Orem, Utah 84057
Phone - 801/378-6327

Burrell S. Whitlow
President
Geotechnics, Inc.
321 Walnut Avenue (P. O. Box 217)
Vinton, VA 24179
Phone - (703) 344-4569; 344-0198

1986

Ed J. Zeigler, Associate
Rummel, Klepper, and Kahl
1035 N. Calvert Street
Baltimore, MD 21202
Phone - 301/247-2260

1986

**EMERITUS MEMBERS OF THE STEERING COMMITTEE FOR THE
HIGHWAY GEOLOGY SYMPOSIUM**

R. F. Baker
Virgil E. Burgat
Robert G. Charboneau
Hugh Chase
A.C. Dodson
Walter F. Fredericksen
John Lemish
George S. Meadors, Jr.
W. T. Parrot
Paul Price
David L. Royster

**38th HIGHWAY
GEOLOGY SYMPOSIUM
LIST OF EXHIBITORS**

A. R. CHAMBERS & SON, INC.

CENTRAL MINE EQUIPMENT CO.

du PONT/REMAI, INC.

ELASTIZELL CORPORATION OF AMERICA

GAI CONSULTANTS, INC.

INTERNATIONAL CONSTRUCTION EQUIPMENT, INC.

KRUPP INDUSTRIES

LANG TENDONS, INC.

MACCAFERRI GABIONS, INC.

MEARL CORPORATION

NICHOLSON CONSTRUCTION COMPANY

PENNSYLVANIA DRILLING COMPANY

PREFORMED LINE PRODUCTS COMPANY

STS D'APPOLONIA LTD.

TENSAR, INC.

VIBRA-TECH ASSOCIATES

LIST OF REGISTRANTS

38th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

ABRAMSON, LEE
 ADAMS, WILLIAM R.
 AMEEL, MICHAEL E.
 ANDERSON, MARK
 ANDERSON, SUSAN L.
 BACHUS, ROBERT C.
 BEERBOWER, R. F.
 BENDO, JERRY
 BINGHAM, WILLIAM D.
 BISLER, ROGER
 BOONE, R. L.
 BOYER, BRUCE
 BRUCE, D. A.
 BRUCE, RICHARD L.
 BRUHN, ROBERT W.
 BUTCHKO, STEPHEN
 BYDLON, B.
 CANANAN, WILLIAM
 CARLSON, DAVID E.
 CHALUVADI, S. V.
 CHANG, CHIEN-TAN
 CHHABRA, GREG
 COWELL, MIKE
 DALEY, JEAN H.
 DASH, UMAKANT
 DAVIES, WILLIAM E.
 DEBUS, C. DENNIS

 DELACIO, JOHN
 DE ROSS, JOHN
 DETMAN, DOUG
 DIVINEY, JOHN G.
 DOYLE, EUGENE P.
 DZIUBEK, JOHN A.
 FAGAN, JOSEPH P.
 FULLERTON, TIMOTHY M.
 GALANKO, W. R.
 GAVETT, ROBERT M.
 GILLILAND, JOHN H.
 GRAY, EVERETT
 GRAY, RICHARD E.
 HAMEL, JAMES V.
 HAMER, GREG
 HAYDEN, MYRON L.
 HAZEN, GLENN
 HEMLOCK, TED
 HENDERSON, BOB
 HERLOCHER, ROY H.
 HILL, ED
 HOWELL, PAUL
 HYNES, J. L.
 IRANI, KHODI
 JANI, CHARLES T.
 KELLER, DONALD
 KELLER, KATHY
 KERN, ROY
 KIEFER, JOHN D.
 KONKLE, MICHAEL A.
 LANE, RICHARD
 LEDNEY, CHARLES
 LEECH, T.
 LEGATSKI, LEO A.
 LEGEZA, E. T.
 LEICHTNER, CHARLES H.
 LONG, MICHAEL

 Parsons Brinckerhoff, One Penn Plaza, New York, NY 10119
 Adams Geotechnical Consultants, P. O. Box 1319, Washington, PA 15301
 Ackenheil Consulting Group, 1000 Banksville Road, Pittsburgh, PA 15216
 L. Robert Kimball & Associates, 615 W. Highland Avenue, Ebensburg, PA 15931
 Ackenheil & Assoc. Geo. Systems, Inc., 1000 Banksville Road, Pittsburgh, PA 15216
 Georgia Tech., School of Civil Engineering, Atlanta, GA 30332
 Penn Dot 11-0, 4 Parkway Center, 875 Greentree Rd., Pittsburgh, PA 15220
 Penn Dot 10-0, Indiana, PA 15701
 N.D. Dot, Box 25201, Raleigh, NC 27611
 Central Mine Equip. Company, P. O. Box 91129, Cleveland, OH 44101
 Penn Dot, Box 459, N. Gallatin Ave. Ext., Uniontown, PA 15401
 Maryland State Highway Administration, 2323 W. Joppa Road, Brooklineville, MD 21022
 Nikon Corporation, P. O. Box 98, Bridgeville, PA 15017
 Ground Improvement Techniques, Inc., P. O. Box 17340, 10 Duff Rd., Pittsburgh, PA 15235
 GAI Consultants, Inc., 570 Beatty Road, Monroeville, PA 15146
 The Tensar Corporation, 1270 Ave. of the Americas, Rockefeller Ctr. 2213, New York, NY 10020
 PA. Turnpike Commission, P. O. Box 8531, Harrisburg, PA 17105
 Dept. of Geology SIU-C, Carbondale, IL 62901
 U. S. Army Corps of Engineers, 1000 Liberty Ave., RM 1926, Pittsburgh, PA 15222
 Penn Dot, 875 Greentree Road, Pittsburgh, PA 15220
 Federal Highway Admin., 6300 Georgetown Pike, McLean, VA 22101
 State of Connecticut, 24 Vega Street, New Britain, CT 06015
 The Tensar Corporation, 1210 Citizens Parkway, Morrow, GA 30260
 Nicholson Construction Co., P. O. Box 98, Bridgeville, PA 15017
 PA. Dept. of Transportation, P. O. Box 2926, Harrisburg, PA 17120
 125 West Green Way Blvd., Falls Church, VA 22046
 Maryland State Hgwy. Admin., Bur. of Soils and Foundations, 2323 W. Joppa Rd., Brooklandville, MD 21022
 Penn Dot, 875 Greentree Road, Pittsburgh, PA 15220
 450 Balconade Drive, Pittsburgh, PA 15236
 Woodward-Clyde Consultants, 1300 Piccard Drive, Rockville, MD 20850
 Gannett Fleming, P. O. Box 1963, Harrisburg, PA 17105
 Krupp Industries, Inc., 1370 Washington Pike, Bridgeville, PA 15017
 Michael Baker, Box 280, Beaver, PA 15009
 Engineering Mechanics, Inc., 4636 Campbells Run Road, Pittsburgh, PA 15205
 Consolidation Coal Company, Consol Plaza, Pittsburgh, PA 15241
 Penn DOT, Box 459, N. Gallatin Ave. Ext., Uniontown, PA 15401
 A. R. Chambers & Son, Inc., 111 35th Street, Pittsburgh, PA 15201
 Penn Dot 8-0, 21st & Herr Streets, Harrisburg, PA 17055
 Kentucky Dept. of Transportation, Wilkinson Boulevard, Frankfort, KY 40622
 GAI Consultants, Inc., 570 Beatty Road, Monroeville, PA 15601
 Hamel Geotechnical Consultants, 1992 Butler Drive, Monroeville, PA 15146
 International Construction Equipment, Inc., 301 Warehouse Dr., Matthews, NC 28105
 University of South Florida, Dept. of Civil Engr., Tampa, FLA 33620-5350
 Ohio University, 147 Stocker Center, Athens, OH 45701
 Sprague & Henwood, Inc., Box 446, Scranton, PA 18501
 Eastern Cellular Concrete, Hazelton, PA
 Penn DOT, Bellefonte, PA 17823
 PA. Dept. of Transportation
 USDA - SCS, 1271 Bordeaux Drive, Lexington, KY 40504
 Colorado Geo. Survey, 1313 Sherman St., #715 Denver, CO 80203
 R & R International, Inc. 3675 Copley Rd., P. O. Box 4383, Akron, OH 44321
 Penn DOT, 1118 State Street, Harrisburg, PA 17120
 PA. Department of Transportation, 1118 State Street, Harrisburg, PA 17120
 Alabama Highway Dept., 1409 Coliseum Blvd., Montgomery, Alabama 36130
 Equipment Corporation of America, P. O. Box 306, Coraopolis, PA 15108
 Kentucky Geological Survey, 311 Breckenridge Hall, Univ. of Ky., Lexington, KY 40506
 A.R. Chambers & Son, Inc., 111 35th Street, Pittsburgh, PA 15201
 Dept. of Transportation, Bur. of Materials-Research, Box 483, Stickney Ave., Concord, NH 03301
 L. Robert Kimball & Associates, 615 W. Highland Avenue, Ebensburg, PA 15931
 Gannett Fleming Transportation Engrs., 875 Greentree Road, Pittsburgh, PA 15220
 Elastizell Corp of America, P. O. Box 1462, Ann Arbor, MI 48106
 PA. Department of Transportation, 3795 Snowden Road, Library, PA 15129
 Earth Support Systems, 821-F King Street, S.W., Leesburg, VA 22075
 Oregon Department of Transportation, 9002 SE McLoughlin Blvd., Milwaukie, OR 97222

LONG, STUART B.
 LOSSMAN, HORST
 LOVELL, C. W.
 LUDOWISE, HARRY
 LUTZ, WILLIAM
 MAJORIS, PAUL
 MARKUNAS, B.
 MARTIN, ARTHUR D.
 MATHIS, HENRY
 McBEAN, ALAN
 McCASLAND, WILLARD
 McDOWELL, S.
 MIKI, HIROSHI
 MILLER, JERRY
 MILLER, S. M. ORBOCK
 MITCHELL, DAVID A.
 MOORE, HARRY
 NEAMAN, D.
 NEWMAN, F. BARRY
 NOVOTNY, THOMAS
 OLMSTEAD, BRUCE
 OLSON, LARRY
 PASCOLI, WILLIAM L.
 PERIN, RICHARD J.
 PETRASIC, KERRY W.
 PHILIPOT, DON
 PORTO, VICTORIA A.
 REEVES, R. BRUCE
 REIL, JAMES W.
 REYNOLDS, CHARLA D.
 RIGGS, CHARLES
 RUDENKO, D.
 RUSH, KEN
 SCHELLING, WILLIAM L.
 SCHULTZ, JOSEPH W.
 SEKELA, JAMES
 SHAKOOR, ABDUL
 SHERMAN, WILLIAM F.
 SHIMKO, ERIC J.
 SKILTON, ROBERT D.
 SMITH, GREG
 STOKOWSKI, STEVE
 STURGES, THOMAS B.
 SULZLE, WARREN
 TAYLOR, ERNIE
 THOMPSON, BERKE L.
 THORNTON, SAM I.
 TURGEON, BETTY J.
 TURGEON, ROBERT
 ULLMAN, R. SCOTT
 UMANSKY, HAROLD
 VASKOV, EUGENE
 VENDLINSKI, JOHN
 VOYTKO, EDWARD P.
 WATTS, CHESTER F.
 WEBSTER, SCOTT
 WEST, TERRY R.
 WETZEL, RICHARD
 WHISONANT, BOB
 WHITEHOUSE, DEREK, H.
 WILLS, WILLIAM. T.
 WILSHUSEN, J. P.
 WISNER, WILLIAM A.
 WITHIAM, JAMES L.
 WRIGHT, EARL
 YOUNG, BRIAN
 ZANG, SCOTT
 ZEIGLER, EDWARD J.

U.S. Army Corps of Engineers, 1000 Liberty Ave., Pittsburgh, PA 15222
 Oman Construction Company, Inc.
 Purdue University, School of Civil Engng., W. Lafayette, IN 47907
 6308 NE 12th Avenue, Vancouver, WA 98655
 Pennsylvania Drilling, 1205 Chartiers Ave., Pittsburgh, PA 15220
 PA. Department of Transportation, P. O. Box 429, Indiana, PA 15701
 Gannett Fleming Geotechnical, P. O. Box 1963, Harrisburg, PA 17105
 Maryland State Hwy., Brooklandville, MA.
 Kentucky Dept. of Highways, Wilkinson Blvd., Frankfort, KY 40622
 Vermont DOT, 133 State Street, Montpelier, VT 05602
 Oklahoma Dept. of Transportation, 321 Sahoma Terrace, Edmond, OK 73013
 PA. Department of Transportation, Dallastown, PA 17313
 Drexel University, Philadelphia, PA 19104
 Southern Illinois University, Dept. of Geology, Parkinson Labs, Carbondale, IL 62901
 Southern Illinois University, Carbondale, IL 62901
 Georgia Dept. of Transportation, 15 Kennedy Drive, Forest Park, GA 30050
 Tennessee Dept. of Transportation, P. O. Box 58, Knoxville, TN 37901
 1057 Tennis Court Cir. #C, Indianapolis, IN 46260
 GAI Consultants, Inc., 570 Beatty Road, Monroeville, PA 15146
 Vibra-Tech Engineers, Inc. 135 Cumberland Road, Pittsburgh, PA 15237
 State of Connecticut, 1107 Cromwell Ave., Rocky Hill, CT 06067
 Olson-Church, Inc. 925 E. 17th Ave. Denver, CO 80218
 Nicholson Construction Company, P. O. Box 98, Bridgeville, PA 15017
 Consolidation Coal Company, Consol Plaza, Pittsburgh, PA 15241
 PA. Dept. of Transportation, 1118 State Street, Harrisburg, PA 17120
 Place-Crete, Inc., 2475 Arbor Blvd., Dayton, OH 45439
 PA. Dept. of Transportation, 1118 State Street, Harrisburg, PA 17120
 Lang Tendons, Inc., 143 N. Newark Road, Toughkenamon, PA 19374
 Vibra-Tech Engineers, Inc., P. O. Box 577, Hazleton, PA 18201
 4625 Fifth Ave., Pittsburgh, PA 15213
 Central Mine Equipment Company, P. O. Box 91129, Cleveland, OH 44101
 Vibra-Tech Engineers, Inc., P. O. Box 577, Hazletown, PA 18201
 PA. Dept. of Transportation, 1118 State Street, Harrisburg, PA 17020
 Maccaferri Gabions, Governor Lane Blvd., Williamsport, MD 21795
 PA. Department of Transportation Dist., 8-0, 765-C Eppley Road, Mechanicsburg, PA 17055
 U.S. Army Corps of Engineers, Federal Bldg., 1000 Liberty Ave., Pittsburgh, PA 15222
 Kent State University, Kent, OH 44242
 Wyoming Highway Dept., Geology Branch, Cheyenne, WY 82002-9019
 PA. Department of Transportation, 1140 Liberty Street, Franklin, PA 16323
 Preformed Line Products, P. O. Box 91129, Cleveland, OH 44101
 The H.C. Nutting Company, 912 Morris Street, Charleston, WV 25301
 Consulting Geologist/Petrographer, Suite A, 10 Clark Street, Ashland, MA 01721
 Pennsylvania Drilling Company, 1205 Chartiers Ave., Pittsburgh, PA 15220
 South Dakota Dept. of Transportation, 700 Broadway East, Pierre, SD 57501
 International Construction Equipment, 301 Warehouse Drive, Matthews, NC 28105
 West Virginia Dept. of Highways, 312 Michigan Ave., Charleston, West Virginia 25311
 Civil Engineering Univ. of Arkansas, Fayetteville, AR 72701
 505 Nottingham Place, Harrisburg, PA 17109
 PA. Department of Transportation, 1118 State Street, Harrisburg, PA 17120
 Ackenheil Consulting Group, 1000 Banksville Road, Pittsburgh, PA 15216
 The Mearl Corporation, P.O. Box 208, Roselle Park, NJ 07060
 Allegheny Co. Dept. of Engr. & Constr., 501 County Office Bldg., Pittsburgh, PA 15219
 Remy, Inc., P. O. Box 511, Nashville, TN 37138
 STS D'Appolonia Ltd., One Monroeville Center, Ste 500, Monroeville, PA 15146
 Radford University, Engineering Geosciences, Radford, VA 24142
 STS Consultants Ltd., 2929-C Eskridge Road, Fairfax, VA 22031
 Purdue University, Dept: Earth & Atmos. Sci., W. Lafayette, IN 47907
 Sprague & Henwood, Inc., Box 446, Scranton, PA 18501
 Institute for Engineering Geosciences, Radford Univ., Radford, VA 24142
 Virginia Department of Transportation, P. O. Box 671, Culpeper, VA 22701
 Maryland State Hwy. Admin., 2323 W. Joppa Road, Brooklandville, MD 21022
 PA. Geology Survey, P.O. Box 2357, Harrisburg, PA 17120
 Fla. Department of Transportation, Box 1029, Gainesville, Fla. 32602
 STS D'Appolonia Ltd., One Monroeville Center, Ste. 500, Monroeville, PA 15146
 Kentucky Dept. of Highways, Geotechnical Branch, Frankfort, KY 40622
 Kent State University, Kent, Ohio 44242
 Michael Baker, Box 280, Beaver, PA 15009
 Rummel, Klepper & Kahl, 48 Alco Place, Baltimore, MD 21227

38th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM TABLE OF CONTENTS

Papers presented Monday morning, May 11, 1987

**AN EMPIRICAL MODEL TO BE USED IN EVALUATING THE POTENTIAL FOR
LANDSLIDING IN ALLEGHENY COUNTY, PENNSYLVANIA**

William R. Adams, Jr., Adams Geotechnical Consultants, Washington, Pennsylvania (HGS-87-1) 1

**EXPLORATION AND ANALYSIS OF A PROPOSED ROADWAY OVER ORGANIC SOILS IN
WESTERN OREGON**

Elizabeth A. Brossard and Michael T. Long, Oregon Department of Transportation, Highway Division,
Salem, Oregon (HGS 87-2) 9

**STABILITY OF SELECTED ROAD CUTS ALONG THE OHIO RIVER AS INFLUENCED BY VALLEY
STRESS RELIEF JOINTS**

Brian T. Young and Abdul Shakoor, Kent State University, Kent, Ohio (HGS-87-3) 15

LANDSLIDE STABILIZATION IN HILLY URBAN TERRAIN

Thomas G. Leech, Gannett Fleming Transportation Engineers, Inc., Pittsburgh,
Pennsylvania; John G. Diviney, Gannett Fleming Geotechnical Engineers, Inc.,
Harrisburg, Pennsylvania; Charles T. Janik, Pennsylvania Department of
Transportation, Harrisburg, Pennsylvania (HGS-87-4) 24

FT. PITT TUNNEL NORTH PORTAL CUT SLOPES REVISITED

Alfred Ackenheil, Ackenheil Consulting Group, Pittsburgh, Pennsylvania (HGS-87-5) 30

**A PRELIMINARY LOOK AT SIMPLE BACK ANALYSIS OF ROCK SLOPE STABILITIES
UTILIZING MICRO COMPUTERS**

Chester F. Watts, Dept. of Geology, Radford University, Radford, Virginia;
Earl Frizzel, U.S. Bureau of Mines, Spokane, Washington (HGS-87-6) 42

GEOLOGICAL HAZARDS ALONG DORCON ROAD AND LR 1094

F. Barry Newman and T.R. Gower, GAI Consultants, Inc., Monroeville, Pennsylvania (HGS-87-7) 50

NEW METHODS OF HIGHWAY STABILIZATION

Dr. Donald A. Bruce and Dennis L. Boley, Nicholson Construction Company, Bridgeville, Pennsylvania
(HGS-87-8) 57

Papers presented Monday afternoon, May 11, 1987

**HIGHWAY CONSTRUCTION IN THE LAKE BED DEPOSITS, SOUTHWESTERN
INDIANA**

Terry R. West, Purdue University, West Lafayette, Indiana; (HGS-87-28) 181

GEOLOGICAL AND GEOMORPHOLOGICAL INVESTIGATION FOR CULTURAL RESOURCE EVALUATIONS

James V. Hamel, Hamel Geotechnical Consultants, Monroeville,
Pennsylvania (HGS-87-27) 73

**NONDESTRUCTIVE TESTING AND EVALUATION METHODS FOR INVESTIGATING THE CONDITION
OF DEEP FOUNDATIONS**

Larry D. Olson, Edward Church and Clifford C. Wright, Olson-Church, Inc., Denver, Colorado
(HGS 87-11) 81

**LESSONS LEARNED FROM EUROPEAN PRACTICE ON THE USE OF STONE
COLUMNS FOR SITE IMPROVEMENTS**

Robert C. Bachus, School of Civil Engineering, Georgia Institute of
Technology, Atlanta, Georgia (HGS-87-12) 88

GROUND MODIFICATION OF HIGHWAY EMBANKMENT FOUNDATION BY DYNAMIC COMPACTION

John G. Diviney, Gannett Fleming Geotechnical Engineers, Inc., Harrisburg,
Pennsylvania (HGS-87-13) 94

CONE-PENETROMETER EXPLORATION OF KNOWN SINKHOLES

Myron Hayden, D. Bloomburg, and S.B. Upchurch, University of South Florida, Tampa, Florida;
Ronald C. Williams, Williams & Associates, Clearwater, Florida (HGS-87-15) 101

CUT SLOPE DESIGN FOR A MAJOR URBAN HIGHWAY IN THE PITTSBURGH AREA

James M. Sheahan, HDR-Richardson Gordon, Pittsburgh, Pennsylvania (HGS-87-16) 108

KARST vs. HIGHWAY DITCHLINES IN EAST TENNESSEE

Harry L. Moore, Tennessee Department of Transportation, Knoxville, Tennessee (HGS-87-17) 117

CURES FOR SLOPE FAILURES IN ARKANSAS

- Sam I. Thornton, Civil Engineering, University of Arkansas, Fayetteville, Arkansas;
W.E. Kirkpatrick, Arkansas Highway and Transportation Department, Little Rock,
Arkansas (HGS-87-18) 124

ANCHORED SOLUTIONS FOR UNSTABLE TOPOGRAPHY

- Charles H. Lechner, Earth Support Systems, Leesburg, Virginia (HGS-87-19) 127

ASSESSMENT OF LANDSLIDE POTENTIAL ALONG ROUTE 3, SOUTHERN ILLINOIS

- S.M. Orbach Miller, William Canavan, and R. Craig Kochel, Dept. of Geology,
Southern Illinois University, Carbondale, Illinois (HGS-87-20) 131

THE USE OF GEOSYNTHETICS TO SUPPORT ROADWAYS OVER SINKHOLE PRONE AREAS

- R. Bonaparte, Geo Services, Inc., Boynton Beach, Florida; R. Berg and S. Butchko,
Tensar Corporation, New York, New York (HGS-87-21) 138

GROUND MAGNETIC STUDIES IN APPALACHIAN VALLEY KARST

- Steven J. Stokowski, Jr., Consulting Geologist/Petrographer, Ashland, Massachusetts (HGS-87-22) 147

ROCK SLOPE MODIFICATION ABOVE THE NORTH PORTAL OF THE MT. WASHINGTON TUNNEL

- Edward P. Voytko, Vincent Scovazzo, and Neil Cope, STS D'Appolonia Ltd., Monroeville, Pennsylvania
(HGS-87-23) 155

THE EFFECT ON HIGHWAYS OF SURFACE SUBSIDENCE RESULTING FROM LONGWALL COAL MINING

- Glenn A. Hazen and Shad Sargand, Civil Engineering Dept., Ohio University,
Athens, Ohio (HGS-87-24) 163

ROCK SLIDE ON I-81, NORTHEASTERN PENNSYLVANIA

- J. Peter Wilshusen and Jon D. Inners, Pennsylvania Geological Survey,
Harrisburg, Pennsylvania; Duane D. Braun, Bloomsburg University, Bloomsburg, Pennsylvania
(HGS-87-25) 171

ROADWAY RELOCATION THROUGH ABANDONED MUNICIPAL DUMPS: A CASE STUDY NEAR HERSHEY, PENNSYLVANIA

- Bernard Markunas, Gannett Fleming Geotechnical Engineers, Inc.,
Harrisburg, Pennsylvania (HGS-87-26) 175

An Empirical Model to be Used in Evaluating the Potential for Landsliding in Allegheny County, Pennsylvania

WILLIAM R. ADAMS, JR.
Adams Geotechnical Consultants
Washington, Pennsylvania

HGS-87-1

INTRODUCTION

ALLEGHENY COUNTY, PENNSYLVANIA is an urban county located in southwestern Pennsylvania which is in the northeastern part of the United States. Physiographically the county lies in a plateau in the mature stage of erosional development with most of the area in slope. The total relief is on the order of 213 m (700 ft) with local relief being typically 61-122 m (200-400 ft).

The county has long been recognized as being in a landslide-prone region (Baker and Chieruzzi, 1959; Briggs and others, 1975; Radbruch-Hall and others, 1982). The geomorphic processes of diastrophism, stress relief, weathering, erosion, and mass-wasting have acted upon the interbedded weak and resistant rock types underlying the county to produce slopes with marginal stability. As a result, numerous slope movements occur naturally on these slopes particularly during times of severe climatic conditions. The problems have been magnified by the actions of man through farming, urbanization, and mining.

Although not generally life-threatening, slope movements in the county are costly to man in terms of both money and time. Some investigators have estimated costs of as much as \$4 million per year (Fleming and Taylor, 1980); however, the lack of complete and accurate cost data probably results in significantly underestimating the magnitude of the landslide problem. Many kinds of indirect costs (such as, reduced property values and loss of productivity due to interruption of transportation system) result from slope movements. These indirect costs are difficult to determine and are seldom, if ever, identified (Adams, 1986).

The causes of slope movements in Allegheny County are generally known and earlier studies (Ackenheil, 1954; Ackenheil and Associates, 1968; Pomeroy,

1982; Pomeroy and Davies, 1975; Winters, 1972) have examined the stability of slopes throughout the county. These studies typically made qualitative presentations of the importance of certain factors in contributing to slope movements, and occasionally used simple descriptive statistical methods to illustrate the importance of individual variables. However, no successful method had been developed to evaluate the potential for slope movement at a site without preparing detailed mapping or performing engineering analyses requiring sampling, laboratory tests, and detailed surveying.

This paper presents an empirical model to be used in evaluating the potential for slope movement at a site during the preliminary/planning stages of a project. The variables are structured so that input data for the model can be obtained from an office study and field reconnaissance. The Delphi method was used to introduce subjective judgment into the importance ratings of the variables involved in slope movements.

The goal was to develop a method based on a process-response model which can be used to identify general characteristics of slope stability. This initial attempt was not expected to account for all possible conditions leading to slope failure.

During the preliminary stages of a project an engineering geologist formulates a qualitative evaluation of the potential for a slope's failure based on his subjective evaluation of the conditions and processes acting at a given site. If the validity of this model can be proven, such an investigator will have a method to improve and quantify his subjective evaluation of the slope's potential for movement. The need to be able to communicate in a more quantitative manner is easily appreciated by any engineering geologist who has tried to present his subjective

evaluation of a site's stability and justify his recommended course of action to project managers, clients, government officials, or others in the decision making process.

The landslide classification scheme presented by Varnes (1978) was used because it represents the best melding of geologic phenomena with engineering principles for Allegheny County. One distinctive feature of the Varnes' classification scheme is the use of the term "slope movement" rather than "landslide" in describing all types of slope-failure phenomena. Since many of the slope failures investigated involved little or no sliding, the term slope movement was chosen to describe all slope failures in the county.

As part of the study, an inventory of approximately 720 slope movements was developed. Approximately 220 of these were investigated in detail through published soils and geologic information and a visit to each site which consisted of a general surface reconnaissance of the slope movement and surrounding area. Detailed information (such as, boring logs) was available for a very limited number of sites.

Data collection on these sites was initially guided by a check list and later by more detailed inventory forms. The pertinent data to be used in the

DEVELOPMENT OF THE MODEL AND APPLICATION OF THE METHOD FOR EVALUATING SLOPE MOVEMENT POTENTIAL

Since the purpose of this study is to develop a method to be used by engineering geologists when evaluating the potential for slope movement at a given site, it was necessary that a model be developed whose elements could be represented in a reproducible, numerical form. The slope-movement problem was thought of as a process-response model with the process elements (causes) separated into two groups of factors. The first group consists of the initial controlling factors (such as, geometry and materials) and the second group consists of the energy factors (for example, geological agents) applied to the initial elements. In this study the response element is considered to be the potential for failure measured on an arbitrary scale without attempting to describe the resulting geometry and material factors. This is an obvious simplification of the actual phenomenon since there is a complex interrelationship among most of the elements. However, it was hoped that this approach would identify general

characteristics and trends.

From previous experience it was known that the material properties and geometrical attributes would not be known adequately enough to develop the model using a deterministic or rational approach; that is, an approach where the model is developed from basic physical laws and all the independent variables controlling the failure mechanisms are identified. The alternative is to use an empirical approach.

With an empirical approach one must establish workable relationships between measurable variables. No attempt is made to describe the internal mechanisms of the system explicitly. Support for this approach results from the belief that even if the potential failure mechanism(s) at a given site are understood, the variables required for a deterministic approach generally could not be measured during the preliminary stages of an investigation.

Since numerous variables were measured for each slope movement used in the study, consideration was given to using some multivariate statistical analysis to begin structuring the conceptual model. Factor analysis and multiple regression analysis were considered to be the most appropriate. However, it quickly became apparent that the limitation upon the development of the slope-movement model would not be the available methods of analysis but the ability to quantify the observable data. Although the data could be quantified, the majority could not be measured on scales suitable for use in the analyses being considered. The lack of complete data sets and multiple variable values, also, created problems in utilizing the methods and the available computer programs. Therefore, it was decided to develop a purely empirical model to describe the slope movement phenomenon in the county.

The method chosen involved the determination of the factors that can contribute to slope instability and forming of them into measurable variables. The variables were rated according to their importance in the slope-movement phenomenon. The importance rating were established using the subjective judgment of a group of experts. Unlike many previous studies that appear to incorporate subjective judgment rather informally, it was decided that the opinions of experts would be formally incorporated into this model using the Delphi method.

In very general terms the Delphi technique can be described as a method for structuring a group communication

process (Linstone and Turoff, 1975) and is used in a variety of fields where hard data are not available or are too costly to obtain. In this study the process involved sending an initial questionnaire to a group of 18 experts on slope movements in Allegheny County. They were asked to rate certain variables that could contribute to slope movement specifically in Allegheny County.

The responses from the first questionnaire also included recommendations from the experts on additional variables that they thought should be considered. All results were summarized and a second questionnaire was sent to the experts.

Since the model being developed is intended to be used in the preliminary stages of a project when time and funds are limited, the variables were structured to minimize and simplify data acquisition. It makes no sense to develop an empirical model that is so complex that acquiring the data for it is as costly and time-consuming as acquiring data for an existing, deterministic method of analysis.

The following variables were established and rated by the experts:

- * SMV(1)-Orientation of slope face
- * SMV(2)-Gradient of slope
- * SMV(3)-Position on slope
- * SMV(4)-Slope configuration
- * SMV(5)-Surcharge on slope

- * SMV(6)-Removal of lateral support
- * SMV(7)-Removal of underlying support
- * SMV(8)-Lateral pressure
- * SMV(9)-Previous movement
- * SMV(10)-Stratigraphic unit(s)
- * SMV(11)-Attitude of bedding
- * SMV(12)-Bedrock discontinuities
- * SMV(13)-Increase in water content and/or change in groundwater level or pressure
- * SMV(14)-Inappropriate human actions

Shown in Fig. 1. are the mean rating values for each variable and the error statistic. Using a 90% degree of certainty, it was found that the actual mean value for the variable ratings of all but one of the variables lies within a value of 1.0 above or below the calculated sample means. The variable SMV(7), removal of underlying support, has an error value greater than 1.0. This is thought to be due, in part, to inconsistencies in the interpretation of the variable by different experts. Unfortunately, this was not detected until the ratings from the last questionnaire were being tabulated, and as a result, no corrections could be made for these inconsistencies.

The process of reducing the number of variables to be incorporated into the slope-movement model was started. The

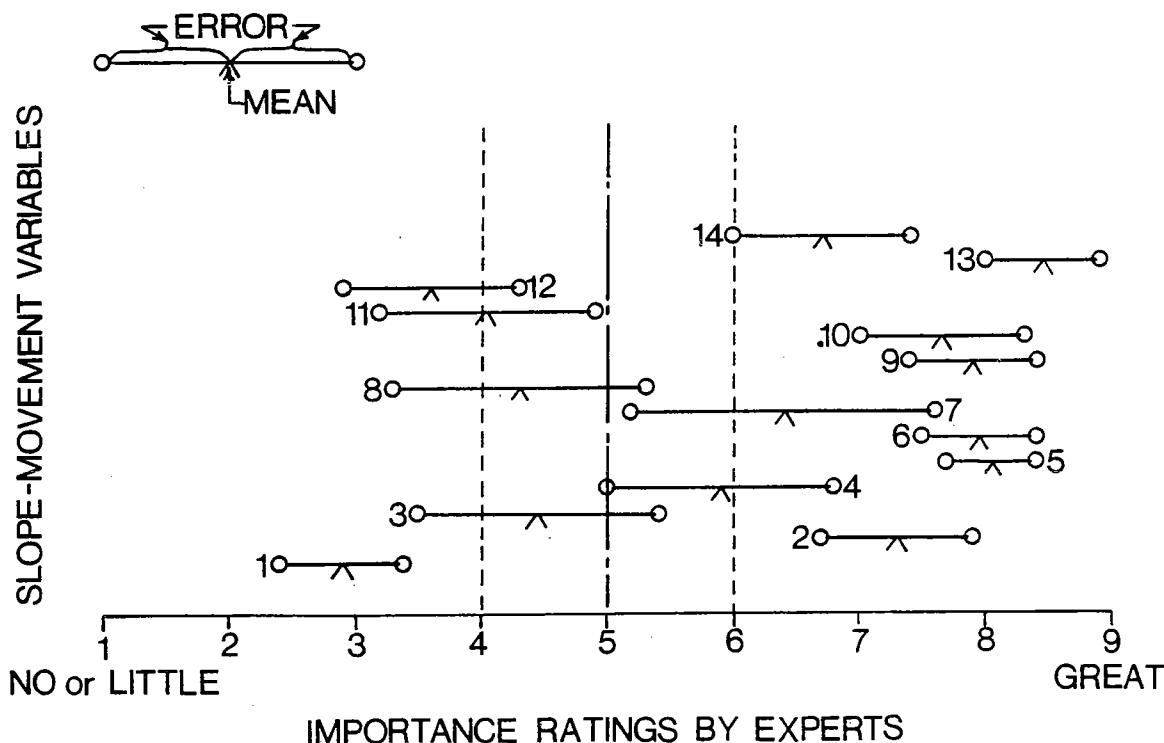


Fig. 1 Ratings for movements in soil.

rating value of 5 was chosen as the value at which the ratings began to indicate that the variable was important in the slope movement process. The confidence interval of 1.0 was then applied to the importance rating of 5 (see Fig. 1).

It was decided that if the mean and confidence interval for a variable were above the rating of 6, the variable should be retained. If the mean and confidence interval fell below the rating of 4, those variables would be excluded from the model. Finally, if all or part of the confidence interval for a variable fell in the range of 4 to 6, additional criteria would be used to evaluate the variable's importance.

The findings of detailed investigations of slope movements included in this study and the findings from other studies in the region were then used to further evaluate the variables to determine if they should be retained. Nonparametric statistical tests were used where possible to assist in making the decision. Examination of the variables in this manner resulted in the retention of 10 of the original 14 variables.

A linear model form was chosen to relate the independent and quasi-independent variables of the slope movement model to a dependent variable representing the stability of the slope. The numerous interactions among the conditions and processes affecting slope stability almost certainly result in a nonlinear relationship. However, it was considered impractical, based on existing knowledge and data available, to attempt to develop a nonlinear relationship for the model. The linear model was chosen, also, because it has been found to be a very good approximation in many situations involving a number of variables (Krumbein and Graybill, 1965).

Using the linear model form and the 10 variables, the model is as follows:

$$\begin{aligned} \text{Slope Movement Stability Rating (SMSR)} = & 7.3 * \text{SMV}(2) + 4.5 * \text{SMV}(3) + 5.9 * \text{SMV}(4) \\ & 8.1 * \text{SMV}(5) + 7.9 * \text{SMV}(6) + 6.4 * \text{SMV}(7) + \\ & 7.9 * \text{SMV}(9) + 7.7 * \text{SMV}(10) + \\ & 8.5 * \text{SMV}(13) + 6.7 * \text{SMV}(14) \end{aligned}$$

The factor in front of each variable is the mean value of the rating of that variable by the group of experts. The Slope Movement Stability Rating (SMSR) represents the response element and will be used to represent the potential for failure of a given slope. The range of 0 to 1 was chosen for the variable values used in the model. A value of 0 represents the absence of the variable's influence, and a value of 1 represents the full presence of the variable's influence.

The SMSRs were calculated for 25 slope movements which had been field visited during the study. The data from these investigations had not been used in any of the tabulations used in developing the model. The results are plotted in Fig. 2 on a 1-cycle logarithmic scale. The range in SMSRs for a given slope movement result from uncertainties in the data.

Since the objective of the study was to evaluate the likelihood of slope failure at a site, zones of low, moderate, and high potential were established. Divisions were made by visual inspection to establish ranges where the majority of the movements occurred within the high potential range, a significantly lesser number occurred within the moderate potential range, and few, if any, occurred within the low potential range. For sites known to be totally within one range or another, 76% were within the high potential range and 24% were within the moderate potential range.

To verify the model, the SMSRs were calculated for 5 slope movements from each of two recent slope movement studies in Allegheny County (Rosenquest, 1978; Yeager, 1981). It was determined that for those slope movements that fell totally within one range or another, 71% were within the high potential range and 29% were within the moderate potential range. This compares well with the results from the 25 slope movements used to establish the ranges for the method.

Extreme conditions were modeled using dummy variables to further try and verify the model. To accomplish this the variables were first separated into two basic groups. One group approximately represented the natural conditions and processes, including water conditions, and the other group represented what could be described as triggering agencies (such as, surcharges).

With all the variables describing the general physical setting given their minimum possible values, two triggering activities were required to be present before the hypothetical site was classified as having moderate potential for movement. All four triggering activities were necessary before the site was classified in the high potential range.

A hypothetical site at the other extreme (that is, a site with the variables representing the general physical setting given their maximum values) was classified as having a moderate potential for failure even without any triggering activities. The site was moved into the high potential

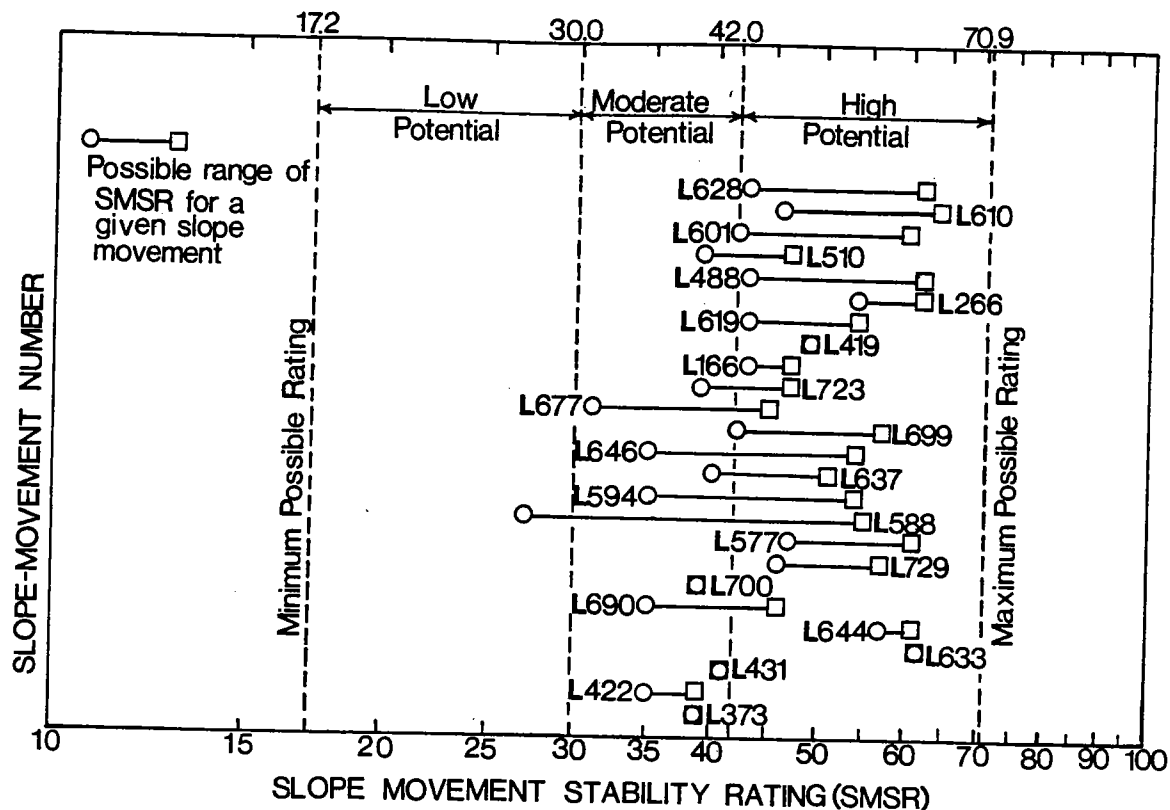


Fig. 2 Ranges for slope-movement potential derived from linear model.

range when only one triggering activity was included. These latter results are not unrealistic or unexpected for a site in a region classified as landslide prone.

Even by examining only these extreme conditions, it is obvious that in the average case a unique set of conditions must be present before a slope will be classified as having a high or even moderate potential for failure. That is, a number of the factors affecting slope stability must be present and exert sufficient influence before a slope is identified as potentially unstable.

The use of the model and method can be illustrated by examining the slope movement in Fig. 3. This slope movement is primarily a flow with some slumping within the mass. The movement occurred shortly after an excavation was made in the slope for construction of the building foundation shown on the left side of the photograph. The shoulder area of the two lane roadway and support for a portion of one lane of the roadway were lost. A water line and gas line were exposed in the scarp of the movement. Prior to disconnecting and rerouting the utilities, the water line ruptured.

A Slope Movement Stability Rating

calculated for the site prior to construction would be 39. As can be seen from Fig. 2, this would have placed the site in the upper portion of the moderate potential range. If a triggering mechanism (such as, removal of lateral support) is included, the SMSR is approximately 47 which is in the high potential range.

If the method being described in this paper had been available prior to development, it could have been used to evaluate the site and proposed construction. The potential problems could have been identified prior to development and a relocation or redesign of the development could have occurred with a lessening in the potential for failure at the site.

SUMMARY AND CONCLUSIONS

It appears that the conceptual model developed in this study shows promise of being able to identify the general conditions and processes that could create a potential for slope instability in soil at most locations within Allegheny County, Pennsylvania. The linear model developed has the following form:

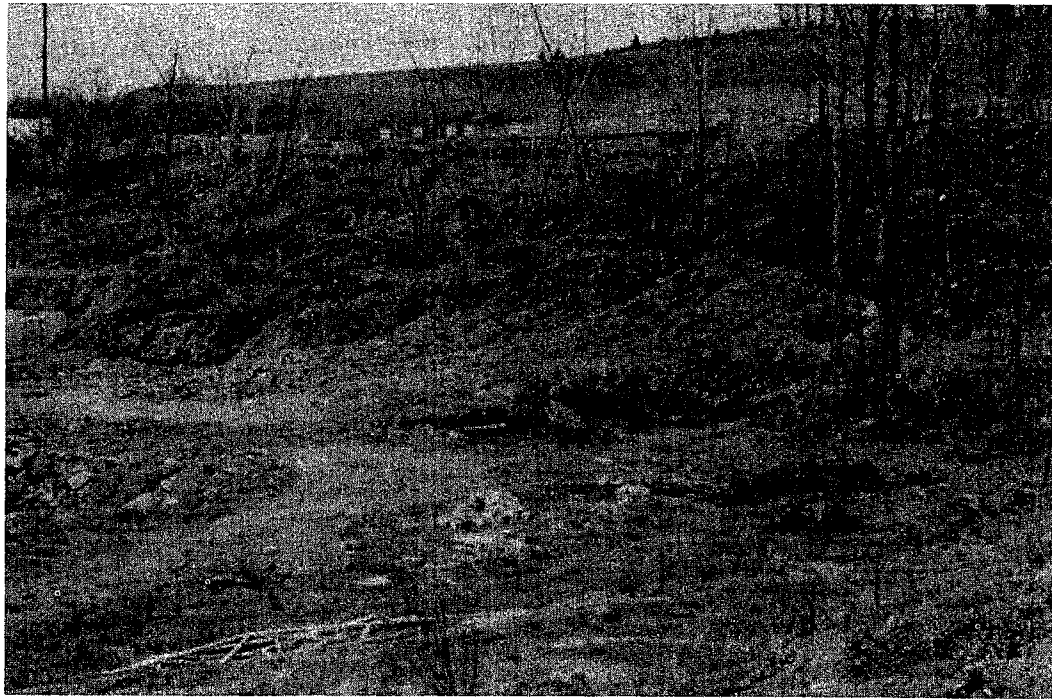


Fig. 3 Slope movement in western Allegheny County.

$$\begin{aligned} \text{Slope Movement Stability Rating (SMSR)} = & 7.3 * \text{SMV}(2) + 4.5 * \text{SMV}(3) + 5.9 * \text{SMV}(4) + \\ & 8.1 * \text{SMV}(5) + 7.9 * \text{SMV}(6) + 6.4 * \text{SMV}(7) + \\ & 7.9 * \text{SMV}(9) + 7.7 * \text{SMV}(10) + \\ & 8.5 * \text{SMV}(13) + 6.7 * \text{SMV}(14) \end{aligned}$$

The slope-movement variables (SMV()) are those found to be the most involved in the slope-movement process in the county. The value of each variable can range from 0 (representing that the influence of the variable is absent) to 1 (representing full presence of the variable's influence). The Slope Movement Stability Rating (SMSR) or response element of this model was used to establish ranges of low, moderate, and high potential for slope movement in soil (Fig. 2).

The method accomplishes the evaluation of slope failure potential, while meeting the restrictions of time and cost placed on it as a procedure to be used in the planning stages of a project. The model relies on data that are generally available from an office study and field reconnaissance of any site in the county. Data associated with large-scale mapping, subsurface investigations, laboratory testing, etc. are not required to evaluate a site when using the model.

The model utilizes the experience and judgment of experts experienced in slope failures in the county. This was accomplished by using the Delphi technique to obtain a consensus from a group of experts on the importance of various factors influencing slope

instability.

It is encouraging that the initial attempts at verifying the model indicated that a number of variables should be present before the potential for slope failure becomes significant at a site. However, even though it appears that the model provides a reasonable method for identifying slope movement potential for typical conditions in the county, it should be considered to be preliminary in nature at this time and only used as a guide by experienced investigators. One limitation of the model is that it may not adequately predict the potential for movement when only one or a small number of the variables are dominant and movement results regardless of the other factors.

It should be emphasized that this study is very definitely biased toward developed areas where man has interacted with the natural setting. The model is not only of a natural phenomenon but of a natural phenomenon that has been subjected to the processes of man.

Besides providing a method of evaluating a site's potential for movement during the planning stages of a project, it also provides the basic structure for a simple and relatively inexpensive method of quantifying an opinion that previously could only be described qualitatively. The method also provides a general approach for others working in other developed regions where similar types of data are available.

RECOMMENDATIONS ON USING THE METHOD AND FUTURE MODEL REFINEMENTS

At this stage of the conceptual model's development, it is recommended that it be used only as a guide to assist an experienced investigator in the evaluation of a given slope's potential for movement. However, the model is intended eventually to be used by both experienced and less experienced engineering geologists during the planning stages of a project when adequate data are not available to perform detailed stability analyses of a site.

In addition to the linear model, the following items should also be utilized:

- * 1:24,000 scale topographic map
- * small-scale geologic map
- * generalized stratigraphic section

These items are available for Allegheny County, Pennsylvania. If other data are available for the site (such as, aerial photographs, construction history, etc.), they also should be consulted.

After a thorough office study, the site should be field visited. It is essential that the field investigation be conducted by someone trained in the principles of engineering geology.

The method is not intended for use by the general public or a professional trained in another field. The reason for this is that many geologic and geotechnical engineering interpretations will have to be made during the field reconnaissance and when using maps, stratigraphic sections, etc.

Once the field visit has been made, the SMSR can be calculated for the site and results can be plotted as shown in Fig. 2 to determine the potential for slope movement at the site. If modifications are planned at the site (such as, placement of fill on the slope) a new SMSR can be calculated reflecting each modification.

It should be noted that although no absolute quantitative evaluation (such as, factor of safety) is suggested, a site with SMSRs in the upper portion of the moderate potential range, for example, should be considered to have a greater potential for failure than a site with a SMSR near the lower end of the moderate range. After the model is verified, the investigator will have a more quantitative way of indicating to others involved in the decision-making process what potential problems there are at the site being studied.

Regarding refinements in the model, perhaps now that the general form of the model has been developed it will be

possible to better define some variables. The inability of the model in its present form to accurately predict the potential for slope movement when one variable or a small number of variables become dominant, also, may be improved by further refinement in the number of levels or steps for variables in the 0 to 1 value range. For instance, instead of assigning a value of 1 for all surcharges, it may be possible to assign lesser values (such as, 0.25, 0.5, etc.) for many of the smaller surcharges and restrict the use of the maximum value of 1 to the condition when this variable is dominant. Also the use of control slopes (that is, slopes for which the actual degree of stability is known) should help to refine the boundaries between the various ratings.

It is important that any refinements maintain the simplicity of the model. If the model becomes so complex that the time and cost to apply it becomes substantial, the value of using an empirical model lessens and it probably would be better to use a more analytical method.

ACKNOWLEDGEMENTS

The material in this paper is drawn largely from a dissertation presented by the writer to the University of Pittsburgh as partial fulfillment of the requirements for the degree of Doctor of Philosophy (Adams, 1986). The writer is indebted to his major advisor, the late Dr. Norman K. Flint, without whose guidance, interest, and patience this study could not have been completed.

Special thanks are due Eugene W. Vaskov, Allegheny County Department of Engineering and Construction, who field visited numerous slope movements with the writer. Mr. Vaskov also reviewed the manuscript of this paper and suggested improvements. The writer, however, takes full responsibility for the findings, opinions, and conclusions expressed herein.

REFERENCES

- Ackenheil, A. C., 1954, A soil mechanics and engineering geology analysis of landslides in the area of Pittsburgh, Pennsylvania [Ph.D. thesis]: Pittsburgh, Pennsylvania, University of Pittsburgh, 117 p.
- Ackenheil, A. C., and Associates, Inc., 1968, Mining and physiography study, Allegheny County, Pennsylvania, for the Board of County Commissioners: Pittsburgh, Pennsylvania, 45 p.

Exploration and Analysis of a Proposed Roadway Over Organic Soils in Western Oregon

ELIZABETH A. BROSSARD and MICHAEL T. LONG
Oregon Department of Transportation, Highway Division
Salem, Oregon

HGS-87-2

ABSTRACT

POOR FOUNDATION CONDITIONS consisting of up to 18 feet of organic soils led to detailed investigation and design for a proposed 25-foot high embankment. The organic soils consisted of highly compressible, low shear strength peat and organic silt located in a federally designated wetlands area. Several design alternatives were considered to minimize impact on the wetlands, effect of post-construction settlement and settlement of existing highway pavement (PCC). A structure, bridging over the organic soils, was selected as the most environmentally acceptable and cost effective alternative.

INTRODUCTION

The project consists of reconstructing on and off-ramps at the Interstate 5/Myberg Road Interchange, 3-1/2 miles south of Portland, Oregon (Figure 1).

The wetlands area is relatively flat and poorly drained. Groundwater levels at the site are generally within 3 feet of the original ground surface. Standing water is common in low areas throughout.

The soils within the area consist of up to 8 feet of peat underlain by as much as 10 feet of organic silt. The peat varied from amorphous (completely decomposed) to fibrous (70% fibers) giving it a spongy appearance, characterized by a strong organic smell. The organic silt included about 25% organics in an amorphous state.

In order to build the roadway 25 feet above the wetlands, several design options were considered. Four embankment design options were evaluated, including: 1) embankments constructed with a 30-foot counterbalance and 10 or 15-foot surcharge fills; 2) an embankment constructed in stages with a 20-foot surcharge fill; 3) an embankment reinforced with three layers of high strength geogrid, with a 20-foot surcharge fill; 4) a 10-foot subexcavation of the organic soils beneath the embankments. In addition, a 1,000 foot bridge option and a 400 foot bridge with subexcavation beneath the embankment were considered. The alternatives were compared on the merits of cost, impact on the wetlands, construction time and complexity, post-construction

performance and effect on existing highway pavement (PCC).

A detailed subsurface investigation was made to determine the extent and properties of the organic soils. During the course of the investigation it was observed that a deep differential bedrock weathering profile was evident on the south end of the project. This condition made it necessary to confirm the stratigraphy with deep borings, in the event a deep pile foundation design was considered. Following completion of the field investigation, settlement and stability analyses were performed to determine the most feasible design alternative.

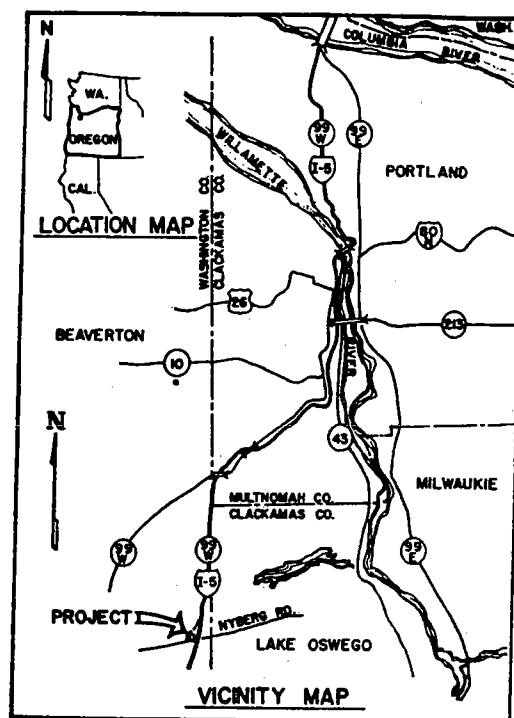


FIGURE 1

BACKGROUND

Two methods had been previously employed for embankment construction through the wetlands area. The methods included: 1) sand drain installation through the organic soils (1953), and 2) subexcavation of the organic soils (1969).

The sand drains were installed to the base of the organic soils. A 2-foot granular blanket was placed over the soil, with the remaining fill placed at a controlled rate. Settlement plates were located within the embankment. Up to 5 feet of settlement was measured in the first 9 months with more than 1 foot of post-construction settlement recorded after 10 years.

The second construction method included subexcavation of the organic soils to a depth of 10 to 15 feet. No problems were encountered during subexcavation below the groundwater table. Negligible post-construction settlement (< 2") has occurred.

Maintenance records indicate that post-construction settlement of approximately 1 foot in ten years has occurred for embankments constructed directly on the organic soils.

GENERAL GEOLOGY

STRATIGRAPHY - The stratigraphic column is presented in Figure 2. The project area,

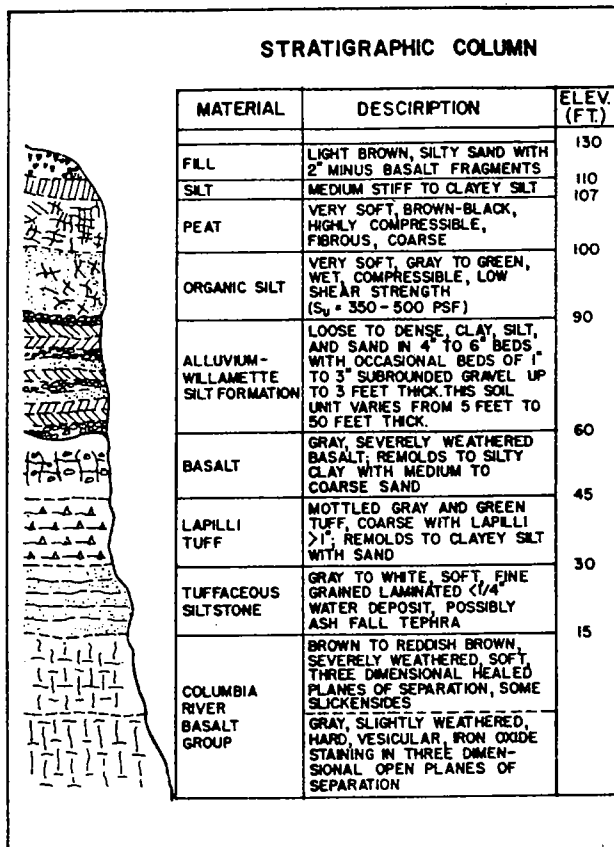


FIGURE 2

located in the Tualatin Valley on the flood-plain of the Tualatin River, is typical of alluvially deposited topography. Previous flood events (possibly the Missoula Pleistocene event) and meander channels have left alluvial deposits of the Willamette Silt Formation. This soil consists of stratified sand, silt, and clay in 4 to 6-inch layers, overlying previous erosional surfaces of the Columbia River Basalt group (Schlicker and Deacon, 1967). Local deposits of undifferentiated weathered basalt and tuffaceous siltstone were observed in the drill core overlying Columbia River Basalt in two locations. These deposits may represent the Frenchman Springs Member of the Columbia River Basalt Group and the Vantage Horizon (Paleosol) which is found overlying the Grande Ronde Member (Beeson, et al., 1985).

In the area of the Nyberg Interchange, the Willamette Silts are overlain by organic soils consisting of 5 to 10 feet of organic silt overlain by up to 8 feet of peat (Figure 3).

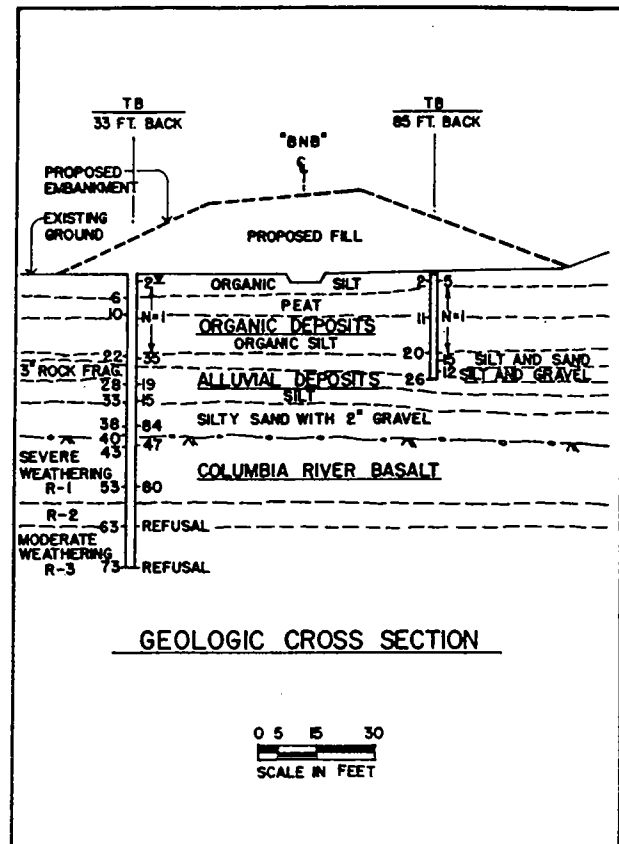


FIGURE 3

STRUCTURE - A north-south trending refraction seismic survey indicated that previous erosion and poor drainage have produced a differential weathering profile along the proposed

fill location. North of the Nyberg Slough, depth to slightly weathered bedrock is less than 40 feet; however, south of Nyberg Slough a severe weathering profile extends through the bedrock beyond 170 feet below the surface due to a previously eroded channel, which is now poorly drained. Drilling exploration confirmed the validity of this model.

The average local mass attitude of the bedrock contact was estimated to be N36°W, dipping 40° southwest. The mass attitude is based on three point calculations from contact zones observed in the drill core, projections made to outcrops in nearby Nyberg Quarry, and comparison to structural contour maps. The mass attitude of the upper surface of the bedrock was calculated in the event an end bearing pile foundation was considered. Direction and angle of dip could be used to determine pile length at any location along the alignment (Hart and Newcomb, 1965).

HYDROLOGY - An unconfined groundwater table is located within 1 to 3 feet of the surface. Groundwater recharge is continuous throughout the year by surface springs on the west side of Interstate 5, Nyberg Slough, and center draining side slopes in the project area. Average seasonal groundwater level fluctuations vary only 3 to 4 feet. The average level has been observed to be approximately 111 feet elevation, which creates 2 to 3 acres of standing water (2 feet deep) during the winter months. The area is drained by the Nyberg Slough. The design flood level for this project is at elevation 123 feet with a recurrence interval of 100 years.

INVESTIGATION

The site plan, Figure 4, shows the extent of subsurface explorations. A total of 9 hand auger holes, 18 in-situ vane shear tests and 1,100 feet of drilling in 12 borings was completed.

Vane shear tests have been determined to give reliable results in peats with high degrees of humification (Gruen and Lovell, 1983). Due to the nonhomogeneous nature of the organic soil layers through the wetlands, it was not possible to develop a consistent strength increase with depth relationship (S_u/P'_o). In general, the test results indicated a variable decrease in strength with depth relationship, which is probably due to increased amounts of fibrous organics closer to the ground surface.

Continuous core samples were obtained between standard penetration test samples. Undisturbed samples of the organic silt and peat were obtained at representative locations in the soil profile with a 2-1/2 inch thin wall Shelby tube. Due to the nature of the organic soils (low shear strength and high moisture contents), an Osterberg piston sampler was used to obtain the undisturbed samples.

The laboratory testing program for the undisturbed samples included a series of classifi-

ication tests which consisted of water contents, Atterberg limits, mechanical grain size analyses, and in-place density determinations. Consolidation tests and triaxial tests were also performed on selected samples. The results of the laboratory tests and field vane shear tests on the organic soils are presented in Table 1.

RESULTS OF LABORATORY AND FIELD
VANE SHEAR TESTS OF ORGANIC SOILS

SOIL (DEPTH RANGE)	1 W _N %	2 Y _N PCF	3 C _c	4 C _α	5 C _{FT} ² /DAY	*CIU* TRIAxIAL TEST		VANE SHEAR TEST S _u PSF
						σ'	C _{PSF}	
PEAT (4'-12')	450 TO 710	70	10.0	.01	1.0	31°	115	400 TO 700
ORGANIC SILT (5'-17')	150 TO 440	75	0.5 TO 6.5	.01 TO .02	1.0	24° TO 29°	—	240 TO 500

- 1 NATURAL MOISTURE CONTENT
- 2 MOIST UNIT WEIGHT
- 3 COMPRESSION INDEX
- 4 COEFFICIENT OF SECONDARY COMPRESSION
- 5 COEFFICIENT OF CONSOLIDATION
- 6 UNDRAINED SHEAR STRENGTH

TABLE 1

ENGINEERING ANALYSES AND CONCLUSIONS

GENERAL - Placement of embankment fills through the wetlands areas would result in excessive ground settlements. A significant amount of settlement would occur due to consolidation of the highly compressible peat and organic silt layers in the upper 18 feet. Analyses were performed for reducing the amount of post-construction settlement with surcharge (pre-load) fills.

The relatively low undrained shear strength of the organic soil presented an embankment stability problem. To maintain stability, special embankment/foundation design would have been necessary, such as reinforcement or counterbalancing the embankment and/or constructing the embankment in stages to take advantage of foundation soil strength gains due to consolidation. These requirements significantly increased the cost of embankment construction, thereby prompting the evaluation of a bridge option over the wetlands area.

SETTLEMENT ANALYSES - Primary and secondary consolidation settlements as a result of fill placement were estimated using laboratory consolidation test data and previous settlement records. Estimates of settlement along the edge of the existing PCC Interstate 5 pavement were made to determine the impact of proposed fill construction. In addition, time rate of settlement analyses were performed. In the settlement analyses, the groundwater table was

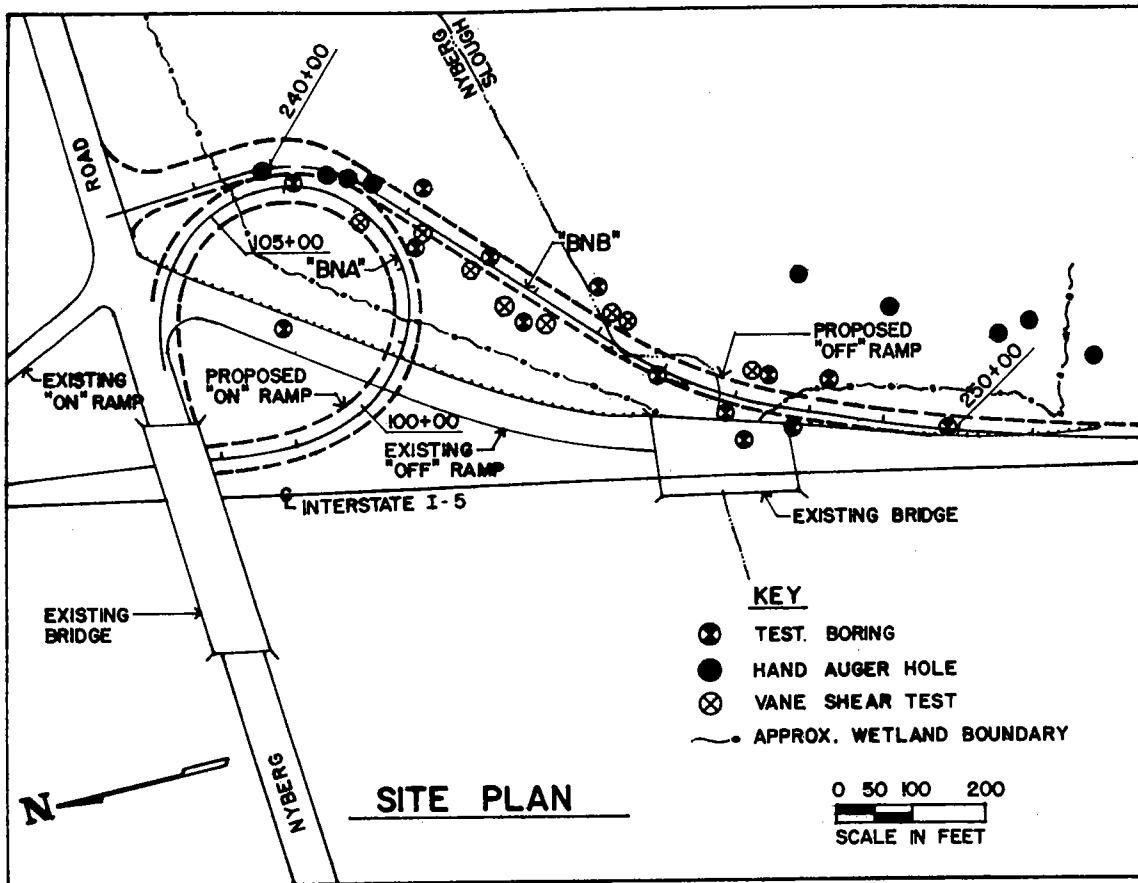


FIGURE 4

assumed to be at the ground surface. The unit weight of the embankment fill was assumed to be 110 pcf.

The total primary settlement was estimated to be about 4-1/2 feet for an embankment height of 25 feet. Primary settlement would be complete in about 4 months because of the reasonably good permeability of the organic soils. Secondary settlement of about 1-1/2 feet would be expected to occur over a 20-year period. It was determined that surcharge fills and/or sub-excavation of the compressible organic soils would be desirable to reduce post-construction settlements.

STABILITY ANALYSES - The stability of an embankment on the wetlands area was evaluated using the STABL 4 computer program (Bishops Modified Method - Critical Arc Search - CIRCL 2 Subroutine). For the reinforced embankment option, the methods described in the FHWA Geotextile Engineering Manual (1984) were used. In the stability analyses, the groundwater was assumed to be at the ground surface.

For stage construction designs, an increase in foundation soil shear strength due to consolidation at the end of each stage was assumed. The corresponding increase in shear strength was determined from relatively conservative

parameters determined from triaxial tests (CIU) on representative organic soil samples and to a lesser degree from the vane shear test results. Gruen and Lovell (1983) indicate that the triaxial compression test reasonably estimates the shear strength of organic soils, including those that are fibrous. Vane shear test results, however, are generally less accurate in soils containing fibrous organics.

DISCUSSION OF DESIGN ALTERNATIVES - Five design alternatives were evaluated to determine the most feasible, cost-effective option.

The counterbalance option with a 10-foot surcharge fill was evaluated as the least complex to construct; however, the area of wetland affected and the potential for excessive post-construction settlement made this option unsatisfactory. A 15-foot surcharge fill was determined to lessen post-construction settlement to a tolerable level, but settlement at the edge of the existing Interstate 5 PCC pavement would have become significant (about 2-1/2 inches).

A stage constructed embankment option was considered. The stage constructed embankment option included a large (20-foot) surcharge fill to reduce post-construction settlement to negligible levels. No special foundation preparation would be required. Large settlement at

the edge of the existing Interstate 5 pavement, lengthy construction time and the complexity associated with controlling rate of fill placement made this option undesirable.

A reinforced embankment option was considered with the intent of decreasing the amount of fill needed for stabilizing the embankment (no counterbalances) and decreasing the area of wetland affected. The reinforced embankment option consisted of reinforcing the base of the embankment with 3 layers of high strength geo-grid placed 12 inches apart. A surcharge fill would be required to reduce post-construction settlement. Disadvantages of this design included a potential for excessive post-construction settlement and a medium to high degree of construction complexity. Increasing the surcharge to lessen post-construction settlement would result in significant settlement to the existing highway PCC pavement.

The 10-foot subexcavation option included removing 10 feet of the organic soils and replacing it with granular material prior to embankment construction. Subexcavation could not be done, however, under the proposed embankment sections adjacent to the existing Interstate 5 fills because of the potential for instability of the existing embankment. With this constraint, the fills adjacent to Interstate 5 would require a counterbalance to maintain stability. This option would have a moderate impact on wetlands, more than the reinforced embankment option, but less than the counterbalance option. Although settlement for the subexcavated embankment section would be tolerable, the settlement of the existing PCC pavement caused by fills placed close to the existing embankment would be excessive.

A 400-foot bridge option and a 1,000-foot bridge option were evaluated. The 400-foot bridge option would include embankments constructed over a portion of the wetlands with 10-foot subexcavation of the organic soils. In either option, the bridge would extend alongside the existing highway embankment in order to minimize settlement of the existing PCC pavement. Comparison of costs of these two options indicated the longer bridge option to be more cost effective and environmentally acceptable. Further benefits of the bridge option include lower pavement grade maintenance costs and a shorter time for construction.

Table 2 represents the alternatives evaluated, with the list of the variables considered. Although the total bridge option cost exceeds the reinforced embankment cost, the bridge alternative was selected as the best design alternative. The decision to select the bridge option was based on 1) minimal impact on the wetlands, 2) short construction time, 3) standard construction procedures, and 4) favorable post-construction performance. One disadvantage of the bridge design would be the construction of a gore point where the existing structure would meet the proposed structure. The impact of a gore point would be lessened by installing and maintaining an attenuator.

SUMMARY

Proposed construction of an on and off-ramp 25 feet above a wetlands area composed of organic soils prompted a detailed investigation. Properties of the foundation materials were evaluated by means of field and laboratory testing. Several design alternatives were analyzed

COMPARISON OF DESIGN ALTERNATIVES

DESIGN ALTERNATIVES		COST	AREA OF WETLAND	CONSTRUCTION TIME	POST CONST. SETTLEMENT
COUNTER-BALANCE	10' SURCHARGE	\$1,194,200	3.5 ACRES	7 MONTHS	9" - 12"
	15' SURCHARGE	\$1,444,800	3.5 ACRES	10 MONTHS	4" - 7"
STAGE CONSTRUCTED EMBANKMENT		\$1,786,400	3 ACRES	12 MONTHS	< 4"
REINFORCED EMBANKMENT		\$1,261,400	3 ACRES	8 MONTHS	9" - 12"
SUB-EXCAVATE 10'		\$1,556,800	3 ACRES	2 MOS. - SUB-EX. 7 MOS. - C.B.	< 2" (NORTH) 9" - 12" (SOUTH)
SUB-EXCAVATE 10' WITH 400' BRIDGE		\$1,551,000	2.5 ACRES	2 MOS. - SUB-EX. 7 MOS. - BRIDGE	< 2" (NORTH) NOT APPL. (SOUTH)
1000' BRIDGE		\$1,443,000	1 ACRE	9 MONTHS	NOT APPLICABLE

TABLE 2

to determine the most feasible approach for crossing the organic soils, including four embankment options, a combination 400-foot bridge/subexcavation option, and a 1,000-foot bridge only option. The alternatives were evaluated in terms of wetlands impact, construction performance and total cost. Following a complete evaluation, the 1,000-foot bridge option was selected as the most feasible and cost-effective design.

ACKNOWLEDGEMENTS

The authors would like to thank all of the ODOT employees involved in this project. Gratitude is especially extended to Constantino Gamolo for drafting the figures. Special thanks to George Machan for his report review and technical design guidance.

REFERENCES

- Beeson, Marvin H., Karl R. Fecht, and Terry L. Tolan, 1985. Regional Correlations within the Frenchman Springs Member of the Columbia River Basalt Group: New Insights into the Middle Miocene Tectonics of Northwestern Oregon, Oregon Geology, Vol. 47, No. 8, August 1985.
- Christopher, B. R. and R. D. Holtz, 1984. Geotextile Engineering Manual, Federal Highway Administration, National Highway Institute, Washington, D.C.
- Gruen, H. A., Jr. and C. W. Lovell, 1983. Use of Peats as Embankment Foundations, Joint Highway Research Project, Indiana Department of Highways, Purdue University, West Lafayette, Indiana.
- Hart, D. H. and R. C. Newcomb, 1965. Geology and Ground Water of the Tualatin Valley, Oregon, U.S.G.S. Water Supply Paper 1697, U.S. Government Printing Office, Washington, D.C.
- Schlicker, R. G. and R. J. Deacon, 1967. Engineering Geology of the Tualatin Valley Region, Oregon. Oregon Department of Geology and Mineral Industries, Bulletin No. 60, Portland, Oregon.

Stability of Selected Road Cuts Along the Ohio River as Influenced by Valley Stress Relief Joints

BRIAN T. YOUNG
ABDUL SHAKOOR
Kent State University
Kent, Ohio

HGS-87-3

ABSTRACT

THE OHIO RIVER between Wheeling, West Virginia and East Liverpool, Ohio flows nearly due south and marks the border between Ohio and West Virginia. State Route 7 in Ohio and State Route 2 in West Virginia run parallel to the river. Both roads have been cut through thick sequences of horizontally bedded, Pennsylvanian age strata consisting of alternating sandstones, thin limestones, shales, claystones, and coals. The sandstones, limestones, and some shales are resistant, whereas the claystones and most other shales are easily eroded, resulting in undercutting. A number of steeply dipping joint sets exist in the sandstones, limestones, and shales, the most prominent being those paralleling the river and known as "valley stress relief joints". The presence of the alternating hard and soft strata, well developed jointing, and steep slopes has resulted in many unstable cuts along these two heavily traveled roads. Rock falls, plane failures, and wedge failures were identified as the common types of slope movement.

Eight sites were selected for in-depth investigations and a detailed line survey was done at each site to obtain structural data. Information regarding joint orientation, spacing, continuity, surface irregularities, infilling material, and ground water was recorded. Stereographic projection techniques were used to identify the kinematically possible modes of failure. Specific gravity and slake durability tests were performed to determine the density and weathering characteristics, respectively, of rock units involved in failure, whereas basic friction angles were measured by Stimpson's method. The density and friction values were used in quantitative analysis of plane and wedge failures by limiting equilibrium approach. The slake durability results were used to evaluate the potential for rock falls caused by undercutting.

This paper presents the results of stability investigations at three of the

sites, namely Mingo Junction and Brilliant in Ohio, and East Steubenville in West Virginia. Plane failures were found to be common at the first two sites, whereas wedge failures were indicated at all three sites. The valley stress relief joints appear to significantly influence the mode of failure. Also, the undercutting of the more resistant units by the soft shale and claystone results in frequent rock falls at these sites.

INTRODUCTION

The rock cuts along the Ohio River between Wheeling, West Virginia and East Liverpool, Ohio are very unstable as evidenced by the numerous "rock-slide" warning signs seen along State Routes 2 and 7. Because both roads are heavily traveled, these unstable cuts could pose a considerable hazard to motorists. The valley stress relief joints, which are well developed in the area, appear to strongly influence the stability in combination with tectonic joints and undercutting.

Ferguson (1967, 1974), Hamel (1971, 1980), and Ferguson and Hamel (1981) have discussed the relationship between valley stress relief jointing and slope stability in the Appalachian Plateau. According to these authors, valley stress relief joints are steeply dipping to vertical fractures which resulted from stress relief accompanying valley formation. Unlike deep-seated regional jointing created by tectonic stresses, valley stress relief joints exist near the valley, do not cut across beds of differing strength, and become less frequent with depth below the valley floor and distance away from the valley wall. The pattern of these joints depends on the thickness and competency of the strata in which they form, the competency of the adjacent strata, and their position in the valley wall. Generally valley stress relief joints are smaller and more closely spaced in the less competent rocks, such as clayey shales, claystones, and coal than in the more competent units such as sandstones and limestones. The formation of valley stress

relief joints can be explained as follows. During downcutting the residual stresses are released normal to the orientation of the river valley. Within the valley walls the weak beds are destressed by stretching. The more competent layers are dragged along and fractured, creating joints paralleling the river valley.

A study was initiated in June 1986 to investigate the slope stability problems along the Ohio River, with special reference to the role of valley stress relief joints. For the purposes of this study, all joints within 20 degrees of the river direction were assumed to be valley stress relief joints.

The specific objectives of the study were:

1. To identify the modes of slope failure within the study area.
2. To investigate the role of valley stress relief joints in causing slope movements along the Ohio River.
3. To subdivide State Routes 2 and 7 into stable, potentially unstable, and unstable sections.
4. To suggest remedial measures.

Eight sites were selected to accomplish the objectives listed above. Figure 1 shows the location of these sites. These sites were picked because of their well developed jointing, their accessibility for measuring joints, and their obvious stability problems. Only three of the eight sites will be discussed in this paper, namely Mingo Junction and Brilliant in Ohio, and East Steubenville in West Virginia. These three sites are representative of the modes of failure common in the study area. The Brilliant cut typically exhibits plane failures, the East Steubenville site has well developed wedge failures, and the Mingo Junction road cut has both types. Rock falls are common at all three sites.

METHODOLOGY

FIELD INVESTIGATIONS - The field investigations for this study involved:

1. A detailed line survey at each site.
2. Construction of a stratigraphic cross-section at each site.
3. Collection of rock samples for laboratory testing.

The detailed line survey was done according to the method proposed by Piteau and Martin (1977). It consists of placing a measuring tape along the outcrop and recording the characteristics of all the joints that intersect the tape within 3 feet above or below it. The relevant joint characteristics include orientation, spacing, continuity, nature of infilling material, nature of surface irregularities, and amount of water

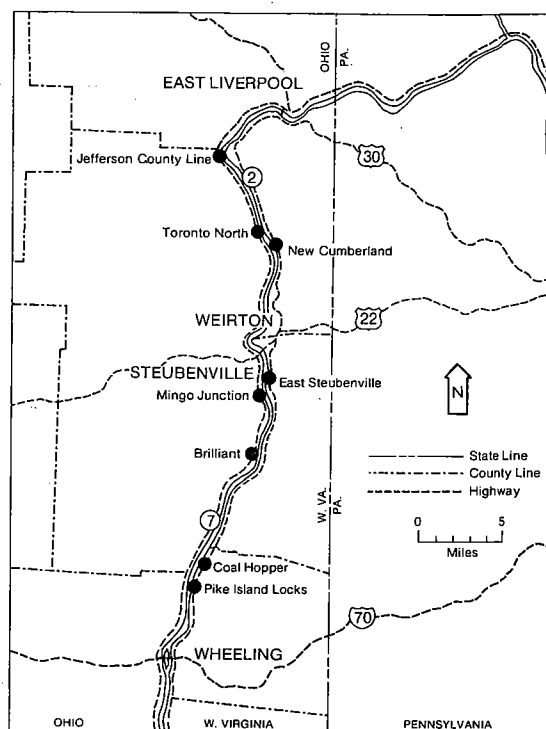


Figure 1: Map showing location of study sites.

present. Because the valley stress relief joints roughly parallel the river valley and thus the road cuts, all joints either three feet above or below the tape were chosen whether or not they intersected the tape. The line surveys were done in resistant sandstones, limestones or shales because the joints were best developed in these units.

The complex stratigraphy of the road cuts in the study area is typical of coal cyclothem. Rock units generally vary in thickness laterally, sometimes pinching out completely. Interbedding of different units is common. A rock unit may change in composition laterally, occasionally becoming a different rock type. In addition to geological variability, the slope parameters, such as slope angle, bench width, and bench filling vary along the length of the road cuts. For these reasons, the stratigraphic cross-sections constructed for each site are generalized and illustrate the characteristics representative of the road cut.

Rock samples were collected for laboratory testing in order to provide the additional data necessary for assessing slope stability at the sites. Representative samples were collected from the rock units along which the line survey was performed as well as from the units involved in undercutting.

LABORATORY INVESTIGATIONS - The laboratory investigations consisted of the following work:

1. Determination of the basic friction angle.
2. Slake durability tests.
3. Determination of specific gravity.
4. Analysis of joint orientation data and calculation of factors of safety.

The basic friction angle (ϕ) was determined for each rock unit on which a joint survey was performed, using Stimpson's method (1981). In this method three cores are placed on a platform, with one core lying above the other two. The platform is inclined until the upper core slides along the length of the two lower cores. The basic friction angle is given by the equation:

$$\text{Friction Angle } (\phi) = \tan^{-1}(1.155 \tan \alpha)$$

where: (α) = angle of inclination of the platform.

The slake durability tests were done according to the procedure outlined by Franklin and Chandra (1972). This index test provides a measure of the erodibility of rock units. The test consists of tumbling in water 10 rock chips, each weighing 40 to 60 g, for 10 minutes at 20 revolutions per minute. For this study the slake durability index was determined for every rock unit that was surveyed by the detailed line method as well as for the underlying unit where it was found undercutting the surveyed unit. This was done in order to compare the slake durability indices of the resistant units in the surveys to those of the underlying non-resistant units. The magnitude of the difference between the indices of the resistant and non-resistant rock units could possibly be related to the degree of undercutting and the resultant slope stability problems.

The specific gravity was determined according to ASTM procedure C-97. The specific gravity values were then converted to densities. The density and basic friction angle of each rock unit surveyed were used in conjunction with the joint orientation data to determine the factors of safety at each site.

The joint orientation data obtained from the line survey were plotted as poles and then contoured by the floating-circle counting method described by Hoek and Bray (1981). Major peak orientations ($>4\%$) were used to define the principal joint sets. The principal joint sets and the basic friction angle for each site were plotted on a stereonet. The Markland test (Markland, 1972) was used to determine the potential modes of failure. Factors of safety were determined using the "friction-only" method discussed by Hoek and Bray (1981). These values were checked by using "ROCKPACK", a computer program furnished by Dr. C.F. Watts of Radford University. The application of this program

in slope stability analysis has been explained in publications by Watts and West (1985, 1986). The program uses the limiting equilibrium approach as described by Hoek and Bray (1981).

STABILITY ANALYSIS

MINGO JUNCTION SITE - This road cut is located along State Route 7, approximately 1 mile north of the town of Mingo Junction, Ohio. The length of the cut is roughly 2200 yards but only the southernmost 1700 yards are well exposed. The slope angle ranges between 80 degrees and vertical for the sandstones and siliceous shales, and 40 to 45 degrees for the claystone at the base. The slope dips to the east and ranges in height from 120 to 140 ft.

As illustrated in the stratigraphic section (Figure 2), a highly weathered claystone, 8 to 30 ft thick, lies at the base of this road cut. An interbedded sequence of white sandstone, clay shale, and silty shale, averaging 20 ft thick, overlies this claystone. A red sandstone of highly variable thickness, outcrops above this interbedded sequence. At two locations this sandstone thickens at its base and rests unconformably on the claystone. Generally the thickness of this unit ranges between 20 ft and 50 ft. The

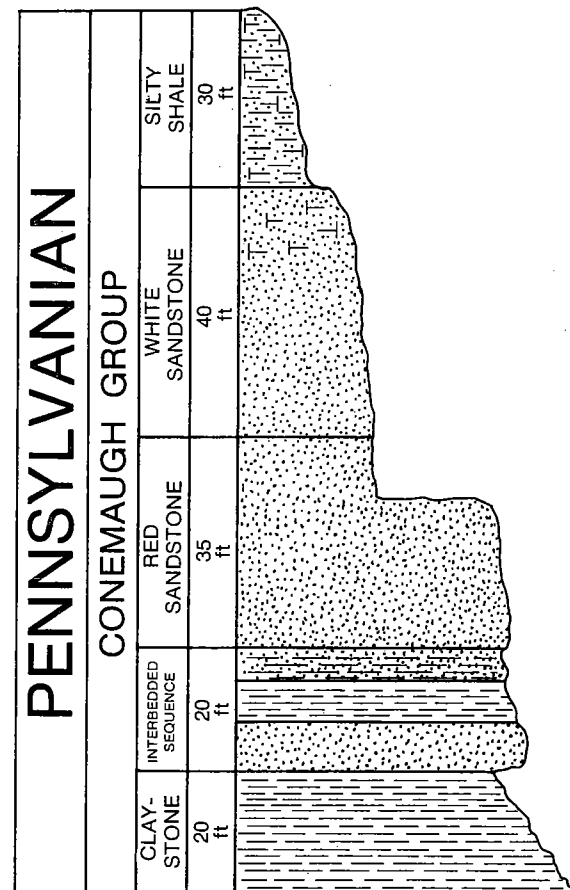


Figure 2: Stratigraphic cross-section at Mingo Junction cut.

uppermost 10 ft of this unit are cross-bedded. This red sandstone grades vertically into a white sandstone which is thickly bedded at its base, becoming more thinly bedded upwards. The more thinly bedded strata are calcareous. The thickness of this sandstone ranges between 20 ft and 40 ft. A brown calcareous silty shale, approximately 30 ft thick, is exposed uppermost at this road cut. This shale is thinly bedded and highly fractured.

The detailed line survey was completed in the white sandstone of the interbedded sequence, and a total of 165 joints were recorded. Table 1 lists the principal joint sets as determined from the contour diagram.

Table 1: Principal Joint Sets at Mingo Junction Cut.

Joint Set	Orientation	Peak Percentage
A	N61°W/70°NE	9
B	N43°W/84°NE	9
C	N22°W/80°NE	7
D	N44°E/85°SE	6
E	N88°W/85°NE	6
F	N52°W/69°SW	6
G*	N04°W/84°SW	5

* possible valley stress relief joints

The friction angle for the white sandstone, determined by Stimpson's method, is 42 degrees. The average density of the white sandstone is 164 pounds per cubic foot (pcf). The friction angle is apparently high. However, because of the steeply dipping nature of the joints in the study area, the high friction angle will not exclude any potential failures. The slope trends N33°W and is close to vertical. The stereogram shown in Figure 3 indicates the potential for 2 plane failures and 11 wedge failures. Table 2 presents the complete information about these failure modes, including their orientations and their factors of safety as determined for dry slope conditions.

The valley stress relief joints are shown by joint set G. It should be noted that the slope direction at this site varies between N33°W and N05°W. At the location of the joint survey, the slope orientation is N33°W which results mostly in wedge failures as indicated in Figure 3 and Table 2. Several of these wedge failures are caused by the intersection of joint set G with other joints. Besides wedge failures, there are good examples of plane failures. The plane failures at this site are very large where the red sandstone thickens as shown in Figure 4. A number of shallow wedges were also observed in this unit. One of these is shown in Figure 5. The wedge and plane failures occurring in the thinner units contribute to numerous rock falls at this site. Undercutting by the

claystone has promoted many failures in the white sandstone. The slake durability index for the claystone is very low (12%) while that for the sandstone is extremely high (98%). This large difference in slake durability has produced 5 to 7 ft of undercutting near the central portion of the site which promotes rock falls.

Two benches are present in this road cut. The lower bench is near the top of the red sandstone. This bench is 30 to 35 ft wide and

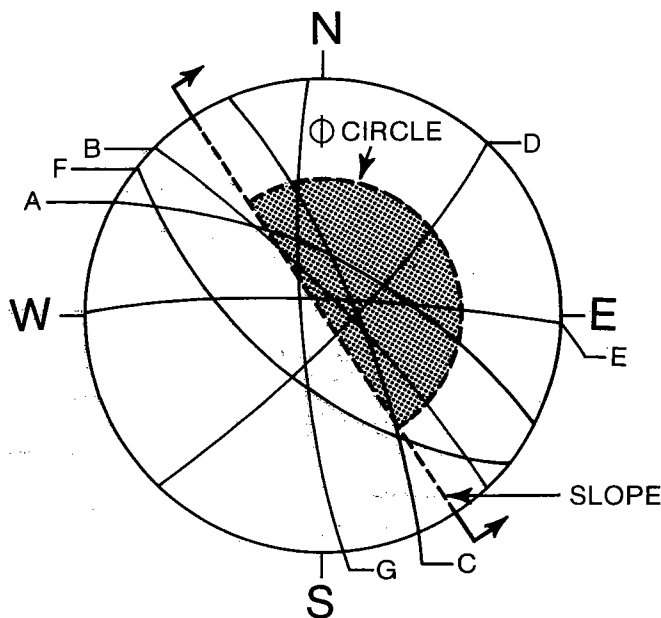


Figure 3: Stereogram of the principal joint sets at Mingo Junction cut.

Table 2: Potential Modes of Failure at Mingo Junction Cut.

Failure Mode	Joint Set(s)	Orientation *	F.S.
Plane (P1)	B	N43°W/84°NE	0.1
Plane (P2)	C	N22°W/80°NE	0.2
Wedge (W1)	B, G	N22°W/73°W	0.6
Wedge (W2)	A, G	N14°W/62°W	0.8
Wedge (W3)	C, G	N10°W/46°W	---
Wedge (W4)	A, C	N04°E/67°E	0.7
Wedge (W5)	B, E	N30°E/84°E	0.1
Wedge (W6)	A, D	N57°E/68°E	0.5
Wedge (W7)	C, E	N64°E/80°E	0.2
Wedge (W8)	A, E	N82°E/60°E	---
Wedge (W9)	B, D	N82°E/82°E	0.2
Wedge (W10)	C, B	S73°E/78°E	0.1
Wedge (W11)	C, D	N76°E/80°E	0.1

* orientations are strike and dip for plane failures, and trend and plunge of the line of intersection for wedge failures.



Figure 4: Large plane failures at Mingo Junction cut.



Figure 5: Shallow wedge failure at Mingo Junction cut. Notice the large rock blocks in the foreground.

is partially covered with rock blocks and shale talus. The upper bench has been cut at the base of the top unit. This bench ranges in width from 20 to 25 ft and has mostly been filled by shale debris sloping at 35 degrees. The ditch along the road has an average width of 20 ft and slopes away from the road.

This site appears most hazardous in the central portion because of the large amount of undercutting. Although the ditch prevents most rock blocks from reaching the road, some were observed close to it.

BRILLIANT SITE - This road cut is located four miles south of the Mingo Junction site and is just north of Brilliant, Ohio. The cut is approximately 950 yards long. The slope angle in the sandstones and siliceous shales ranges from 80 to 85 degrees, whereas the less resistant shale slopes at 45 degrees. The slope has an approximate height of 120 feet and dips to the east.

Sandstones constitute the predominant rock type in this cut. As illustrated in the stratigraphic cross-section (Figure 6), a light red, thickly bedded sandstone lies at the base. This is overlain by a 3 ft thick clay shale. A white sandstone, similar in thickness to the lower sandstone, overlies the clay shale. This sandstone is irregularly bedded. The total thickness of these three units averages 35 ft. A pale blue shale, approximately 25 ft thick, overlies the white sandstone. Above this unit, a massively bedded sandstone is exposed. This sandstone is 35 to 45 ft thick. Uppermost lies a 20 ft thick layer of interbedded shale and sandstone.

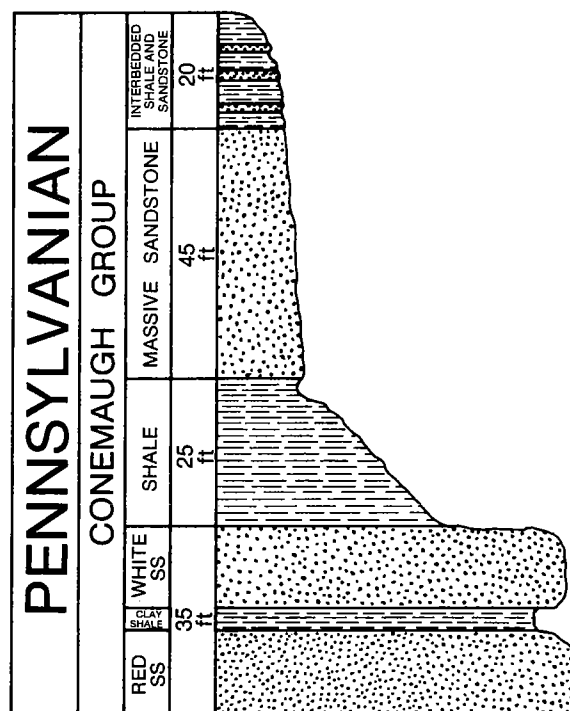


Figure 6: Stratigraphic cross-section at Brilliant cut.

The detailed line survey was done in the red sandstone exposed at road level. A total of 212 joints were recorded. Table 3 lists the principal joint sets determined from the contour diagram of this site.

Table 3: Principal Joint Sets at Brilliant Cut.

Joint Set	Orientation	Peak Percentage
A*	N05°W/85°NE	12
B	N32°E/86°SE	6
C	N32°W/86°NE	6
D	N55°W/85°NE	5
E*	N12°E/81°NW	4

* possible valley stress relief joints

The basic friction angle for the red sandstone is 38 degrees and the average density is 161 pcf. Again, the friction value is apparently high. The slope trends N06° E and the red sandstone unit dips roughly 85 degrees to the east. Table 4 lists the potential failures determined from the stereogram for this site (Figure 7). Assuming dry slope conditions, factors of safety were calculated for each failure (Table 4).

Table 4: Potential Modes of Failure at Brilliant Cut.

Failure Mode	Joint Set(s)	Orientation *	F.S.
Plane (P1)	A	N05°W/85°NE	0.1
Wedge (W1)	A,D	N55°E/83°E	0.1
Wedge (W2)	B,D	N74°E/83°E	0.1
Wedge (W3)	A,B	S82°E/84°E	0.0
Wedge (W4)	A,C	S20°E/71°E	0.0

* orientations are strike and dip for plane failures, and trend and plunge of the line of intersection for wedge failures.

Joint sets A and E represent the valley stress relief joints at this site. According to Table 4, these joint sets result in failures P1, W1, W3, and W4. The zero values of factors of safety for W3 and W4 suggest that these are wedge-shaped rock falls. Joint set E dips into the slope and could possibly result in toppling failure but evidence for such a failure was not found in the field. Several examples of well developed plane failures can be seen at this site. The one shown in Figure 8 happened in the massive sandstone. The largest rock block, on the right side of the figure has a volume of nearly 9000 cubic feet. Another plane failure occurred within two months after completing field work at this site. In this failure, a large rock block measuring 144 cubic ft, landed within 20 ft of where the joint survey

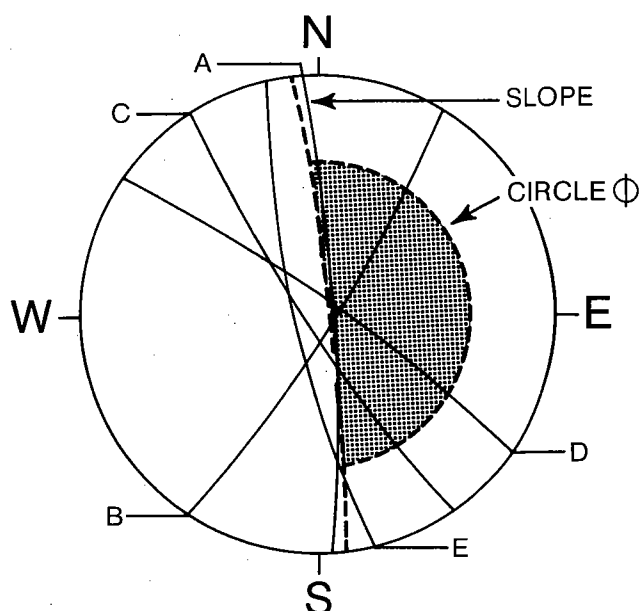


Figure 7: Stereogram of the principal joint sets at Brilliant cut.



Figure 8: Large plane failure at Brilliant cut. Notice the near right-angle joints in the upper right and the large rock blocks in the foreground.

began. No large wedge failures were observed at this site but smaller wedge failures as well as small plane failures have produced many rock falls. Undercutting of the white sandstone near the base of this cut is very pronounced. The white sandstone has an extremely high slake durability rating (98%), whereas the underlying clay shale has a low

rating (29%). This difference in slake durability has produced 3 to 6 ft of undercutting, which undoubtedly contributed to the second plane failure mentioned above.

A bench is present above the white sandstone. The width of the bench varies between 15 ft in the southern portion and 50 ft in the central portion of the road cut. Shale talus fills the southern area of the bench. The bench appears capable of catching most rock blocks except in the south where a number of rock blocks rest in the ditch. The ditch, although inclined away from the road, is only 12 to 15 ft wide. The southern section of the road cut probably poses the most danger to motorists.

EAST STEUBENVILLE SITE - This road cut is located on State Route 2 in West Virginia, across the river from Steubenville, Ohio. It is approximately 1200 yards long, with 900 yards exposed south of the East Steubenville Bridge and 300 yards exposed north of it. The cut is located predominantly in a silty, blue to green, siliceous shale. The slope angle in this unit varies between 85 degrees (dipping east) and vertical.

The stratigraphy is relatively simple at this site (Figure 9). Lowermost is a claystone, 2 to 10 ft thick, with the thickest exposures located north of the bridge. Above the claystone is the silty, siliceous shale mentioned above. The thickness of this highly fissile shale is fairly uniform and is roughly 90 to 100 ft. Above this shale is a thickly bedded sandstone with well developed cross-bedding. This unit is approximately 20 ft thick.

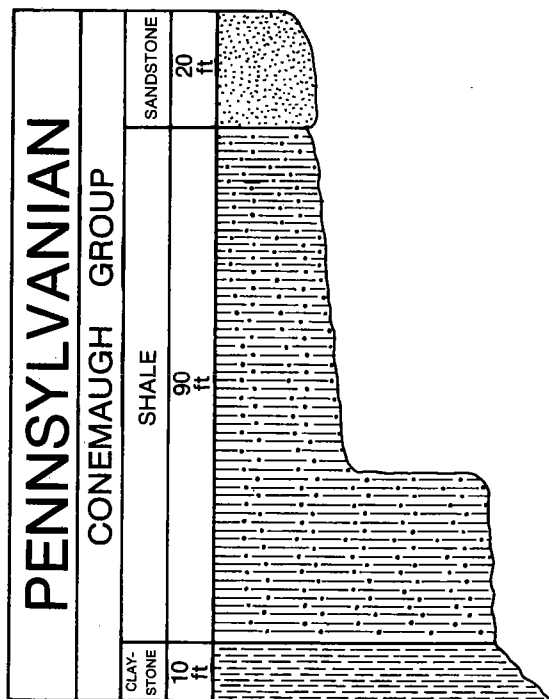


Figure 9: Stratigraphic cross-section at East Steubenville cut.

The line survey was done in the shale at this site. A total of 307 measurements were recorded. Table 5 lists the principal joint sets determined from the contour diagram.

Table 5: Principal Joint Sets at East Steubenville Cut.

Joint Set	Orientation	Peak Percentage
A	N56°W/84°SW	14
B	N32°E/85°NW	7
C	N24°E/86°NW	8

The shale has a friction angle of 39 degrees. This relatively high value can be attributed to the siliceous nature of the shale. The average density of the shale is 170 pcf. The slope trend varies from N14°W to N20°E over the length of the road cut. The slope trends N14°W at the joint survey location in the southern end of the cut. This value and a vertical slope angle were used in the stereogram for evaluation of failure modes (Figure 10). Table 6 lists the potential failures determined from Figure 10. Assuming dry slope conditions, factors of safety were calculated for the potential failures as shown in Table 6.

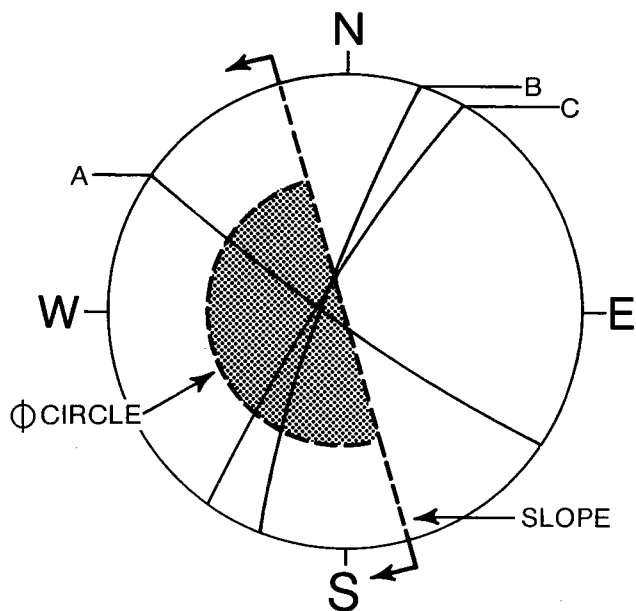


Figure 10: Stereogram of the principal joint sets at East Steubenville cut.

Table 6: Potential Modes of Failure at East Steubenville Cut.

Failure Mode	Joint Set(s)	Orientation *	F.S.
Wedge (W1)	B,C	N37°W/84°W	0.1
Wedge (W2)	A,C	S76°W/83°W	0.1
Wedge (W3)	A,B	S60°W/83°W	0.2

* orientations are strike and dip for plane failures, and trend and plunge of the line of intersection for wedge failures.

No valley stress relief joints within the requisite 20 degrees of the river orientation (generally north-south) were determined from the stereonet contour diagram. However, as seen in Figure 11, some large plane failures were observed at this site. These plane



Figure 11: Plane failure at East Steubenville cut.

failures occur where the road trends nearly parallel to joint set C (N24° E/86° NW). The dominant wedge failures W2 and W3, are formed by combinations of joint set A with joint sets B and C. These are large wedges as illustrated in Figure 12. The shale, due to its siliceous nature, has an extremely high (99%) shale durability rating, whereas the claystone completely disintegrates in water giving a zero rating. However, undercutting

at this site is minimal for probably two reasons. First, south of the bridge, shale talus covers the claystone thereby preventing erosion. Second, where the claystone is well exposed north of the bridge, failure of the shale proceeds at a rate equal to the undercutting because of the closely spaced jointing in the shale.



Figure 12: Wedge failure at East Steubenville cut.

A bench exists in the shale about 40 ft above the road level. This bench varies in width from 30 ft in the north to 50 ft in the south. Shale fills the bench to varying degrees. This bench seems to catch most rock fall blocks from the sandstone layer. A higher bench is present in the shale north of the bridge but it is mostly filled. The Highway Department has placed New Jersey Barriers along the berm to prevent the shale from reaching the road. The shale, although highly susceptible to wedge failures, probably poses less hazard to motorists because it's fissile nature causes the wedges to fall apart upon sliding. Rock fall blocks from the sandstone unit are infrequently seen in the ditch.

DISCUSSION

Based on the analysis of data obtained from the detailed line surveys, it is obvious that the valley stress relief joints influence

slope stability at the Mingo Junction and Brilliant sites. At the Mingo Junction cut, the valley stress relief joints result in wedge and plane failures, depending on the trend of the slope. At the Brilliant cut, these joints give rise to plane failures. The valley stress relief joints are not encountered at the East Steubenville site, according to analysis of the detailed line survey. The work by Ferguson and Hamel (1981) indicates that the shales destress by stretching instead of fracturing. This is a possible explanation for the absence of valley stress relief joints at this site. Also, the river takes a sharp meander here which may have caused a change in the orientation of the valley stress relief joints, thereby making them indistinguishable from the tectonic joints.

The other joint characteristics at these sites, namely spacing, continuity, nature of infilling material, nature of surface irregularities, and amount of water present were recorded and evaluated. In general, the joint spacing is less in the non-competent rocks (shales, coal) and greater in the competent units (sandstones, limestones). Unlike regional joints, the valley stress relief joints are not continuous and thus do not cut across units of differing strength. Many of the joints have no infilling material. If infilling material does exist, it is usually soil or detritus, but some joints are filled with calcite. In the majority of cases, the infilling material is too thin to alter the basic friction angle. Most surface irregularities are small and do not seem to increase the basic friction angle. Water appears to be a problem at these sites where the cuts have been excavated through pre-existing drainage channels. Upslope from these cuts, water enters these channels and drains down the slope face. The water which percolates into the ground near the channels, exits out the slope face. This water contributes to slope instability by freeze-thaw action.

Due to the variable geology and structure at these sites, protection measures in the form of shaped ditches, catch fences, removal of loose blocks, and periodic cleaning of the benches may be the best remedial measures. Stabilization methods, such as rock bolting may be necessary in the massive sandstone at the Brilliant site. This is especially true in the southern portion where the bench is narrow. Shotcreting the claystone and clay shale units would prevent undercutting at those sites where it poses a problem.

Finally, an important objective of this study is to categorize State Routes 2 and 7 into stable, potentially unstable, and unstable sections. The work to accomplish this objective will be completed in the near future.

REFERENCES

- Ferguson, H.F., Valley stress release in the Allegheny Plateau, Bull. Assn. Engr. Geol., 4(1), 63-71 (1967).
- Ferguson, H.F., Geologic observations and geotechnical effects of valley stress relief in the Allegheny Plateau, Proc.: Natl. Res. Engr. Meeting, Jan. 21-25, Los Angeles, Ca., ASCE, 31 p. (1974).
- Hamel, J.V., The slide at Brilliant cut, Proc.: Thirteenth Symp. Rock Mech., Univ. of Ill., 487-510 (1971).
- Hamel, J.V., Geology and slope stability in western Pennsylvania, Bull. Assn. Engr. Geol., 17(1), 1-26 (1980).
- Ferguson, H.F., and J.V. Hamel, Valley stress relief in flat-lying sedimentary rocks, Proc.: Intl. Symp. Weak Rock, Sept. 21-24, Tokyo, Japan, 1235-1240 (1981).
- Piteau, D.R., and D.C. Martin, Detail line engineering geology mapping method, Rock Slope Engineering, Part D, Fed. Hwy. Adm., Ref. Manual FHWA-TS-97-208, 29 p. (1977).
- Stimpson, B., A suggested technique for determining the basic friction angle of rock using core, Intl. J. Rock Mech. Min. Sci. & Geomech. Abstr., 18, 63-65 (1981).
- Franklin, J.A., and R. Chandra, The slake durability test, Intl. J. Rock Mech. Min. Sci. & Geomech. Abstr., 9, 325-341 (1972).
- Hoek, E., and J. Bray, Rock slope engineering, 358 p., Inst. of Min. & Metal., London, (1981).
- Markland, J.T., A useful technique for estimating the stability of rock slopes when the rigid wedge sliding type of failure is expected, Imperial College Rock Mechanics Research Report No. 19, 10 p. (1972).
- Watts, C.F., and T.R. West, Electronic notebook analysis of rock slope stability at Cedar Bluff, Virginia, Bull. Assn. Engr. Geol., 22(1), 67-85 (1985).
- Watts, C.F., T.R. and West, Discontinuity significance index and electronic data collection for rock slope stability studies, Bull. Assn. Engr. Geol., 23(3), 265-278 (1986).

Landslide Stabilization in Hilly Urban Terrain

THOMAS G. LEECH

Gannett Fleming Transportation Engineers, Inc.

Pittsburgh, Pennsylvania

JOHN G. DIVINEY

Gannett Fleming Geotechnical Engineers, Inc.

Harrisburg, Pennsylvania

CHARLES T. JANIK

Pennsylvania Department of Transportation

Harrisburg, Pennsylvania

HGS-87-4

AFTER A PERIOD OF SUSTAINED PRECIPITATION ON June 5, 1986, a landslide developed in hilly terrain along the bounday of a major construction work zone of a Pennsylvania (I-279) Interstate highway (locally known as the East Street Expressway). The landslide, known as the Howard Street Landslide, caused severe structural damages to seven nearby residential structures and placed key infrastructure utilities including underground water, gas, sewer, and telephone facilities in jeopardy of failure. (Figure 1) This paper outlines the conditions which precipitated the landslide, describes the

mechanics of the slide, and outlines the techniques employed to (1) temporarily arrest the landslide motion and (2) provide for long term stability. Extensive laboratory and field investigations were undertaken to determine in-situ conditions and to establish design parameters. A computer simulation of potential failure modes was developed to provide guidelines for final design. Long term stabilization included an extensive anchor wall system with anchorage derived in claystone, and specified work sequenced to insure viability of the endangered residences and infrastructure utilities.

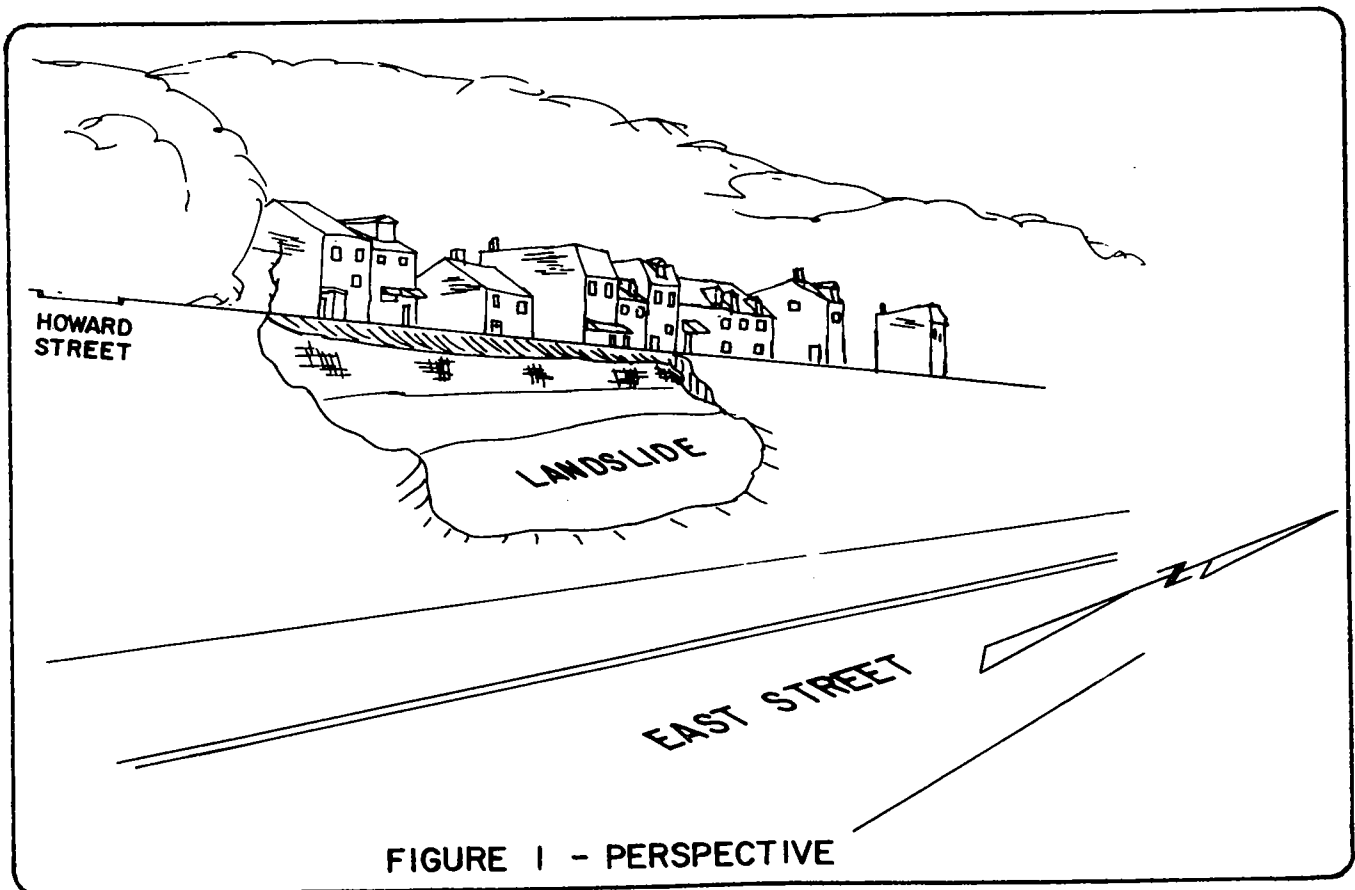


FIGURE 1 - PERSPECTIVE

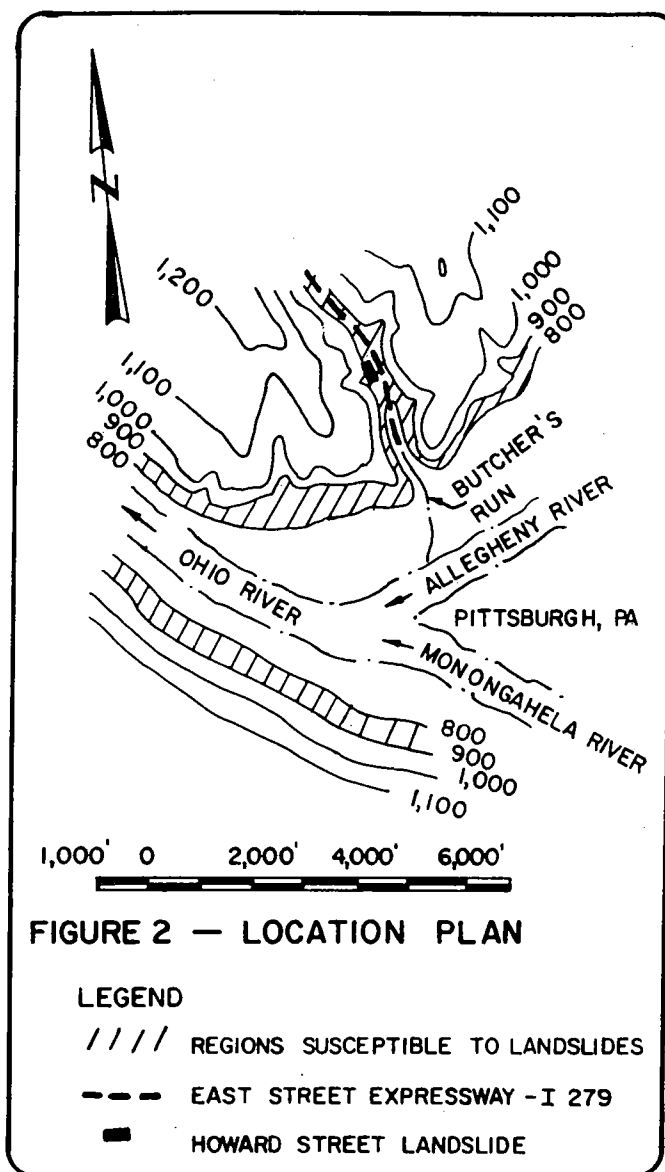
GEOGRAPHICAL/HISTORICAL SETTING

Howard Street is a mature residential street located along the western boundary of a major interstate construction work zone in the City of Pittsburgh (Figure 2). The interstate follows the axis of a major Allegheny River tributary, named Butcher's Run, a name dating over 100 years and derived from early German meat packing enterprises. The Pittsburgh region is located in the Appalachian Plateau, characterized by broad, flat hilltops intersected by a myriad of small streams draining into one major river basin, the Ohio River, located approximately 500 feet below the plateau. The forces of erosion have produced rolling hill tops, meandering streams, ravines, and steep sided valleys.

From the Allegheny River, northward, the interstate construction follows the narrow East Street Valley - the one time eastern border of the City of Allegheny, [sic] Pittsburgh. Early settlement, c.1800's, prospered by farming, denuding the steep hillsides of vegetation. This period was followed by waves of immigration as the regions rich coal fields were exploited for industrialization. By the early 1900's farmlands were subdivided and the region was dotted with ethnic communities, clinging to the sides of steeply sloping valleys.

SITE GEOLOGY

Bedrock of the region comprises cyclic sedimentary rocks of the Conemaugh Formation (mid Pennsylvanian age). Typically, the units are composed of alternating layers of shale, sandstone, limestone, coal and claystone. There is considerable lateral and vertical variation within each layer. Within the upper regions of the East Street Valley, conspicuous sandstone layers, with good stability, occur. Within the lower regions of the East Street Valley, individual beds, known as the Birmingham Shale and Pittsburgh Red Beds, occur. (Figure 3) These beds are principally comprised of claystones, i.e. indurated clays, characterized by bright red coloring and rapid decomposition on outcrop. Soil units overlying the bedrock are typically complex structures falling within two distinct boundaries. The soil boundary immediately above claystone bedrock, comprises residual soils derived entirely from the weathering of underlying claystone. Above the residual soil layer is colluvium comprised of fragments of shale, sandstone, clay and coal. This colluvial layer is derived from the weathering of bedrock at higher elevations and from the transporting of this soil mass, downslope, through the force of gravity. Colluvium can extend



from 5' to 30'. in depth. Typical side hill slopes vary from 2H:1V to 1H:1V, increasing with elevation. In the lower regions of the valley, the red bed areas are particularly susceptible to landslides. Landslides within this region, are frequently observed at the contact zone between residual soils and bedrock, within residual soils, within colluvial soils, and infrequently within bedrock along slickensided planes. In the early 1900's, closely spaced homes sprung up along the steep slopes of the valley. Homes were carved into the hillsides, and within hillside rivulets were placed fills of cinder and brick. The residences along Howard Street are typically stepped into the hillside with partial support on virgin soil and on loose fill.

LANDSLIDE DESCRIPTION

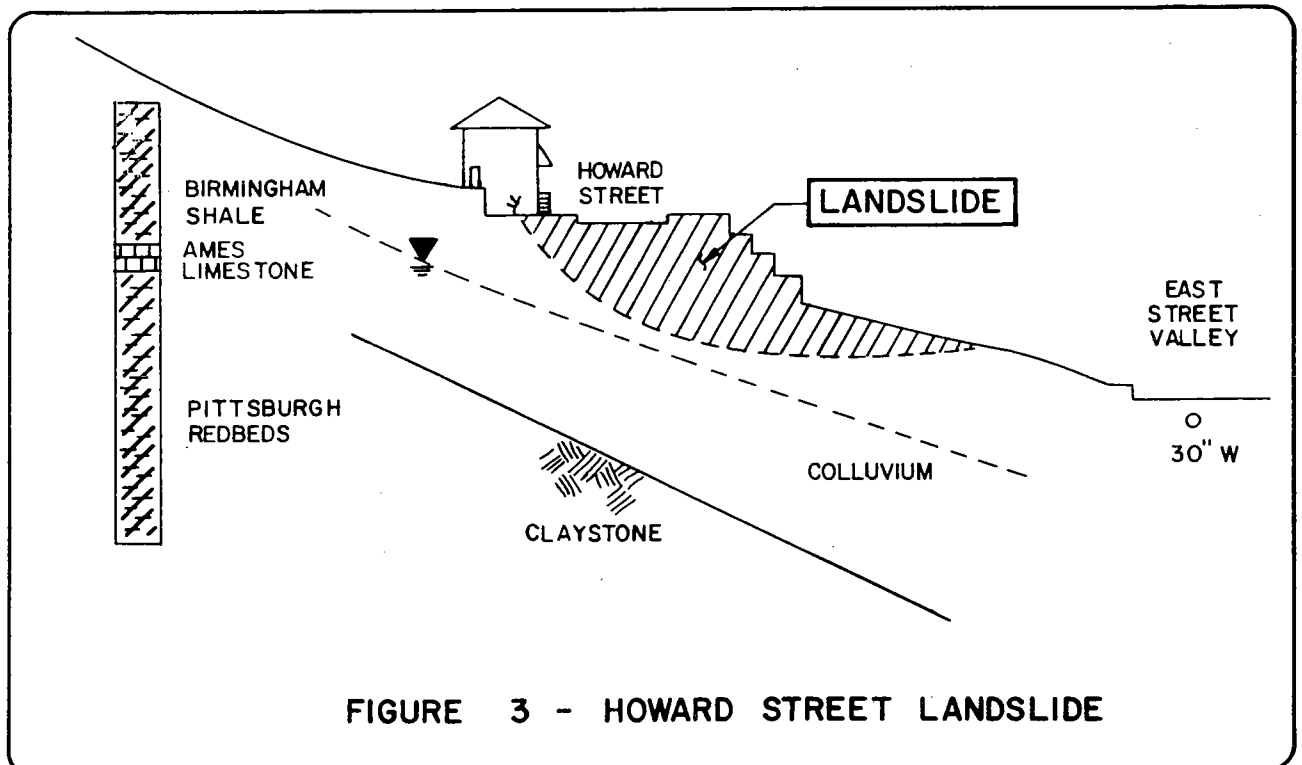
From May 30, 1986 to June 5, 1986, an unusually heavy period of precipitation occurred in the Pittsburgh region with record flooding recorded in several small drainage basins immediately north of the Allegheny River. On the morning of June 6, 1986, major cracking developed in the foundations of several houses along the western side of Howard Street. This cracking was accompanied by an open tension crack in adjacent empty lots along the scarp line of the slide. The cracking essentially split each affected house in two with the portion of the house fronting the street displaying noticeable leaning in the direction of the slide. Howard Street forms the western border of the interstate construction project. All residences east of Howard Street were acquired for project right-of-way and had been removed several years prior to the start of construction. Two months prior to the slide, construction activities in the vicinity of Howard Street had begun, with only clearing/grubbing operations and minor construction bench work occurring prior to the initiation of the slide. There was no documented record of landslide activity in this area. Based upon surface indications, and a program of inclinometer monitoring, the failure surface of the slide was defined. A test drilling program was conducted to identify materials within and below the failure surface. The major characteristics of

the slide are summarized as follows:

- The slide occurred within a geometrically square region of approximately one acre.
- Principal slide movements occurred along a curved slip surface with a constant radius of 200'; maximum depth of the slide was 25'.
- Slope movements of maximum velocity of 0.2"/week were observed. For approximately two months, these movements continued until temporary stabilization measures were employed.
- At the scarp line the slip surface proceeded through fill materials to a depth of approximately 10'; the remainder and majority of the slip surface lay within colluvium.
- The average long term groundwater surface lay below the slip surface.
- Utilizing the ordinary method of slices, an average residual friction angle of 12.5° (with $f.s. < 1.0$) was calculated along the slip surface by back-analysis.

On the basis of sensitivity studies and an analysis of construction events, three major factors were identified as the major contributing elements to the initiation of the landslide.

- The zone of soil within the slide mass had existed at or near a balance point for many years; the age of the residences is 90 years. Although landslide activity was not previously documented in this area, a pre-existing failure plane likely existed.

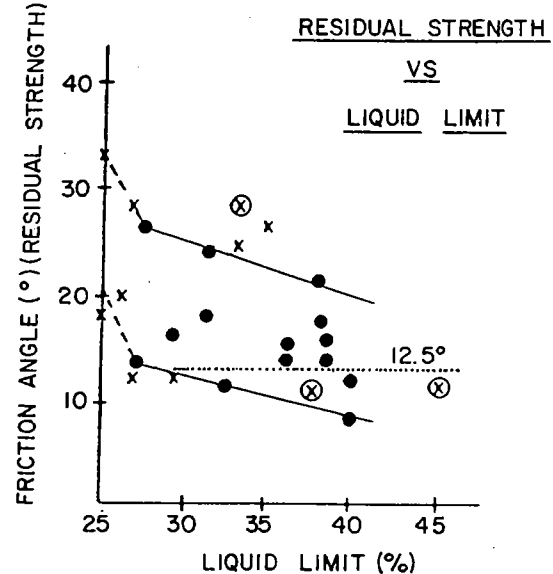


- ° Preliminary construction activity at the toe of the landslide reduced the ability of the slide mass to remain stable. (Calculations utilizing the observed average residual strength indicated a f.s.=1.07 prior to construction activity.)
- ° The presence of groundwater, associated with the heavy rainfall, is believed to be most significant in effecting a temporary increase in seepage forces and a permanent reduction of shear strength of the soil along the slip surface. Leaks in old (> 50 years) sewers and waterlines were also considered to be aggravating factors.

FIELD AND LABORATORY INVESTIGATIONS

A detailed program of field and laboratory investigations were performed to determine in-situ soil and bedrock properties. Direct shear testing to determine peak and residual strength parameters was based on the U.S. Army Corps. of Engineer's procedures. Undisturbed samples were extracted from the landslide region and in adjacent areas. A comparison of the residual soil strength parameters obtained in this investigation are compared to a baseline study conducted by Hamel and Flint for landslide prone regions in the Pittsburgh area. (Figure 4) Residual parameters within the slide mass correlate well with the baseline study and with the average strength parameter calculated by back-analysis along the active slip surface.

FIGURE 4 - RESIDUAL STRENGTH COMPARISONS



LEGEND

- - FROM HAMEL AND FLINT STUDY, 1969.
- x - FROM HOWARD STREET LAB TESTING.
- ⊗ - FROM HOWARD STREET LAB TESTING WITHIN SLIDE ZONE.
- - AVERAGE RESIDUAL STRENGTH PARAMETER CALCULATED ALONG SLIP SURFACE.

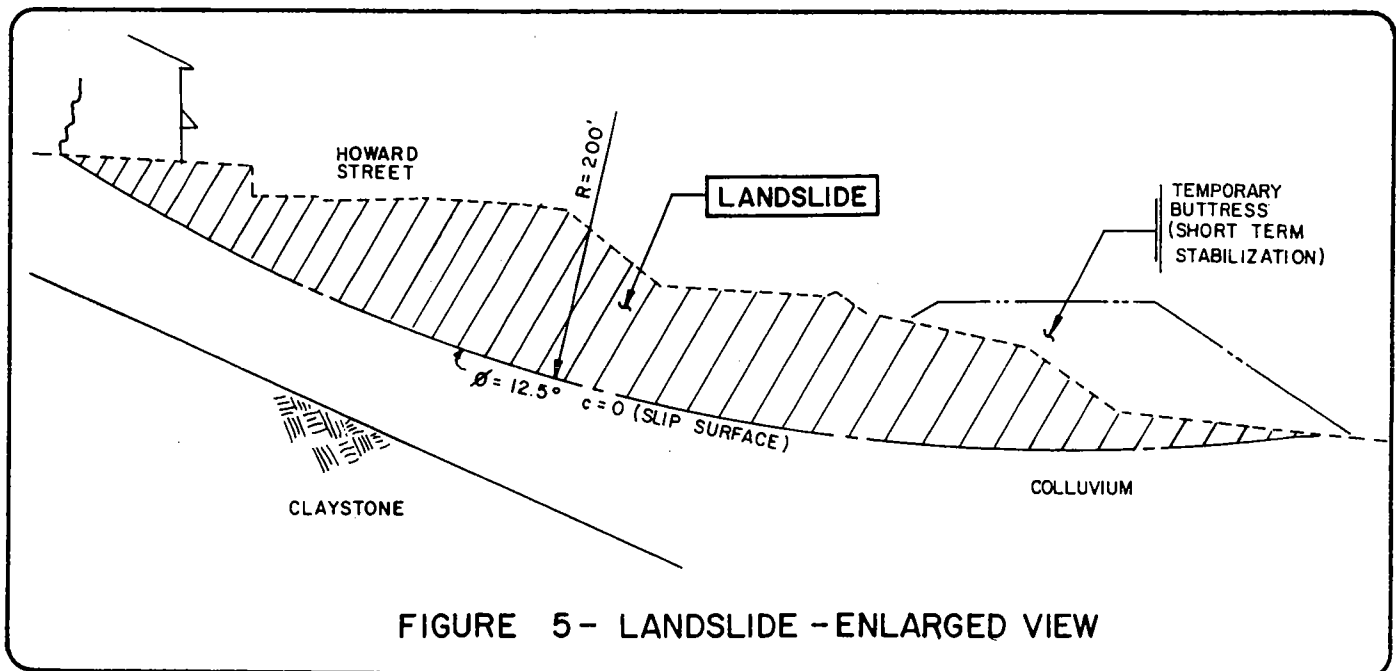


FIGURE 5 - LANDSLIDE - ENLARGED VIEW

SHORT TERM STABILIZATION PROCEDURES

An immediate and effective stabilization procedure was required. An ineffective procedure could have easily increased the movement of the slide with further damage to residences and utilities above and below the slide. One major utility near the toe of the slide was a major watermain serving two reservoirs representing approximately 25% of the water needs for the City of Pittsburgh. A precise quantification of the limits of the slide was necessary before any corrective action could take place. It was only after the precise circular character of the slide was established that placement of a temporary stabilizing buttress over the toe of the slide was enacted. A calculated safety factor of 1.25 was established for this condition. The placement of this buttress at the critical location was fully effective in arresting the motion of the landslide. (Figure 5)

LONG TERM STABILIZATION PROCEDURES

Having arrested the motion of the landslide, a modification to previously established construction procedures was mandatory to provide:

- ° the passage of the expressway along the valley floor in the immediate vicinity of the temporary buttress;
- ° the relocation of key infrastructure

utilities in the vicinity of Howard Street; and

- ° the maintaining of balance achieved by arresting the motion of the landslide.

After a study of several design alternatives the following design and construction procedures were established. (Figure 6)

- ° Install a soldier pile tieback wall immediately east of Howard Street, i.e. along the established western project right-of-way.
- ° Design the wall of sufficient rigidity to fully resist the active motion of the landslide. In addition the wall should have sufficient rigidity to support the entire soil mass above the rock contact zone applying the previously calculated average residual strength parameter to the entire soil mass. Design parameters (based upon laboratory testing) included:
 - . $\phi = 12.5^\circ$, $c=0$ for soil and claystone with core recoveries $< 70\%$;
 - . $\phi = 20^\circ$, $c=800$ psf for claystone with core recoveries $> 70\%$.
- ° Derive the entire support of the wall thru inclined rock anchors, and thru deep vertical sockets embedded in competent unweathered claystone (ie. claystone with core recoveries $> 70\%$).
- ° Fit the top of the wall with a special corridor to support the relocation of major infrastructure utilities.

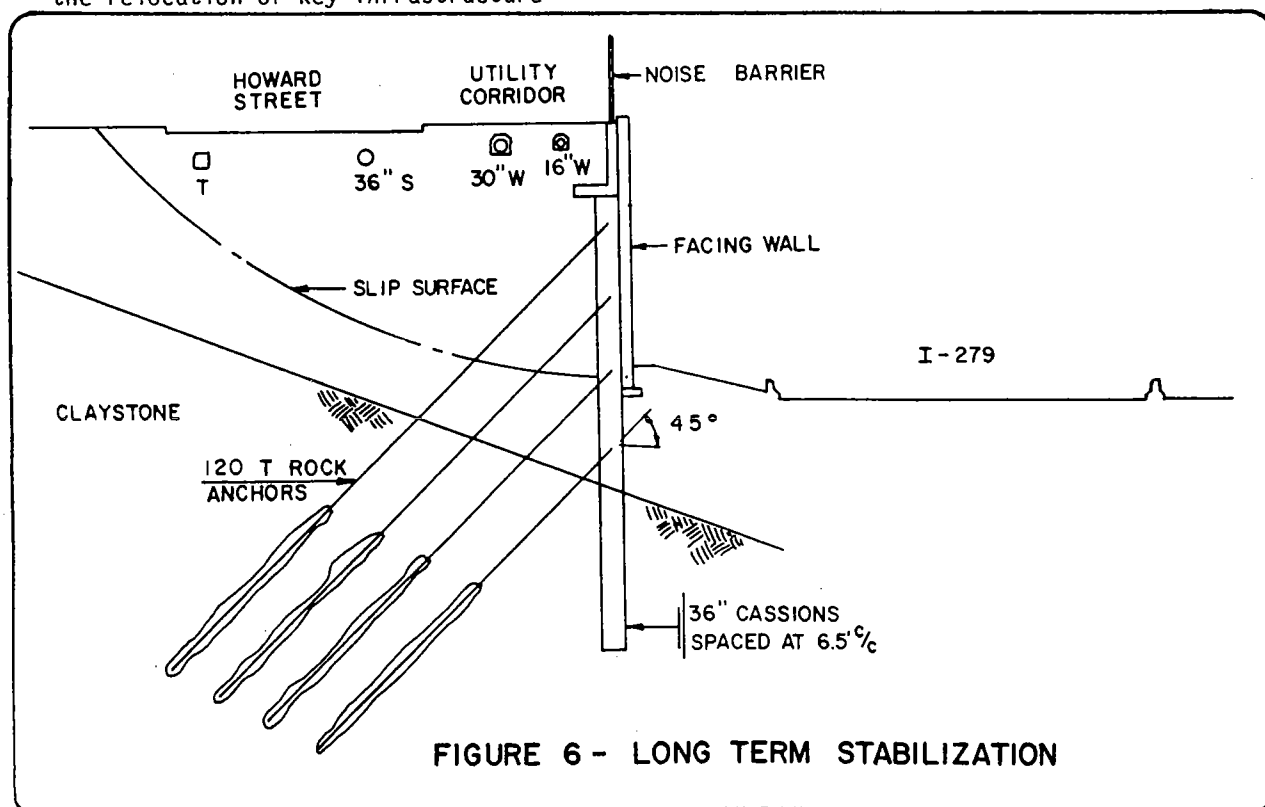


FIGURE 6 - LONG TERM STABILIZATION

- Install the pile components of the wall in checkerboard sequence - so as to not upset the balance effected by the temporary buttress.
- Upon completion of the wall, remove the temporary buttress and proceed with mainline roadway construction.

For final wall design, rigorous analyses were undertaken to establish potential failure modes and assess anchorage requirements. The analyses included the use of random search techniques employing Bishop's method for circular failure surfaces and Janbu's method for sliding block surfaces. The analyses were performed using computer program STABL4, adapted for use with tieback anchor walls. Anchorage was designed to assure that across each potential failure surface the $f.s. > 1.0$ with anchor loads taken at the working load and that the $f.s. > 1.5$ with anchorages stressed to 80% of the ultimate strength of the anchor (1.5 x working load). Potential failure surfaces investigated included surfaces along the rock/soil interface, deep seated circular surfaces in soil, and plane surfaces along the active wedge. Passive pressure resistance attributable to soil was not permitted in stability calculations.

Final design produced a wall system with maximum vertical exposure of 45' utilizing four rows of 120 ton rock anchors in claystone supported by piles spaced at 6.5' on centers. Either concrete encased steel pile sections or 36" reinforced concrete caissons were permitted as the pile member, with the latter section chosen for construction. The permanent anchors were installed through each pile and were installed at an angle of 45° to ensure that the bond zone lay within the right-of-way. Other principal design features included shotcrete support of soil between piles, continuous preformed drainage material between caissons, and an architecturally treated concrete facing wall cast over the soldier pile tieback installation.

CONCLUSIONS

The arresting of movement of an active landslide can be accomplished provided that a precise quantification of the geometry of the slide can be established. Surface indications, alone, are insufficient to characterize a slide when residential structures and infrastructure utilities are involved. A precise determination of slip surface geometry and rate of slide movement (via

inclinometer observation) are necessary to provide corrective actions.

When engaging in large scale construction projects in landslide prone regions, the potential for slides at any stage of construction exists regardless of knowledge of past history. This factor is greatly compounded in established hillside communities where the original groundscape is disguised by fills of variable composition and in locations where the soil overburden is comprised of heterogeneous colluvial materials. When a slide is encountered, a competent, immediate, and systematic evaluation is a necessary first step prior to outlining a program of corrective action.

ACKNOWLEDGMENTS

The material in this paper is drawn from a study conducted under the direction of the Pennsylvania Department of Transportation; the findings of the study were presented in the report entitled "L.R.1021-Section 3C, Howard Street Landslide Studies."

REFERENCES

1. Baker, R. F. (1954). "Analysis of Corrective Actions for Highway Landslides", Transactions, ASCE, Volume 119, Paper No. 2691, pp. 665-689.
2. Cleaves, A. B., "Landslide Investigations, A Field Handbook For Use in Highway Location and Design", U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., 1961.
3. Hamel, J.V. and Flint, N.K. (1969). "Analysis and Design of Highway Cuts in Rock: A Slope Stability Study on Interstate Routes 279 and 79 near Pittsburgh, Pennsylvania," Report by University of Pittsburgh to Pennsylvania Department of Highways and U.S. Department of Transportation, Bureau of Public Roads, 130 pp.
4. McGlade, W. G. et al., "Engineering Characteristics of the Rocks of Pennsylvania", Bulletin EG1, Pennsylvania Geological Survey, Harrisburg, PA. 1972.
5. Root, A. W. (1955). "Correction of Landslides and Slipouts," Transactions, ASCE, Volume 120, Paper No. 2738, pp. 281-289.
6. Wagner, W. R. et al., "Geology of the Pittsburgh Area", General Geology Report G-59, Pennsylvania Geological Survey, Harrisburg, Pa., 1970.

Ft. Pitt Tunnel North Portal Cut Slopes Revisited

ALFRED ACKENHEIL
Ackenheil Consulting Group
Pittsburgh, Pennsylvania

HGS-87-5

BACKGROUND

ABOUT FIFTY YEARS AGO, the famous city planner, Robert Moses, of New York was invited to come to Pittsburgh and prepare a plan for an improved highway transportation system through the downtown area of Pittsburgh.

SOUTH HILLS TRAFFIC - There was no easy way for traffic using the William Penn and Lincoln highways to the east and west to get through the City of Pittsburgh. By the 1930's, the Liberty Tubes were already heavily congested because of the population growth in the South Hills area. The "T" intersection at the south end of the old Point Bridge was a bad bottleneck for traffic to the south and west.

FEDERAL FUNDS - With the passage of the Interstate Highway Bill in the 1950's, it became possible for the Moses plan and dream to come true. Federal funds provided money on a 90/10 basis for the construction of the Fort Pitt Bridge and Tunnel.

FORT PITT TUNNEL AND BRIDGE - The Parkway along the Monongahela River became part of the scheme, and its interchanges with the Fort Pitt Bridge eliminated the bottleneck at the "T" intersection at the south end of the Point Bridge. The Fort Pitt Tunnel provided a much needed second link to the South Hills from the Golden Triangle area. In addition, it became the main route to the new Greater Pittsburgh International Airport.

TUNNEL DESIGN FEATURES - The final plans for the tunnel construction called for two tunnels about 3,500 ft. in length with two 12-ft. traffic lanes in each tunnel. The tunnels were 60 ft. apart on centers. Each tunnel was roughly 29 ft. wide and 35 ft. high. The top of the tunnel was a circular arc with a ventilation system in this area above a false ceiling. Ventilation buildings were located at both the north and south end of the tunnel. The tunnel and drainage sloped to the south at about 1:100. It was estimated that the future traffic volume would be about 100,000 vehicles per day. The design of the tunnel was awarded to Michael Baker, Jr., Inc. Consulting Engineers. The tunnel contractor

was Merritt, Chapman, and Scott Corporation. The construction contract for the tunnel was for about \$12 million.

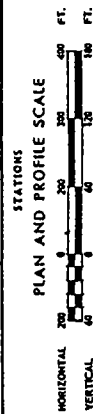
GEOLOGY

STRATIGRAPHY - The general geologic section through the tunnel and at the north and south portal areas is shown by Dwg. 1. The Pittsburgh Coal Seam outcrops at about Elev. 1050 on the north portal slopes, and at about Elev. 1000 on the south portal slopes. The tunnel roof rock changes from a gray indurated clay or claystone at the north end to a carbonaceous silty shale at the south end. The tunnel floor begins with a carbonaceous clay shale at the north end and passes through a gray silt shale, and then becomes a gray claystone at the south end. These sedimentary rock strata are part of the Conemaugh and Monongahela Formations of the Pennsylvanian System of rocks.

The north portal area had a very steep slope. Building construction on Mt. Washington extended out to the edge at the top of the slope. The south portal area was a more gentle slope with fewer buildings. This enabled about 300 ft. of open cut construction in place of the more expensive tunnel construction.

In the north and south portal areas, the competent siltstones and sandstones were somewhat variable in thickness and extent, and sometimes graded into weaker shales in a relatively short distance. The softest rocks were the thick strata of indurated clays or claystones near mid-height of Mt. Washington. There were no natural benches or terraces associated with fluvial-glacial materials or alluvium in the portal areas. The soils were a combination of residual or colluvial clayey deposits with a relatively thin mantle over the top of bedrock.

JOINTS AND FAULTS - The rock strata dipped to the south 20 degrees east at a rate of 1:600. The tunnel was located at the north end of the Nineveh syncline with its axis trending north 45 degrees east. No major fault due to the regional folding of the rocks



GENERALIZED GEOLOGIC SECTION THROUGH TUNNEL DWG. 1

occurred at the tunnel location. Shear joints produced by the synclinal folding action were visible on the walls of each tunnel. These shear joints are essentially vertical with a strike north and south 45 degrees to the axis of the syncline. The shear joints were mostly hairline cracks.

HARDNESS, SHEARING STRENGTH, AND PERMEABILITY - The rocks were divided as follows: (a) Hard Rocks: sandstone, siltstone, limestone; (b) Medium Rocks: shale, coal; (c) Soft Rocks: claystone or indurated clays. This same classification relative to hardness would apply to shearing strength and resistance to sliding. In terms of permeability, the rocks were classified as follows: (a) High Permeability: sandstone, limestone, and the mined out coal seam; (b) Medium Permeability: siltstones and shales; (c) Low Permeability: claystones.

OTHER LANDSLIDES - There have been a variety of landslides along the face of Mt. Washington in the vicinity of the north portal dating back as early as 1868. The types of slides were generally rock slumps or rock falls. Some of these landslides were of major proportions carrying houses down the hillside and then moving across the four-lane Pennsylvania Railroad track and down onto Carson Street. Normally, these slides were in the weathered rock zone and carried the thin colluvial soil mantle along. At Sycamore Street, near the north end of the Pittsburgh Rapid Transit System tunnel, there have been repeated rock falls over the years. Recently this entire area has received remedial action with tiebacks and gunnite.

NORTH PORTAL CUT SLOPE: FIRST DESIGN

CUT DETAILS - The first cut slope design for the original contract is shown on Dwg. 2 and called for a bench 22 ft. wide at about Elev. 855 in the gray indurated clay. Then a 2:1 (vert:horiz) slope was to be cut to about Elev. 900. This cut slope would have a base of dark gray to carbonaceous silt shale and a cap rock of gray sandy siltstone and gray sandstone. At the top of this slope, a second bench 30 ft. wide would be constructed. From that point upward, the design section called for another 2:1 cut to be made through the red/gray indurated clay and topping out in the gray siltstone at about Elev. 950.

NORTH PORTAL EXCAVATION - Excavation of the north portal slopes was begun in the fall of 1957. At the same time, the tunnel excavation begun at the south end was nearing the north end. Inside the tunnel, open crevices were being encountered in the roof and sides of the northbound and southbound tunnels as the headings approached the north portal face.

Two landslides of significant size occurred during the excavation of the cut slopes for the north portal. On February 23, 1958, a rock slide of 15,000 cu.

yds. occurred in the dark gray to carbonaceous silt shale between Stations 3+50 and 6+50 above the tunnel. The slide material was removed and the cut slopes revised. On March 24, 1958, a second slide movement of about the same magnitude occurred between the same two stations with a head scarp beginning at about Elev. 900. The movement appeared to be along the same failure surface as the previous slide. The March landslide was a rock slump-type slide with a rotary motion. The slip plane extended from the upper bench at Elev. 900 down to the lower bench at Elev. 855 (See Dwg. 2).

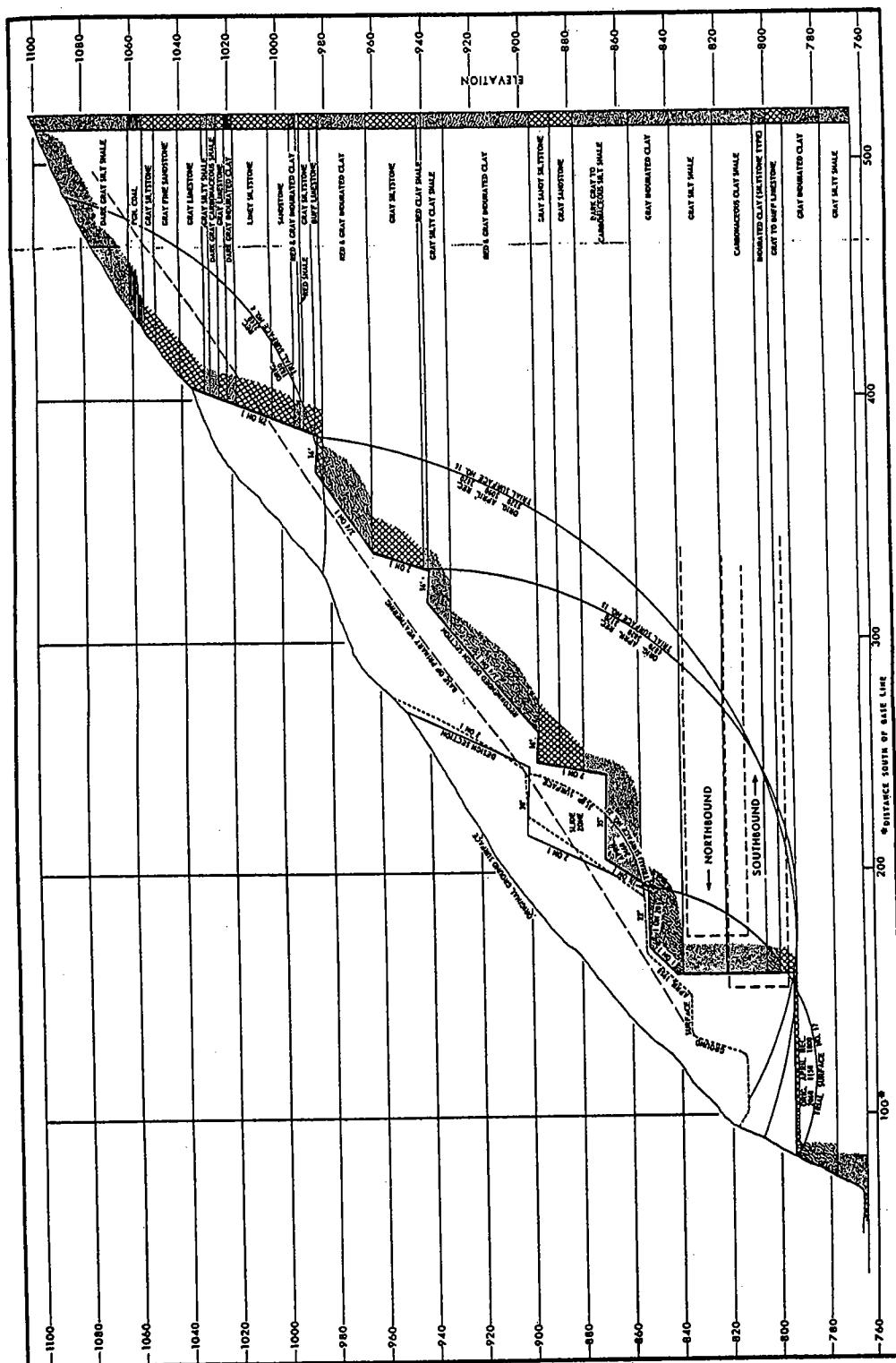
CREVICES INSIDE THE TUNNELS AT THE NORTH END - A study was made of the crevices inside the tunnel to determine what, if any, relationship there was with the rock slides on the surface. A location plan of the crevices is shown by Dwg. No. 3. These crevices were joints in the bedrock trending parallel to the valley walls of the Monongahela River. The crevices or rock breaks extended for a distance of about 350 ft. into the hillside from the portal face. Some of the crevices were filled with mud and weathered shale, and carried a steady dripping of groundwater.

JOINT DETAILS - Because of the skew of the tunnel, these joints were about 70 degrees from the tunnel axis. The dip was variable with an average of 77 degrees from the horizontal. Major joint zones occurred about every 75 ft. The crevices extended for an undetermined distance upward into the roof. They had openings of about 6 in. in width on the average, although some of them were up to 20 in. in width and some contained solid rock sheets. One large joint encountered during the north portal excavation was covered with calcite about 1-1/2 in. thick, indicating long-term water percolation.

JOINTS RELATED TO LANDSLIDES - Both the northbound and southbound tunnels contained similar crevices. The crevices were monitored for widening and further movement for a period of weeks after the March landslide. No further movement was observed. The crevices observed in the tunnel were identified as stress relief joints or sheeting joints. They were created as the valley rock wedge was eroded by the Monongahela River in the vicinity of the Point. This wedge extended from the top of the peneplain at Elev. 1200 down to the bedrock in the riverbed at about Elev. 650. The presence of the stress relief joints existing in the hillside was a contributing factor to the rock slides of February and March 1958. They, therefore, had to be considered in the redesign of the excavation cut slopes to reduce the risk of future landslides.

NORTH PORTAL CUT SLOPE: SECOND DESIGN

TUNNEL SECTION AND EMERGENCY RAMP SECTION - Two sections of the north portal



STABILITY AND GEOLOGIC SECTION IN VICINITY
OF TUNNEL ENTRANCE AT STATION 5+00
DWG 2

Top Row	17.5	1.8	1/2	1		
1/2 Row Row	17.8	1.8	1/2	1		
2	18.1	1.8	1/2	1		
1/2 Row Row	18.4	1.8	1/2	1		
Bottom Row	18.7	1.8	1/2	1		

PLAN OF CREVICES IN TUNNEL FROM STATION
1072+40 TO 1074+71

area were investigated for possible future landslides. These sections were at Station 5+00 along the centerline of the tunnel and Station 8+00 in the Emergency Ramp area. The geology at these two sections are shown on Dwg. Nos. 2 and 4. Three ground profile conditions were used for the stability analyses. They were: (a) Original - the original ground surface prior to any construction; (b) Design No. 1 - the ground surface at the time of the March 24 landslide; (c) Design No. 2 - the ground surface as recommended subsequent to the March 24 landslide.

BRILLIANT CUT SLIDE - A study was made of the rock slump that occurred in 1940 at the Brilliant Cut on the Pennsylvania Railroad near Highland Park in Pittsburgh. This slide took place through rock strata very similar to those in the north portal area in the Fort Pitt Tunnel. The slump at Brilliant Cut was one of the largest rock slumps in the Pittsburgh area. It also was associated with a sheeting joint failure related to the Allegheny River and the adjacent tributary valley. Since the rock slump movements were very close to circular arcs along the slip surface, this shape of slide was used as the analysis both for the Brilliant Cut failure and the trial sections at the Fort Pitt Tunnel. At Brilliant Cut, the average shearing stress at failure was about 3000 psf. The slopes at the north portal area were designed to keep the average shearing stresses below this failure value at Brilliant Cut and to reduce the stresses for the revised design slopes to be about 10-20% lower than those which existed in the original Fort Pitt portal slopes prior to any construction. The average shearing stress for different trial sections are shown on Dwg. No. 2 at Station 5+00 and Dwg. No. 4 at Station 8+00.

EMERGENCY RAMP AREA - In the Emergency Ramp area, the factor of safety was set lower since this particular portion of the north portal area would have only occasional traffic and was not located upslope from the Ventilation Building. A comparison of the two excavation sections shows the considerable difference in the amount of rock to be removed between the two areas.

CUT SLOPE VS. LITHOLOGY - Recognition was given to the various rock strata lithology, shearing strength, and durability by using different rock cut slopes for different materials. For the harder, more durable rock, slopes of 2.5:1 to 7:1 (vertical to horizontal) were used. For the soft claystone and indurated clay .75:1 was used. A bench width of 14 ft. was established for the benches with the exception of the first two levels of benches where wider benches were employed where the previous rock slides had occurred.

CONSTRUCTION OF CUT SLOPES: SECOND DESIGN - After the March 24, 1958 landslide, the second cut slope design used flatter slopes in the vicinity of the tunnel entrance and the

Emergency Ramp area. The excavation of the new cut slopes was performed by working from the top down. This was done by shooting and bulldozing the material downslope and developing the various cut slopes and benches.

HORIZONTAL DRAINS - On benches at Elev. 805, 855, and 940, a total of about fifteen 3 in. diameter horizontal drains on 50 ft. centers were drilled to provide an additional factor of safety. These horizontal drains outletted to the north on the portal face. The drains extended about 400 ft. into the hillside from the face of the cut. The purpose was to reduce the risk of hydrostatic pressure build-up in the stress relief joints.

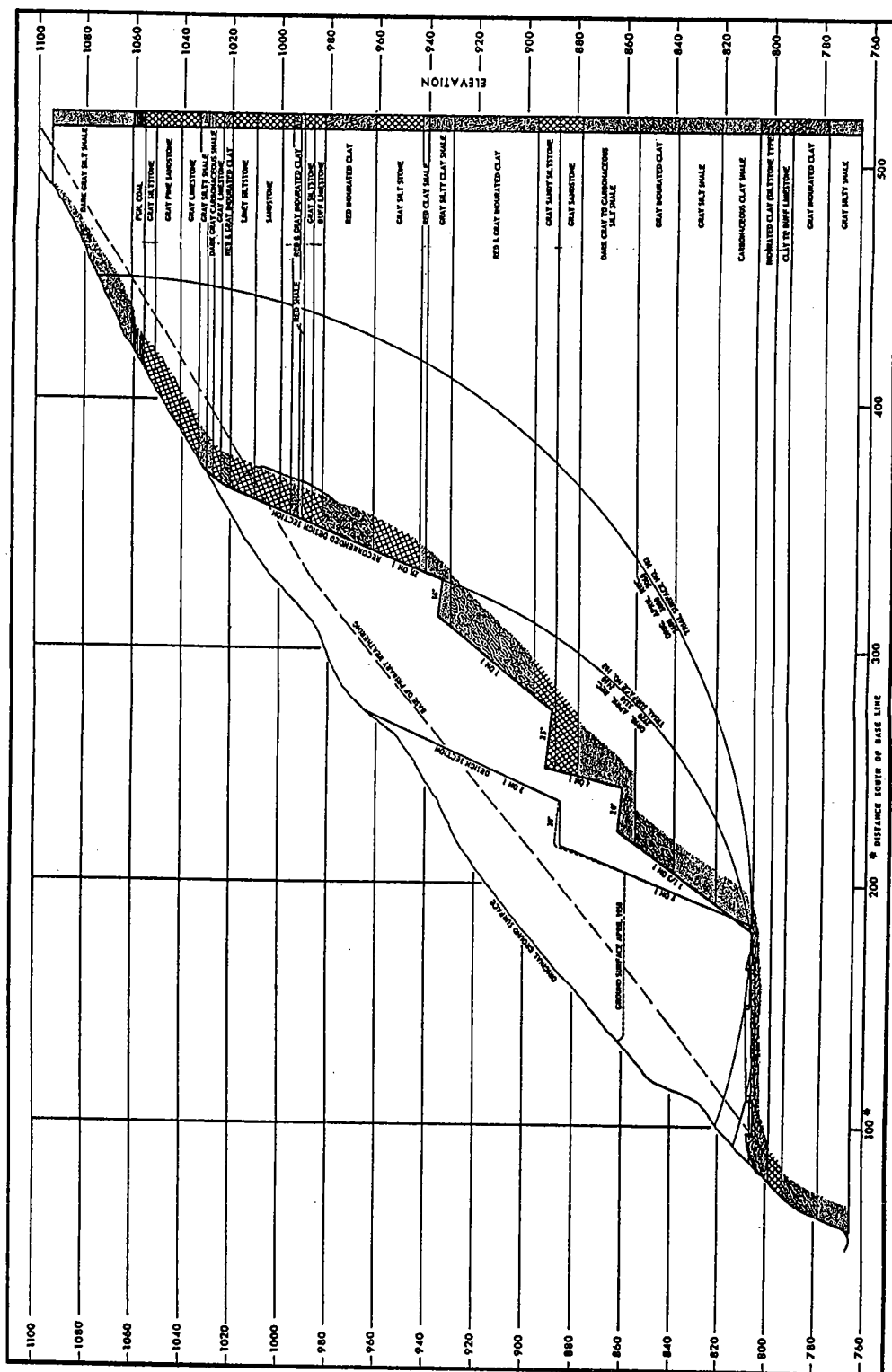
WATERPROOFING - The benches were covered with a bituminous coating to reduce the amount of surface water that might enter the crevices and joints on the tops of the benches. Drainage systems were recommended for the interior of the tunnel to carry off any water that might be encountered in the roof or sidewalls at various joints. The tunnel was built with a double concrete lining with a copper water stop between the two linings. Beneath each tunnel, there was a large floor drain to pick up any water beneath the tunnel and carry it out at the south end. The tunnel itself acted as a large horizontal drain.

NORTH PORTAL CUT SLOPE REVISITED IN 1987

Most geotechnical projects are one-time investigations of a site and the preparation of a technical report providing conclusions and recommendations. Unfortunately, on less than half of the projects is the geotechnical consultant involved in the construction phase and hardly ever after construction. He has little time and money available to go back on his own and look at a project during construction or after it is completed. To go back 20 to 30 years later is quite rare. This paper presents the data, conclusions, and recommendations for the rare case when a project was revisited.

SEVEN CONSIDERATIONS - When the north portal cut area was revisited, seven aspects of the original study were re-examined. They are the following: (1) Cut Slopes; (2) Horizontal Drains; (3) Stability Analysis; (4) Bench Maintenance; (5) Surface Drainage; (6) Internal Tunnel Drainage; and (7) Vegetation and Erosion.

CUT SLOPES - The condition of each cut slope was examined. The north portal was once again divided into two sections: (a) The main section over the tunnels behind the Ventilation Building at Stations 0+00 to 5+00, and (b) The Emergency Ramp area between Stations 5+00 to 10+00. For purposes of quick review, the enclosed tables have been prepared. The benches and cut slopes have been identified by number and letter; by approximate elevation and width; and by rock lithology. The observed condition when the cut slope was



STABILITY AND GEOLOGIC SECTION IN VICINITY
OF EMERGENCY RAMP AT STATION 8+00
DWG 4

TABLE 1: CONDITION OF BENCHES 0+00 TO 5+00

<u>BENCH</u>	<u>APPROX. ELEV.</u>	<u>WIDTH</u>	<u>MATERIAL</u>	<u>CONDITION</u>
1	855	22 ft.	Gray indurated clay	Clear, accessible grass covered, no bituminous covering visible.
2	875	35 ft.	Dark gray carbonaceous silt shale	Clear on west section. Blocked by small trees at about 5+00. Transition road from Bench 1 carrying seepage with boggy area at Bench 1 intersection.
3	895	16 ft.	Gray sandy siltstone	West end choked with siltstone talus and vegetation for 250 ft.
4	940	16 ft.	Gray silty clay shale	Entrance on west end of bench choked by siltstone talus.
5	985	16 ft.	Buff limestone	Entrance at west end is clear, but sandstone rock falls block bench after 150 ft.
6	1035	16 ft.	Gray limestone	Not in original design. Generally blocked by talus and vegetation.

TABLE 2: CONDITION OF CUT SLOPES 0+00 TO 5+00

<u>CUT SLOPE</u>	<u>FROM</u> <u>TO</u>	<u>SLOPE V:H</u>	<u>MATERIAL</u>	<u>CONDITION</u>
A	855 - 875	1.5:1	Carbonaceous silty shale	Thick vegetation mat, no rock exposure.
B	875 - 895	7:1	Gray sandstone, siltstone, silty shale	Covered by vegetation mat, few trees, broken rock, possible slide movement just west of cribwall near Station 5+00.
C	895 - 940	.75:1	Red-gray claystone	Vegetation mat, no rock exposure
D-E	940 - 985	2:1 .75:1	Gray siltstone and gray indurated clay	Gray siltstone rock falls from stress relief joints at western end. Gray claystone, moderate weathering, partially vegetation covered.
F	985 -1020	2.5:1	Sandstone and limey siltstone	Differential weathering causing rock overhangs. Broken fractured rock at west end, possible slide movement.
G	1020 -1015	2.5:1	Various thin shale and limestone layers	Differential erosion and undercutting.
H	1015 -1035	2.5:1	Claystone, carbonaceous shale, siltstone, limestone	Undercutting of more resistant layers.

TABLE 3: CONDITION OF EMERGENCY RAMP AREA BENCHES 5+00 TO 10+00

<u>BENCH</u>	<u>APPROX. ELEV.</u>	<u>WIDTH</u>	<u>MATERIAL</u>	<u>CONDITION</u>
E1	805	50 ft.	Carbonaceous clay shale	Numerous rock falls from about Elev. 1000. Some units 8 ft. X 6 ft. X 2 ft. in size. Concrete pavement broken. Guardrail broken.
E2	855	20 ft.	Dark gray carbonaceous silt shale	Eastern section covered by partial talus covering. Trees in planter at about Station 6+00. Bench blocked.
E3	895	25 ft.	Gray sandy siltstone	Bench blocked by talus and trees.
E4	940	16 ft.	Gray silty clay shale	Bench blocked by talus and trees.

TABLE 4: CONDITION OF EMERGENCY RAMP AREA CUT SLOPES 5+00 TO 10+00

<u>CUT SLOPE</u>	<u>FROM</u> <u>TO</u>	<u>SLOPE V:H</u>	<u>MATERIAL</u>	<u>CONDITION</u>
EA	805 - 855	1.33:1	Carbonaceous clay shale, gray silt shale, gray indurated clay, dark gray to carbonaceous silt shale	Rock surface generally exposed eastern section. Some vegetation west of Station 5+00.
EB	855 - 895	4:1	Dark gray to carbonaceous silt shale, gray sandstone, and gray sandy siltstone	Rock exposed. Not available for close examination.
EC	895 - 940	1:1	Red and gray indurated clay	Partially covered by siltstone and sandstone talus from above. No apparent significant erosion of claystone.
ED	940 - 1030	2.5:1	Gray siltstone, red indurated clay, gray limestone, gray siltstone, sandstone, and limey siltstone	In this section, the slope has been eroded in the lower half creating undercutting and rock falls of the harder units at the top of the section.

revisited in 1987 has been briefly stated.

HORIZONTAL DRAINS - Design II prepared after the February and March landslides called for about 15 horizontal drains (4 in. diameter) to be drilled at three different bench levels in the portal cut area. These benches were at Elev. 805, 855, and 940. The purpose of the drains was to intercept the sheeting joints to prevent hydrostatic pressure build-up.

When the site was revisited in April 1987, only two drain openings were visible. All the rest were covered by talus or vegetation. One of these two drains was dry, but the other one was carrying about 10 gallons per hour flow.

STABILITY ANALYSIS - No major rockslide has occurred in the primary section of the north portal cut slope since the 1958 Design II and construction. This primary portion of the north cut slope over the tunnel and behind the Ventilation Building is between Stations 5+00 to Station 10+00.

One rockslide involving about 200 cu. yds. plus a number of individual rock unit falls have occurred in the north portal cut slope above the Emergency Ramp area since 1958. This possibility was anticipated in the original design since a lower factor of safety was used.

BENCH MAINTENANCE - In 1958, the bench surfaces were coated with a bituminous covering to help shed water off the north portal cut area. Any vertical joints in the rock of the bench surface were plugged with the bituminous material. When the site was revisited in 1987, no bituminous covering was visible on any of the benches. Instead the benches were covered with vegetation or clay soils and talus.

Tables 1 through 4 describe in detail the condition of the various benches. A number of the benches had their entrances blocked in such a way as to prevent maintenance by a high-lift or dozer. This blockage was primarily rock talus and soil materials. There were also a number of trees of about 6 in. diameter. The benches were also blocked at other locations besides the entrance to the bench. At a number of locations, particularly in the Emergency Ramp area, the benches were completely full from the front to the back of the bench with steeply sloping talus material. The front end of the benches were generally in good condition with no significant loss.

SURFACE DRAINAGE - Surface drainage along the various benches had been blocked in a number of locations due to the accumulation of talus and erosion material. Since the benches are sloped to drain into the hillside, this creates the possibility of ponding during heavy precipitation. No examination was made of the drainage facilities above the north portal cut slopes. The central corrugated metal pipe drain appears to be in good condition and still functioning.

INTERNAL TUNNEL DRAINAGE - The broken rock

section between the northbound and southbound tunnels encountered in 1958 has created seepage problems in the ceiling and walls of the southbound tunnel during the past 29 years. The roadway pavement at the entrance to the southbound tunnel is frequently wet and a number of ceramic tiles have fallen off the walls and ceiling. These have been replaced from time to time.

A water-catchment system has been installed in the ventilation chamber above the ceiling of the southbound tunnel. This system picks up the water coming in through the cracks and joints in the tunnel lining and carries it out through a 4 in. diameter PVC pipe to an area back of the Ventilation Building. The main drains beneath the floor in the northbound and southbound tunnels drain to the south. Because the northbound tunnel is at a higher elevation than the southbound tunnel, it is possible that some of the water coming into the southbound tunnel could be from this floor drainage system.

VEGETATION AND EROSION - The north portal cut area in 1958 was primarily rock with some thin soil cover. In the past 29 years, a number of shrubs and trees have grown up on the slopes. Some were volunteers, some were placed in the timber crib planting areas. Sizes ranged up to about 6 in. in diameter.

Only a small amount of erosion of the rock slopes has occurred. The claystone slopes, which are most susceptible to erosion have been well protected by the vegetation mat which has grown on these slopes. At some locations, the soil fines from the weathered claystones has accumulated sufficiently to partially block a bench.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

MAJOR LANDSLIDES; TWO, THEN NONE, BUT - Two major rock slides occurred in the north portal cut slope area of the Fort Pitt Tunnel prior to April 1958. Some of the contributing factors were: (a) cut excavation at the toe of the slope; (b) sheeting joints; (c) hydrostatic pressure; (d) weak rock types; and (e) too steep a slope.

After the April 1958 redesign and subsequent construction, no major landslide has occurred in the past 29 years. Changes that probably reduced the risk of major landslides were: (a) removal of rock load and flattening of slope; (b) horizontal drains reducing hydrostatic pressures in sheeting joints; and (c) tunnel drainage.

But rock slopes involving claystones can lose stability under long-term stress. The model Brilliant Cut on the Pennsylvania Railroad strained slowly for years, then suddenly sheared. The Fort Pitt Tunnel north portal contains a similar geologic section.

DUAL FACTORS OF SAFETY: LEFT AND RIGHT - The left half of the north portal cut slope is designed with a lower factor of safety than

the right half. The left half above the Emergency Ramp area has a design factor of safety of about 1.15 vs. about 1.35 for the main cut area behind the Ventilation Building and over the tunnel. Some minor slides of up to 200 cu. yds. have occurred at the top of the cut slope above the Emergency Ramp.

ROCK SLOPES/BENCHES VS. ROCK TYPES - The cut slopes were originally selected for construction feasibility and durability with age. With the passage of 29 years, the slope/bench conditions probably have worked out better than expected. No benches have been lost, but most benches need to have accumulated talus removed to protect against downhill rock rolls. Bench drainage has been impaired.

HORIZONTAL DRAINS VS. COST BENEFIT - Drilling the horizontal drains was expensive. Some carried only small quantities of water. But small quantities can be important relative to hydrostatic pressure build-up in sheeting joints. After 29 years, only two drains are still visible, but the one carries enough water to be significant relative to rock slides. The other horizontal drains should be located, uncovered, and freed up for proper drainage outlet and evaluation.

MONITORING A \$10,000,000 RISK - If the Fort Pitt Tunnel carries 100,000 vehicles per

day, it would be very costly if temporarily closed due to a major rock slide. If each driver and vehicle had to re-route 15 minutes each way per day, the labor value and travel expense could easily reach \$10,000,000 over a three month period while the tunnel was shut-down for remedial measures. It would be penny wise and pound foolish to not pay for quarterly monitoring now of the north portal for landslide danger. The Brilliant Cut slide displayed evidence for months before it failed.

GEOTECHNICAL CARE FROM CRADLE TO GRAVE - Because of the high risks involved, geotechnical work has the highest professional liability risk of all design professionals. In 1965, no insurance company, including Lloyds of London, would offer insurance, and in 1968 the Association of Soil and Foundation Engineers was founded to provide this insurance coverage.

One of the goals of the Association of Soil and Foundation Engineers is to seek to provide a continuous project involvement of geotechnical services from initial investigations through construction and into the post-construction monitoring phases in order to reduce risks. This cradle to grave care can be very valuable to an owner when a project is revisited by the geotechnical engineer.

A Preliminary Look at Simple Back Analysis of Rock Slope Stabilities Utilizing Microcomputers

DR. CHESTER F. WATTS

Radford University

Radford, Virginia

E. M. FRIZZELL

Bureau of Mines, Spokane Research Center

Spokane, Washington

HGS-87-6

ABSTRACT

AN EXAMINATION OF TECHNIQUES utilizing personal computers to determine the shear strength of rock mass discontinuities after failure is presented. The origins of the personal computer programs ROCKPACK and BACKPACK are traced, and the capacities of the two programs are briefly described. Several methods of selecting shear strength values of cohesion (c) and friction angle (ϕ) are discussed. Each method may be utilized in conjunction with the BACKPACK code. The most reliable and desirable method requires more than one failure in the same rock mass to be identified. An example using a major slope failure in a copper mine is discussed; this example includes the development of a range of relationships between cohesion and friction obtained by using BACKPACK. The value of a one-point back analysis in providing useful information on rock strength, especially when used in conjunction with sound engineering judgment, is discussed.

INTRODUCTION

Back analysis of rock slope stability is the determination of discontinuity after shear strength failure within a rock mass. Back analysis is calculated by solving the limiting equilibrium stability equation for the strength parameters, cohesion, and friction angle, rather than calculating for the factor of safety. The equation assumes that limiting equilibrium conditions existed at the time of failure, or that, in other words, failure occurred when the safety factor fell to 1.0. It requires that slope geometry and related conditions at the time of failure be known precisely. This concept is useful as a supplementary means of obtaining strength values for use in the design of excavations in rock masses having already failed at least once.

A series of microcomputer programs analyzing rock slopes were written by C. F. Watts while he was a graduate student at Purdue University (1).*

*Underlined numbers in parentheses refer to items in the list of references at the end of this report.

Initially tested on a 280-foot-high rock slope in southwest Virginia, the software was used to predict successfully a rock slide that partially closed a highway (2-3). Refined and named ROCKPACK, for ROCK Slope Stability Computerized Analysis PACKage, the program capabilities are briefly described below. Modification of these programs was done in cooperation with personnel of the Spokane Research Center of the U.S. Bureau of Mines and has resulted in a second software package called BACKPACK, for BACK Calculation and Artificial Support Analysis PACKage. BACKPACK has applications in mining and highway construction where simple back analyses of discontinuity strength parameters are needed and is introduced here along with the preliminary results of a sample study. General guidelines for utilizing simple back analysis results are also provided.

THE SHEAR STRENGTH CONCEPT

Shear strength refers to the maximum shear stress that can be resisted along a potential sliding surface. In the case of an actual slope failure, it is the shear stress along the sliding surface at the time of failure. Shear strength is generally estimated by laboratory testing of samples collected from the rock mass that are representative of the potential sliding surface. A normal stress is applied across a discontinuity in the sample, and the shear stress required to cause movement along the discontinuity is measured. A series of shear tests are conducted at increasing normal stresses in order to determine the relationship between shear strength and normal stress. This relationship is often approximated by a linear equation (fig. 1) which expresses shear strength (τ) as a function of normal stress (σ_n), cohesion along the failure plane (c), and friction angle (ϕ). Therefore,

$$\tau = c + \sigma_n \tan \phi. \quad \text{Eq. (1)}$$

Cohesion is the intercept of the curve with the shear stress axis and is, therefore, the shear strength when the normal stress is 0. The friction angle is indicated in figure 1 and is the component of shear strength that increases as normal stress increases. Although approximated by a straight line, the actual shear strength

relationship is not linear (4), and is probably more closely represented by the curve in figure 2.

DETERMINATION OF DISCONTINUITY SHEAR STRENGTHS

A problem universal to those who analyze the stability of rock excavations is the selection of the shear strength values of cohesion and friction angle. The methods utilized will often depend on the resources available. Engineers and geologists are often forced to make educated guesses and assumptions based on experience to arrive at strength parameters. In such cases, conservative uncertainty may result in slopes that are overdesigned. The educated guess method is probably used more often than many would care to admit.

Additional strength estimates may be obtained by placing representative rock samples in a shear testing device, usually in the laboratory, and determining the shear strength along a discontinuity at varying normal stresses. These laboratory results are quite valuable. However, relating the values obtained from testing small samples in a laboratory to the actual full-scale rock mass is not always easy. Sometimes a simple cohesion-equals-zero assumption is made when designing a slope, even after laboratory testing, thus providing an overly conservative analysis (fig. 3).

Finally, if a failure has actually occurred in a rock mass, or if failures can be induced, then useful discontinuity strength estimates may be obtained by back calculation. Back calculation of a single failure is basically a large-scale shear test at one normal stress. It requires that the pre-failure geometry of the slope, external forces, and water conditions be known if the normal and shear stresses on the failure surface at the time of failure are to be calculated as accurately as possible. These stresses provide some useful information about shear strengths along discontinuities in the rock mass.

REVIEW OF ROCKPACK SOFTWARE CAPABILITIES

The original ROCKPACK software provides detailed stability analyses of geologic structures that can lead to sliding failures in rock excavations. ROCKPACK presently consists of 12 programs that perform the following 6 functions:

1. Format on-site geologic data for transmission and analysis.
2. Display collected data in tabular form along with cumulative rectangular discontinuity orientation plots.
3. Create stereonet plots of discontinuity orientation data.
4. Perform graphical kinematic tests for potential plane and wedge failures.
5. Contour discontinuity orientation data on rectangular and stereonet plots to assist in cluster recognition.

6. Calculate safety factors for potential plane, wedge, and toppling failures.

The programs are based primarily upon limit equilibrium theory as presented in a text by Hoek and Bray entitled "Rock Slope Engineering" (5). ROCKPACK is applied by the Oregon State Highway Division to analyze the stability of road cuts in columnar-jointed basalts (6). ROCKPACK is also being used by Radford University, in cooperation with the U.S. Geological Survey, to study ancient giant landslides in Virginia and by five universities for research and instruction in rock slope stability. Numerous engineering consulting firms have the software to analyze the stability of roadcuts and building excavations.

BACKPACK SOFTWARE CAPABILITIES

E. R. Wirt, an engineering geology student at Radford University, began BACKPACK programming during the summer of 1986 in cooperation with personnel from the Bureau. BACKPACK software provides approximations of rock mass strength characteristics through back analyses of existing plane and wedge failures. It also determines optimum rock anchor tensions and installation angles and calculates idealized rock anchor installation patterns for stabilization of coherent rock masses. The programs utilize limit equilibrium theory and include the effects of external loading, water pressures, rock bolt or cable forces, and earthquake or blasting accelerations. The BACKPACK software presently consists of the programs listed below:

1. BACKONE - provides back calculations of planar slope failures to characterize the strength of the rock mass in terms of an apparent cohesion and friction angle in accordance with limit equilibrium theory, if the pre-failure slope geometry is known.
2. PLANE2 - analyzes potential plane failures by determining optimum angles for rock anchor installation in order to produce maximum support for given anchor tensions. The program utilizes a feature that loops through specified anchor angles and tensions and displays the results in tabular and graphical form.
3. BOLTPAT - utilizes results from 1 and 2 above to design rock bolting patterns where bolt spacings on the rock face and required drilling lengths are estimated for a specified rock mass.
4. BACKWEDG - provides back calculations of wedge-shaped slope failures to characterize the strength of the rock mass in terms of an apparent average cohesion and average friction angle in accordance with limit equilibrium theory, if the pre-failure geometry is known.
5. COMFWEDG - calculates a safety factor against translational sliding of a rock wedge formed by the intersection of discontinuities in the rock mass. The program utilizes limit equilibrium theory based upon equations presented in the text by Hoek and Bray (5) and includes the influence of water pressures, external loads, and rock anchors. It permits different cohesion

and friction parameters on the two failure surfaces.

6. BOLTWEDG - is identical to COMPWEDG except that it is modified to permit calculations of the optimum tension of the total number of anchors and the orientation required to produce a desired safety factor for a specified rock wedge.

BACK ANALYSES OF ROCK SLOPE FAILURES

Although laboratory testing of samples for shear strength is a valid engineering tool and provides valuable data, it cannot precisely represent the many complex interactions within an actual rock mass. For this reason, back analyses of the shear strength parameters of cohesion and friction angle from existing failures provide another useful approach when designing excavations in the same or similar rock masses. Back-calculated shear strengths are often lower than the shear strengths determined by laboratory testing (7).

An ideal back analysis would result in the determination of apparent cohesion and friction angle values for discontinuities in the rock mass, thereby defining an equation like Equation 1 for shear strength at varying normal stress. Unfortunately, that is not possible when only a single failure has occurred because only one point on a shear strength curve would be given. At least two failures in the rock mass are required to construct a back analysis strength line from which cohesion and friction angle could be estimated. Nevertheless, a "one-point" back analysis can provide general strength information when properly understood.

The greatest difficulty in back-calculating cohesion and friction angle values is that a single equation cannot be solved for two unknowns. Hence, one or the other must be approximated in order to solve for the remaining one, unless multiple failures have occurred in the rock mass. Four methods of dealing with this problem are described below. All of these methods may be utilized in conjunction with the BACKPACK software. Method 4 is probably the most reliable and hence the most desirable, although conditions are seldom optimal.

1. Use laboratory testing to approximate cohesion and then back-calculate the apparent rock mass friction angle from a single failure.
2. Use laboratory testing to approximate friction angle and then back-calculate the apparent rock mass cohesion from a single failure.
3. Loop through a range of reasonable friction angles, back-calculate the cohesions that would be appropriate for limiting equilibrium for the single failure, make a graph of the results, and rely on past experience and engineering judgment to select reasonable cohesion and friction angle values.
4. Back-calculate and plot the shear strength at each failure if more than one failure can be identified within the same rock mass, and if the normal stresses on the failure surfaces at

failure are different from one another. The resulting curve may be used to approximate BOTH the rock mass cohesion and the friction angle. Unfortunately, it is often difficult to clearly identify more than one failure within the same rock mass.

Note that the strength values obtained by back analysis are only useful for those discontinuities in the rock mass having physical characteristics comparable to those of the back-calculated failure surface.

EXAMPLE OF BACK ANALYSIS OF PLANAR FAILURE

The Nacimient Mine in northern New Mexico was chosen for a test of back analysis of a planar failure using the BACKONE program in the BACKPACK software. The principles are applicable to highway roadcuts. A Canadian Centre for Mineral and Energy Technology (CANMET) report entitled "A Case History of Support at Nacimient Mine" (8) provided data for a very general back analysis. The report also included the results of back analyses performed for the mine by engineers for comparison.

The following generalized data were entered into BACKONE. They are believed to represent the site conditions closely but may vary slightly.

SLOPE HEIGHT.....150 ft
INCLINATION OF SLOPE FACE.....40°
INCLINATION OF UPPER SLOPE FACE.....0
INCLINATION OF POTENTIAL FAILURE PLANE.....29°
HORIZONTAL ACCELERATION DUE TO BLASTING
OR EARTHQUAKE.....0
BOLT TENSION per foot along slope.....0
BOLT ANGLE.....0
COHESIVE STRENGTH OF FAILURE SURFACE.....UNKNOWN
FRICTION ANGLE OF FAILURE SURFACE.....UNKNOWN
SAFETY FACTOR ASSUMED.....1.0
DENSITY OF ROCK.....148 pcf
DENSITY OF WATER.....62.4 pcf
AMOUNT OF DISCONTINUITY SATURATED.....0
ADDITIONAL VERTICAL SURCHARGE.....0

The strength parameters of COHESION and FRICTION ANGLE are the unknowns to be characterized for the rock mass described above. This was calculated by assuming values for cohesion and solving for friction angle following the computer prompts of the BACKONE program. Equation 2 (fig. 4) is the back analysis equation used to calculate friction angle from cohesion. Thus,

$$\tan \phi = \frac{FS(W \sin \theta - T \sin \beta + V \cos \theta) - cA}{(W \cos \theta - U - V \sin \theta + T \cos \beta)} \quad \text{Eq. (2)}$$

where ϕ = friction angle of failure surface for given c value,

c = assumed cohesion along failure surface,

FS = assumed safety factor at failure (usually 1.0),

W = weight of sliding mass,

θ = dip angle of failure surface,

T = tension in rock bolts or cables,

β = inclination of bolts or cables with respect to perpendicular to failure surface.

V = water pressure in vertical tension crack,

U = water pressure on failure surface,

A = failure surface area.

This equation is derived from

$$FS = \frac{cA + (W\cos\theta - U - V\sin\theta + T\cos\beta) \tan\phi}{W\sin\theta - T\sin\beta + V\cos\theta} \quad \text{Eq. (3)}$$

In accordance with limit equilibrium theory, the maximum theoretically possible friction angle will occur when the cohesion equals 0. Conversely, the maximum possible cohesion will occur when the friction angle equals 0 (fig. 5). Although it is not likely that either condition would be exactly equal to 0, solving for them does provide a range within which the cohesion and friction angle for the rock mass must fall if the slope geometry and geologic conditions of the model are accurate. The intersection, or "pivot point," shown in the figure represents the shear and normal stresses on the failure plane at the time of failure.

Substituting various cohesion values in the computer program and assuming the limiting equilibrium condition safety factor of 1.0 at the time of failure, the following range of relationships between cohesion and friction angle was obtained from BACKONE:

Cohesion (psf)	Friction Angle (degrees)
0	29.00
200	25.87
400	22.56
600	19.09
800	15.50
1000	11.71
1200	7.80
1400	3.89
1597	0.00

This analysis indicates that cohesion for the rock mass as defined above could be expected to range between 0 and 1,597 psf and that the friction angle should range between 0 and 29°. Analyses of additional failures in the same rock mass, if any were available, would help narrow the range of strength values. Otherwise, comparisons with laboratory values as well as sound engineering judgment must be utilized.

ONE-POINT BACK ANALYSIS

As the back calculation of a single failure provides a shear strength value for only one normal stress, and since the normal stresses on other potential failures in the rock mass may differ vastly from those on the back-analyzed failure surface, one might question the usefulness of even making a "one-point" back analysis. However, even a one-point back analysis can provide useful strength information.

As previously mentioned, the normal and shear stresses calculated for the failure plane at failure form a pivot point through which all lines representing possible shear strength equa-

tions for the back-calculated failure must pass for all possible cohesion and friction angle values (fig. 5). The implications of this pivot point can be quite useful.

Figure 6 illustrates a Confidence Diagram from a One-Point Back Analysis based on the Nacimient Mine data. A similar diagram could help characterize the shear strength of rock mass discontinuities in a highway roadcut where a single failure has already occurred. Where normal stresses on discontinuities in this rock mass are at the pivot point (2,900 psf), then the shear strengths along those discontinuities are theoretically equal to the shear stress represented by the pivot point (1,597 psf). As discontinuities having normal stresses above and below the pivot point are examined, the uncertainty with regard to the back-calculated shear strength increases. For example, at a normal stress of 2,000 psf, shear strength could actually be any value between 1,109 psf and 1,597 psf, as indicated in figure 6. Similarly, at a normal stress of 4,000 psf, shear strength could be any value between 1,597 psf and 2,217 psf.

In many cases, discontinuities in designed slopes will involve normal stresses very close to those existing at the time of the back-analyzed failure; hence the range of back-calculated shear strengths will be small enough for design purposes. For cases where normal stresses are quite different from those around the pivot point, the usefulness of the range of back-calculated shear strength equations represented in figure 5 becomes less clear. Several reasonable procedures are illustrated in figures 7 to 10, but each must be used in conjunction with sound reasoning and engineering judgment.

METHOD 1. (Figure 7) A value for cohesion obtained from laboratory testing or previous experience could be plotted on the shear stress axis, and a line representing shear strength drawn from it through the pivot point. The resulting strength equation could be determined by solving for friction angle by hand or by using the BACKONE program. The shear strength equation could then be solved for any normal stress. The weakness of this approximation lies in its dependence on obtaining a value for cohesion.

METHOD 2. (Figure 8) If the discontinuities in question consist of "clean" rock, meaning that no clay infilling material is present, then a cohesion-equals-zero assumption might be appropriate. For normal stresses below that of the pivot point, shear strengths can be calculated that are conservative, but still fairly close to actual values. However, at normal stresses above that of the pivot point, shear strength calculations indicate strengths greater than the actual values. Designing slopes using those values could be hazardous.

METHOD 3. (Figure 9) If failure was obviously controlled by a clay seam or by a weak shale stratum, a friction-equals-zero analysis might be appropriate. However, the figure indicates

that at normal stresses below the pivot point, overly optimistic strength calculations are possible if clays do not dominate in the area of the discontinuities.

METHOD 4. (Figure 10) Finally, a conservative approach could be taken where a cohesion-equals-zero assumption is made for those normal stresses below the pivot point, and the friction-equals-zero assumption is made for normal stresses above the pivot point. Most slopes would probably fall in the range below the pivot point, and the calculated strength values would be fairly close to actual values. The slopes that fall above the pivot point would tend to be large slopes and the extra margin of safety could perhaps be justified. This is especially true in highway roadcuts built for the motoring public, where the characteristics of large rock masses are uncertain but must be considered.

Using Method 1 above, and based on an interpretation of data reported by Seegmiller (8), Watts chose to use a cohesion of 200 psf. This resulted in a back-calculated friction angle of 25.87°. If a more conservative approach had been desired, such as for a highway roadcut, Watts would have recommended that these cohesion and friction angle values be used only for slopes having normal stresses below the pivot point, and that the shear stress of the pivot point (1,597 psf) be used as the maximum shear strength for slopes having normal stresses above the pivot point. This is a combination of Methods 1 and 4 above.

CONCLUSIONS

Personal computers using software such as ROCKPACK have been shown to be extremely valuable in analyzing geologic field data and rock slope stability. Back analyses of existing failures in a rock mass, such as those provided by BACKPACK software, can provide estimates of the strength parameters of the apparent cohesion and friction angle in discontinuities having physical characteristics similar to the back-analyzed failure surface. Multiple failures in a given rock mass provide the greatest information for determining cohesion and friction angle. However, it appears that one-point back analyses can provide a range of useful infor-

mation, even in instances of a single failure, when the analysis is used in conjunction with sound engineering judgment. Additional examination of these simple back analysis concepts and comparison with other procedures are planned at the Nacimientito Mine during 1987.

REFERENCES

1. Watts, C. F. Development, Testing, and Evaluation of a Microcomputer System for Rapid Collection and Analysis of Geologic Structure Data Related to Rock Slope Stability. Ph. D. dissertation, Dep. of Geosciences, Purdue Univ., West Lafayette, IN, 386 pp. (1983).
2. Watts, C. F., and T. R. West. Electronic Notebook Analysis of Rock Slope Stability at Cedar Bluff, Virginia. Bull. of the Assoc. of Eng. Geol., v. XXII, No. 1, pp. 67-85, (Feb. 1985).
3. Watts, C. F., and T. R. West. Discontinuity Significance Index and Electronic Data Collection for Rock Slope Stability Studies. Bull. of the Assoc. of Eng. Geol., v. XXIII, No. 3, pp. 265-278, (Aug. 1986).
4. Barton, N. R. A Relationship Between Joint Roughness and Joint Shear Strength. Proc. Int. Symp. on Rock Fracture, Nancy, France, pp. 1-8, (1971).
5. Hoek, E., and J. W. Bray. Rock Slope Engineering. Revised 3rd edition, The Inst. of Min. and Metall., London, England, 358 pp., (1981).
6. Long, M. T., R. J. Barrows, and D. C. Turner. IDEAL SYSTEM-Instrument Data Electronically Acquired and Logged. 23rd Symp. on Eng. Geol. and Soils Eng., Logan, UT, 24 pp., (Apr. 6-8, 1987).
7. Da Gama, C. D. Back Analysis of Slope Failures in the Ceracado Uranium Mine (Brazil). Third Int. Conf. on Stability in Surface Mining, Vancouver, June 1981, ch. 32, ed. by C. O. Brawner, (Ref 13), pp. 745-771, (1982).
8. Seegmiller, B. L. A Case History of Support at Nacimientito Mine. CANMET Rep. 76-27, Canada Centre for Miner. and Energy Technol., Ottawa, 51 pp., (1976).

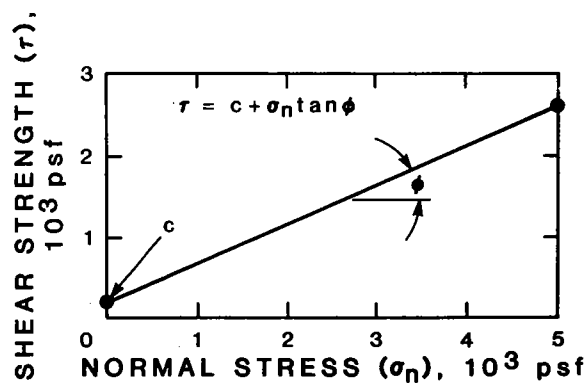


FIGURE 1. - Generalized discontinuity shear strength relationship.

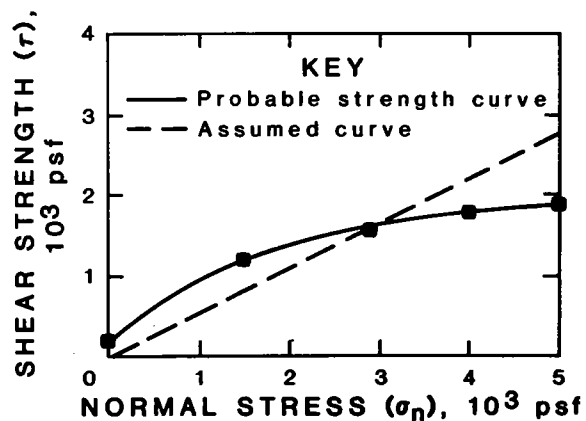


FIGURE 2. - Comparison of the probable discontinuity strength curve with a typical strength curve assumed for design purposes.

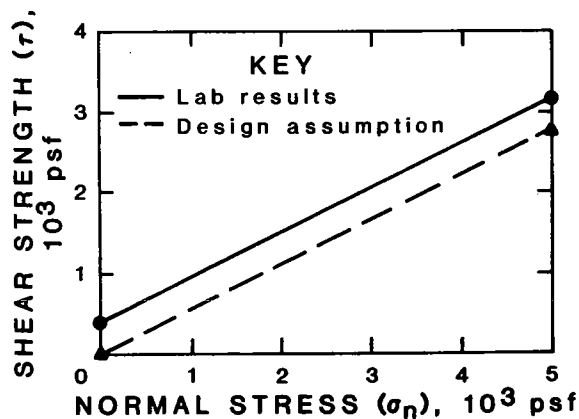


FIGURE 3. - Comparison between discontinuity strength laboratory results plotted as a straight line and the same results following the "cohesion-equals-zero" assumption.

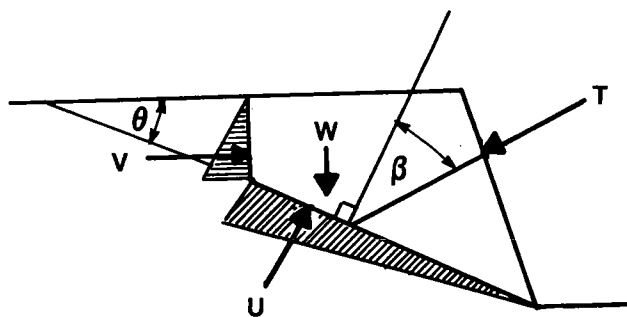


FIGURE 4. - Explanation of equation 2.

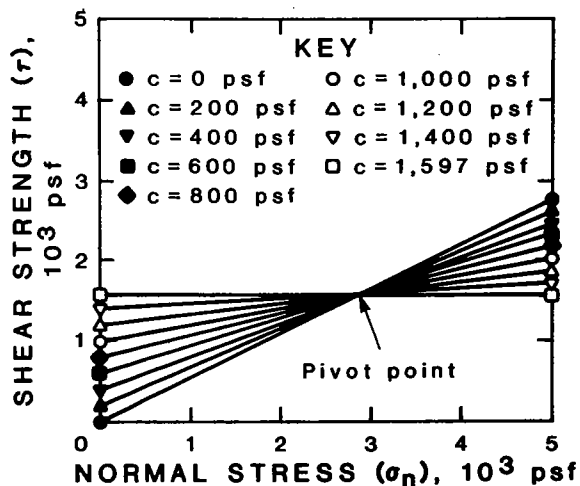


FIGURE 5. - The range of possible shear strength relationships for the back-analyzed failure surface in the Nacimientto Mine.

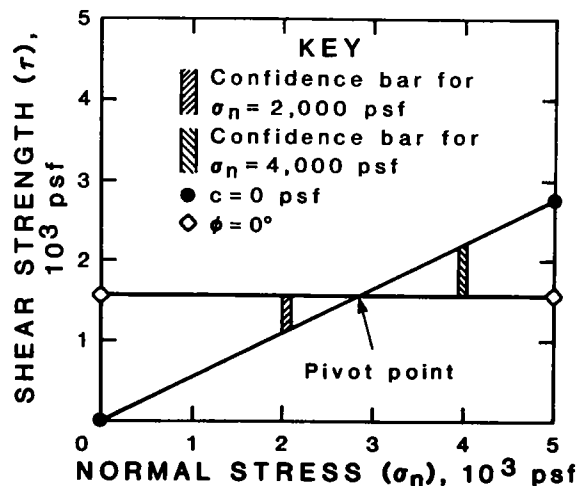


FIGURE 6. - Shear strength confidence diagram illustrating confidence bars for normal stresses of 2,000 and 4,000 psf for the back-analyzed failure surface in the Nacimientto Mine.

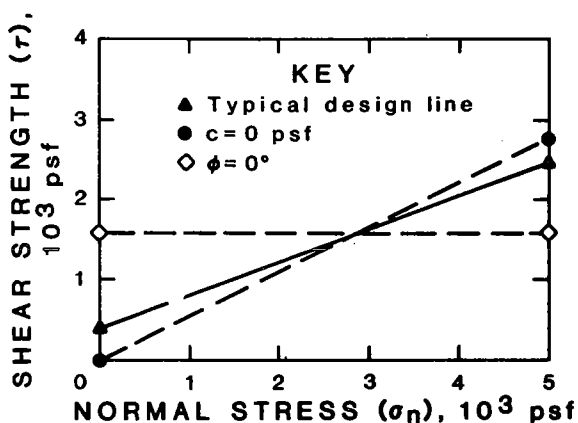


FIGURE 7. - Method 1 shear strength selection diagram. Solid line represents a typical shear strength relationship having a low c -value and falling within the $c = 0$ and $\phi = 0^\circ$ boundaries.

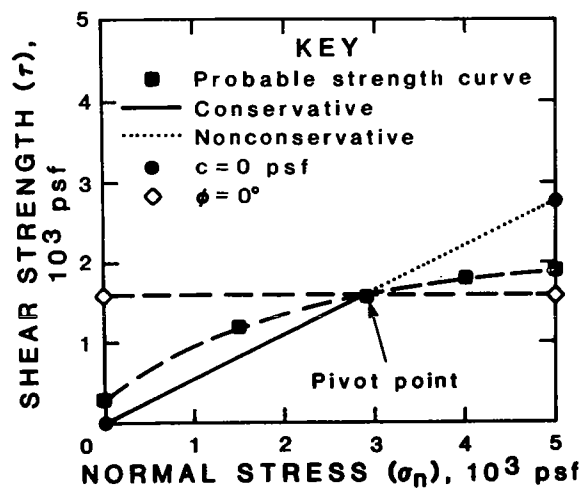


FIGURE 8. - Method 2 shear strength selection diagram. Solid line represents a $c = 0$ shear strength relationship illustrating conservative design values below the pivot point; dotted line represents $c = 0$ nonconservative values above the pivot point.

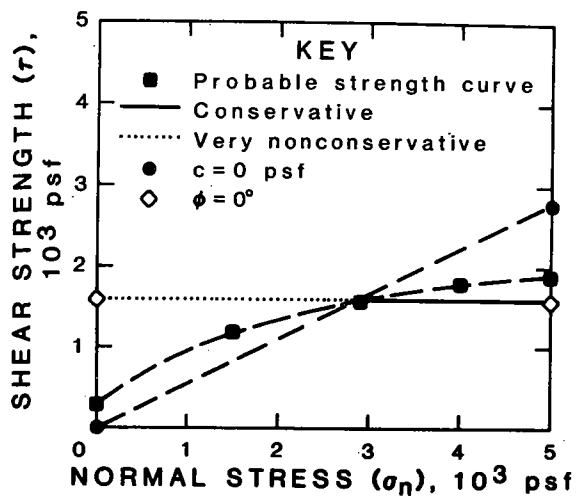


FIGURE 9. - Method 3 shear strength selection diagram. Dotted line represents a $\phi = 0$ shear strength relationship illustrating nonconservative design values less than the pivot point; solid line represents $\phi = 0$ conservative values greater than the pivot point.

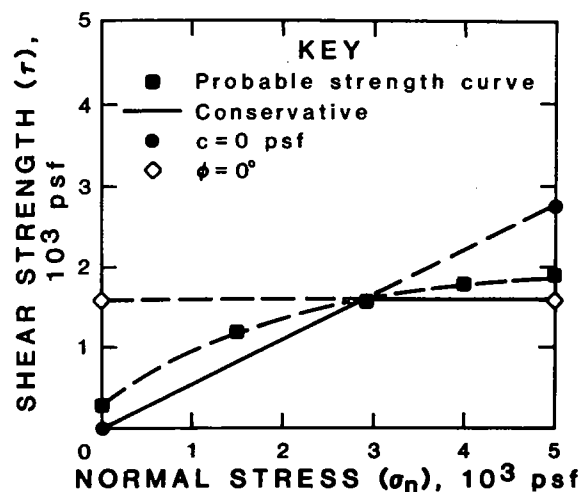


FIGURE 10. - Method 4 shear strength selection diagram. Solid lines represent a combined $c = 0$ and $\phi = 0$ shear strength relationship, resulting in conservative design values for normal stresses both less than and greater than the pivot point.

Geological Hazards Along Dorcon Road and L.R. 1099

F. BARRY NEWMAN
THOMAS R. GOWER
GAI Consultants, Inc.
Monroeville, Pennsylvania

HGS-87-7

INTRODUCTION

THIS PAPER DESCRIBES studies and resultant solutions for geologic hazards for two highway projects in western Pennsylvania. Key aspects of the projects include soil excavations on steep slopes, large rock excavations, soil- and rock-anchored retaining walls, landslide correction, subsurface stabilization of coal mines, and preloading to control settlement. Observations during and after construction are presented.

The first project to be discussed is the construction for improvements to the existing two-lane Dorcon Road (L.R. 61047, Section A00, about 1.1 miles) and the new construction for adjacent completion of the four-lane Warren Bypass (L.R. 1094, Section B00, about 0.6 miles) at Warren, Pennsylvania. The second project involves the construction of approximately 1.8 miles of the four-lane L.R. 1099, Section 01A; the relocation of 1850 feet of L.R. 64176; and the relocation of two access ramps connecting TR 201 to I-70 in Rostraver Township, Westmoreland County, Pennsylvania.

Currently, the construction along Dorcon Road and the Warren Bypass is nearly complete. The construction on the L.R. 1099 project is in its early stages.

GAI Consultants, Inc. was retained by the Pennsylvania Department of Transportation to design these two projects. GAI also conducted the studies described herein. The final resolution of the geologic issues resulted from the combined efforts of GAI, the Department and in some cases contractors in the form of alternates to improve the ease of construction.

DORCON ROAD AND WARREN BYPASS

PROJECT DESCRIPTION - The location of the project is shown on Figure 1. The initial portion of the Warren Bypass was constructed around the south western side of Warren. The new 3300-foot link between the previously existing highway and Dorcon Road completes the Bypass. Improvements to the initial 6000 feet of Dorcon Road to help maintain proper flow of

traffic from the Bypass included widening to provide minimum paved lanes of 12 feet and minimum shoulder widths of 4 to 10 feet and relocation of a portion of an existing railroad track near the connection of the Bypass with Dorcon Road.

Retaining walls up to about 25 feet high were required at several locations to accommodate the proposed construction without undermining support for the soil slopes below the adjacent Oakland Cemetery. Additional smaller walls were required near drainage structures and a wall was used to stabilize an existing landslide near the east end of the section on Dorcon Road.

An original goal was to maintain traffic throughout construction. This ultimately was not possible due to space constraints.

GEOLOGY - Dorcon Road and the Warren Bypass are located on the south valley wall of the Allegheny River. The natural soils present consist of recent alluvium and glacial deposits overlying sedimentary sandstone and shales of the Pocono Group of the Mississippian Period. The bedding planes dip slightly to the southeast at an approximate slope of 1/2 percent. The glacial soils are Early Wisconsinan Titusville till and kame terrace outwash deposits. The till which is along the total length of both sections is a light gray to a weathered yellow brown silt to clayey silt, moderately to sparsely pebbly. The western portion of the project area is also covered by a kame terrace deposit of poorly sorted sand and gravel formed by meltwater streams flowing between the glacier and the south valley wall.

Geologic maps indicate that the soil slopes above and below Dorcon Road east of the kame terrace deposits contain prehistoric and recent landslides. Site reconnaissance prior to the start of drilling confirmed the presence of these landslides and the necessity of not disturbing or aggravating existing sliding conditions with the proposed construction activities. The Department directed that the design of landslide corrective measures should be limited to the

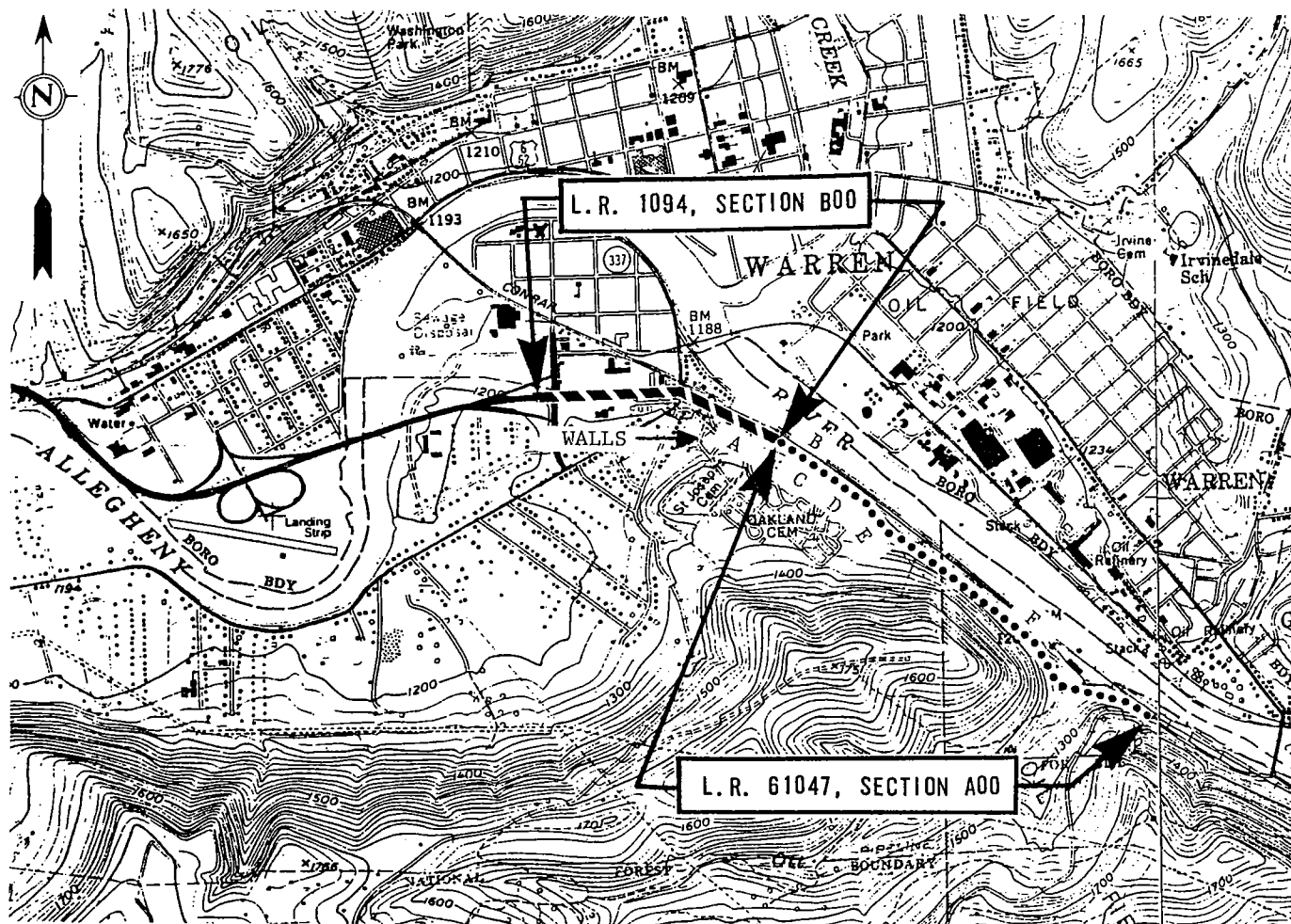


Figure 1. Location of Warren Bypass (L.R. 1094) and Dorcon Road (L.R. 61047)

one active slide on the riverside of the highway near the east end of Dorcon Road.

The geology dictated that very little fill be placed on the downhill side of the road for stability reasons. Thus, excavation into the hillside above the road was required for widening, and the original grade of the highway was maintained as much as possible. It was agreed that retaining walls for these excavations should be no larger than necessary to maintain the stability of the existing slopes.

A total of 36 borings were drilled along the the alignment during the winter of 1984. The borings were arranged generally in sections perpendicular to the slope where highway widening and/or realignment were required. The next sections describe typical subsurface conditions at wall locations. A total of 6 major walls plus several gabion walls were constructed.

WALLS B AND C - Figure 2 shows the conditions typical at several walls. Much of the soil on the uphill side of the road was the kame terrace sand and gravel deposits having an angle of shearing resistance of

about 40 degrees. Wall B was designed as a conventional cantilevered wall in an area where no slope was present below the wall and a small amount of fill was added above the wall. Wall C was a driven soldier beam and treated wood lagging wall with a precast concrete panel facing. This type of wall was used below all of the uphill cut areas. The basic concept was recommended and designed by GAI to provide support for the slopes as the excavation proceeded. Earth Support Systems, Inc., who built the walls provided the final wall designs as an alternate bid. The shorter walls were cantilevered, and up to two rows of grouted soil anchors were used for the higher walls.

GABION WALLS - Several gabion walls were installed along the downhill side of the highway in shallow swales above steep slopes to permit widening and/or for drainage structures. The maximum height of these walls was about 10 feet and they were designed as conventional gravity walls.

LANDSLIDE CORRECTION WALL F - Figure 3 shows the typical section of Wall F used to correct a landslide in cohesive soils

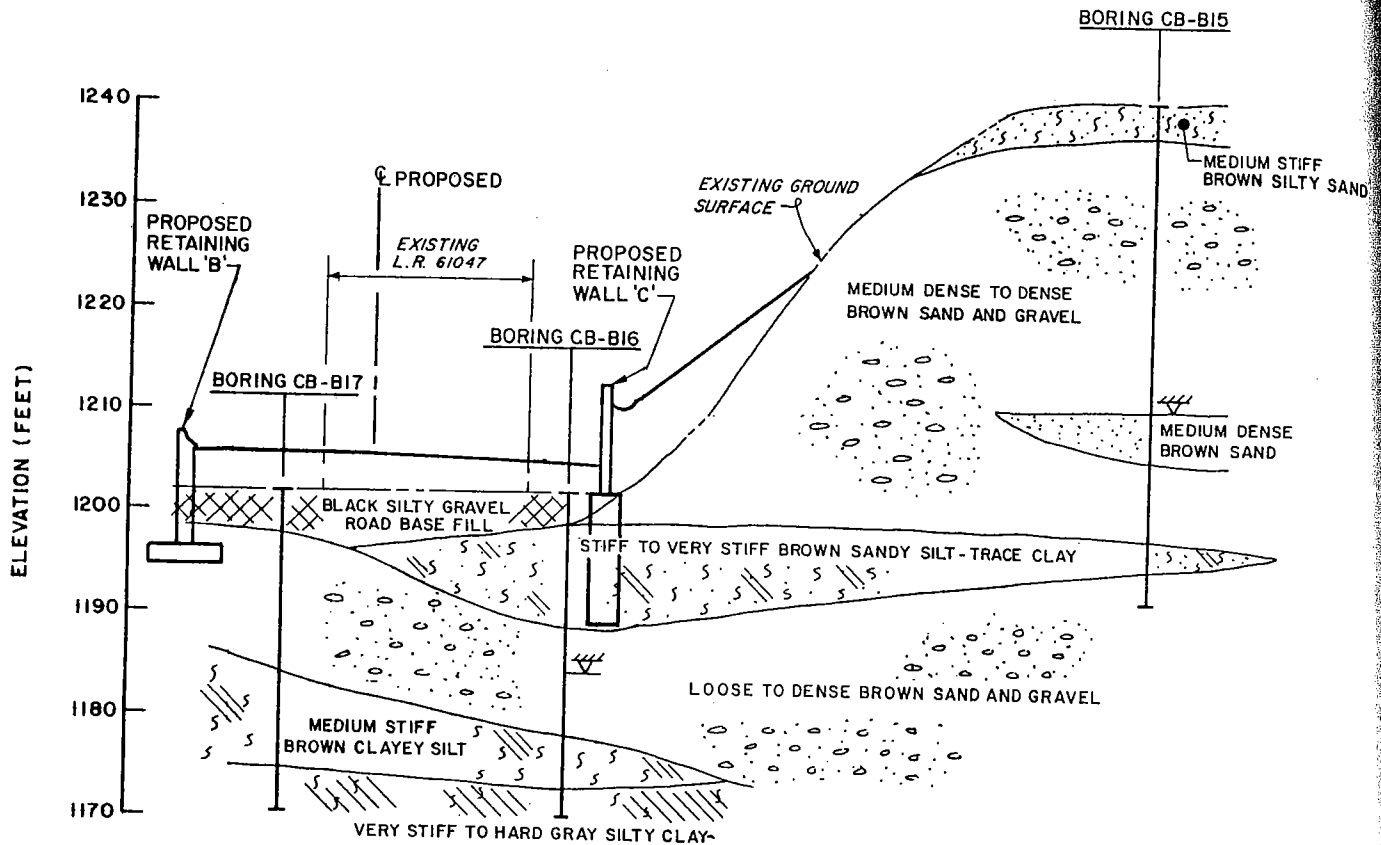


Figure 2. Geologic Section at Walls B and C on Dorcon Road

affecting the road. The wall was chosen over simply overexcavation and backfill to assure stability in this area and to limit the impact on traffic during construction. The entire slope below this portion of Dorcon Road extending to the railroad track below showed signs of sliding movements. A small gravity wall at the toe of this slope above the railroad track is also marginally stable. Therefore, overexcavation of the soils below the new wall unloaded the lower portion of the slope improving its stability. While computations indicated that the wall did not have to be extended into rock, it was for additional safety relative to a toe failure. The soldier beams for this wall had drilled pier foundations, and two rows of rock anchors were required in the excavated zone.

OBSERVATIONS DURING CONSTRUCTION -

Generally the construction was carried out as planned in 1986 and the walls are performing satisfactorily. One problem that is to be corrected is shallow sloughing of the slope above one of the walls. Actual slope geometry was slightly higher in this area than the wall was designed to retain. Thus, the wall is to be extended slightly higher. Work was halted for this past winter, and seeding and drainage above the walls is to be completed this year.

L.R. 1099

PROJECT DESCRIPTION - The proposed four-lane highway will connect the existing Donora-Monessen Bridge with Interstate 70 as shown on Figure 4. The route crosses a hill top, a major slag dump, Speer's Run valley and a second major hilltop before ending in the Monongahela River valley. The geotechnical studies for approximately 1/4 of the current alignment were conducted in 1970 by another consultant. GAI conducted studies for the remainder of the alignment in 1985 and 1986.

Proceeding from south to north, a 100-foot high cut will be required through the first hilltop. The fill crossing the Speer's Run valley will be 100 feet high and will be underlain by an approximate 1155-foot long culvert to carry Speer's Run. The existing highway (L.R. 64176) adjacent to Speer's Run will be relocated upslope and pass through the upper portion of the embankment below a proposed bridge for the four-lane highway. The four-lane highway will then proceed through a 140-foot high cut and finally join the south end of the Donora-Monessen Bridge.

GEOLOGY - The natural soils in the hilly terrain of most of the route are primarily residual and colluvial cohesive soils weathered from the underlying rock strata. The

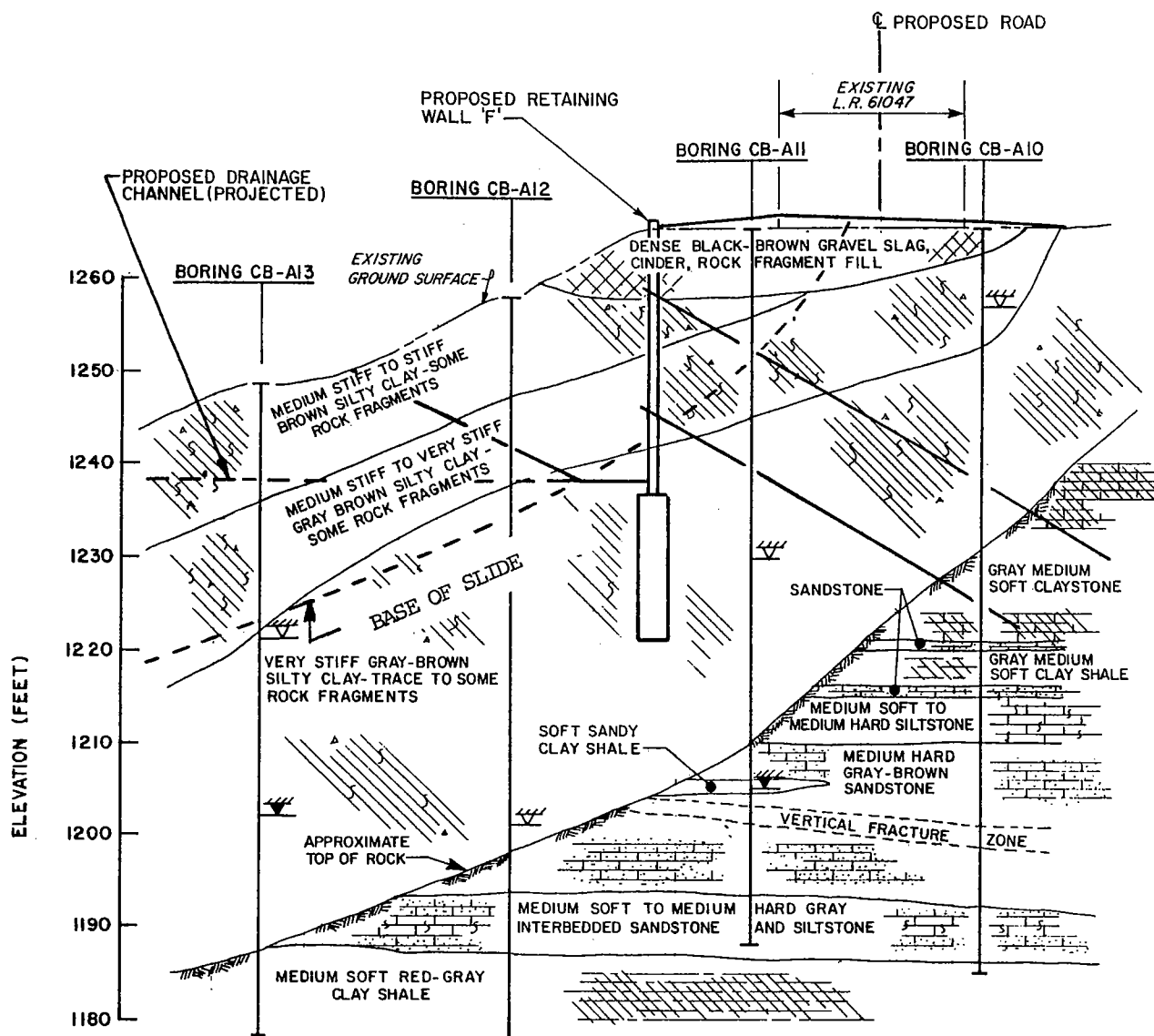


Figure 3. Geologic Section of Landslide Correction Wall F on Dorcon Road

access ramps to I-70 also encounter the Carmichaels Formation, a Pleistocene age, non-glacial soil deposited by the Monongahela River in abandoned sections of its valley. This soil formation is made up of sand, silt and clay with varying amounts of gravel and boulders. Recent alluvium is also present in the small drainage basin valleys crossed by the route.

The rock is sedimentary in origin and of the Pennsylvanian age Pittsburgh Formation of the Monongahela Group. The Pittsburgh Formation is bounded by the Pittsburgh Coal at its base and extends to the top of the Benwood Limestone. The primary members of the Pittsburgh Formation in ascending order above the Pittsburgh Coal are the Pittsburgh Sandstone, Redstone Limestone and Coal, Fishpot Limestone, Sewickley Coal and Benwood Limestone. The proposed grade of L.R. 1099 Section 01A crosses all of these rock units.

The rock strata dip 2 to 3 degrees to the southeast.

The entire route has been undermined by deep mining in the Pittsburgh Coal. The Redstone Coal has been strip mined near the Donora-Monessen Bridge portion of the route. No other mining has been detected in the area.

A total of 75 borings were drilled for the new alignment for this project during the fall of 1985. They were located along the centerline and provided cross sections where cuts and fills were planned. Additional borings were made later for final design of selected structures.

All mining below the route is abandoned. In the previous study report, the undermining in the area toward the Donora-Monessen Bridge where the Pittsburgh Coal outcrops was addressed. There is typically 140 feet of rock between the finished highway subgrade and the top of the mined seam below most of the new

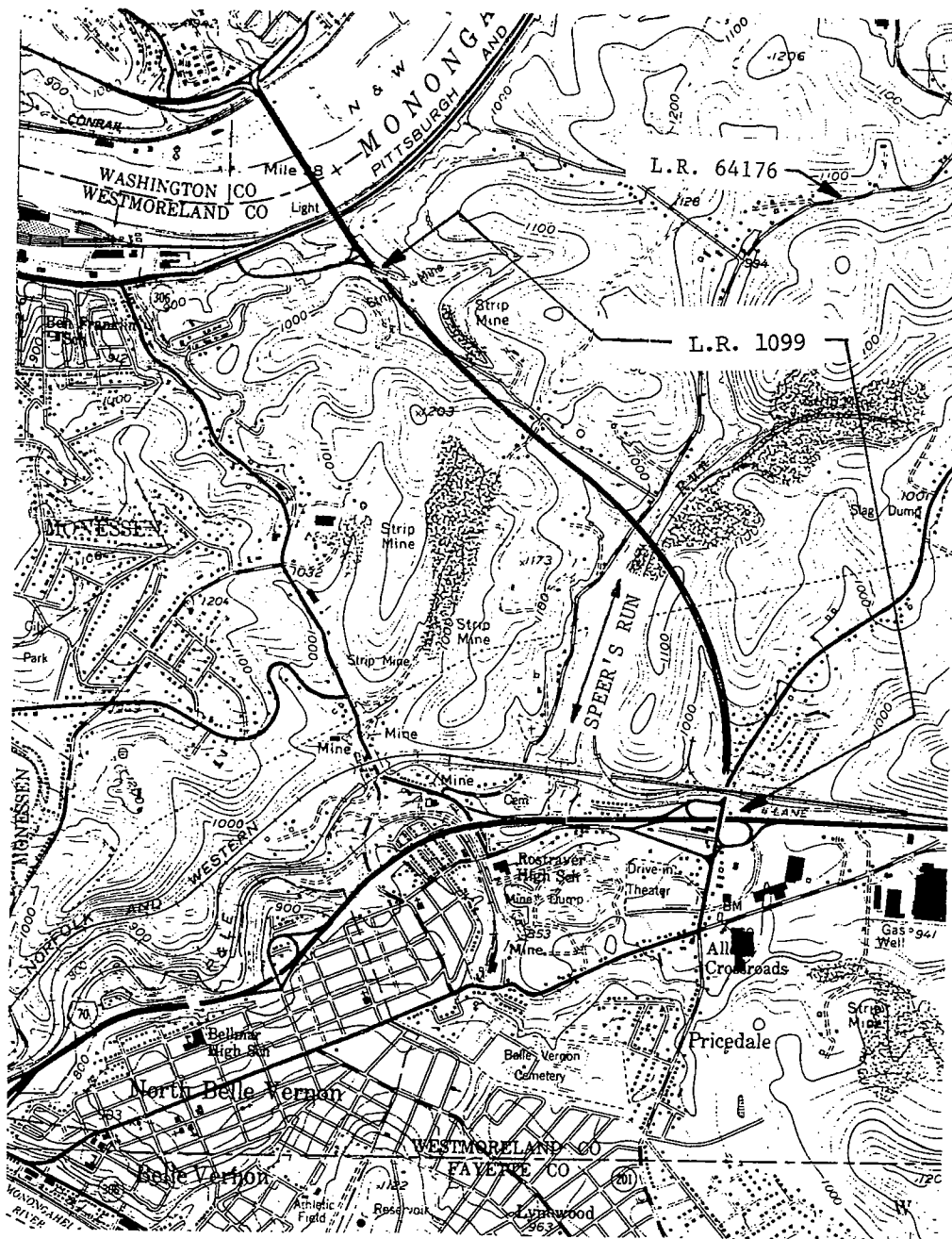


Figure 4. Location of L.R. 1099

section. The exception is the low point of Speer's Run Valley where the coal mine has only 40 feet of rock cover. The coal was extracted using the room and pillar method of coal mining and based on the available mine maps and drilling information, the mine roof has not collapsed. Of the five borings drilled through the mine in Speers Run Valley, all encountered large voids (4.5 to 10 feet) and gob filling. It was concluded that there is a significant probability of mine subsidence in this area where the thickness of the overlying proposed fill will be about 100 feet and the rock above the mine is only 40 feet thick.

HILLTOP CUTS - The geology of the first hilltop cut is shown on Figure 5. The rock types encountered in this area were primarily argillaceous limestone of the Benwood Limestone. The upper portion of the cut is interbedded with thin layers of soft shale and claystone, and the lower portion is generally harder and less broken. It was originally desired to provide a constant 1.5 horizontal to 1 vertical (1.5H:1V) slope in this rock because of its relatively broken condition and its tendency to weather into boulders that can readily fall down steeper cut slopes. However, an existing transmission tower at the top of the slope precluded this design. Thus, the cut slope geometry shown on Figure 5 was

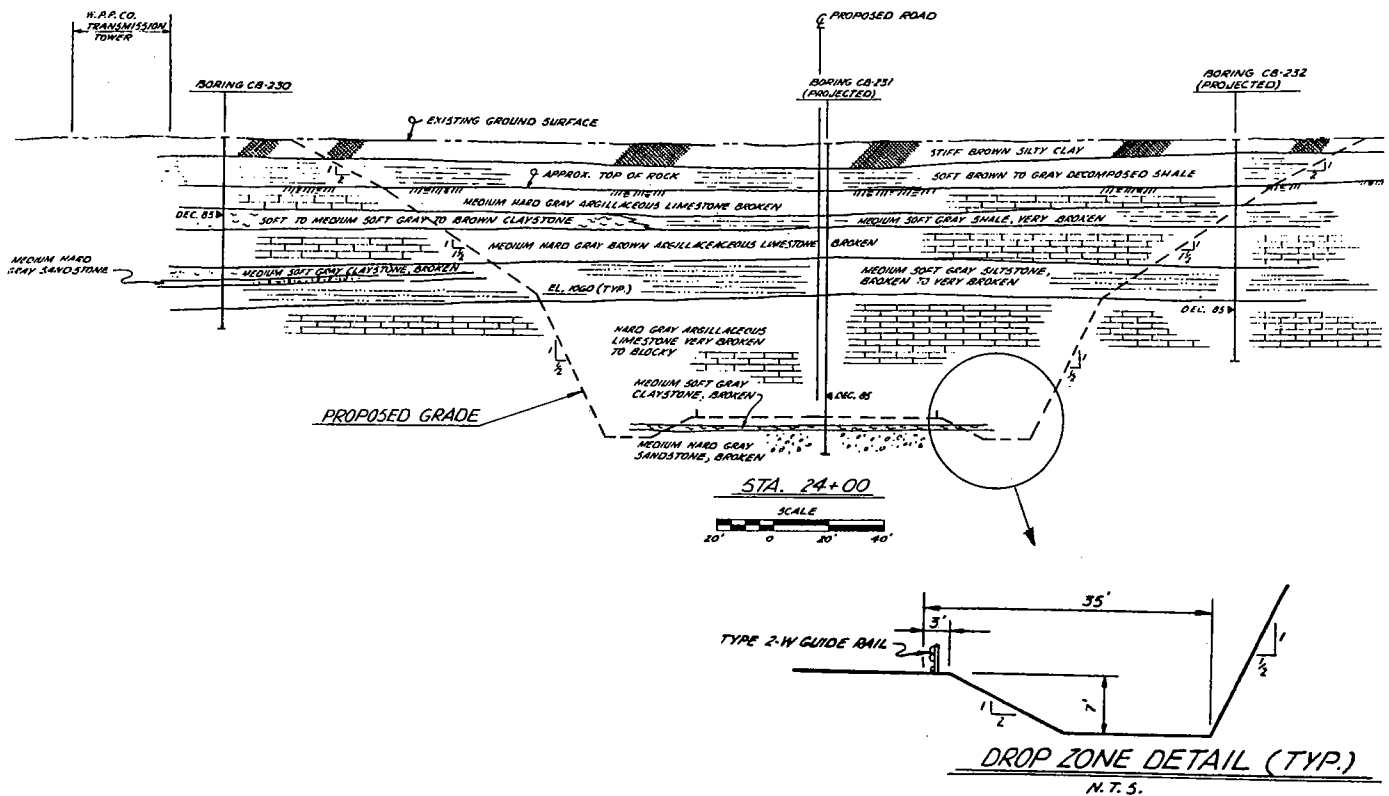


Figure 5. Geologic Section and Drop Zone Detail
for First Major Cut for L.R. 1099

selected to provide a drop zone at the base of the cut while maintaining sufficient distance for foundation stability between the top of the proposed cut slope and the existing tower. The proposed configuration may require occasional cleaning of the drop zone. However, the ditch geometry was sized to prevent falling rock from bouncing onto the pavement.

The geology of the second hilltop cut nearer the Donora-Monessen Bridge is similar to the first, but there were no specific constraints at the top of the cut slope. Thus, the entire rock slope is being excavated at 1.5H:1V. The permanent soil cut slopes are to be 2H:1V.

SUBSURFACE STABILIZATION - Figure 6 shows the subsurface conditions below the proposed 100-foot high embankment in the Speer's Run area. Considering that mine subsidence could close the culvert carrying Speer's Run or damage the highway bridge at the top of the fill, GAI recommended that the entire mined zone below these proposed facilities be filled with low strength concrete or grout. The perimeter of grout protection below the culvert and bridge was established based on a 1:1 slope in soil and a 15° slope in rock.

The probable effects of potential subsidence damage outside this zone where cover is greater were judged to be less critical and special protective measures were not employed. The outcrop of the deep-mined Pittsburgh Coal near the Donora-Monessen Bridge is being overexcavated and backfilled

where cover is less than 30 feet to address the potential for subsidence in this transition zone.

PRELOADING - The proposed bridge crossing the relocated L.R. 64176 is to be a single span with reinforced earth abutments setting atop the 100-foot thick fill. It was desired to provide a mechanism to assure that the settlement of the bridge and its abutments due to the long term consolidation of the underlying embankment would be tolerable. GAI recommended that the entire zone of bridge construction initially be filled with embankment material (preloaded) and monitored for settlement. Both settlement monitoring points and horizontal slope indicator tubing have been included in the construction program. The preloaded area is estimated to settle six inches during the first six months following embankment construction. The actual data obtained will be used to determine when to remove the preload and construct the lighter bridge and abutments.

Settlement of rock fill composed of potentially degradable rock is harder to estimate (magnitude and rate) than soil fill. Thus, the zone below the bridge is to be filled with soil or shale compacted to a dense soil fill for more uniform, predictable behavior.

The specifications for the reinforced-earth wall abutments require that only No. 57 durable aggregate be used to construct the walls. This requirement is aimed at assuring

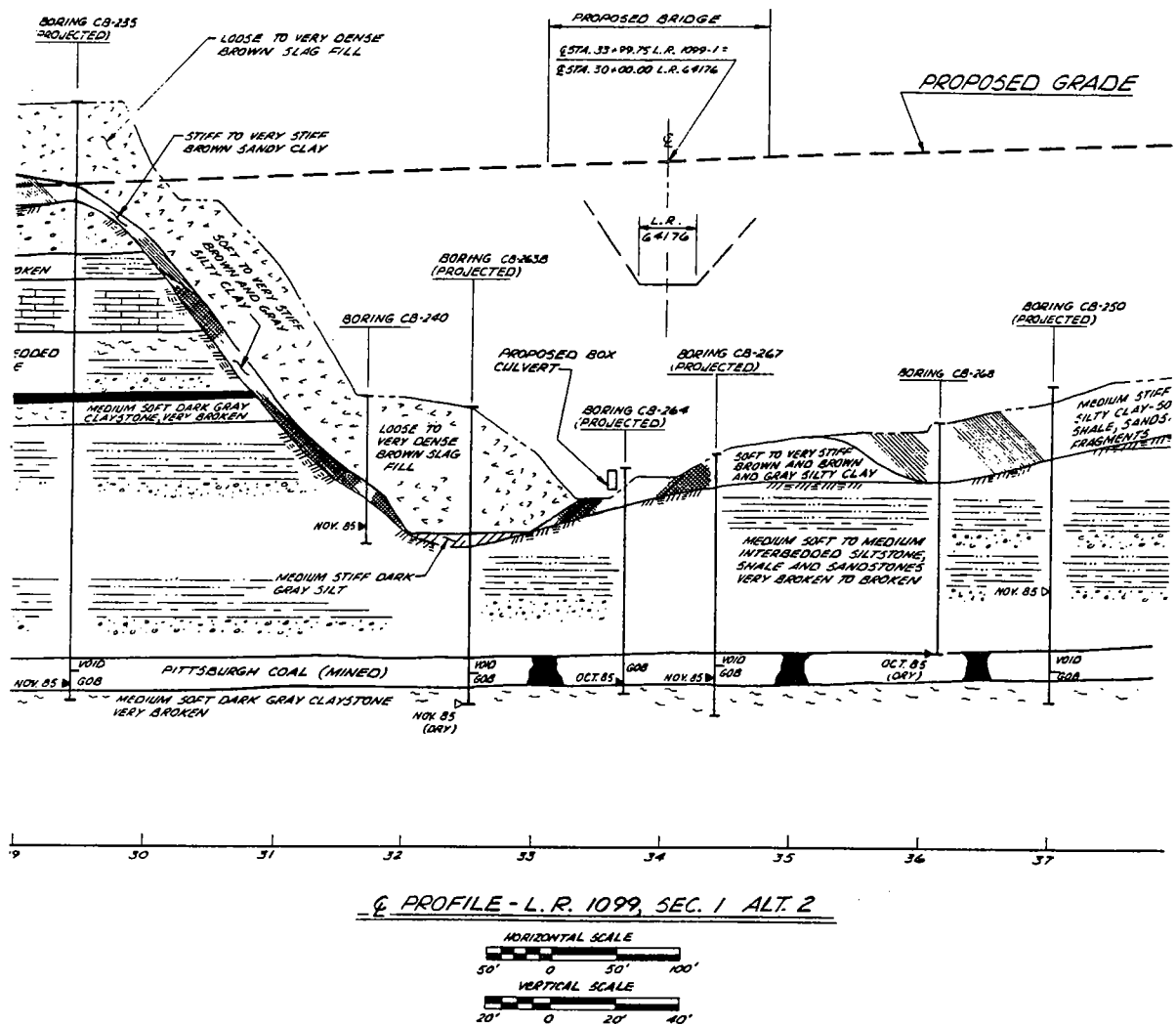


Figure 6. Centerline Profile Showing Mine Below 100-foot High Embankment and Zone for Proposed Bridge for L.R. 1099

that settlements of the abutment walls and bridge footings will occur during construction when adjustments can be made. The only remaining long term settlement should be due to the settlement of the embankment under its own weight. This should be predictable and tolerable at that point in time based on the preload settlement data.

OBSERVATIONS DURING CONSTRUCTION - A portion of the grouting is complete at this time and work is in progress to undercut the Pittsburgh seam near the Donora-Monessen Bridge. The major excavations and fills will not occur until later.

Drilling for the grouting program initially encountered only coal-in-place. At that point, the entire program was being questioned. Subsequent drilling disclosed about 50 percent voids confirming the advisability of the program.

CLOSURE

The method used to select solutions to the geologic hazards along these highway projects was common sense application of basic principles of engineering geology and geotechnical engineering. While there are several possible variations to approaches to many of the issues, we believe that the selected options offer a reasonable compromise between technical acceptability, economy and overall project objectives.

New Methods of Highway Stabilization

DONALD A. BRUCE and DENNIS L. BOLEY
Nicholson Construction Company
Bridgeville, Pennsylvania

HGS-87-8

1. INTRODUCTION

Most highway engineers are by now well acquainted with the use of prestressed rock and soil anchorages in slope stabilization programs. Such anchorage systems may be applied:

- directly to the slope face above and/or below the highway (Reference 1)
- to stabilize primary retaining structures such as piled or slurry trench walls (Reference 2)
- to stabilize existing structures affected by changed geotechnical or structural parameters (Reference 3)

In addition, through the efforts of researchers, constructors and suppliers alike, and via the medium of conferences such as this, other techniques are gaining wide recognition; for example, the use of stone columns, dynamic consolidation and geotextiles is reviewed elsewhere in this Symposium.

The aim of this paper is to highlight three other groups of options for stabilizing slopes in appropriate conditions:

- insitu reinforcement (by nails, micropiles and dowels)
- large-scale drainage (by trenches and large diameter wells)
- ground treatment (by jet grouting)

Although each group is relatively new to American practice, each represents a powerful tried and tested solution with several years of application in Western Europe and the Far East.

Each group is introduced in turn, and typical applications are described. Key references are provided for the reader who wishes to pursue the details of design and construction.

2. INSITU REINFORCEMENT

From the early 1970's, engineers in Western Europe and Western Canada have exploited the special advantages of slope stabilization by insitu reinforcement (Reference 4). Insitu reinforcement

comprises inclusions placed in the soil to maintain equilibrium under the soil self weight loading, and surcharge loading on the soil. Today, one of the methods - soil nailing - is one of the fastest growing geotechnical processes in the States. This extraordinary national upsurge may be dated from the execution of the application of soil nailing to stabilize the excavation for the foundations of the PPG Building, here in downtown Pittsburgh, in 1982 (Reference 5).

2.1 Insitu Reinforcement Techniques - There are three main categories of insitu reinforcement techniques used to stabilize soil slopes and excavations. These are *nailing*, *reticulated micropiling*, and *dowelling*.

In *soil nailing*, the reinforcement is installed horizontally or subhorizontally so that it improves the shearing resistance of the soil by acting in tension (Figure 1a).

Reticulated micropiles are steeply inclined in the soil at various angles both perpendicular and parallel to the face (Figure 1b). The overall aim is similar to soil nailing, namely to provide a stable block of reinforced soil which supports the unreinforced soil by acting like a gravity retaining structure. In this technique the soil is held together by the multiplicity of reinforcement members acting to resist bending and shearing forces.

Soil dowelling is applied to reduce or halt downslope movements on well defined shear surfaces (Figure 1c). The slopes treated by dowelling are typically much flatter than those in soil nailing or reticulated micropile applications. Gudehus (Reference 6) has shown that the most efficient way to improve mechanically the shearing resistance on a weakened shear surface through the soil is to use relatively large diameter piles which combine a large surface area with high bending stiffness. The diameter of a soil dowel is thus generally far greater than that of a soil nail or micropile (typically up to 6").

2.2 Selecting Insitu Reinforcement - Although there are fundamental differences in the mechanical action of these three insitu reinforcement techniques, there are circumstances where more

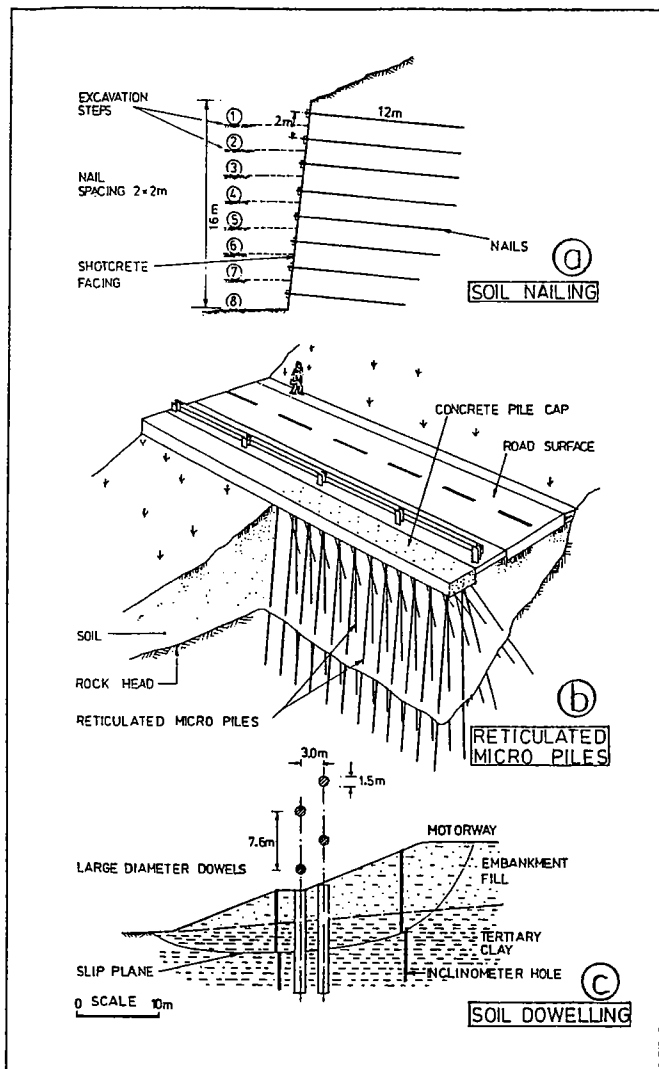


Figure 1 The family of insitu soil reinforcement techniques. (Bruce and Jewell, 1986)

than one may be applied to slope stabilization as illustrated in **Figure 2**. The following points should be considered when choosing the appropriate insitu reinforcement technique.

Laboratory experiments have shown the influence of the inclination and properties of reinforcing members on the shearing resistance of reinforced soil (Reference 7). These indicate that the reinforcement gives the best increase in strength when it is angled across the potential rupture surface in soil so that the reinforcement is loaded in tension. At other orientations in the soil, the reinforcement provides less benefit, and can even reduce the shearing resistance of the soil mass if it is forced to act in compression.

The conclusion, therefore, is that in applications where a steep slope is to be excavated in a homogeneous granular soil, it is most efficient to place the reinforcement through the face in a

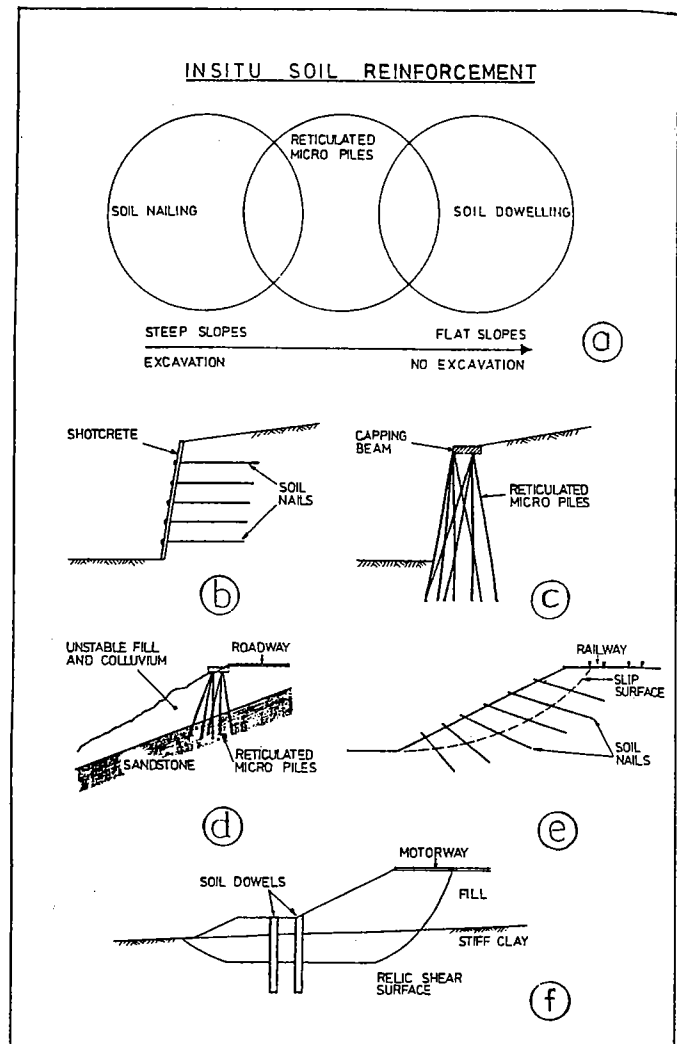


Figure 2 Overlap of insitu soil reinforcement applications (b) and (c) in excavations, (d) and (e) for general slope stabilization, and (f) to stabilize residual slips in clay. (Bruce and Jewell, 1986)

direction close to the horizontal, as in **Figure 2b**. To stabilize the soil with reinforcement placed in substantially vertical directions (**Figure 2c**) will require a much higher density of reinforcement. For this type of application soil nailing is likely to be more cost-effective than reticulated micropiling.

In marginally stable granular or scree slopes when stability must be improved, but where excavation is not foreseen, then either soil nailing or reticulated micropiling would be appropriate. Where drilling equipment cannot be placed on the slope, reticulated micropiling would be best (**Figure 2d**). Where access is not problematical, either technique could be applied (**Figures 2d and 2e**), with economic considerations being decisive.

In flatter clay slopes where stability is governed by a well defined shear zone, larger diameter soil dowels would be most appropriate (**Figure 2f**). (Also References 8 to 10).

2.3 Local Examples of Insitu Reinforcement -

2.3.1 Soil Nailing: Cumberland Gap: Bearing in mind that the classic PPG soil nailing contract in Pittsburgh was to secure a deep excavation, the work on US 25E at Cumberland Gap (Reference 11) represents a milestone in slope stabilization by this method. This FHWA project was conducted to stabilize a slope near the Kentucky portal of a pilot tunnel under the Cumberland Gap.

The parent rocks in the area are a series of shales, sandstones, and siltstones that have been differentially weathered to form a soil mantle ranging in thickness from 5 to 20 feet. Beneath the soil overburden is weathered rock of varying degrees of competence and thickness. In general, competent bedrock is encountered at a depth of from 10 to 40 feet.

The design of the 40-foot-high section of the wall, based on the studies of University of California at Davis (Reference 12), resulted in a design as shown on Figure 3. The spacing of the nails was a grid 5-foot square with an assumed nail diameter of 0.58 foot. The upper 3 or 4 rows of reinforcing members were to be #8 bars, and the lower rows, #10 bars - all grade 60 steel. (During construction, #11 bars were substituted for the #10 bars because of availability.) Up to 8 rows of nails were installed, and a total of 335 nails were installed in the 9,000 square feet cut.

Prior to construction, two vertical test bars were installed and load tested to verify design assumptions on interfacial bonds. During construction, randomly selected nails were tensioned to 90% yield, as quality assurance.

The wall has been instrumented with three different types of load and movement measuring devices:

1. slope inclinometers on and behind the slope
2. strain gauges on 16 bars
3. electronic distance measuring points on bar heads

Instrument groups 1 and 3 are in agreement, showing maximum total head movements of 1/4 to 3/8 inch. This is less than 0.1% of slope height--a performance within the "typical" range of 0 to 0.3% identified by Bruce and Jewell (Reference 4). Strain gauge data have proved more difficult to decipher, but frost-related fluctuations in head load have been detected. The FHWA continues to monitor the installation and to analyze its performance.

This was the first use by the FHWA of nailing to solve a combination of design and environmental problems, and was recognized by an award in the "Excellence in Highway Design," 1986 Biennial Awards of the Federal Highway Administration (Category VIII - Highway Improvements in Federally-owned Lands).

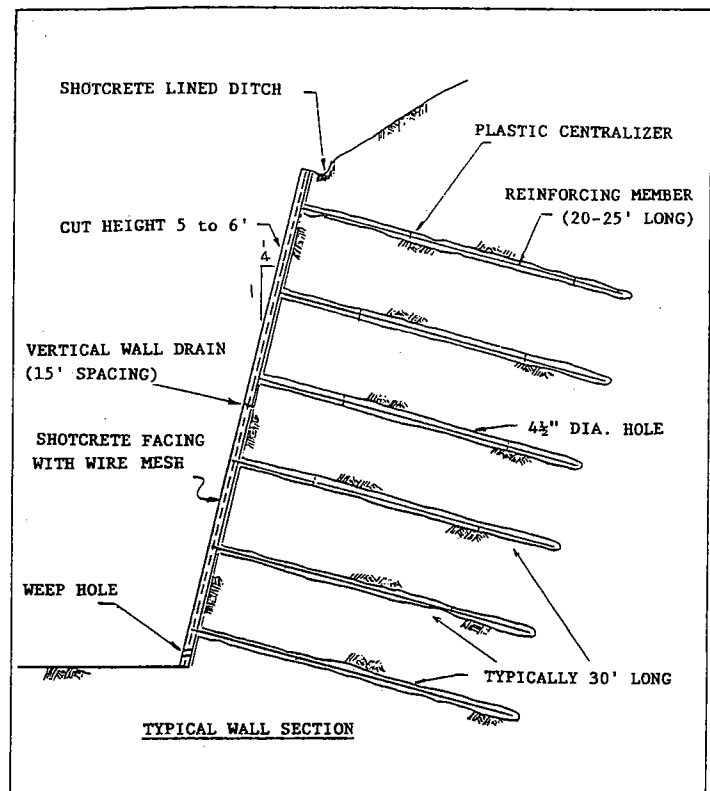


Figure 3 Details of Cumberland Gap soil nailing (Nicholson, 1986)

2.3.2 Reticulated Micropiling: LR69: Legislative Route 69 between Vandergrift and Leechburg, PA runs along the Kiskiminetas River. Much of it is cut into the steep hillside along the bank of the river, with the cut material used as downhill side fill to provide the highway bench. In a number of places this embankment material proved unstable and slides occurred.

One such slide repair was designed by The Pennsylvania Department of Transportation (PennDOT) as a tie-back caisson wall. This project was bid in the Summer of 1984 with bids accepted on the "as designed" wall or contractor designed alternatives. The authors' company was successful based on an alternate design (Figure 1b). This type of wall is referred to as a Type "A" INSERT Wallsm (Reference 13).

The 310 feet long slide had previously been repaired by filling and repatching, as the closeness of the river prohibited placing a fill buttress below. Given its role as a major traffic route, detours would have proved lengthy and inconvenient and so the stabilization method had to allow passage of one lane of traffic throughout.

The design height for both the as-designed wall and the insert wall, was 34 feet. The spacing on the inserts (Figure 4) was chosen to achieve the

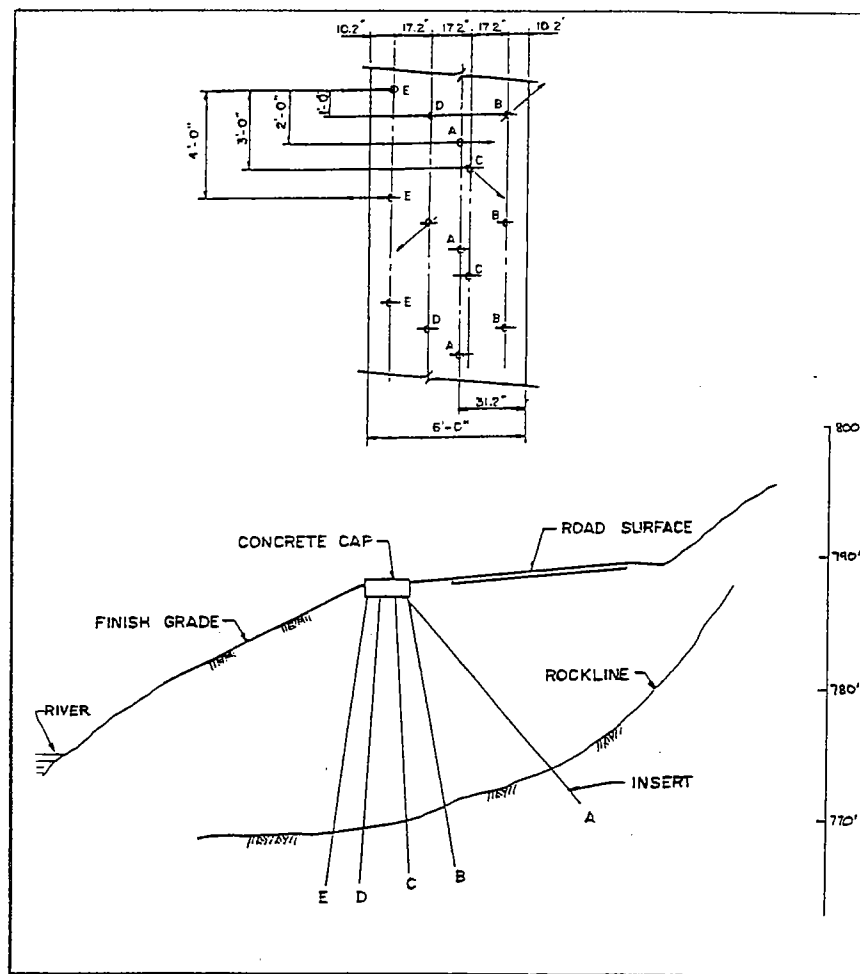


Figure 4 Details of LR69 insitu reinforcing - Type A wall
(Boley and Crayne, 1985)

maximum width at the top of rock for stability purposes. It was determined that the minimum density would be one reinforcing unit per 20 square feet at the top of rock. The resultant width of the gravity mass at the top of rock was approximately 20 feet. The stabilized mass thus achieved was, however, still potentially unstable with respect to overturning during maximum rapid drawdown conditions following a flood. For this reason, an additional row of tension members was added (Row A, Figure 4), of full design load 80 kips. These additional tension members gave a minimum factor of safety of 1.24 during rapid drawdown conditions and a factor of safety against overturning during steady-state seepage conditions of 1.95. A density of 1.33 nr 6" dia. holes per lineal foot of wall satisfied all the stability criteria, when equipped with #11 or #14 rebar, grouted with a neat cement mix to 40 psi excess pressure.

All the inserts were extended into rock a minimum of 15 feet to ensure that they would function as shear keys to thus prohibit sliding of the gravity mass and also to allow the transfer of any axial load, whether in tension or compression, into bedrock. Special diesel hydraulic drilling rigs were

used to ensure fast penetration through both soil and rock in one pass.

Monitoring of the slide by slope indicators during 1984 showed movement of the slide at a depth of 30 feet of about 0.5" during a seven-month period. The slope indicator readings taken between August and November at 30 feet depth showed additional movement of about 3.75". On about November 10, 1984, after the concrete cap had been placed and the drilling of the inserts just started, some additional movement occurred. A section of roadway along almost the entire length of the cap moved vertically downward about 4" and a crack about 1" wide opened up in the middle of the road surface. No movement or cracking of the cap was detected so installation of the inserts in the slide area was accelerated to provide support as soon as possible. About a week later, after installation of about 40 of the inserts, movement appeared to have stopped. No further movements of the roadway have been detected and other slope indicator readings taken to date have shown little or no further movement.

Savings to the State of about 15% of the total contract price as foreseen using the conventional design were generated. The cost per foot of roadway for the repairs, not including other roadway items such as drainage, paving and traffic control, is about \$1,000 to \$1,500 per foot of roadway. The exact cost depends on many factors including the size of the project, depth to stable material or rock, and composition of the sliding soil.

3. DRAINAGE

An unfavorable groundwater regime is a common cause of slope instability. Pore pressure decreases the effective stress level, and may reduce the shearing resistance of the material along a potential slip surface to a degree where unacceptable movement and failure may occur. In such conditions it is logical and effective to reduce the groundwater potential, thereby reducing pore pressures and increasing the effective stress and the factor of safety against sliding. Drainage is most effective (Reference 14) when placed as close as possible to the potential failure zone, when influencing the largest possible portion of the zone, and if placed in the direction of maximum slope inclination. Stabilization by drainage is appropriate to materials of both high and low permeabilities, as change in flow net patterns is also very important (Reference 15).

Small diameter drains inclined just above horizontal are commonly used to drain steep slopes or faces, while in other, usually temporary, applications, vertical dewatering wells are used to provide the same function within their pump out influence zones.

However, such solutions may not be practical or economic where slopes are shallow and cover large areas, or where urban structures or underground services otherwise preclude their use. Two excellent systems which are being used routinely in Western Europe, and which have major potential for application here are:

- Drainage Trenches (for shallower slides in open areas), and
- Drainage Wells (for slides of all depths in urban and rural areas).

3.1 Drainage Trenches - In conditions where surface access is good and continuous and the depth to the potential failure plane is less than 30 feet, then drainage trenches are economic and practical. Shallow conventional diaphragm wall equipment and techniques are used to form vertical continuous panels in the soil, but without the use of bentonite slurry. These panels are backfilled with an appropriately graded granular fill material, in a sequence which ensures perfect continuity between adjacent panels. The trenches are arranged in an interconnecting "herringbone" pattern (e.g., Figure 5) generally at right angles to the contours (they have no shear strength to resist lateral thrust) and are later covered with the natural soil to conceal

their presence. At the downslope end of the system, the water collected along its length flows, by gravity, into a collector drain or simple discharge well.

The rationale of design has been addressed by Stanic (Reference 14), while the application in conjunction with the companion technique of drainage wells, is described by Bianco and Beligni (Reference 16) and Bianco (Reference 17).

A good recent example was seen by the authors at a site near Ancona in Eastern Italy. In a rural farming area of rolling hills, the development of a very large areal slip was threatened in a thickness of 15-33 feet of new cohesive colluvials and alluvials overlying similar but stronger and stabler materials of Middle Pliocene age. The piezometric level (about 10 feet below the surface) was largely responsible for the marginal stability of an area of about 250,000 ft.² (600' down slope and up to 450 feet wide). It was calculated that a reduction of the piezometric level by 20 feet would increase the factor of safety against slippage to an acceptable minimum of 1.30. The slope height difference down the 600' length was about 110 feet.

Panels 30 feet deep, about 30" wide and 7 feet long were excavated in sequence with a hydraulic rope suspended grab. The leading edge of each successive panel was temporarily protected by a full height protective steel plate, later removed upon backfilling of the adjacent panel. Gravel was placed in each trench to about 5 feet from the ground surface and topped with a geotextile sheet. Soil backfill was then graded over. In this way about

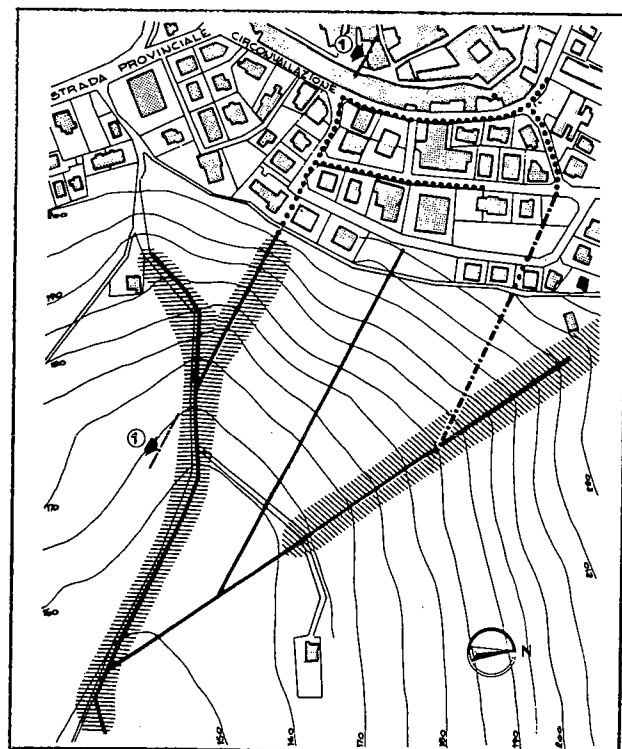


Figure 5 Stabilization of landslide at Monte S. Giusto, Italy by drainage trenches (and deep wells) (Bianco, 1986)

80,000 ft.² of drainage trenches were formed in a number of interconnecting "arms" aggregating about 2,500 lineal feet of trenches. At the bottom of the slope, the system discharged into a horizontal outlet drain.

Daily productions per machine can reach over 2,000 ft.² of trench per day, but can be as low as 300 ft.² per day in collapsing trench conditions where biodegradable slurry must be used as support. In this particular project, the approximate cost of slope stabilization was about \$2.50 per square foot of slope.

3.2 Drainage Wells - A series of major landslides which caused extensive structural damage and tragic loss of life in the Ancona-Tresenda area of Italy in the early '80's spurred the specialist geotechnical contractor, Rodio, to devise new systems of large slope stabilization for urban areas. Large diameter vertical wells are excavated by grab or auger to full depth which may be well over 60 feet. These can either be backfilled completely with filter material, or can be constructed with a combination of materials to provide both drainage and insitu reinforcement of the soil dowell type (Figure 1c). The key element of the system, however, is the horizontal interconnecting collector drain: this is a small diameter plastic pipe linking adjacent wells at depth and installed immediately after well drilling but before backfilling (Figure 6).

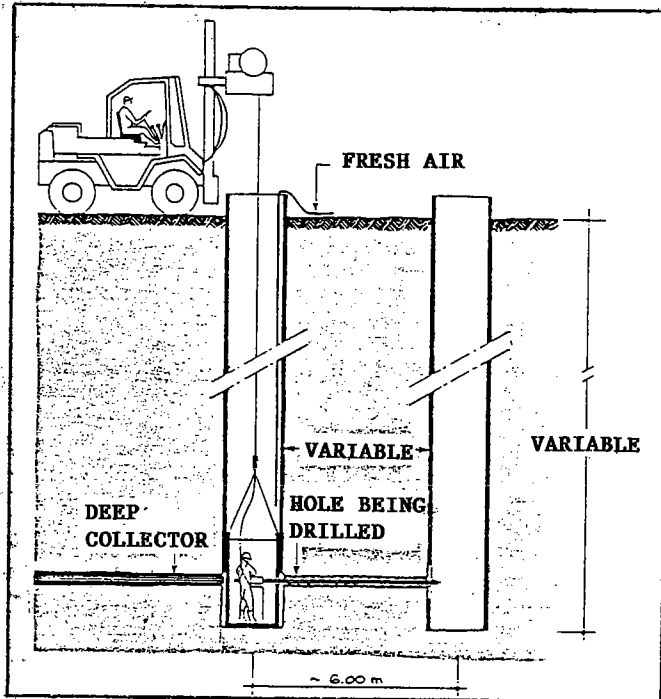


Figure 6 Drilling horizontal collector pipe between adjacent deep drainage wells (Bianco, 1986)

Wells are typically placed at distances of around 15-20 feet, but applications with up to 60 feet spacing have been recorded. Spacing is dictated primarily by the hydrogeological requirements (Reference 16) but practical considerations such as the site surface access restraints (especially in urban areas) and the accuracy with which the horizontal collector can be drilled are also limiting factors.

There are three types of wells: (1) normal full section drainage, (2) inspectable/controllable, and (3) reinforced.

1. **Normal Full Section Drainage (Figure 7):** A shaft about 4 feet diameter is drilled, and a temporary steel liner placed. A small purpose built drilling rig is then lowered to the base and a 4-1/2" hole drilled horizontally to penetrate the adjacent well, at a level about 4 feet above the base. A 3" pvc pipe is then placed in the hole and the drill removed. A 3'-4' thick concrete base plug is then cast. After setting, the well is backfilled with filter material as the temporary liner is extracted. The filling is stopped 4'-5' below the surface and covered by geotextile. The rest of the hole is then filled with soil or concrete, depending on the environment of the well.

2. **Inspectable/Controllable (Figure 8):** A slightly larger shaft (say 5 feet diameter) is formed and a 4 feet diameter permanent liner (of corrugated galvanized steel) is placed to full depth. The outer annulus is filled with filter aggregate. The horizontal drilling and placing of the pvc tube is then executed as before, except that the tube has a valve system attached inside the well. A concrete base plug, top hatch arrangement and ladder are placed following removal of the drilling equipment.

Such wells are located between groups of 3-10 normal drainage wells, at changes of direction or intersections of "runs," and at final discharge locations. They permit the total system through-flow to be monitored, and controlled if necessary, either manually (by operating the valves) or automatically (by a preset float arrangement).

3. **Reinforced (Figure 9):** Where structural strength is also required, then wells of this type can be integrated into the system, or substituted for the standard inspectable well. A 7 feet diameter hole is formed to the total depth required for structural stability, a 6 feet diameter galvanized lining tube placed, with filter material placed. The standard 4 feet diameter liner is then homed to the required depth for drainage, carrying an outer reinforcement cage. The resulting inner annulus of 1-foot width is then concreted, as is the basal plug. The horizontal collector drain is then drilled, as before. In addition, further "fans" of normal, unconnected, drainage holes can also be drilled from the shaft at any elevation or direction dictated, for example, by the need to penetrate under certain buildings or into particular strata.

In particular circumstances, the concrete upper section of well can be thickened to accommodate the

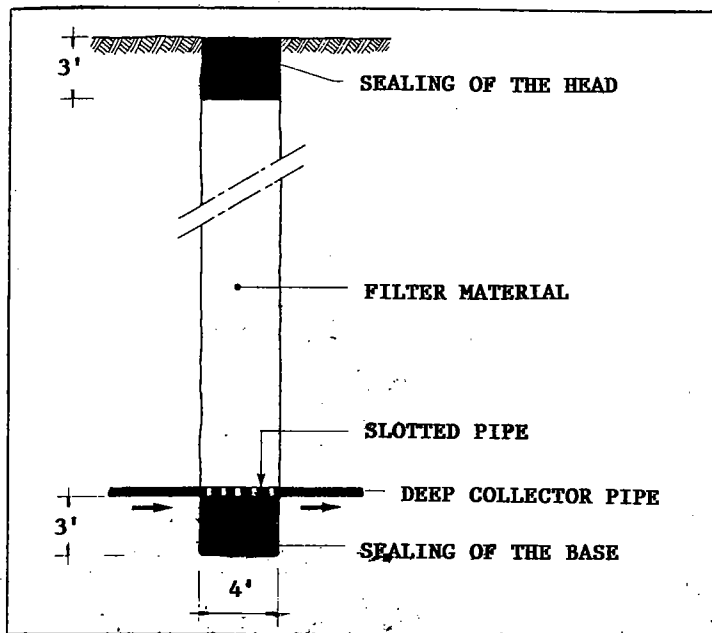


Figure 7 Normal full section drainage well (Bianco, 1986)

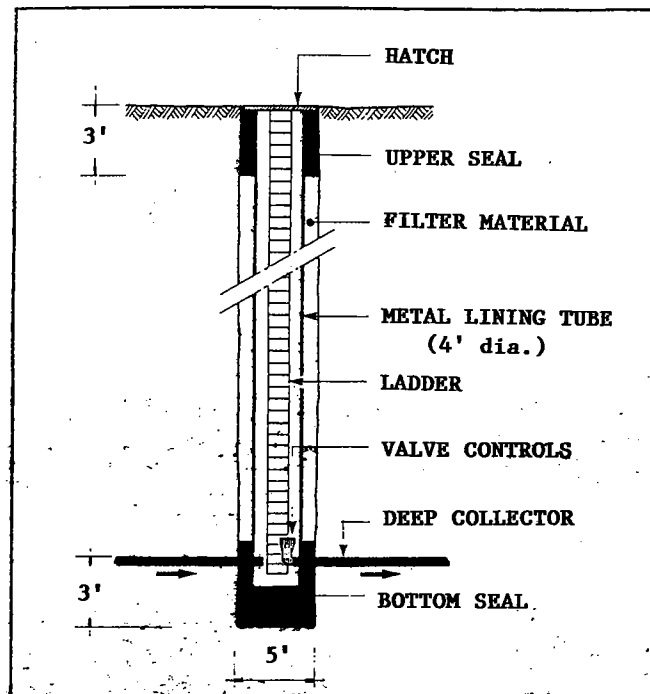


Figure 8 Inspectable/controllable well (Bianco, 1986)

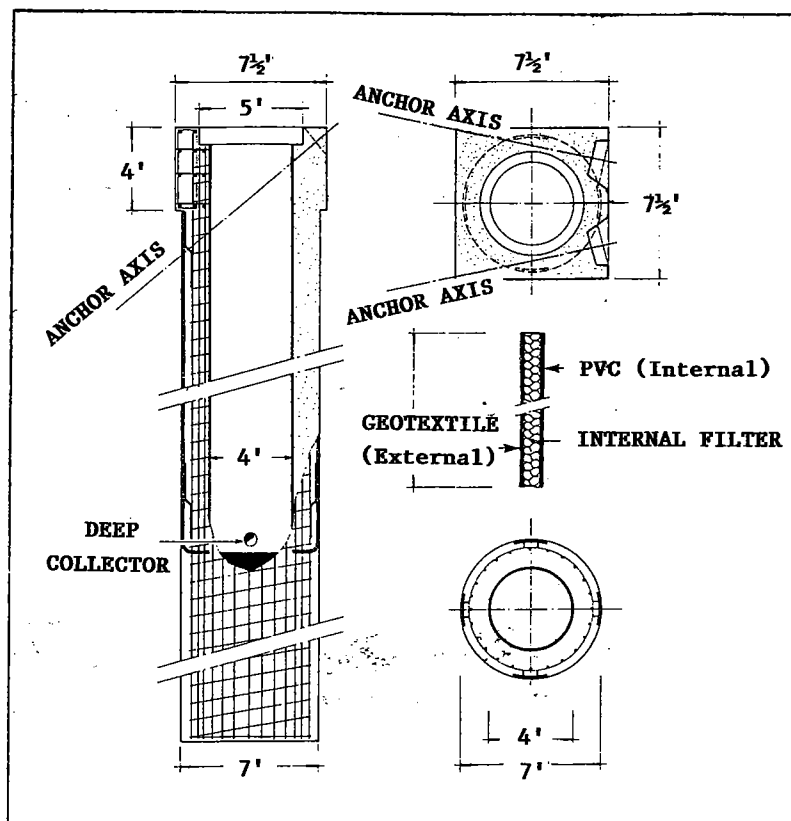


Figure 9 Reinforced well, with anchorages (Bianco, 1986)

stresses imposed by prestressed ground anchors installed to resist overturning forces. The edge distance between such structural wells is typically 0.5-3.0 times diameter, to ensure an efficient "arching" effect in the intermediate ground, as well as reflecting hydrological factors.

Design criteria - based on the classic formulae - and excellent data on six recent large and successful case histories in Italy are provided in References 16 and 17. Most of the examples were executed in the middle of urban developments with minimal disturbance to the environment.

All these case histories confirm the following major advantages of the method:

(1) Its facility for continual monitoring allows it to be limited or expanded during construction or in service in direct response to performance.

(2) It is ideal for even densely populated areas, providing minimal disruption to services, adjacent structures, and the environment. It can, therefore, be used to safeguard all kinds of structures including bridges, roads, and buildings.

(3) After installation, operational and maintenance costs are minimal: the system requires no external power source.

(4) The method of drainage is natural and self-regulating, responding by gravity feed to daily and seasonal changes in the groundwater characteristics. As there is no "forced pumping," there is no danger of instabilities due to rapid drawdowns. Manual regulation of a system's inspectable wells can further ensure optimal groundwater/stability conditions.

(5) The inspectable and structural wells permit the drilling of further families of drains, allowing the controlled dewatering of particular areas and horizons, as desired.

(6) No fines are extracted from the ground, and so no surface settlements, or clogging of the drainage system is possible.

(7) Observation and monitoring of the concrete of the reinforced wells will indicate significant ground strains.

(8) The installation does not require cuts to be made in potentially unstable slopes and will, therefore, not compromise the stability of very delicate areas during construction.

(9) It is extremely cost effective, taking over from drainage trenches (where applicable) at depths of around 30 feet.

4. GROUND TREATMENT

It is common (e.g., Reference 18) to identify four basic methods of ground treatment by grouting: hydrofracture, compaction, permeation, and replacement (Figure 10). For various reasons, the first three have little relevance or application within the framework of this conference. Furthermore, it is likely that the most contact highway engineers have had with grouting is in association with bulk infill (e.g., in areas of old coal mines), slabjacking (to inject under loosened concrete road slabs) or injection of post-tensioning ducts or anchorage holes (in concrete structures).

However, the fourth category of ground treatment, as listed above, replacement grouting -

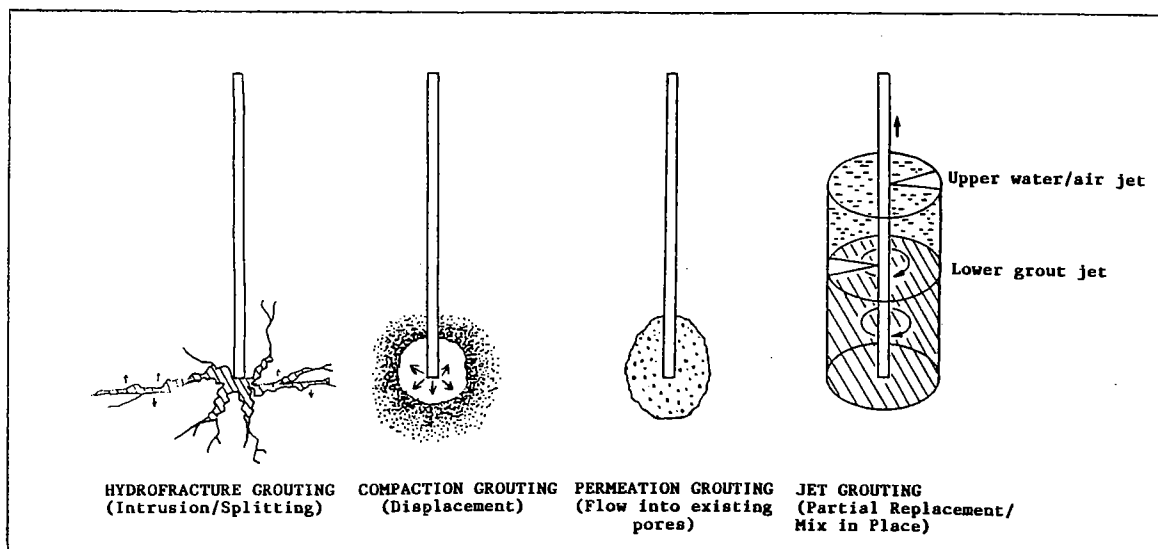


Figure 10 Basic types of ground treatment by grouting

usually referred to as jet grouting - is a tool of extreme potential and power, already proven in Europe and the Far East for over ten years.

Its development was fostered by the need to treat reliably and thoroughly soils from gravels to clays in areas where major environmental controls were strongly exercised over the use of chemical (permeation) grouts and allowable ground movements. In one particular form (Figure 11), the ground around the drill string is fragmented by a very high pressure (up to 8,000 psi) horizontally directed water and air jet, and largely expelled from the hole. The cavitated zone so formed is simultaneously filled from below with a cement-based grout, which does, of course, incorporate some of the native ground. A simpler variant eliminates the air/water cavitation and instead uses only the high pressure grout jet to cavitate and eject, as well as inject.

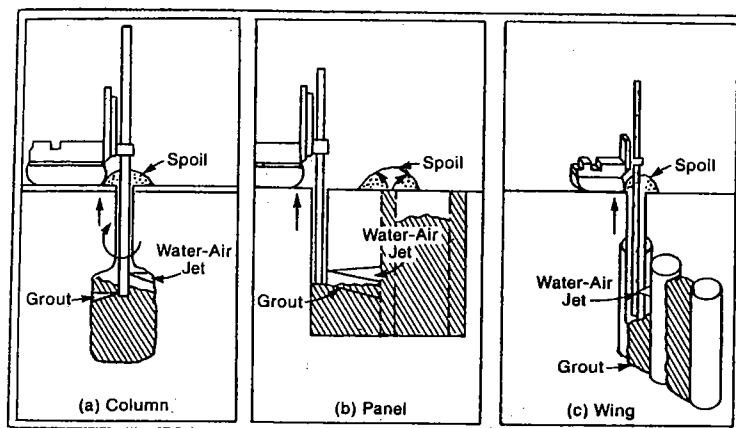


Figure 11 Jet grout construction (Bruce, 1987)

Major advantages of the technique include:

(1) Its applicability across a wide range of soil types (Figure 12), whereas other types of grouting (e.g., by permeation) may be very expensive or simply not practical.

(2) The relatively high strength and durability of the treated "soilcrete" mass: unconfined crushing strengths may range from 700 to 2,000 psi in sandy/gravelly soils, and 250 to 500 psi in cohesive deposits.

(3) Treatment is usually accomplished only by using cement grouts. Other materials (e.g., flyash) can, however, be readily incorporated in response to the target ground parameters after treatment.

(4) The system can be easily applied in areas of very restrictive access conditions or other tight environmental restraints.

Major case histories and test programs have been reported from around the world in the last five years, particularly (References 18, 19 and 20), and today jet grouting is the favored technique of specialist contractors in Western Europe and Japan. There are also indications that it is now about to seriously threaten, in the States, the dominance that compaction grouting for settlement control has previously enjoyed.

The following are the main applications of jet grouting, those of particular relevance to highway construction being illustrated in Figures 13 to 25.

1. Underpinning and Protection of Existing Structures

- (1) Enhancement of piled foundations (Figure 13)
- (2) Support of raft foundations (Figure 14)
- (3) Support of strip footings (Figure 15)
- (4) Support of isolated footings (Figure 16)

2. Waterproof Diaphragms

- (1) Under dams
- (2) Around shafts and other deep excavations (NB In these applications the bases can also be grouted to resist inflow, and uplift forces) (Figure 17)

3. Tunnels

- (1) Ground consolidation by vertical injection of full face from ground surface.
- (2) Ground support of portals and tunnels by sub/supra horizontal "umbrella" around excavation (Figure 18)

4. Support in New Construction

- (1) As load bearing piles for concrete structures (Figures 19 to 21)
- (2) As support for embankments on soft ground (Figure 22)

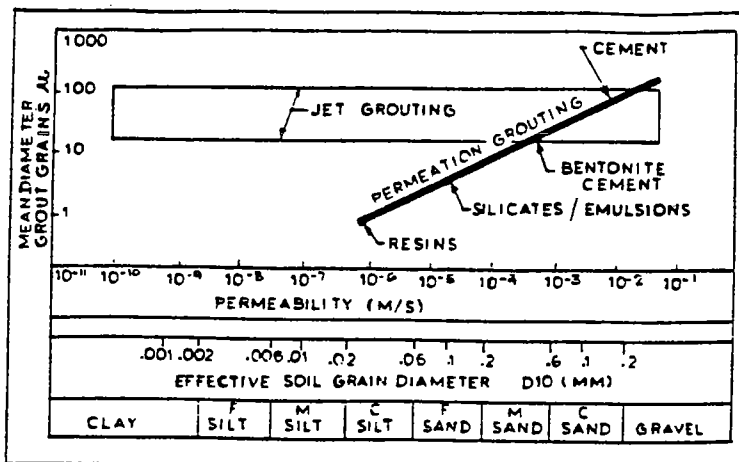


Figure 12 Groutability of soils in relation to grout and soil properties (Bruce, 1987)

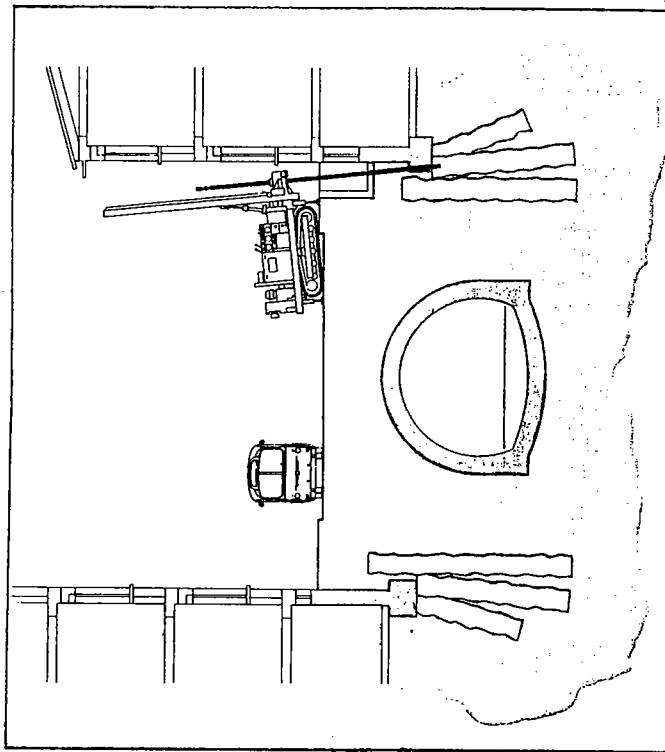


Figure 15 Support of strip foundation (Courtesy Pacchiosi Ltd.)

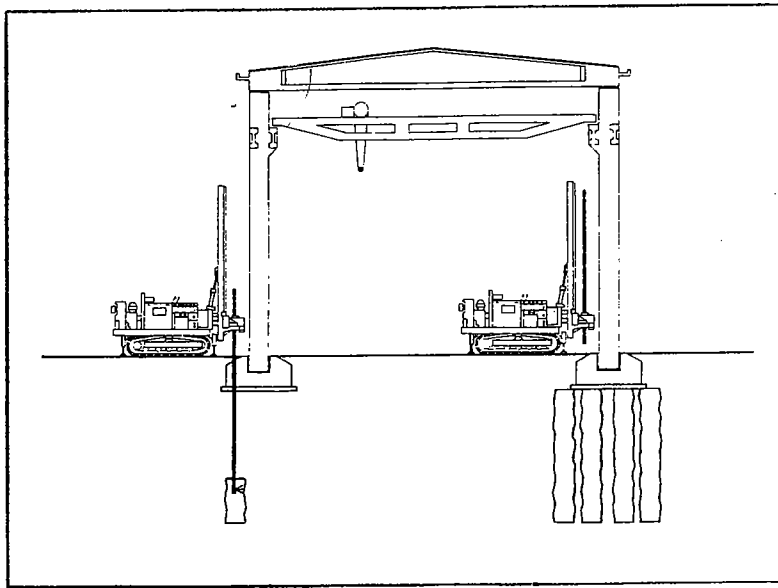


Figure 16 Support of isolated footings (Courtesy Pacchiosi Ltd.)

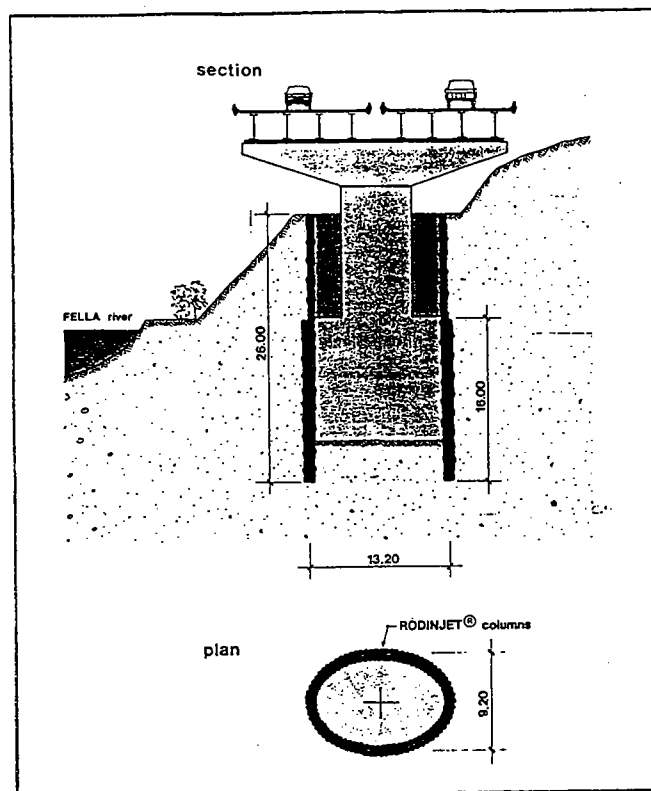


Figure 17 Shaft construction for excavation of pier foundations, Chiusaforte, Italy (Courtesy, Rodio and Company)

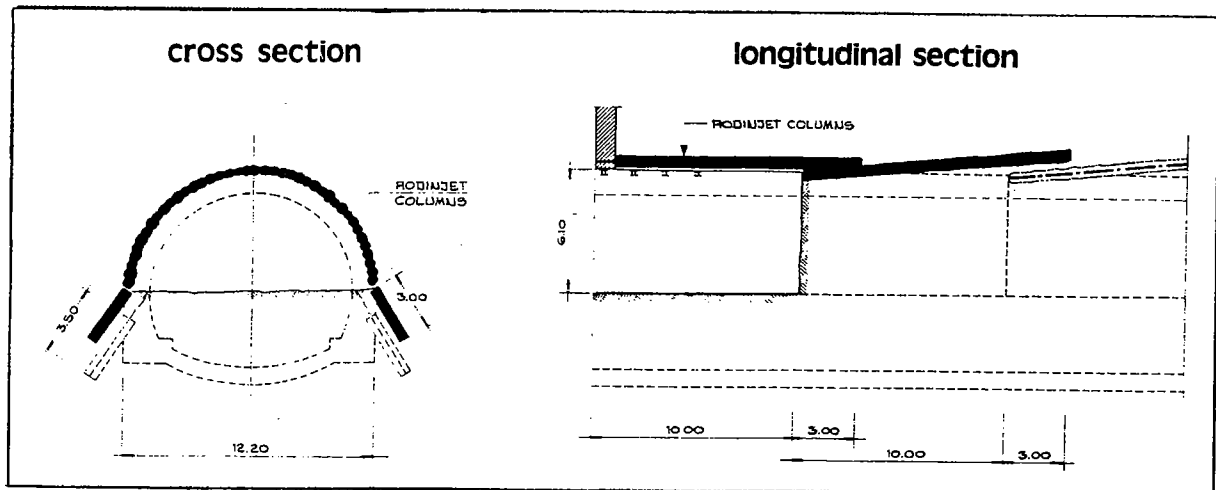


Figure 18 Ground support for tunnel excavation, Moggio Udinese Railway, Italy (Tornaghi and Cippo, 1986)

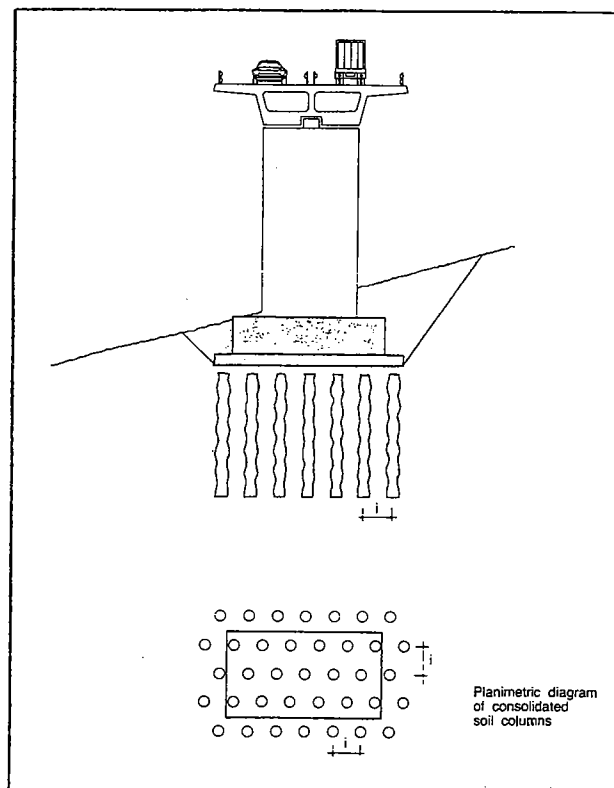


Figure 19 Columns as piles for piers (Courtesy Pacchiosi Ltd.)

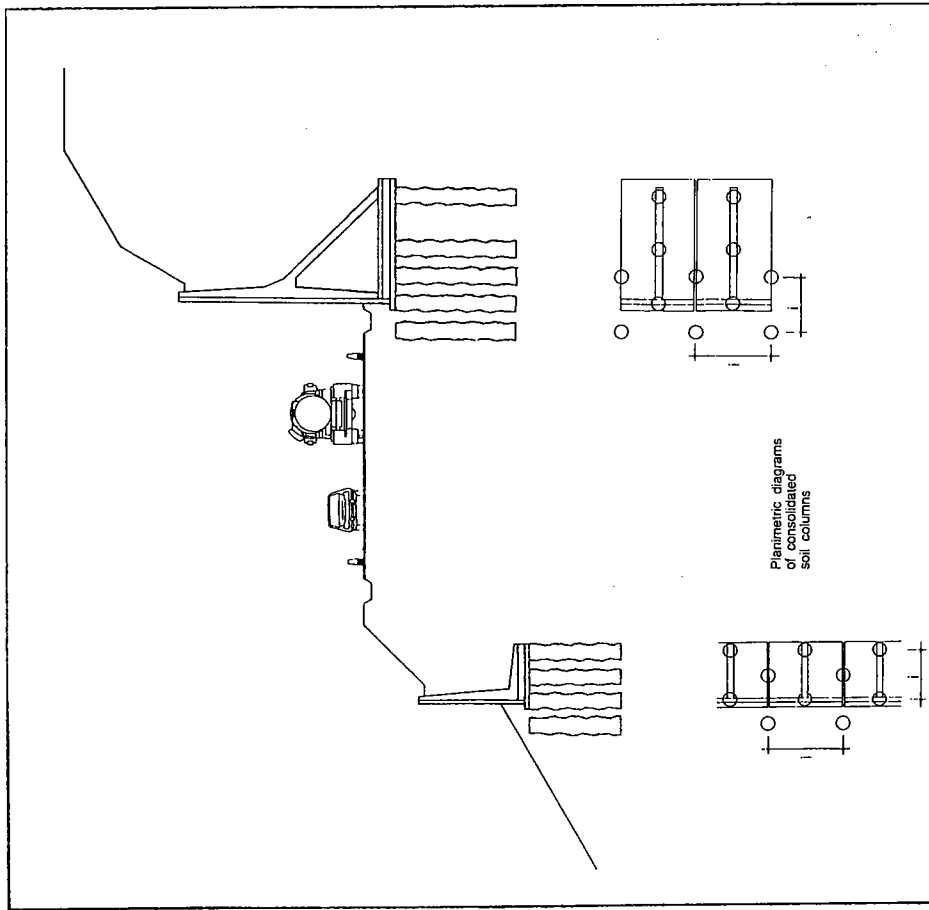


Figure 20 Columns as piles for retaining walls (Courtesy Pacchiosi Ltd.)

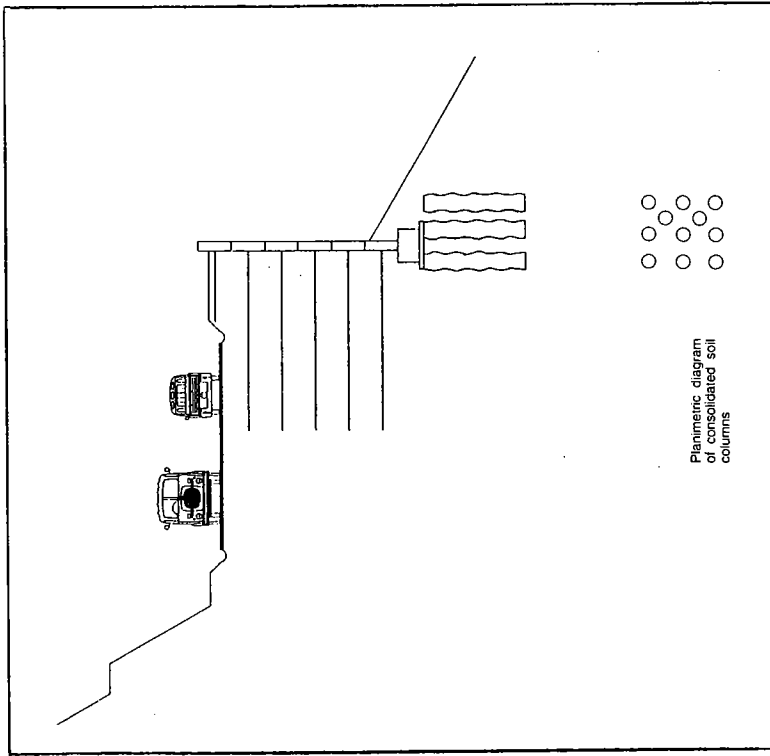


Figure 21 Columns as piles for reinforced earth wall (Courtesy Pacchiosi Ltd.)

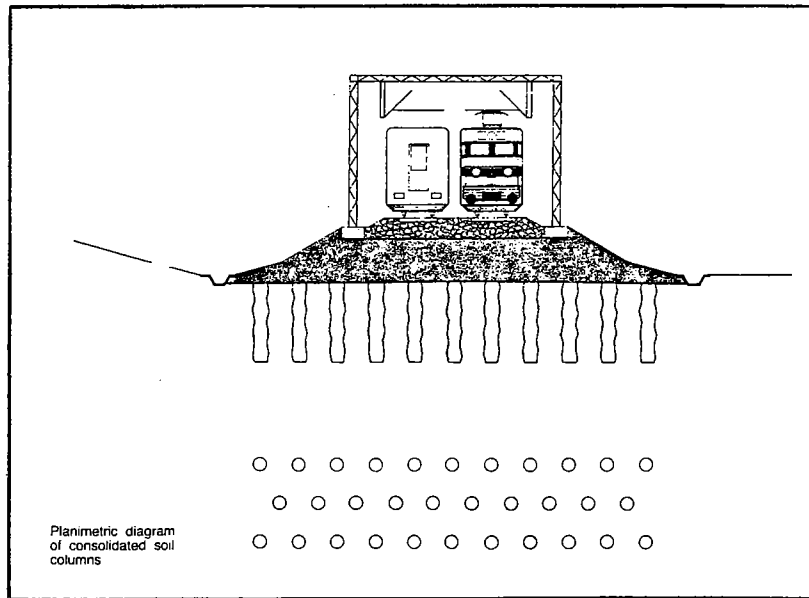


Figure 22 Columns as support for embankments (Courtesy Pacchiosi Ltd.)

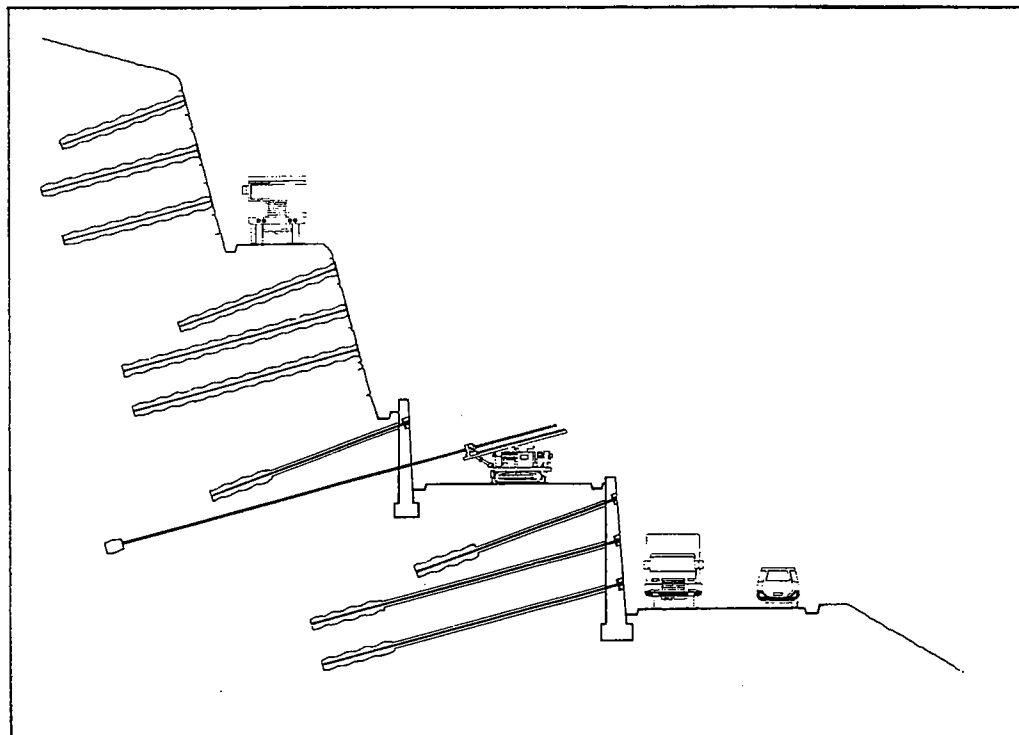


Figure 23 Slope stability, together with anchorages (Courtesy Pacchiosi Ltd.)

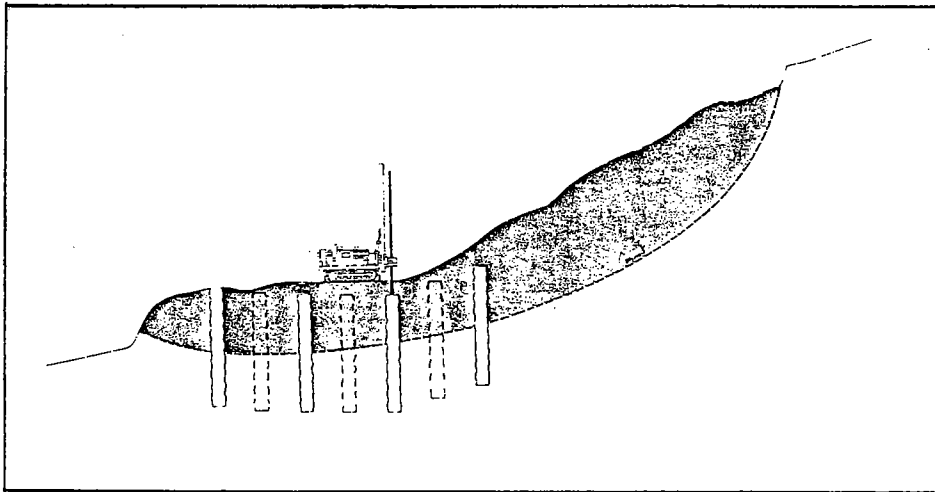


Figure 24 Shear keys in shallow slopes (Courtesy Pacchiosi Ltd.)

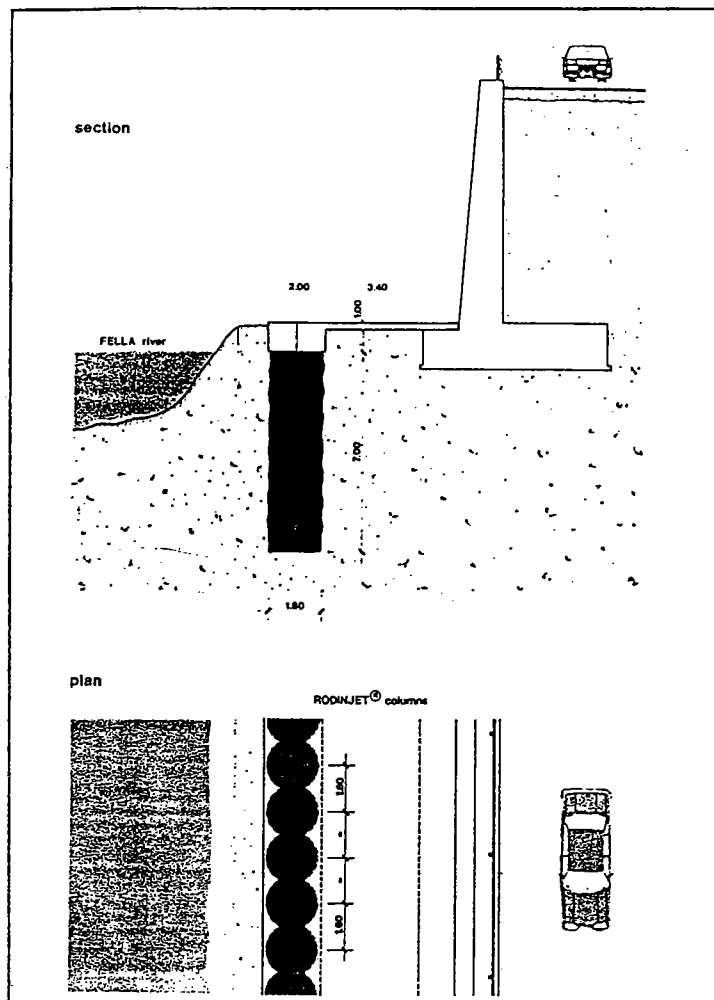


Figure 25 Protection of structure against river scour, Udine-Carnia-Tarvisio Motorway, Italy (Courtesy Rodio and Company)

5. Slope Stability

- (1) With anchorages or soil nails for steep cuts (Figure 23)
- (2) As shear keys for shallow slopes (Figure 24)
- (3) As protection against scour (Figure 25)

5. CONCLUSIONS

The techniques of insitu earth reinforcement have been demonstrated successfully in many parts of the United States, and one of these techniques - soil nailing - is one of the fastest growing geotechnical processes in the country. Although the benefits of groundwater lowering to enhance slope stability have long been practiced, the novel Italian methods of trenches and wells, interconnected at depth, afford the designer a new dimension. These techniques are of special relevance in large-scale slope stability problems which impact directly on urban developments. Ground treatment has not been a commonly applied tool of the highway engineer to date. However, the recent developments in jet grouting technology in Western Europe and the Far East could well revolutionize the way certain problems in underpinning, slope stability and deep foundations are solved in this country.

ACKNOWLEDGEMENTS

The authors have pleasure in acknowledging the assistance of their colleagues at Nicholson Construction Company and in Ing. Giovanni Rodio, Milano, Italy. They also thank the organizing committee for the opportunity of presenting the paper.

REFERENCES

1. Weatherby, D. E. and Nicholson, P. J. (1982). Tiebacks Used for Landslide Stabilization. ASCE Seminar for Construction Excavations, April, 17 pp.
2. Wycliffe-Jones, P. T. and Nicholson, P. J. (1981). New Methods for Anchoring Marine Bulkheads. Deep Foundations Institute Annual Conference, St. Louis, MO, 61 pp.
3. Bertolina, V.E. and Mannella, F. A. (1986). Design and Construction of a Tie Back Bridge Abutment for the Clairton-Glassport Bridge. Third Annual International Bridge Conference, Pittsburgh, PA. June 2-4, 23 pp.
4. Bruce, D. A. and Jewell, R. A. (1986-1987). Soil Nailing: Application and Practice. Ground Engineering, Vol. 19(8), pp. 10-15, Vol. 20(1) pp. 21-38.
5. Nicholson, P. J. and Boley, D. L. (1986). Soil Nailing Supports Excavation. Civil Engineering, April 1986, pp. 45-47.
6. Gudehus, G. (1983). Design Concept for Pile Dowels in Clay Slopes. "Discussion, Spec. Session 5, Proceedings, 8th European Conference on Soil Mechanics and Foundation Engineering, Helsinki, Vol. 3, 1182.
7. Jewell, R. A. (1980). Some Effects of Reinforcement on the Mechanical Behavior of Soils. Ph.D. Thesis, University of Cambridge.
8. Baker, R. F. and Yoder, E. J. (1958). Stability Analysis and Design of Control Works in Landslides and Engineering Practice. E. B. Eckel, ed., HRB Special Report 29, pp. 189-216.
9. Verrier, G. and Merlette, P. (1981). Confortement des Remblais Ferroviaires - Une Technique Particuliere: Le Clouage. Special Revue Travaux, March, pp. 76-81.
10. Gudehus, G. and Schwarz, W. (1984). Stabilisierung von Kriechhangen durch Pfahldubel. Vortrage der Baugrundtagung Dusseldorf, pp. 669-681.
11. Nicholson, P. J. (1986). Insitu Earth Reinforcement at Cumberland Gap, US 25E. ASCE/PennDOT Conference, Harrisburg, April 17, 1986, 21 pp.
12. Shen, C. K., Herrmann, L. R., Romstad, K. M., Bang, S., Kin, Y. S. and De Natale, J. S. (1981). An In Situ Earth Reinforcement Lateral Support System. Dept. of Civil Engineering, University of California, Davis. Report No. 81-03, March, 130 pp. NSF Report No. NSF/CEE-81059.
13. Boley, D. L. and Crayne, L. M. (1985). INSERT Wall Saves Roadway. Highway Building, Fall, pp. 14-16.
14. Stanic, B. (1984). Influence of Drainage Trenches on Slope Stability. Jour. Geot. Eng., ASCE, 110 (11), November, pp. 1624-1636.
15. Nonveiller, E. (1981). Efficiency of Horizontal Drains on Slope Stability 10th ICSMFE, Stockholm, 3, pp. 495-500.
16. Bianco, B. and Belligni, D. (1983). Il Drenaggio Profondo per il Consolidamento di Aree Urbane in Frana. Estratto Dalli Rivista Tecnica Dell'Ance, "L'Industria Delle Construzioni," Dec. pp. 47-57.
17. Bianco, B. (1986). Il Consolidamento della Frana di Fontivegge in Perugia: Problemi Technico-esecutivi. XVI Convegno Nazionale di Geotecnica, Bologna, 14-16 May, 10 pp.
18. Mongilardi, E. and Tornaghi, R. (1986). Construction of Large Underground Openings and Use of Grouts. Proc. Int. Conf. on Deep Found., Beijing, September, 19 pp.
19. Tornaghi, R. and Cippo, A. P. (1985). Soil Improvement by Jet Grouting for the Solution of Tunnelling Problems. Proc. 4th Int. Symp. Min. and Met., "Tunnelling 1985," Brighton, March 10-15, pp. 265-276.
20. Bruce, D. A. (1987). Tunnel Grouting--An Illustrated Review of Recent Developments in Ground Treatment. Proc. Int. Conf. "Foundations & Tunnels - 87," London, March 24-26, pp. 156-172.

Geological and Geomorphological Investigations for Cultural Resource Evaluations

JAMES V. HAMEL
Hamel Geotechnical Consultants
Monroeville, Pennsylvania, and
GTech, Inc.
Pittsburgh, Pennsylvania

HGS-87-27

INTRODUCTION

STATE AND FEDERAL AGENCIES and private owners are required by law to consider the effects of construction on all historic and prehistoric sites, structures, and objects eligible for inclusion in the National Register of Historic Places. Federal legal mandates include Section 106 of the National Historic Preservation Act of 1966 (Public Law 89-655), Executive Order 11593, and regulations of the Advisory Council on Historic Preservation (Pennsylvania Historical and Museum Commission, 1986).

Under the National Historic Preservation Act, each state operates a State Historical Preservation Office (SHPO) which oversees development in areas of historical or archeological significance. In Pennsylvania, for example, this is done by the Bureau for Historic Preservation of the Pennsylvania Historical and Museum Commission in Harrisburg.

The National Historic Preservation Act has been in effect for 20 years but it has been strictly enforced for only the past decade (Chapple, 1987). Guidelines and procedures have been developed for historical and archeological investigations in connection with construction projects (Pennsylvania Historical and Museum Commission, 1986; Chapple, 1987).

Considerable experience has been gained in conducting these investigations but cases still occur where adversarial positions develop and where projects are delayed or terminated, often at considerable cost.

Historical and archeological studies for construction projects are often termed "cultural resource evaluations." Many of these studies have significant geological and geotechnical components. The portions of cultural resource evaluations dealing with geology and soils are sometimes called "geomorphological surveys" though they include much more than classical geomorphology, i.e., the branch of geology dealing with landforms (Thornbury, 1954).

The purpose of this paper is to present an overview of cultural resource evaluations for engineering geologists and geotechnical engineers with emphasis on geologic aspects of these evaluations.

OVERVIEW OF CULTURAL RESOURCE EVALUATIONS

GENERAL - The draft guidelines for project review given by the Pennsylvania Historical and Museum Commission (1986) are representative of those used by SHPO. The SHPO must first be notified of the project type, size, and location in order to determine whether it occurs on or near a previously recorded, potentially significant, historic or prehistoric archeological site or in an area with potential for significant archeological sites. Areas where previous activities have destroyed all potential archeological sites or where sufficient archeological surveys have been conducted may require no further investigation. Otherwise, the type and extent of cultural resource evaluation depends on the known or potential archeological sites in the project area and data available on these sites.

PHASES - Cultural resource evaluations, if required, may include one or more of three phases:

- Phase I - Minimum level of cultural resource evaluation where there is insufficient background data but potential for a significant archeological site; involves literature review, interviews, reconnaissance, and limited excavation (subsurface testing) to locate all surface and subsurface (stratified) archeological sites in the project area.
- Phase II - Undertaken if Phase I indicates potential for significant archeological sites; involves more detailed field testing (excavation)

to determine significance and extent of archeological sites in the project area.

Phase III - Undertaken if Phase II indicates archeological sites meeting criteria for inclusion in National Register of Historic Places; involves development of a plan to mitigate all adverse effects of construction on significant archeological sites in the project area (typically means a full-scale archeological investigation).

Phase I evaluations are performed on many projects. Of these projects, it has been estimated that only some 15% to 20% require Phase II investigation and perhaps 1% require Phase III mitigation (Chapple, 1987).

PERSONNEL - Some organizations and agencies have geologic and engineering personnel involved in cultural resource evaluations. For the most part, however, these evaluations appear to be directed, performed, and, at the SHPO, reviewed by archeologists, anthropologists, agronomists, historians, environmental scientists, or others. These personnel usually have good intentions and zealous attitudes toward historical and archeological preservation but limited training and experience in geology, engineering, and construction.

It is to be expected that archeologists and other non-geologic, non-engineering personnel will continue to have major roles in cultural resource evaluations in the future and that these personnel will gain experience in geologic and engineering matters along the way. In the meantime, it is important to communicate with these personnel in the course of cultural resource evaluations, to demonstrate to them the value of geologic and engineering methodologies and techniques in their work, and to make them aware of the limitations imposed on this work by geological and geotechnical conditions.

TYPICAL PRACTICE - As noted above, the portions of cultural resource evaluations dealing with geology and soils are often conducted by archeologists and other non-geological, non-engineering personnel. A typical "geomorphological survey" for Phase I of a cultural resource evaluation might include review of a U.S. Geological Survey Topographic Quadrangle map, a U.S. Department of Agriculture soil survey report, and some very basic geologic literature on the project area. Little attention appears to be paid to

site-specific aerial photography, topographic mapping, or geologic and engineering data which may have been developed in the course of project investigation and design.

Field reconnaissance during Phase I places emphasis on historical and archeological features including the search for artifacts. This reconnaissance may yield some information on surficial geology and soils.

Phase I often includes excavations (field testing) in areas of potential archeological significance. These excavations are typically shallow by engineering standards, though guidelines for their depths are meager. In some locations, the ground surface is plowed and harrowed with farm implements, then hoed or sifted in quest of artifacts. Shovel or auger probes 50 cm square or round to approximately 50 cm depth on grid spacings of 5 to 30 m are common. (Archeologists use metric or SI units.) Materials from the shovel or auger probes are sifted through 6 mm ($\frac{1}{4}$ in.) screens. Phase I sometimes includes 1 m square holes in locations of particular potential or test pits or trenches excavated with backhoes. Logging of these Phase I excavations is often inadequate by geological and geotechnical standards.

These Phase I "geomorphological surveys" may suffice for certain projects, i.e., those with minimal subsurface considerations relative to cultural resources, but such surveys are not generally adequate for projects with significant subsurface concerns. The latter include projects where soil ages, fluvial erosion and deposition, previous site grading, or locations and depths of cultural resource excavations (for buried or stratified archeological sites) are critical.

Phase II may include additional plowing and harrowing and shovel or auger probes. Phase II typically involves "deep testing," i.e., excavation and scrutiny of 1 m or 2 m square holes at locations identified during Phase I. Guidelines for the depths of Phase II excavations are virtually non-existent. These excavations are generally extended down to "culturally sterile soil," i.e., soil without artifacts. In some cases, bedrock or groundwater controls excavation depths. Better geological and geotechnical information from Phase I will obviously improve Phase II excavation efforts.

SHORTCOMINGS - Typical practice with regard to geological and geotechnical aspects of cultural resource evaluations has a number of deficiencies, including:

- * General lack of scientific rigor and technical standards.
- * Limited perception of geologic processes, engineering practices, and construction considerations including grading and ground disturbance effects and their implications.
- * Heavy reliance on U.S. Department of Agriculture soil surveys conducted for agricultural purposes and extending to depths of only 1.5 to 2 m (5 to 6 ft).
- * Lack of correlation of soil strata exposed in cultural resource excavations with those characterized in previous geological and geotechnical investigations and those observable in surface exposures.
- * Lack of survey control for cultural resource excavations relative to project mapping (stations, coordinates, and particularly elevations).
- * Little emphasis on stratigraphic context in geologic and soils work despite the heavy emphasis placed on stratigraphic context in archeological work.
- * Lack of appreciation for the fact that features of geological and geotechnical significance typically extend beyond the limits of an archeological site.
- * Overly optimistic and, in some cases, erroneous identifications of dark colored materials in the ground as "charcoals" or "paleosols."
- * Tendency to place excessive weight on discovery of only one (or a very few) "artifacts" at a site, particularly in view of the potential for both natural and man-made transport processes.

SUGGESTIONS FOR IMPROVEMENT - There appears in general to be considerable room for improvement in geological and geomorphological portions of cultural resource evaluations. The fundamental questions here are essentially the same as those in other areas of geological or geotechnical investigation:

* Where to explore or excavate

* How deep to explore or excavate

Geological and geotechnical guidance can help here just as it does in investigations for slopes, foundations, and borrow areas. In some cases, this guidance can even assist in locating archeological sites (Gardner and Donahue, 1985).

A broad geotechnical approach is suggested in connection with cultural resource evaluations. As in other areas of engineering geology and geotechnical engineering, emphasis should be placed on developing the geotechnical framework of the project area, i.e., those elements of geology and geometry critical to the project (Hamel, 1983). When the geotechnical framework of an area is understood and appreciated, subsurface aspects, including those related to cultural resource evaluations, are clarified and simplified. In addition, consideration of the cultural resource evaluation relative to the geotechnical framework of the project should foster the use of geological and geotechnical data from other project phases in the cultural resource evaluation.

Many public works projects, including transportation projects, evolve over a period of years with considerable geological and geotechnical information developed during the planning and design stages. Much of this information should be available by the time cultural resource programs are developed. Preliminary assessment of this information by experienced engineering geologists and geotechnical engineers can assist greatly in determining the need for additional geological and geotechnical investigations for cultural resource purposes and in planning these additional investigations where they are required. Properly planned and executed geological and geotechnical investigations for cultural resource investigations will also contribute information useful in geotechnical design and construction.

INVESTIGATION TECHNIQUES

GENERAL - Most of the techniques applicable to geological and geomorphological investigations for cultural resource evaluations are similar to those employed in other areas of engineering geology and geotechnical engineering. These include review of background information, field reconnaissance, subsurface exploration, and analysis of soils and landforms. In addition, age dating of soils is required in some cultural resource evaluations.

BACKGROUND INFORMATION - Background materials to be reviewed in connection with cultural resource evaluations are virtually the same as those for other geotechnical endeavors. These materials include topographic maps of both small (U.S. Geological Survey) and large (project mapping) scales, geologic and mining maps and reports, aerial and terrestrial photographs, and hydrologic and flood data (mainly for flood plain sites). U.S. Department of Agriculture soil survey maps and reports can be reviewed but they typically have limited value because of their agricultural emphasis and shallow depth of soil exploration. If previous construction or mining activities have occurred in the project area, reports, maps, drawings and other information on these activities should be obtained and reviewed. Subsurface information available from such projects and, where available, from earlier phases of the subject project, should be reviewed. As noted above, considerable subsurface information often exists on a project by the time it reaches the stage of a cultural resource evaluation.

Reports from previous regional and site-specific cultural resource investigations in the project area should of course be reviewed. These reports may not clarify geological and geotechnical conditions but they will give insight into cultural resource investigation procedures and aid in communication with cultural resource personnel.

RECONNAISSANCE - Reconnaissance for cultural resource purposes is virtually the same as that for other geotechnical endeavors and is best done in Spring or Autumn when neither vegetation nor snow obscures features. Items to be examined include topography, landforms, soil and rock exposures, surface water bodies (e.g., lakes, ponds, swamps, streams), and groundwater expressions (e.g., wet areas, seeps, springs). Man-made features are particularly important in reconnaissance for cultural resource evaluations. These features may be related to structures, grading (excavation and fill placement), and previous or existing construction, mining and mineral extraction, or industrial activities.

During reconnaissance, efforts should be made to discuss area and site histories with local people including amateur historians and archeologists, collectors of artifacts, and children. Children playing and exploring in an area tend to locate sites yielding artifacts and this information is passed down through successive generations as part of childhood lore.

SUBSURFACE EXPLORATION - Test pits and trenches excavated with backhoes are probably the most useful techniques of subsurface exploration in connection with geological and geotechnical portions of cultural resource evaluations. These pits and trenches are fast and economical. They yield a great deal of useful information including soil samples for analysis and age dating, and, in some cases, depths and elevations of bedrock and the water table. Stratigraphy and other pertinent features of soils exposed in test pits should always be logged in geotechnical terms to facilitate comparison with other subsurface information on the project area. Pedological logging of soils is useful in many cases, especially if pedological techniques are to be used in determinations of soil ages. In all cases, soil faces and pertinent soil details exposed in test pits should be photographed using 35 mm color print film.

Appropriate safety precautions must be taken and pertinent safety regulations must be followed in test pit work. This is particularly important where personnel enter test pits. Test pits should be backfilled upon completion.

ANALYSIS AND PRESENTATION OF RESULTS - Data obtained in geological and geomorphological investigations for cultural resource evaluations are analyzed in terms of established principles of engineering geology and geotechnical engineering. These analyses often involve concepts of stratigraphy, weathering profiles, geomorphology, and glacial and fluvial processes as well as geologic and engineering common sense.

Results of these analyses are presented in the traditional manner in reports with logs, maps, plan drawings, and cross-sections. Large scale topographic maps or plan drawings of the project area available from previous engineering work can be annotated with geological and geotechnical features required for interpretation of site conditions relative to cultural resource evaluations. Cross-sections should in general be drawn with equal horizontal and vertical scales to illustrate pertinent surface features, landforms, and soil and rock strata or zones.

Both plan and cross-section drawings for cultural resource evaluations are less detailed than those for geotechnical design and construction. The purpose of these drawings is to depict the geotechnical framework of the project area and particularly those elements of the geotechnical framework relevant to the cultural resource evaluation. These drawings are conceptual and interpretive in the sense of geologic drawings rather than detailed in the sense of engineering drawings. The key idea in these drawings, as in the related report, is to communicate essential features of site geology and geotechnical history to non-geologists and non-engineers. In this regard, the famous KISS principle of engineering ("Keep It Simple, Stupid") should be kept firmly in mind.

CULTURAL CHRONOLOGY AND SOIL AGE DATING

CULTURAL CHRONOLOGY - Considerations of cultural chronology require determination of absolute and relative ages of soils in certain cultural resource evaluations. Some broad aspects of cultural chronology must be presented before discussing techniques for assessing soil ages.

In North America, the last four centuries correspond to historic time. This time interval poses relatively few problems in cultural resource evaluations because of historic records and surficial remnants.

Prehistoric time in North America extends back from four centuries ago. There is considerable uncertainty regarding the chronology and distribution of people in North America during prehistoric time, particularly in earlier prehistoric time.

Most archeologists presently agree that prehistoric people migrated from Asia to North America via a land bridge across the Bering Strait during the latter part of the Pleistocene Epoch (sometime during the last 60,000 years) but archeologists disagree on exactly when this occurred (Fladmark, 1986). Abundant evidence indicates that humans inhabited both North and South America by about 12,000 years ago but evidence for earlier human habitation remains controversial (Fladmark, 1986). Some evidence suggests that humans occupied the Yukon more than 40,000 years ago (Irving, 1987). Other evidence suggests that humans

had reached southern Chile more than 13,000 years ago and perhaps as early as 33,000 years ago (Dillehay, 1987).

Until recently, it was generally thought that the earliest people arrived in what is now the northeastern United States about 12,000 years ago at the end of the Pleistocene Epoch. The classical tabulation of prehistory of the Upper Ohio Valley showed the Paleo-Indian period extending back about 10,000 years (Table 1, Mayer-Oakes, 1955). Recent work at the Meadowcroft Rockshelter 79 km (49 mi.) southwest of Pittsburgh, Pennsylvania, has shown that prehistoric people occupied the Upper Ohio Valley at least 16,000 years ago and perhaps as early as 19,000 years ago (Adovasio, *et al.*, 1983; Adovasio and Carlisle, 1986). The first five periods in Table 1 are still accepted but the Paleo-Indian period is now generally extended back to some 16,000 years ago.

These data indicate that Holocene and Late Pleistocene times have prime importance in cultural resource evaluations. Techniques for soil age determinations must be suitable for these times and particularly for the last 10,000 to 20,000 years.

SOIL AGE DATING - Techniques for soil age dating have been reviewed by Colman, *et al.* (1986). Of these techniques, those with the greatest application and potential in cultural resource evaluations are radiocarbon dating, rates of soil development (in the pedological, rather than geotechnical, sense), and stratigraphic analyses. Geomorphic analyses also have major application and potential in cultural resource evaluations.

Radiocarbon dating is well known and widely used in both archeology and geology. The maximum age that can be reliably measured is on the order of 40,000 years. A sample containing at least 5 gm of organic material is necessary for ordinary radiocarbon dating. Smaller samples can be dated using extended count and particle accelerator procedures. These techniques for smaller samples require longer times and are more expensive. The critical thing with all these radiocarbon dating techniques is obtaining a sufficient number of samples in appropriate stratigraphic contexts. Such samples are generally hard to find. Radiocarbon dates from a single sample or only a few samples are always suspect because of inherent limitations in radiocarbon dating and potential for sample contamination (Adovasio and Carlisle, 1986). Contamination potential is particularly great in areas with coal-bearing rocks and histories of coal mining and processing.

TABLE 1 - PREHISTORY OF THE UPPER OHIO VALLEY

(after Mayer - Oakes, 1955, p. 8)

<u>Period</u>	<u>Characteristics</u>	<u>Approximate Time</u>
Historic	European contact	A.D. 1600 - present
Late Prehistoric	Village-dwelling farmers	A.D. 900 - A.D. 1600
Middle Woodland	Hopewellian burial mounds	100 B.C. - A.D. 900
Early Woodland	Adena burial mounds	1000 B.C. - 100 B.C.
Archaic	Shell-heaps and camp sites	3000 B.C. - 1000 B.C.
Paleo-Indian	Nomadic hunters of animals now extinct	8000 B.C. - 3000 B.C.

Note: The Paleo-Indian period is now generally extended back to approximately 14,000 B.C. on the basis of Meadowcroft Rockshelter (Advasio, et al., 1983; Advasio and Carlisle, 1986).

Pedological analyses have also been used in both archeology and geology. Pedology deals with fundamental aspects of soil science and involves much more than the information given in U.S. Department of Agriculture soil survey reports. Pedological analyses include studies of stratigraphy, structure, weathering profiles, geochemistry, and clay accumulations in materials comprising the upper portions of what are generally considered soils in the engineering sense. These analyses can provide both relative and absolute estimates of soil age (Shlemon, 1985; Jacobson, 1985, 1987). Pedological techniques have potential for much wider application in cultural resource evaluations as well as certain other areas of engineering geology.

Stratigraphic and geomorphic techniques of soil age dating are well known to geologists. These techniques involve assessment of erosional, depositional, and weathering features to establish patterns, genesis, and relative ages of soils and landforms so that they can be placed within established increments of Quaternary time.

SOME PRACTICAL SHORTCUTS

SITE DISTURBANCE - Previous site disturbance is a key issue in cultural resource evaluations. If it can be shown through historical, geological, and geotechnical investigations that a site has been disturbed in recent times by natural geologic processes or by man through grading, construction, or mining activities, then there may be no need for additional cultural resource studies. The type and extent of site disturbance, if any, therefore merits thorough investigation.

BEDROCK AND WATER TABLE - In most cases, bedrock and the water table establish lower bound elevations (upper bound depths) on soil zones of cultural significance relative to prehistoric people. Bedrock and water table data should be available from project design investigations. These data will aid in planning cultural resource evaluations, particularly in areas of shallow bedrock and shallow groundwater.

CONTINUOUS SLACKWATER DEPOSITION - Continuous slackwater deposition corresponding to a single ponding episode in a fluvial or lacustrine environment precludes the possibility of a stratified archeological site within the body of a soil deposit. Demonstration of continuous slackwater deposition through sedimentological, stratigraphic, geomorphic, and geotechnical investigations can help to establish depths required for cultural resource excavations.

COMMENTS ON "DEEP TESTING"

DEPTHS OF EXCAVATIONS - In areas with potential for buried or stratified archeological sites, questions sometimes arise regarding depths of excavations required in "deep testing." Consideration of the geotechnical framework of the site, developed through geological and geotechnical investigations conducted for project design as well as cultural resource evaluations, should resolve these questions. The important considerations here are bedrock and groundwater elevations and the depositional history of the site prior to the earliest known entry of prehistoric people to the area.

SAFETY - Engineering and construction personnel are cognizant of safety requirements related to working in soil excavations. Archeological personnel are not always cognizant of these requirements or of the scheduling and cost implications of archeological work in excavations with depths greater than 1.5 m (5 ft). More communication in these regards would be desirable for all concerned.

INSPECTION - In order to avoid (or resolve) potential controversies regarding soil conditions and excavation depths in "deep testing," an inspection procedure analogous to that employed in foundation construction should be considered. This procedure would involve inspection of the sides and bottoms of major (or selected) cultural resource excavations by qualified soils personnel (engineering geologists or geotechnical engineers) from the staff of the project owner (or sponsoring agency) or from a consultant organization. The inspector would assess soil conditions exposed in the excavation relative to soil conditions determined from borings and test pits and determine whether archeologically sterile soil had been reached within the context of the geotechnical framework of the site.

An inspection procedure of this type is logical, straightforward, and consistent with other areas of geotechnical practice. Provisions for such inspection could be included in contracts (and contract monitoring) for cultural resource investigations. Only intermittent inspection would be required because of the slow pace of cultural resource excavations relative to foundation construction. The intermittent inspections could be done when major or critical cultural resource excavations reach intermediate or planned depths, questions arise concerning the need to excavate deeper, or disputes occur regarding the nature or extent of sterile soil (Hamel, 1987).

CONCLUSIONS

Geological and geomorphological investigations for cultural resource evaluations provide a new set of challenges for engineering geologists and geotechnical engineers. Most of these challenges relate to communication and coordination with cultural resource personnel, e.g., archeologists and anthropologists. Basic geological and geotechnical work required for cultural resource evaluations is the same as that required to develop the geotechnical framework of a site for purposes of design and construction. Considerations of geologic time (Late Pleistocene to Holocene) have much greater importance in cultural resource evaluations than in most ordinary

geotechnical work. For cultural resource evaluations, these time considerations must be woven into the geotechnical framework of the site.

Geological and geomorphological investigations for cultural resource evaluations are geotechnical in the broad sense and, like other geotechnical work, they are highly site-specific. Their type and extent should be determined early in the formulation of cultural resource requirements in order to optimize acquisition and use of geotechnical data, maximize efficiency and cost effectiveness, and minimize construction delays.

ACKNOWLEDGEMENTS

Many of the concepts presented here were derived from work on projects for the Pittsburgh District, Corps of Engineers. Discussions with F. Gensler, L. Moskovitz, C. Weiser, and J. Purdy of the Pittsburgh District Planning Division; R. Jacobson of the U.S. Geological Survey; and J. Donahue of the University of Pittsburgh stimulated thoughts on these concepts. The opinions presented herein are solely those of the writer, however, and not to be considered those of the above-mentioned individuals and organizations.

This paper was typed by Gerry Marting.

REFERENCES

1. Pennsylvania Historical and Museum Commission, Draft Guidelines for Archeological Survey and Mitigation, Harrisburg, PA (1986)
2. Chapple, A., "When The Past Gets in The Way of The Future: Archeology & Engineering," Engr. Times 9, 2, 10 (1987)
3. Thornbury, W. D., Principles of Geomorphology, p. 1, John Wiley & Sons, Inc., New York, NY (1954)
4. Gardner, G. D. and J. Donahue, "The Little Platte Drainage, Missouri: A Model for Locating Temporal Surfaces in a Fluvial Environment," Archeological Sediments in Context, J. K. Stein and W. R. Farrand, eds., University of Maine at Orono, 69-89 (1985)
5. Hamel, J. V., "Fundamentals of Geotechnical Engineering Practice," Geotechnical News, 1, 2, 11-12 (1983)

6. Fladmark, K. R., "Getting One's Berings," *Natural History*, 95, 11, 8-9 (1986)
7. Irving, W. N., "New Dates From Old Bones," *Natural History*, 96, 2, 8-13 (1987)
8. Dillehay, T. D., "By The Banks of The Chinchihuapi," *Natural History*, 96, 4, 8-12 (1987)
9. Mayer-Oakes, W. J., "Prehistory of The Upper Ohio Valley; An Introductory Archeological Study," p. 8, *Annals of Carnegie Museum*, Pittsburgh, PA (1955)
10. Adovasio, J. M., *et al.*, "Meadowcroft Rockshelter and The Peopling of The New World," *Quaternary Coastlines and Marine Archeology*, P. M. Masters and N. C. Fleming, eds., Academic Press, New York, NY, 413-439 (1983)
11. Adovasio, J. M. and R. C. Carlisle, "Pennsylvania Pioneers," *Natural History*, 95, 12, 20-27 (1986)
12. Colman, S. M., *et al.*, "Dating The Upper Cenozoic Sediments in Fisher Valley, Southeastern Utah," *Geol. Soc. Amer. Bull.* 97 1422-1431 (1986)
13. Shlemon, R. J., "Application of Soil Stratigraphic Techniques to Engineering Geology," *Bull. Assoc. Engr. Geol.* 22, 2, 129-142 (1985)
14. Jacobson, R. B., "Spatial and Temporal Distributions of Slope Processes in The Upper Buffalo Creek Drainage Basin, Marion County, West Virginia," Ph.D. Diss., The Johns Hopkins Univ., Baltimore, MD (1985)
15. Jacobson, R. B., "Pedological Studies at Grays Landing Terrace, Fayette County, Pennsylvania," *U.S. Geol. Surv. Open File Rep.* 87-128 (1987)
16. Hamel, J. V., "Geological, Geomorphological, and Geotechnical Investigation; Right Bank Terrace, Grays Landing Lock and Dam Project, Monongahela River, Pennsylvania," Report to Pittsburgh District, Corps of Engineers (1987)

Nondestructive Testing and Evaluation Methods For Investigation and Condition of Deep Foundations

LARRY D. OLSON, EDWARD O. CHURCH and CLIFFORD C. WRIGHT
Olson-Church, Inc.
Denver, Colorado

HGS-87-11

NONDESTRUCTIVE TESTING (NDT) methods for evaluating the integrity of deep foundations have been under active research, development and application since the mid 1960's. The most common NDT methods use sonic and nuclear density techniques. These techniques were adapted from geophysical exploration and mechanical vibration testing methodologies to test drilled pier and driven pile foundations. Case histories are presented herein to illustrate applications of three of the sonic techniques.

The research and development of the NDT methods was spurred by failures of deep foundations. The failures were generally due to defects in the foundations, although there were other causes of foundation damage such as earthquakes. The defects in the foundations were the result of problems that occurred during pier drilling and pile driving operations. Between 5% and 10% of the deep foundations that were tested with nondestructive methods overseas were found to be defective in a study by Preiss and Shapiro (1981). Their statistics most likely came from sites with difficult subsurface conditions where problems were suspected during construction.

The authors' experience in NDT of deep foundations has shown performance problems with drilled piers and driven piles are much less frequent than implied above when "good" and "bad" sites are lumped together. Another factor that undoubtedly contributes significantly to the generally good performance of deep foundations is the conservatism in design and material properties. This factor mitigates the effects of less severe defects. Regardless, failures do occur in deep foundations and such failures can be catastrophic and costly for critical structures such as bridges and high-rises.

Nondestructive tests of deep foundations are typically performed for three project types: 1. quality assurance for critical structures and/or difficult subsurface conditions, 2. investigation of foundation conditions when problems are suspected during construction, and 3. investigation of foundation conditions for older structures for renovation purposes. The type of project, importance of each foundation,

desired information and access conditions for testing must all be considered in developing an NDT program to meet project objectives. Depending on the nature of a project, destructive testing, such as coring or excavation, is frequently done in conjunction with nondestructive testing.

The four main sonic methods are: stress wave, impulse response, crosshole sonic logging, and parallel seismic. Each of these methods is discussed in the following section. Case history presentations follow to illustrate applications of the first three methods. Statistical and engineering considerations for NDT of foundations are then addressed, followed by a summary.

SONIC NONDESTRUCTIVE TESTING METHODS

The stress wave, impulse response, sonic logging and parallel seismic methods provide information on the integrity and length of piers and piles. The impulse response method also provides data relating to soil-foundation interaction conditions. Although the NDT methods are most commonly applied to concrete foundations, some of the methods are also applicable to timber and steel piles.

The above NDT methods involve very low strain measurements from the sonic excitation. Consequently, these methods cannot provide data on the capacity of deep foundations. Load tests and use of the dynamic pile analyzer will provide data on the integrity and capacity of deep foundations, but at much greater costs. The low strain sonic NDT methods are discussed below.

STRESS WAVE - Principles of a geophysical seismic reflection survey have been adapted for NDT of deep foundations with the stress wave (SW) method. The method involves striking the head of the foundation to generate a stress (compression) wave that travels down the foundation until it is reflected from the foundation toe or an irregularity back to the surface (Fig. 1).

An oscilloscope or signal analyzer is triggered at the moment of impact and the vibration at the head is monitored with an acceler-

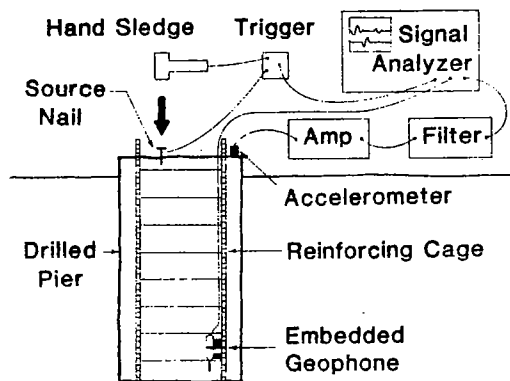


FIG. 1 - STRESS WAVE METHOD

ometer on the surface to identify the arrival time of reflected stress wave(s). If the stress wave velocity (V_c) is known, then the depth of the reflector (D), toe and/or defect, is calculated by multiplying the velocity by the arrival time (t) and dividing this quantity by 2 as indicated in Eq. (1). Filtering of the surface receiver signal is generally necessary to remove noise from surface and shear waves at the foundation head to permit clear identification of the reflected stress wave.

$$D = (V_c * t) / 2 \quad (1)$$

Stress wave velocity can be measured directly if geophone receivers are embedded during construction for quality assurance purposes (Fig. 1). In cases where geophone receivers are not embedded, stress wave velocity can be accurately estimated by measuring the ultrasonic pulse velocity through the foundation head. The estimated stress wave velocity is assumed to be 90% of the pulse velocity (Hearne, Stokoe and Reese, 1981). Stress wave velocity typically ranges between 11,000 and 14,000 feet per second (fps) in sound foundation concrete. Capabilities and limitations of the SW method are discussed in more detail by Hearne, Stokoe and Reese (1981), and Olson and Thompson (1985).

IMPULSE RESPONSE - This method involves applying dynamic modal testing techniques developed for mechanical engineering purposes to deep foundations. A building institute in France (CEBTP) adapted the forced vibration response technique to evaluate the integrity of drilled piers and driven piles in the 1960's (Stain, 1984). The technology has now developed to the point where the equipment needed to perform the impulse response (IR) test is portable and consists of a signal analyzer, impulse hammer (built-in force transducer), and a geophone (Fig. 2).

The test consists of hitting the top of the foundation several times with the impulse hammer and measuring the vibration response of the foundation with the geophone. The impact force and the vibration response are averaged and processed with the signal analyzer to deter-

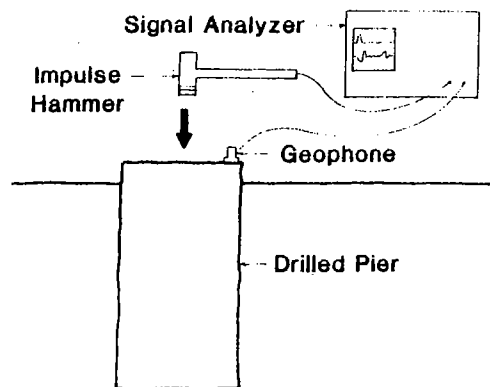


FIG. 2 - IMPULSE RESPONSE METHOD

mine the dynamic frequency response of the soil-foundation system. Reflected stress wave energy is indicated by resonant frequency peaks in the records. Reflector depths (D) are calculated by dividing the stress wave velocity (V_c) by twice the average change in frequency between resonant peaks (df) as indicated in Eq. (2). Reflector depths are compared to foundation depths to decide if the reflection is from the toe or a defect.

$$D = V_c / (2 * df) \quad (2)$$

In addition to predicting reflector depths, the IR test also measures dynamic stiffness at the foundation head. Dynamic stiffness reflects the soil-foundation interaction conditions and may be thought of as a spring constant with units of kips/inch. Identical foundation members will be much less stiff in soft soil than in bedrock. Defects such as necking and soil inclusions in drilled piers and breaks in driven piles also result in comparatively lower dynamic stiffnesses (Olson and Church, 1986). Correspondingly, enlargements and bulbs in piers will produce comparatively higher dynamic stiffnesses.

The IR test can characterize the nature of defects, such as necking or bulbing, on a comparative basis for similar foundations. Although dynamic stiffness measured by the IR test is a low strain measurement, correlations with static stiffness at low loads in axial load tests indicate dynamic stiffness is typically 1 to 2 times the static stiffness. Such correlations have been used to make reasonable predictions of foundation settlement under working loads. However, the IR test does not predict the capacity of deep foundations.

CROSSHOLE SONIC LOGGING - This method is an extension of sonic logging in geophysics and was also developed by CEBTP. The method involves installing two or more PVC or metal pipes in drilled piers during construction and then subsequently performing crosshole sonic logging (CSL) between all tube pairs as indicated on Fig. 3.

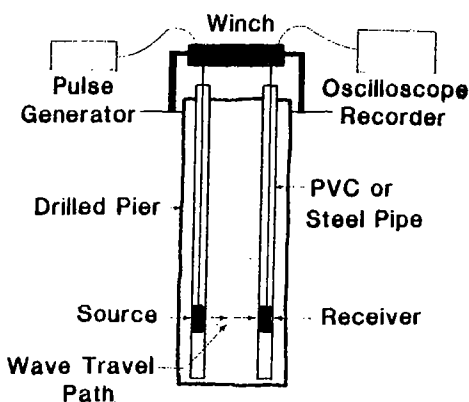


FIG. 3 - CROSSHOLE SONIC LOGGING METHOD

A CSL test consists of lowering a sonic compression wave energy source into one of the water-filled pipes and a receiver tool into another pipe. The tools are then simultaneously withdrawn from the bottom of a pier and a record made of the time for the stress wave to travel from the source to the receiver. The sonic signal is continuously displayed on an oscilloscope screen and a photograph taken of the screen to provide a permanent record. Defects are indicated by increases in wave travel time since defective concrete with soil, gravel and bentonite inclusions has a much slower wave velocity than sound concrete. The sonic signal may be lost entirely if the defect is severe.

From two to four pipes are typically installed in drilled piers with diameters of four feet or less. Larger diameter piers and diaphragm walls may require pipes to be located up to 10 or more feet apart and additional pipes may be needed. The CSL method provides high quality measurements of compression wave velocity at vertical intervals of 1 inch although even finer interval measurements can be performed if necessary. Because of the costs of installing the pipes, the CSL method is generally used for quality assurance purposes on critical foundations and concrete diaphragm walls. The method provides excellent data on the interior concrete quality of these foundation elements, but no data on soil-foundation interaction.

PARALLEL SEISMIC - This method is normally used when questions arise about the integrity and length of deep foundations after a structure has been completed and the tops of foundations cannot be readily exposed for SW or IR testing. The parallel seismic (PS) method involves drilling a borehole immediately adjacent to a foundation to a depth at least a few feet below the depth of the foundation toe as indicated on Fig. 4. A closed end tube may need to be inserted in the borehole if caving is a problem.

The borehole/tube is filled with water prior to lowering a hydrophone receiver to the bottom of the borehole/tube. The hydrophone is

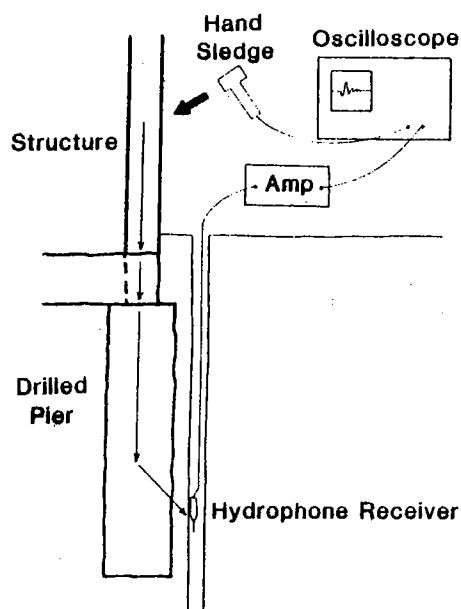


FIG. 4 - PARALLEL SEISMIC METHOD

raised in increments of 1 to 2 feet and the foundation head or structure is struck to send a compression wave down the foundation. The travel time for the wave to travel to the receiver is recorded at the depth intervals and a profile of signals is built up. Provided the soil velocity is slower than the concrete velocity, the bottom of the foundation can be readily identified. The PS method is particularly useful in resolving disputes concerning the bottom elevations of piers and piles after structures have been erected.

CASE HISTORY NDT INVESTIGATIONS

The following case histories are from a forensic NDT investigation of drilled pier integrity at Site A and a quality assurance NDT investigation of pier integrity at Site B. Defects were identified in foundations by the NDT methods at both projects. Results of stress wave and impulse response tests are presented for sound and defective drilled piers at Site A. Crosshole sonic logging test results are presented for sound and defective portions of a cast-in-place pier at Site B. Background information on subsurface conditions, foundation construction, and the structures is also presented.

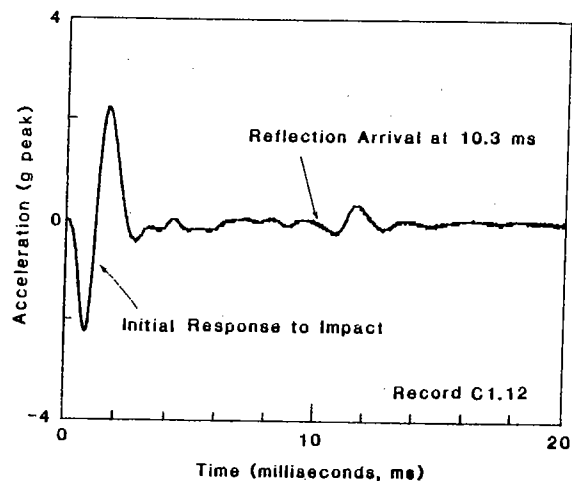
SITE A - Because of excessive settlement of a three-story concrete frame office building, the existing footings were underpinned with drilled piers in the building basement. Subsurface conditions consisted of 60 feet of interlayered silts and sands underlain by sedimentary sandstone and claystone bedrock. Footing settlements were attributed to significant "loosening" of the silt and sand soils due to a rising ground water table over the 14-year life of the building.

Concerns arose regarding the integrity of the piers during concreting operations because

of problems in placing the concrete. An NDT program involving SW and IR tests was performed of selected piers to investigate whether defects existed in the piers or not. The tests were performed on top of beams and haunches that were attached to footings and walls, respectively, and the drilled piers. A total of 8 piers were tested with the SW and IR methods. The NDT results indicated four of the piers were sound and four of the piers were defective.

Drilled Pier Construction - Straight shaft piers drilled into the bedrock were selected to underpin the existing footings below walls and columns in the basement. The piers were drilled with an indoor drilling rig with 2.5 foot auger flights. A 16 inch I.D. steel casing (1/8 inch wall thickness) was advanced with the auger until the casing was seated in the bedrock surface. Casing sections were welded together every 4 feet. The design penetration into bedrock was then drilled below the bottom of the permanent casing and the hole flushed with a 3 inch water hose until the water running out the top of the casing was relatively clean. A steel reinforcing cage was next installed in sections the full depth of the pier. Concrete was placed using a concrete pumper and a tremie to pump concrete from the bottom of the pier up to displace the water in the hole.

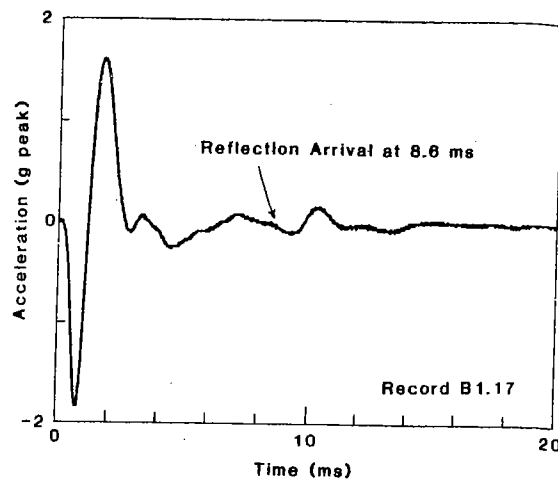
Stress Wave Test Results - Stress wave test records are presented for a sound pier on Fig. 5 and a defective pier on Fig. 6. Examination of the records shows the large initial response to the impact followed by a relatively quiet time and then the increase in amplitude and phase change associated with the arrival of the reflected stress wave energy.



Using Eq. 1 with $V_c = 12,000$ fps and $t = 10.3$ ms
 $D = 62$ feet

FIG. 5 - STRESS WAVE RECORD FOR SOUND PIER

A reflector depth of 62 feet was predicted for the sound pier based on the reflection arrival at 10.3 milliseconds (ms) as indicated on Fig. 5. This agrees closely with the



Using Eq. 1 with $V_c = 12,300$ fps and $t = 8.6$ ms
 $D = 53$ feet

FIG. 6 - STRESS WAVE RECORD FOR DEFECTIVE PIER

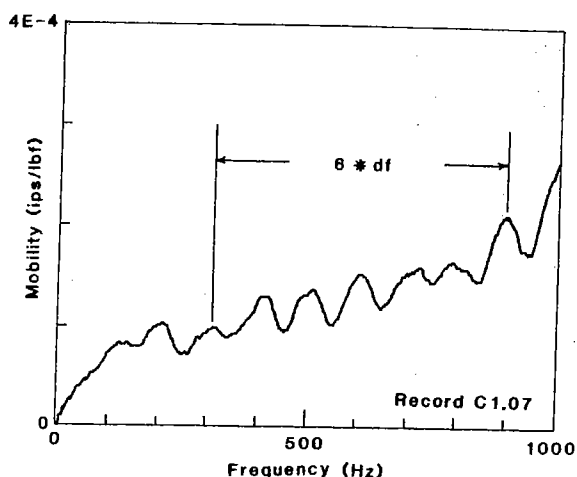
drilled depth of 61.5 feet logged for this pier. Consequently, the reflector is believed to be the pier toe and this pier is considered sound.

A reflector depth of 53 feet was predicted for the defective pier based on a stress wave reflection arrival at 8.6 ms as indicated on Fig. 6. The defective pier was permanently cased to a depth of 58.5 feet with a drilled depth of 63.5 feet. The predicted reflector depth was 5.5 feet above the bottom of the casing. Consequently, this pier was considered defective.

Impulse Response - The 8 piers were also tested with the IR method as a check on the reflector depths identified from the SW results, and in an attempt to characterize the nature of the defects by measuring dynamic stiffness. Impulse response mobility plots are presented for the sound pier on Fig. 7 and for the defective pier on Fig. 8. The mobility plots express the dynamic response of the drilled pier and haunch systems over the frequency range of interest. The peaks in the plot are resonances due to reflections of compression wave energy.

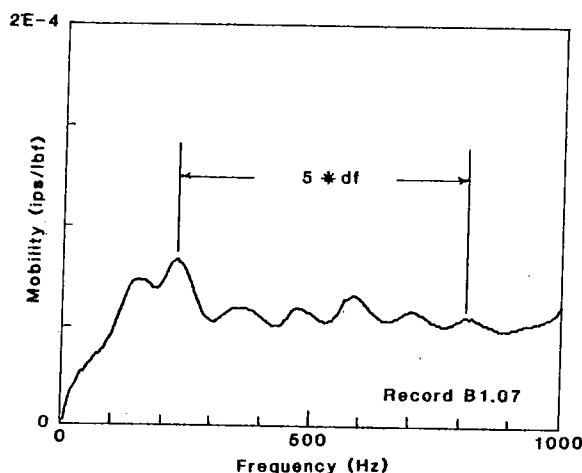
Examination of the mobility plot for the sound pier indicates a reflector depth of 63 feet based on an average change in frequency of 95 Hz (Fig. 7). This compares well with the reflector depth of 62 feet predicted for the SW record of the sound pier (Fig. 5). A reflector depth of 53 feet was predicted from the mobility plot for the defective pier based on an average change in frequency of 116 Hz (Fig. 8). The same reflector depth was predicted from the SW record for the defective pier (Fig. 6).

Dynamic stiffness is measured at low frequencies from the slope of the initial straight line portion of a mobility plot. The slope of this line can be used to calculate the "flexibility" of the pier head. Dynamic stiffness is the inverse of flexibility. A dynamic stiffness of 6,050 kips/inch was measured for the



Using Eq. 2 with $V_c = 12,000$ fps and $df = 95$ Hz
 $D = 63$ feet

FIG. 7 - IMPULSE RESPONSE MOBILITY PLOT FOR SOUND PIER



Using Eq. 2 with $V_c = 12,300$ fps and $df = 116$ Hz
 $D = 53$ feet

FIG. 8 - IMPULSE RESPONSE MOBILITY PLOT FOR DEFECTIVE PIER

sound pier. A lower dynamic stiffness of 5,200 kips/inch was measured for the defective pier as would be expected due to the defect.

However, dynamic stiffnesses were much greater for piers under haunches in comparison to piers under beams. This indicates the top conditions of the piers had a substantial effect on dynamic stiffness. Consequently, the IR tests were unable to characterize the nature of the defects in the piers because of the unknown effects on dynamic stiffness from the haunches and beams. This is not an unusual test result for piers with columns, walls, or grade beams constructed above them. Load tests of defective piers may be performed to determine their capacity.

SITE B - A 50 year old hydroelectric power house tilted 1.5 feet as a result of flooding that washed out two dams on either side of the power house. The hydraulic and scour forces of the flood waters undermined the 8 feet thick mat foundation and caused the

tilting. However, the stiffness of the mat foundation minimized damage to the power house and repairs were feasible if the power house could be releveled for turbine operation.

Repair plans called for cast-in-place piers to be installed on the downhill side of the power house to serve as the reaction for hydraulic jacks to releve the structure. Crosshole sonic logging of the piers was specified in repair plans as a quality assurance measure because of difficult subsurface conditions. Subsurface conditions consisted of 300 feet of glacially deposited sand, gravel, cobble and small to large boulders with ground water almost at the ground surface.

Pier Construction - Prior to installing the repair piers, three test piers were installed to shake-down construction procedures and perform an axial load test to optimize the design length for the repair piers. The test piers were constructed by vibrating a 3 feet diameter heavy steel casing 45 feet into the ground with a vibratory pile driving hammer. Sand and gravel were removed from the casing with an underwater air lift procedure. Larger cobbles and boulders were removed with a clam-shell and a churn drill to break up boulders. Once the casing had been cleaned out, a reinforcing cage was inserted with PVC tubes attached to the inside of the cage for cross-hole sonic logging. Concrete was then pumped from the bottom of the casing up with a slick-line tremie pipe until the casing was full of concrete to the ground level. The vibratory pile hammer was then used to vibrate and pull the casing out of the ground.

Crosshole Sonic Logging - Each pier had three PVC tubes installed in a triangular pattern inside the reinforcing cage with a nominal spacing of 18 inches between the tubes. The PVC tubes were 1.5 inches diameter Schedule 40 with glued joints and bottom caps to prevent concrete intrusion into the tubes. The tubes were filled with water prior to testing.

Crosshole sonic logs were run between each of the three pairs of PVC tubes in Test Pier 2. A test record for tubes 2 and 3 is presented on Fig. 9. The faint vertical white line on the left hand side of the photograph of the oscilloscope screen indicates a time of 0 microseconds when the pulse generator excited the sonic source. The next white vertical line represents the first arrival of the wave energy sensed by the receiver. The oscillating white vertical lines to the right of the initial arrival are from the continuing vibrations of the receiver. A shift to the right of the first arrival time indicates a longer travel time, a slower wave velocity and comparatively poorer quality concrete.

A clear, consistent arrival time of about 140 microseconds is apparent between the depths of 5.5 and 9.0 feet in Fig. 9. However, the signal is completely lost between 2.0 and 5.5 feet which indicates a severe defect in this zone. The sonic signal arrives at a longer

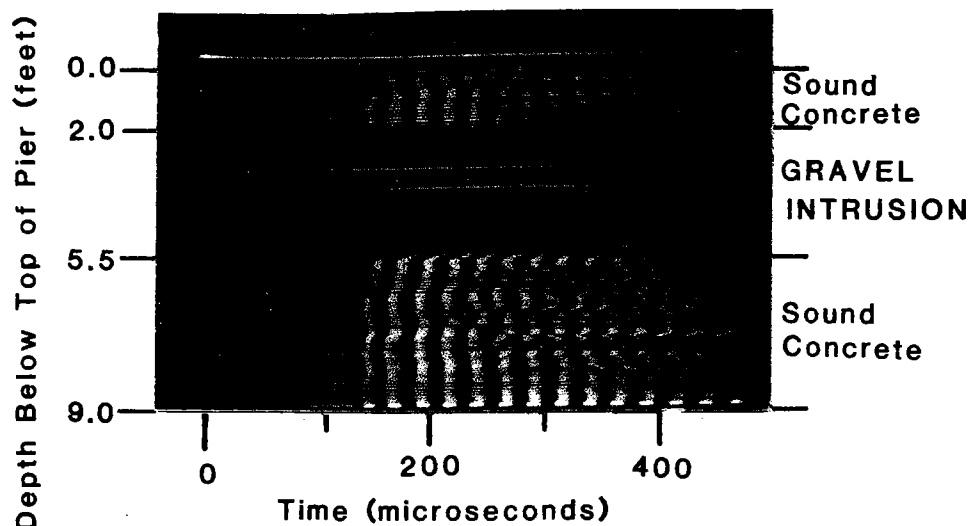


**FIG. 9 - CROSSHOLE SONIC LOG FOR
TEST PIER 2 BETWEEN TUBES 2 AND 3**

time of 160 microseconds, but is received again between the depths of 0 to 2.0 feet. Average wave velocities in the concrete between tubes 2 and 3 were calculated as 10,300 fps in the 0 to 2 feet zone and 11,750 fps in the 5.5 to 9 feet zone. This velocity range is indicative of good quality concrete (except for the major defect) considering the 2 day age of Test Pier 2 at the time of logging.

Unpublished records for sonic logs between tubes 1 and 2, and tubes 1 and 3 in Test Pier 2 indicate similar complete losses of sonic signals between depths of 1.9 to 5.5 feet and 2.3 to 6.4 feet, respectively. Unpublished records also indicate sound concrete above and below the major defect zone in Test Pier 2.

After discussing the construction technique with the contractor, it was predicted that sand, gravel and cobbles had intruded into upper portions of Test Pier 2 because of insufficient concrete head at the time the casing was removed. Excessive use of the vibratory hammer during casing removal may have further loosened the sand and gravel near the ground surface.

The defect near the top of Test Pier 2 was subsequently confirmed by coring three 2 inch diameter holes into the top of the pier. The cores encountered uncemented gravel at depths slightly greater than 2 feet. The cores could not be advanced further because of the loose gravel.

The construction procedure was modified for Test Pier 3 so that sufficient concrete head was maintained to prevent the gravel intrusion. The use of the vibrator to pull the casing was also minimized. Crosshole sonic logging of Test Pier 3 indicated lower velocity concrete between the depths of 6 and 11 feet. The lower velocity zone was interpreted as poorer quality concrete, but not a significant defect. Test Pier 3 was successfully load

tested which confirmed this interpretation.

The repair piers were to be installed below the 8 feet thick mat foundation under 20 feet of water. Consequently, it was decided to install a permanent 42 inch diameter casing in the upper 10 feet of each repair pier. The permanent casing provided for improved control of concrete elevations during placement and minimized the problems of soil intrusion in this critical zone. Crosshole sonic logging of repair piers indicated sound concrete. The power house was successfully releveled at 98% of the hydraulic jacks capacity with good pier performance.

STATISTICAL AND ENGINEERING CONSIDERATIONS

How many deep foundations should be tested with the NDT methods to check integrity? The answer depends on several variables including whether the foundation system is redundant (many piles with a pile cap) or non-redundant (single large drilled pier for a bridge), quality assurance versus forensic situations when problems are suspected, degree of foundation construction observation, and difficulty of subsurface conditions. The decision as to how many foundations to test is ultimately an engineering judgement. However, consideration of the problem from a statistical viewpoint can be an aid to the engineer.

Preiss and Shapiro (1979 and 1981) have presented the statistical probability calculations necessary to estimate the number of piles to be tested on a project. They have prepared charts to determine the number of piles to be tested (n) for a given probability that take into account: the number of defective foundations allowed (N_a); the number of defective foundations found from testing (N_d); and the total number of foundations (N).

This type of probability chart is presented on Fig. 10 for a site with 100 foundations. As an example of how to use this chart, if one wants to be 90 percent certain of having no more than 10 defective foundations in a group of 100 foundations, then 20 foundations would need to be tested and found sound as indicated on Fig. 10. Changes in the total number of piles or the acceptable probability level would require development of a different chart. However, it should be understood that if all of the foundations need to be sound, then every foundation must be tested.

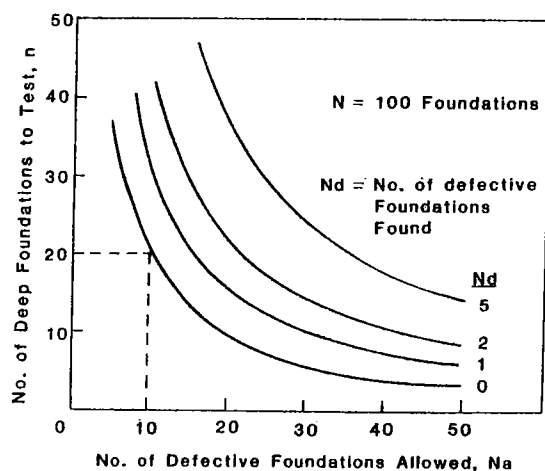


FIG. 10 - 10% PROBABILITY RELATIONSHIPS
(AFTER PREISS AND SHAPIRO, 1979)

The French building specification DTU 13.2 provides for a 20% increase in working stress for deep foundations if minimum testing requirements are met. Their specifications include designations for different deep foundations as to the NDT methods and numbers of tests to be performed. Apparently, their specification has taken the approach of reducing some of the inherent design conservatism if appropriate nondestructive and destructive tests are performed and the tests indicate sound foundation conditions.

SUMMARY

Nondestructive testing of deep foundations can be of excellent benefit both technically and economically for quality assurance, forensic, and renovation purposes. The most suitable NDT methods depend on subsurface conditions, foundation type, desired information, and foundation access.

Case histories were presented that illustrate applications of three sonic methods: stress wave; impulse response; and, crosshole sonic logging. Both of the case history NDT investigations resulted in considerable economic savings in investigation costs over conventional destructive tests such as coring, excavation or load testing. The quality assurance investigation program for the hydroelectric power plant saved hundreds of thousands of dollars by identifying problems in the construc-

tion of the test piers prior to construction of the repair piers.

The decision as to the number of deep foundations to be tested is ultimately an engineering judgement. Statistical analyses can be a valuable aid in arriving at the number of deep foundations to be tested. Key factors in successful implementation of NDT integrity investigations during construction are the experience of the engineers in applying the NDT methods and the interaction of the project engineer, contractor and NDT engineer.

ACKNOWLEDGEMENT

The crosshole sonic logging at Site B was performed by Olson-Church, Inc. in conjunction with Testconsult/CEBTP of England.

REFERENCES

- Preiss, K. and Shapiro, J., "Statistical Considerations in Pile Testing," Congress Mecanique als Sols, 1981.
- Hearne, T.M., Stokoe, K.H., and Reese, L.C., "Drilled-Shaft Integrity by Wave Propagation Method," ASCE Geotechnical Engineering Journal, Vol. 107, No. 10, October, 1981.
- Olson, L.D., and Thompson, R.W., "Case Histories Evaluation of Drilled Pier Integrity by the Stress Wave Propagation Method," Session Proceedings Publication Drilled Piers and Caissons II, ASCE National Spring Convention, Denver, Colorado, May, 1985.
- Stain, R.T., "Integrity Testing of Drilled Shafts," Foundation Drilling Magazine, ADSC, Part I-Vol. XVIII, No. 12, February, 1984, Part II-Vol. XVIII, No. 13, March/April, 1984.
- Olson, L.D. and Church, E.O., "Survey of Non-destructive Wave Propagation Testing Methods for the Construction Industry," Proceedings of the 37th Annual Highway Geology Symposium, Helena, Montana, August 20-22, 1986.

Preiss, K. and Shapiro, J., "Statistical Estimation of the Number of Piles to be Tested on a Project," presented at meeting of the RILEM Commission on Nondestructive Testing, Stockholm, 1979.

Lessons Learned From European Practice on the Use of Stone Columns for Site Improvements

ROBERT C. BACHUS
School of Civil Engineering
Georgia Institute of Technology
Atlanta, Georgia

HGS-87-12

THE USE OF SOIL and site improvement techniques for modifying unstable topography has been considered and/or used at a steadily increasing rate in the recent past. The stone column technique for improving the bearing capacity and settlement characteristics of soft ground was first introduced here from Europe for highway applications, the technique has gained acceptance on other commercial projects. As the theme of this conference deals specifically with the construction of highways on unstable topography, it seems fitting to look in retrospect on the numerous lessons learned from stone column use in Europe.

Both vibro-replacement and vibro-displacement methods of stone column construction were in widespread use in Europe prior to their introduction to the U.S. market. The author had the opportunity to interview numerous European contractors and engineers experienced in the design and construction of stone columns as part of a FHWA study a few years ago. These visits were undertaken specifically to learn of the advantages as well as the limitations of the improvement technique. The results of these visits, coupled with the updated results of recent projects, will be presented in this paper.

Specific topics of discussion will include applications where the technique is best suited as well as those where numerous engineering problems have developed. A discussion of construction procedures and realistic expectations for performance of the stone column improved site will also be presented. A review of the response of stone column subjected to vertical and lateral loads, potential load distribution and failure mechanisms, use in improving the stability of slopes, and reduction of liquefaction potential is presented. Finally, a comparison of the behavior of conventional and rigid stone columns will be developed.

Introduction The use of stone columns to stabilize weak soils has been used since the 1950's. This use has, however been concentrated in Europe and has only used in

the U.S. since the late 1970's. A research report was prepared for the FHWA in 1983 to provide a then-current review of the present knowledge of the use and behavior of stone columns (Barksdale and Bachus, 1983). As part of this project the author had the opportunity to visit all of the major European contractors and engineers involved in the design and construction of stone column stabilized soils. Since that time the technology has spread to the United States and, while not considered common practice as yet, many engineers have considered the use of the technique as a means of solving a wide range of geotechnical problems. Some of the current uses of stone columns in the U.S. are in line with applications from Europe; some European practices have yet to be accepted in this country. The major thrust of this paper will be to summarize the stone column technique of stabilization, to describe the application of the technique and the philosophy of its use with particular emphasis placed on European practice, and to discuss the role that stone columns have played in U.S. practice.

Stone Column Construction Stone column construction involves the partial replacement of unsuitable subsurface soils with a compacted vertical column of stone that usually completely penetrates the weak strata. Replacement is initiated when jetting water is used to remove the unsuitable soil in a process known as vibro-replacement. An alternative method of "replacement" involves displacing the unsuitable soil laterally into the sides of the vertical hole using a lateral vibration and displacement; this method is known as vibro-displacement. Once the vertical shaft is created, the hole is incrementally backfilled in 1 to 4 ft (300-1200 mm) lifts with stone which is densified by the use of a vibrating probe. As the probe penetrates the stone, not only is the stone densified, but the vibration pushes the stone into the walls of the hole, thus increasing the lateral pressure on the stone.

The vibrating probe used in hole formation and stone densification was developed in 1935 for the compaction of granular, non-cohesive soils (Steuerman, 1939). Although each specialty contractor identifies their vibrator by a slightly different name and each has slightly different characteristics, the term vibroflot or poker is frequently used in description of the equipment. The concept of the current generation probes is very similar to the originally developed design. Rotation of eccentric weights within the body of the probe, using either electric or hydraulic power causes lateral vibration at the tip of the probe. During vibro-replacement (the wet method) the vibroflot opens a hole by jetting, using large quantities of water under high pressure. Using vibro-displacement (the dry method) air may be utilized to assist the probe in displacing the native soil laterally as it is advanced into the ground. In both methods the weight of heavy follower tubes attached above the probe and the vibration of the probe aid in advancing the hole and compacting the vertical column of stone. As originally designed the probes were supported by any of a wide assortment of large cranes. During the authors visit to Germany in 1981, a relatively new concept of equipment design was demonstrated. This system, called a Vibrocat by the GKN Keller Co., essentially involves a self contained probe and crane which generally operates using the dry method and bottom feeding of the stone.

The probe typically varies in diameter from 12 to 18 inches (300-460 mm) depending on the individual contractors' equipment. Due to soil erosion and lateral compaction, the excavated hole is slightly larger than the probe. The diameter of the completed stone column varies from typically 2 to 3.5 ft (600-1070 mm) depending on the strength of the subsurface soils and the energy applied at each elevation of the column. Original systems called for the probe to remain in the hole at all times to assure stability of the upper reaches of the hole not yet filled with stone. The probe is, however, routinely removed to facilitate placement of the stone. Newer generation probes use the bottom feed method of stone discharge which allows the probe to remain in the hole most all the time. This is particularly true in the vibro-displacement method of construction. Successive lifts of stone are placed and compacted until a column of stone has been formed up to the ground surface of the native soil. In an additional novel method of construction, concrete or cement-mixed aggregate has been supplied to the bottom feed units in place of the standard stone. Construction of the column is essentially identical but the final product is a much stiffer and stronger "rigid" stone column.

Tables which provide an extensive summary of

the characteristics of the probes used to form the columns, the installation characteristics of the columns, and the construction considerations for the vibro-constructed stone columns were provided in the original FHWA report and reflect characteristics at the time the concept was introduced into this country some 10 years ago. The following summary of these important characteristics and considerations are provided in the following paragraphs.

For the five European contractors actively involved in stone column construction ten years ago, all used relatively similar probes which generally weighed between 2 and 4 tons, were approximately 15 feet (4.5 m) long, 12-22 in (30-55 cm) diameter, and capable of exerting between 12 and 28 tons of lateral force. Differences surfaced in the method of operation and the frequency of vibration. GKN Keller and Vibroflotation used electric motors while Cementation, Bauer, and Franki preferred hydraulic motors. These units provided between 1000 and 4000 rpm. In general, the contractors use operating frequencies between 2000 and 3000 rpm, depending on the size of the vibrator. For small vibrators higher frequencies are needed to achieve proper column construction. The use of higher frequencies provides increased lateral thrust at the expense of bearing wear. Electric motor vibrators need to be constantly cooled.

All of the European contractors used the equipment to construct stone columns which ranged from 2 to 3.5 ft (0.60-1.0 m) in diameter. Columns were constructed at a rate of 30-60 feet per hour (9-15 m/hr) using stone which was typically 0.38-3.0 in (1.0-7.5 cm) diameter. The larger stone was used in the softer soil sites. Water was the preferred method of forming the hole generally because the contractors felt it provided a "surer" column. This confidence was balanced by the on site mess and problem with waste water disposal associated the wet-method.

Acceptable performance from a stone column stabilized site is dependent upon the quality control during construction and the consideration given by the contractor and engineer concerning the use of stone columns at a given site. Quality control includes assurance that the hole is thoroughly cleaned before introducing the stone, and that water is kept flowing during penetration of the stone. The column is constructed in 1-2 ft (0.3-0.6 m) lifts which are repenetrated by the vibrator. Compaction continues until a "suitable" resistance to penetration is achieved. This current or pressure is measured by an ampere meter or a hydraulic gauge and is generally site specific; it is evaluated during the initial stages of the project. Extreme care is followed in cases

where organic soils are encountered, particularly peat layers which are greater than about 3 feet (1 m). Penetration and column construction must advance rapidly in silty soils to avoid undermining of the upper soils by erosion.

It is interesting to note that in preparation for this paper a review of current practice, concerns, and equipment remains essentially unchanged. A few notable exceptions are worthy of mention (Dobson, 1987):

1. The dry method using bottom feed units is very popular. The reasons for this are the related to the problems associated with waste water disposal, cleanliness of the site, and current application of the technique which will be discussed in succeeding sections.

2. Use in the U.S. is now dominated by GKN Hayward Baker. The units which are the workhorses in Europe are the 80 horsepower probes, while the U.S. has utilized the larger 165 horsepower unit.

3. There have been significant improvements in automatically recording the compaction amperage and the quantity of stone used during construction of the column.

4. There has been a tendency to move away from the construction of rigid stone columns. Reasons for this include the different mechanism of load-deformation response when compared to conventional stone columns as will be presented subsequently.

Behavior of Stone Columns The stone column technique of ground treatment has proven successful in improving slope stability of both embankments and natural slopes, increasing bearing capacity, reducing both total and differential settlements, increasing the time rate of settlement, and reducing the liquefaction potential of sands. Stone columns have been used to support relatively lightly loaded structures overlying otherwise unsuitable soils. However, before an extensive discussion of the use of stone columns can proceed, some comment on the reasons that stone columns behave in typical service conditions is warranted.

In stone column construction, usually 15 to 35 percent of the weak soil volume is replaced by stone. The presence of the column thus creates a composite material of lower overall compressibility and higher shear strength than the native soil alone. Confinement, and thus stiffness of the stone, is provided by the lateral stress within the weak soil. Lateral pressures and column diameter can be controlled to some extent by the amount of stone placed at a given elevation and the amount of vibration imparted during construction. Upon application of vertical stress at the ground surface from

application of the service loading condition, the stone column and the weak soil may move downward at the same rate. Due to the stiffness differences an important stress concentration in the column will result. This same axial load applied to the top of the column produces a large bulge to a depth of 2 to 3 diameters beneath the surface. Thus, a characteristic "carrot" shaped column results which will increase the confining pressure further within the clay; the confinement of the stone also increases. An equilibrium state is eventually reached resulting in reduced vertical movement of the composite system when compared to the untreated soil. It has been shown that in stone column groups which are loaded over a large area, the individual columns bulge significantly less than individual columns. This response leads to higher loads per column in the group (Kaffezakis, 1983).

The mechanism of stress concentration on the stone column is an extremely important feature when considering both vertical and lateral load response. As discussed previously, the vertical response results in a net reduction of stress on the softer and more compressible subsurface soils. The result is a reduction of settlement and an increase in ultimate load carrying capability. The response to lateral loads is similarly affected. The concentration of stress on the column results in a higher normal stress carried in the column shaft. Under this increased component the lateral load carrying ability of the frictional stone material increases dramatically. Laboratory studies of the basic mechanism, however, show that the ultimate lateral load response is in fact coupled with the strength of the softer, unreplaced soil. If the soil is too soft or if the column is too stiff, the soil will tend to flow around the column rather than mobilizing the strength of the stone column (Bachus and Barksdale, 1984). Representative vertical and lateral load response of untreated and stone column treated soil is shown in Figures 1 and 2 respectively. Figure 1 is from a full scale vertical load test. Note that the effect of the stone column treatment is to provide increased stiffness and a slightly higher load carrying capability. The response of the stone column foundation is not, however, similar to a pile subject to vertical stress in that the onset of the limiting stress is not particularly well defined, providing somewhat of a ductile load-deformation response. The data shown in Figure 2 is from a laboratory scale model study of response of sand columns to lateral loading. In this figure the experimental curve is shown (#2) as well as a response assuming that just the clay is providing the resistance to loading (#1). Obviously, the clay alone does not provide complete resistance. An analytical procedure was

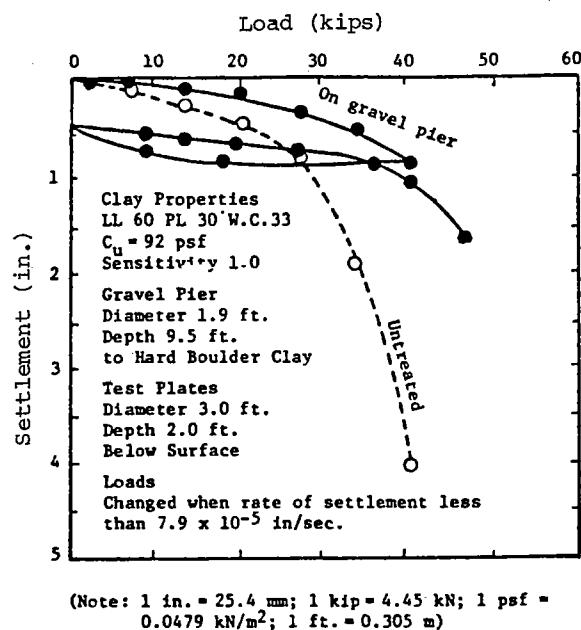


Figure 1. Vertical Load-Deformation Response

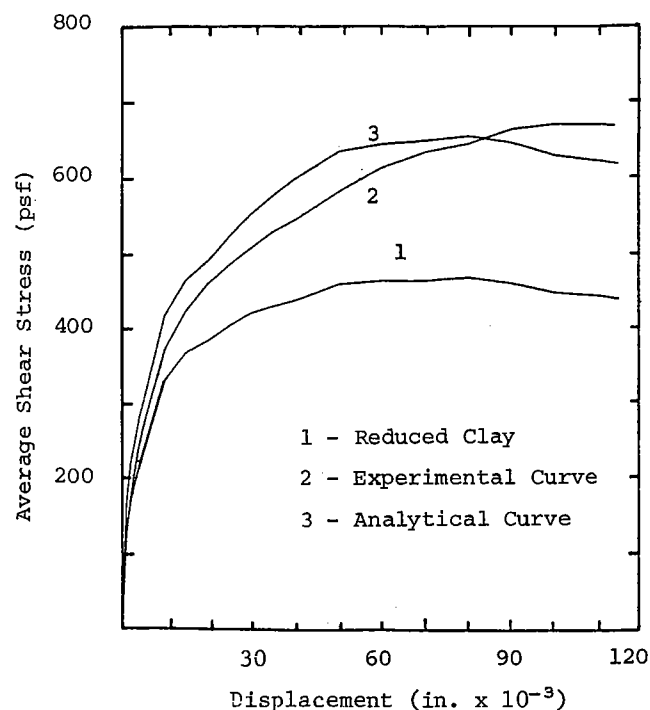


Figure 2. Lateral Load-Deformation Response

developed to account for a concentration of vertical stress atop the sand and to include this component of shear resistance to the composite system (Enchayan, 1982). The result of this analytical approach is also shown (#3); the agreement is very encouraging.

The ultimate load carrying capability and settlement response of the stone column treated foundation could be the title of a paper all to itself. A detailed presentation is beyond the scope of this paper, however, a brief mention of the concepts used in analysis is warranted. In considering the ultimate load carrying capacity, Greenwood (1970) and Wong (1975) considered the lateral load from the stone column mobilizing the passive resistance of the surrounding soil. Hughes and Withers (1974) and Darye and Nagaraju (1981) are among the engineers who utilized cavity expansion theory to predict the ultimate response. These methods seem to yield similar predictions for single stone columns but underpredict the group capacity and are discussed in detail in the FHWA report (Barksdale and Bachus, 1983). For predicting the settlement behavior of stone columns, techniques by Baumann and Bauer (1974), Goughnour (1979), and Barksdale (1981) all utilize the concept of stress concentration factors on the stone column and the one-dimensional consolidation response of the soil/column composite for evaluation. The methods all seem to predict reasonably well the settlement response of the system. The predicted reduction in settlement as a function of stone column spacing and stress concentration factor is shown in Figure 3.

Little fundamental work has been done on the response of rigid stone columns, but it can be shown that because of the stiffened column, a large stress concentration factor is expected. Thus, the column attracts a large amount of load. Unfortunately, this type of column will not show a gradual or ductile response typical of conventional stone columns because there is little movement within the column itself prior to failure. Behavior is rather like an unreinforced concrete pile, i.e. brittle. As mentioned previously, there seems to be a movement away from the use of rigid stone columns in practice today.

Applications for Stone Columns When considering the application of stone column foundation schemes, the information regarding the mechanisms of behavior must be considered. A number of "rules-of-thumb" were developed at the time of the authors visit to Europe that seem to still be in use today. Design loads on stone columns typically vary from 20 to 50 tons, depending to a certain extent on the strength of the weak soils. In practice, typically 15 - 35 percent of the unsuitable soil is either displaced or replaced by the stone. The reasons for this can be seen by the review of Figure 3 and economic considerations. Thus the concept is truly a stabilization technique and not just a weak alternative to a pile foundation system.

Specific applications and limitations can be considered as lessons learned from years of experience in Europe and the U.S. A

NOTE: a_s = fraction of soil tributary to column replaced by stone

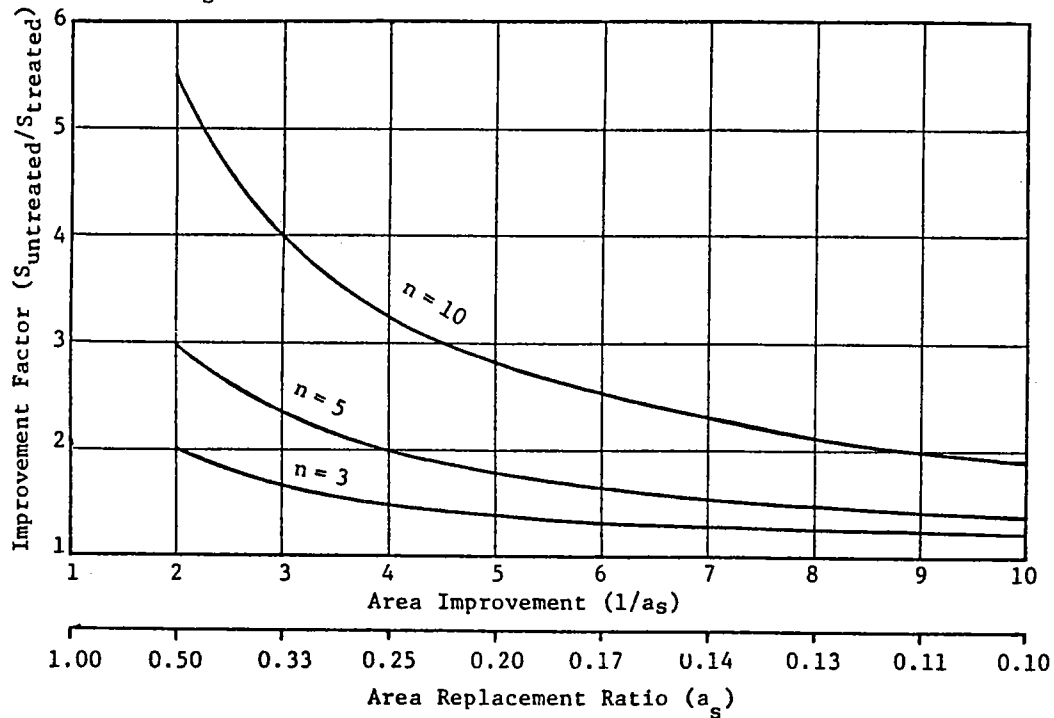


Figure 3. Typical Reduction in Settlement Obtained Using Stone Columns

table of applications and limitations of the stone column technique was presented in the original FHWA report as being typical of sentiments 10 years ago. Specific comment about these applications and limitations are further considered with a view of where the concept has tended.

1. Structural Support - A relatively popular technique for supporting typically 3 to 5 story structures in Europe. While it has been used in the states, it has been slow to catch on. Note that these applications are for relatively light loads and loading will generally be carried by groups of columns rather than a single unit. The use has probably been to control deformations rather than to prevent collapse.

2. Tank Farms - An ideal use of the concept because the loads are relatively light and the structure will probably tolerate some deformation. A common use in Europe that again has had little acceptance in the states, probably due in part to the cost of mobilization for a specific job.

3. Bridge Abutments - Stone columns have been used throughout Europe for the support of bridge abutments as an alternative to shallow foundations. The philosophy in the U.S. is apparently much more conservative as there is little consideration of this use. In cases where it has been used, a Reinforced Earth abutment has generally been incorporated into the design because of the flexibility

and ductility of the entire system.

4. Embankments and Bridge Approach Fills In Europe this still remains one of the more popular uses of the concept. The use offers a number of practical advantages. At the edge of the embankment they are used for stability considerations of the underlying soil. The use under the embankment will tend to reduce settlement and can in fact be used to gradually reduce settlements even further as the fill approaches the relatively rigid highway abutment. In addition to the considerations for strengthening and settlement reduction, the use of stone columns beneath embankments offers a provision for lateral drainage and thus can significantly speed up the time rate of consolidation. This application has been used with some success in the U.S. and remains one of the shining stars for the concept. It is interesting to note that the current use of geosynthetics beneath embankments over soft soil is about where stone columns were 10 years ago. Economics will play a large part in the long term acceptance of both of these procedures.

5. Liquefaction Resistance - Because of the lack of seismic activity in Europe, this application is not very widespread. It has however found tremendous acceptance

in other parts of the world, to include the United States. It has been shown to be an economic alternative to dynamic consolidation for deep deposits of soils ranging from loose sands to soft silty sands. In this application the role of the stone column is not completely realized as the densification ability is the desired long term goal of the effort. It is expected that this trend will continue and that the concept will remain a viable consideration for reducing liquefaction potential of soils

Summary Stone columns offer unique opportunities for improving the engineering performance of soils ranging from soft clays to loose sands. They offer a valuable alternative to other stabilization schemes which are used on these soils. Specifically, they offer 1) an increase in ultimate strength, 2) a reduction in total settlement, 3) a tendency to equilibrate settlements across the site, thus reducing differential settlements, 4) a tendency to speed up the time rate of consolidation, and 5) the ability to reduce the liquefaction resistance of loose sands and silty sands. Stone columns seldom render settlements to negligible levels and should not necessarily be considered a pile foundation. Rather they provide a means of stabilization. Despite this rather impressive list of credentials, the use of stone columns has been slow to catch on in the U.S. For example, typical figures for 1986 show that all vibro work in the U.S. amounted to approximately \$10 million, which was approximately the same as the amount expended in England. Usage in Germany for the same period amounted to approximately \$15 million. Given the size of the respective countries and the amount of total construction in each, stone columns have played a relatively minor role in the states. Reasons for this undoubtedly include the lack of familiarity with the concept, the relatively few number of contractors capable of installing the system, and the relative cost of mobilization and construction compared to alternative means of stabilization. The current consideration and popularity of geosynthetics appear to be attracting some of the potential stone column market. Stone columns appear to have an edge when considering alternatives for reducing liquefaction potential.

Acknowledgement

The information reported in this paper is largely taken from the final report submitted to the Federal Highway Administration as part of Contract No. DTFH61-80-C-00111. Dr. Richard Barksdale co-authored this report with Dr. Bachus and Mr. Jerry DiMaggio served as technical monitor for the project. Their assistance is appreciated.

References

- Bachus, R.C. and R.D. Barksdale, "Vertical and Lateral Load Behavior of Model Stone Columns", Symposium on Insitu Soil and Rock Reinforcement, Paris, 1984.
- Barksdale, R.D., "Site Improvement in Japan Using Sand Compaction Piles", Georgia Institute of Technology Report, July, 1981.
- Barksdale and Bachus, "Design and Construction of Stone Columns - Volume 1", Report No. FHWA/RD-83/026, Federal Highway Administration, McLean, Virginia, 1983.
- Baumann, V. and G.E.A. Bauer, "The Performance of Foundations on Various Soils Stabilized by the Vibro-Compaction Method", Canadian Geotechnical Journal, November, 1974, pp. 509-530.
- Datye, K.R. and S.S. Nagaraju, "Design Approach and Field Control for Stone Columns", Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, 1981.
- Dobson, T., GKN Hayward Baker, Personnel Communication, 1987.
- Enchayan, P., "Small Scale Laboratory Study of the Behavior of Stone Columns Beneath Embankments of Soft Clay", Special Research Problem Report, Georgia Institute of Technology, Atlanta, Georgia, August, 1982.
- Goughnour, R.R. and A.A. Bayuk, "Analysis of Stone Column - Soil Matrix Interaction Under Vertical Load", Proceedings, International Conference on Soil Reinforcement: Reinforced Earth and Other Techniques, Volume 1, Paris, 1979, pp. 271-278.
- Greenwood, D.A., "Vibroflotation: Rationale for Design and Practice", Methods of Treatment of Unstable Ground, F.G. Bell, ed., Newness-Buttersworth, London, 1975, pp. 189-209.
- Hughes, J.M.O. and N.J. Withers, "Reinforcing of Soft Cohesive Soils with Stone Columns", Ground Engineering, Volume 7, No. 3, May, 1974, pp. 42-49.
- Kaffezakis, G., "A Model Test Investigation of the Behavior of Stone Columns", Special Research Problem Report, Georgia Institute of Technology, Atlanta, 1982.
- Steuerman, S., "A New Soil Compaction Device", Engineering News Record, July 20, 1939.
- Wong, H.Y., "Vibroflotation - Its Effect on Weak Cohesive Soils", Civil Engineering (London), No. 824, April, 1975, pp. 44-67.

Ground Modification of Highway Embankment Foundation by Dynamic Compaction

JOHN G. DIVINEY

Gannett Fleming Geotechnical Engineers, Inc.
Harrisburg, Pennsylvania

HGS-87-13

INTRODUCTION

IN URBAN AREAS, landfills, loose man-made deposits, and building rubble deposited in low lying areas have often precluded the economic commercial development of sites. However, these same sites are frequently chosen as locations for highway alignments due to favorable right-of-way costs and alignment constraints. Such alignment constraints along the East Street Valley Expressway (Interstate 279) in Pittsburgh, Pennsylvania resulted in the requirement that a 1,000-foot length of highway embankment be constructed over an abandoned city dump. Settlement and stability beneath the 55-foot high embankment were a concern due to a thickness of dump material which ranged as deep as 50 feet. To minimize embankment foundation settlements and increase the factor-of-safety against foundation instability, a dynamic compaction ground modification treatment was employed prior to embankment placement.

Dynamic compaction, or impact densification, is the ground modification technique of applying high, dynamic energy impacts at the ground surface to densify foundation soils at depth. The high energy impacts are achieved by repeatedly dropping heavy weights, typically 6 to 20 tons, onto the ground surface from heights ranging from 30 to 120 feet. The technique, which is attributed to the Romans and was patented by Louis Menard in the late 1970s, is now employed by several ground modification contractors.

SITE DESCRIPTION

The site of the abandoned city dump is located near the northern end of the East Street Valley approximately 4 miles north of downtown Pittsburgh. Based upon an evaluation of several series of aerial photographs, the dumping of refuse at the site began prior to 1949 and continued until at least 1977, making the age of the dump material at the time of construction between 34 years and 6 years. The aerial photos show that dumping activities progressed from both sides of a narrow valley toward the center of the valley, with the most

recent refuse deposits occurring near the center of the valley. By 1977 approximately 9 acres were covered by the dump. (Figure 1)

Borings obtained at the site indicate that the dump material is generally construction debris consisting of a heterogeneous mix of soil, brick, wood, concrete, slag, shale, rock, cinders, tar paper, roots, and coal. Also, field observations at the time of the borings indicated the presence of large metal objects such as automobiles, refrigerators, and barrels at various locations. The presence of decaying organic material was noted in only one boring, by the generation of methane gas, and groundwater levels were found to be at or below the bottom of the dump.

The aerial photos indicate that refuse material was placed in the dump in an uncontrolled manner, i.e. without compaction. This lack of compaction is evident upon examination of boring logs which show the presence of voids and a wide range of standard penetration test N-values. (Figure 2)

The thickness of the dump material is variable across the site, with a maximum thickness of 50 feet observed in the borings. (Figure 2)

DESIGN AND CONSTRUCTION CONSIDERATIONS

Highway construction across the abandoned dump required that 6 lanes of Interstate highway and 2 lanes of local road be built on embankments approximately 55 feet high (Figure 2), with the base of the embankments encompassing approximately 7 acres of the 9-acre dump site.

Associated with the highway construction was the construction of a 36-inch diameter combined sewer through the dump site to replace existing sewers which were not designed for the highway embankment loads.

The original design of the highway in the 1960s had proposed that the dump material be completely removed from beneath the embankment. While this treatment would have been the most positive method for crossing the dump site, complete removal and replacement would

have been extremely expensive, with an estimated cost of \$3.1 million. An alternative to the original design was proposed in 1983 and was built in 1985 and 1986; it consisted of:

- Leave all dump materials in place at the site.
- Install the 36-inch combined sewer in a 500-foot long, 6-foot diameter tunnel excavated in rock beneath the dump site to avoid deep trench excavations through the site. (Figure 1)
- Perform dynamic compaction on the surface of the dump to (1) collapse near surface voids to prevent localized settlements due to raveling, (2) minimize differential settlements which would result from lateral variability of dump material, and (3) increase the strength of the dump material and consequently the factor-of-safety against deep foundation failure. (Figure 1)
- Provide a staged sequence of dynamic compaction operations on steep, existing slopes of dump material. (Figures 1 and 3)
- Install inclinometer casing prior to embankment construction to monitor possible slope instability where dump material is not confined by the valley walls and sliver embankment is to be constructed. (Figures 1 and 3)
- Install settlement platforms at the base of the embankment to monitor the magnitude and time rate of foundation settlements during embankment construction. (Figure 1)
- Construct the proposed embankment across the dump site to an elevation approximately 3 feet higher than proposed subgrade elevation to allow for anticipated settlements.
- Allow the constructed embankment to set for a preload period of at least 2 years before adjusting to subgrade elevation and paving.

The cost of the alternative treatment was approximately \$1.3 million, including tunnel construction, with a cost savings of \$1.8 million being realized over the complete removal treatment.

Design calculations predicted that the dynamic compaction alternative would provide the following improvements to the site:

- Overconsolidate the upper 25 feet of dump material.
- Reduce the height of dump material by 7.5 feet.
- Reduce the primary foundation settlements due to embankment loading from a 5- to 7-foot range without dynamic compaction to a 2.0- to 2.7-foot range for the worst case cross section.
- Increase the factor-of-safety against deep foundation failure by at least 15 percent to approximately 1.5.

DYNAMIC COMPACTION REQUIREMENTS

Dynamic compaction construction specifications and the criteria for evaluating the effectiveness of the work were developed by examining numerous case histories of similar projects compiled by Mayne, et.al. [Ref. 4], and by utilizing data and procedures proposed by Lukas in the 1984 Draft Dynamic Compaction Manual prepared for the Federal Highway Administration [Ref. 3]. Because of heterogeneous subsurface conditions, a prescriptive specification, rather than a performance specification, was prepared detailing the exact construction procedures to be followed. With the prescriptive specification any extra work, beyond that specified, required to achieve the desired end product would be paid on a time and materials basis. It was concluded that a performance type specification would have been unworkable in the field and unfair to the contractor given the extremely variable site conditions.

The construction specifications required:

- Three primary passes with print locations on a 25-foot grid pattern; alternating print locations with each pass. (Figure 4)
- A weight of 20 tons dropping from a height of 80 feet for primary passes to produce high energy impacts and ground improvement at depth.
- Dropping the weight 10 times at each print location.
- Releveling the site after each pass by bulldozing between prints.
- One ironing, or finishing, pass using an overlapping print pattern with reduced energy per drop to produce low energy impacts and improve ground near the surface which was disturbed by the primary passes and leveling.
- Compaction with conventional equipment after the ironing pass.

EVALUATION OF DYNAMIC COMPACTION WORK

Several methods were employed by the engineer to evaluate the effectiveness of the dynamic compaction work and to determine the need for additional work by the contractor; these included:

- Standard penetration tests and pressure-meter tests obtained in borings taken before and after dynamic compaction operations.
- Measurements of settlement induced by the dynamic compaction work.
- Evaluation of the depth of improvement after the dynamic compaction work.
- Measurement of crater volumes during dynamic compaction operations.

Case history data [Ref. 4] was used to establish the criteria to gage effectiveness, and dynamic compaction operations were considered

successful if all of the following were achieved:

- Standard penetration tests:

$$(N_{\text{final}}) \geq 2 (N_{\text{initial}})$$

$$(N_{\text{final}}) > 8$$

Pressuremeter modulus and limit pressure:

$$(E_{\text{final}}) \geq 2 (E_{\text{initial}})$$

$$(PL_{\text{final}}) \geq 2 (PL_{\text{initial}})$$

(Note: Only data from the upper 25 feet of the dump were considered; only the lowest two thirds of the measurements were used for comparison to discount the influence of bricks, concrete, etc.)

- Reduction in height of dump $\geq 15\%$
- Improvement to upper 25 feet of dump.

Additionally, print volumes were monitored during dynamic compaction construction to immediately identify soft foundation conditions where additional work would be required. During each pass, volume data were plotted on a plan view to identify prints where extreme variations (higher volumes) existed in the data. These anomalous prints were then treated by leveling and performing five additional weight drops before proceeding with the next pass. (Figure 4)

The data obtained to evaluate the dynamic compaction procedures indicated that the ground modification technique was generally effective. (Table 1) Inconsistent results were evident at some locations, however, due to the extreme variability in dump materials and in the depth of the dump.

FIELD INSTRUMENTATION

Upon completion of the dynamic compaction work, settlement platforms were installed on the existing ground surface at random locations (Figure 1) to monitor foundation settlements caused by embankment placement. The settlement data indicated high rates of settlement during and immediately after embankment construction, with the rate of settlement decreasing after completion of construction. (Figure 5) The magnitude of the measured settlements varied significantly between platforms, ranging from 0.5 to 1.3 feet and averaging 0.9 feet; this variation was attributed to the variations in the depth of dump material at each platform.

Slope inclinometers were also installed after the completion of the dynamic compaction work. These instruments were used to monitor possible lateral subsurface movements in the vicinity of the steep, existing slopes of dump material. (Figures 1 and 3) Although it was anticipated that the staged sequence of dynamic compaction would increase the strength of the dump material sufficiently to prevent a deep foundation failure caused by embankment loading, the effectiveness of this treatment was not certain.

Some of the slope inclinometers did show evidence of lateral foundation movements during and after embankment placement. (Figure 6) These movements may have been the results of buckling of the inclinometer casings during large distortion settlements, or may have been an early indication of slope instability. To ensure against the possibility of slope instability a small buttress, or berm, was placed against the embankment during the spring of 1987.

TABLE 1

Borings	SPT Navg.		PRESSUREMETER Eavg.				Height Reduction (%)
	Initial (bpf)	Final (bpf)	Initial (bars)	Final (bars)	Initial (bars)	Final (bars)	
4 & 4A	10	15	82	79	7.1	--	14
5 & 5A	6	37	75	318	5.5	>6.6	38
6 & 6A	9	14	--	--	--	--	82
7 & 7A	5	16	*	36	<0.2	>3.6	7
9 & 9A	8	40	19	201	<1.7	>6.3	14
10 & 10A	9	15	10	18	2.2	>4.7	35
11 & 11A	7	19	*	185	<0.2	>4.7	17
AVERAGE	--	--	--	--	--	--	21.6

*Too low to record.

CONCLUSIONS

Ground modification of the I-279 embankment foundation by the dynamic compaction technique was generally effective in densifying the dump material and resulted in a savings of \$1.8 million when compared to a fill and replacement scheme.

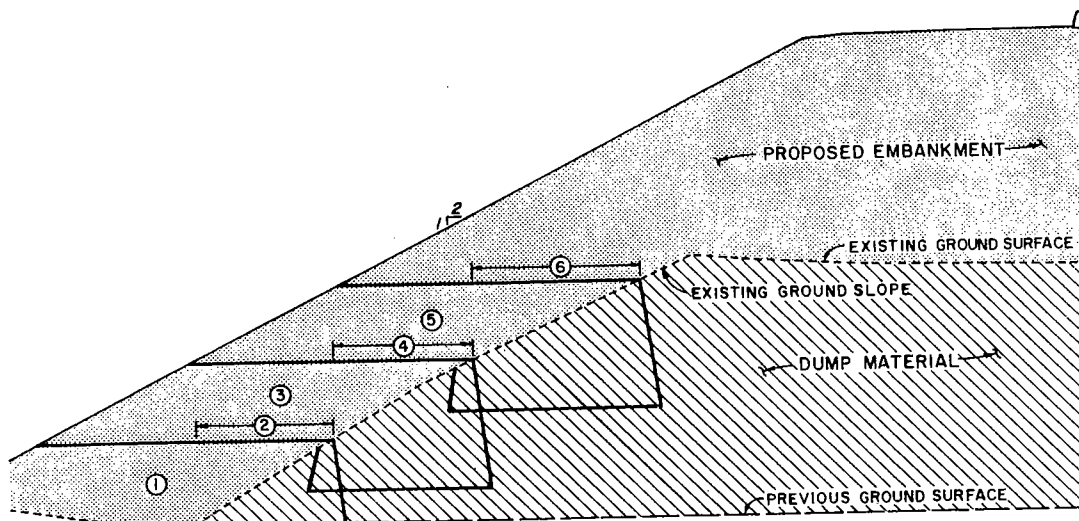
This ground modification technique can be used in areas with highly variable subsurface materials provided that the work is monitored by testing and observations, and that the construction specifications allow for additional densification where weak deposits are encountered. A prescriptive specification, rather than a performance specification, appears to be the best method for implementing dynamic compaction work in such areas.

ACKNOWLEDGEMENTS

The data presented in this paper were obtained during design, construction, and construction investigations for the Pennsylvania Department of Transportation. The writer gratefully acknowledges that agency for allowing the data to be published.

REFERENCES

- Gannett Fleming Corddry and Carpenter, Inc., "Supplemental Soils and Geological Engineering Report, Dump Area, LR 1021, Section 3D, Allegheny County", Commonwealth of Pennsylvania, Department of Transportation, May 1983.
- Gannett Fleming Corddry and Carpenter, Inc., "LR 1021, Section 3D, Allegheny County, Dynamic Compaction Evaluation - Dump Area, Final Report", Commonwealth of Pennsylvania, Department of Transportation, January 1986.
- Lukas, R. G., "Draft Dynamic Compaction Manual", Federal Highway Administration, Washington, DC, March 1984.
- Mayne, P. W., et.al., "Ground Response to Dynamic Compaction", Journal of Geotechnical Engineering, ASCE, Vol. 110, NO. 6, June 1984, pp. 757-774.



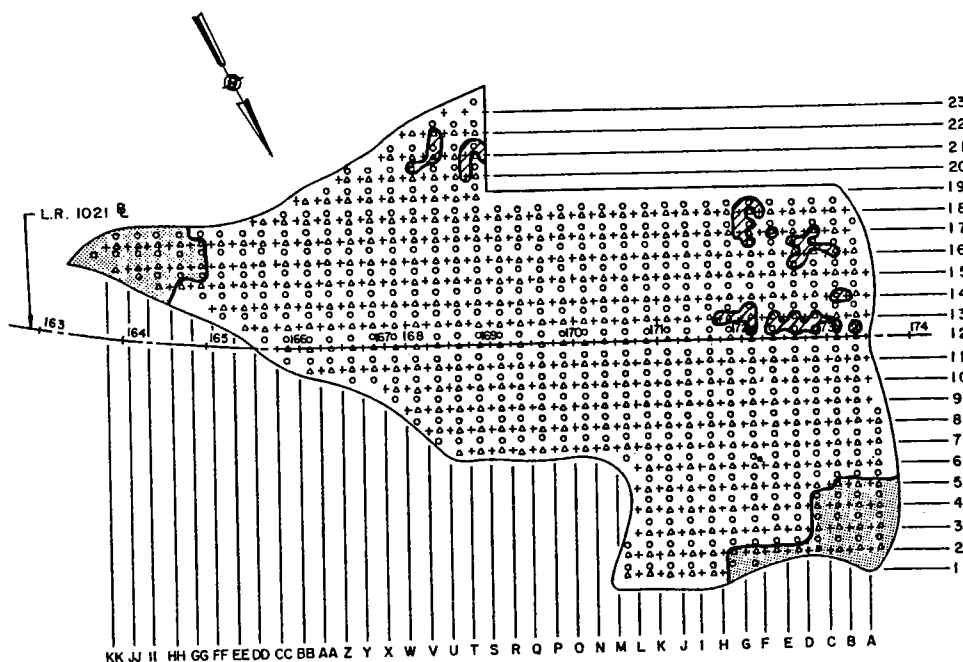
SEQUENCE OF OPERATIONS

- ① Construct Compacted Embankment
- ② Perform Dynamic Compaction On Embankment Surface
- ③ Construct Compacted Embankment
- ④ Perform Dynamic Compaction On Embankment Surface
- ⑤ Construct Compacted Embankment
- ⑥ Perform Dynamic Compaction On Embankment Surface

LEGEND

- Surface Limit Of Dynamic Compaction Work.
- ▭ Extent Of Densification Within Dump Material.

Figure 3 - Typical Cross Section of Dynamic Compaction Treatment on Existing Slopes



LEGEND

- + FIRST PASS PRINT LOCATION
- o SECOND PASS PRINT LOCATION
- x THIRD PASS PRINT LOCATION
- ▨ ANOMALOUS PRINTS (5 extra drops performed at these locations)
- ▩ PRINTS ELIMINATED (dump material not encountered)

Figure 4 - Dynamic Compaction Construction

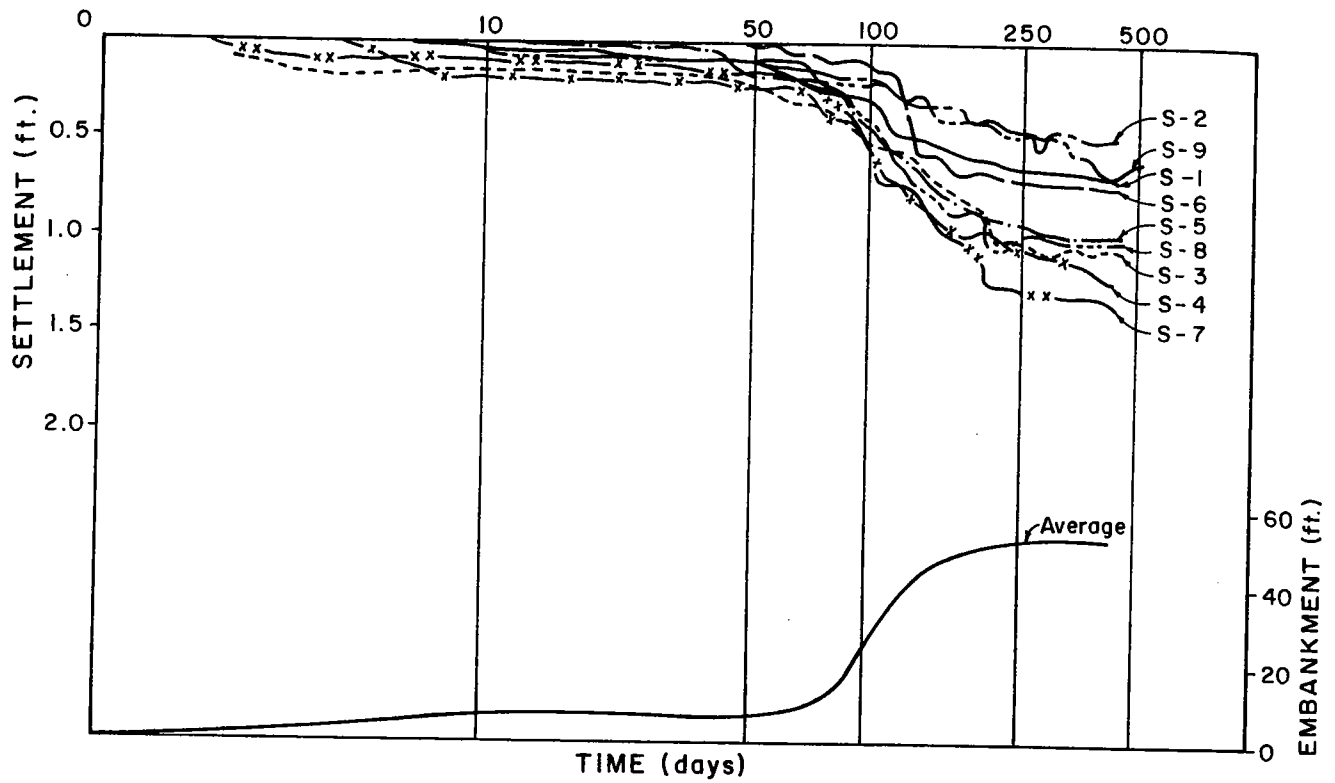


Figure 5 - Settlement Platform Plots

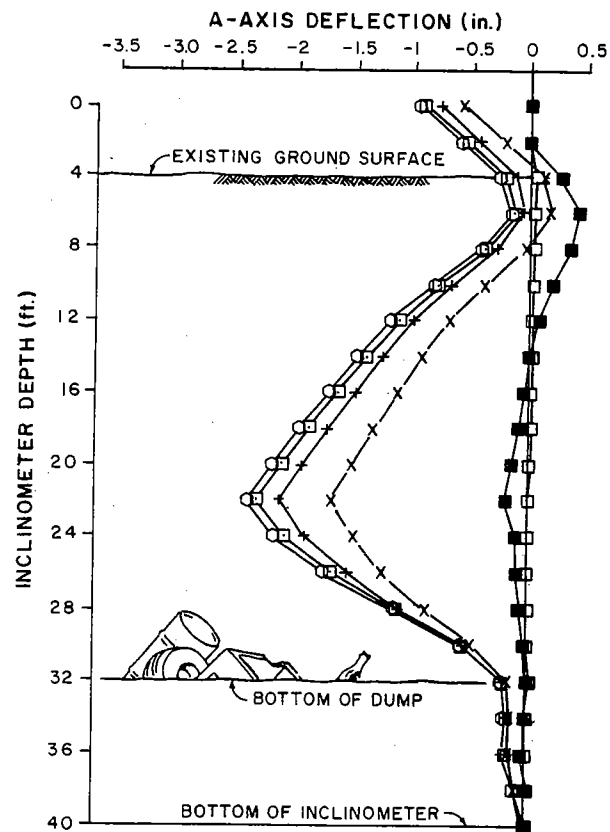


Figure 6 - Typical Slope Inclinator Movement

Cone-Penetrometer Exploration of Known Sinkholes

RONALD C. WILLIAMS

Williams & Associates

Clearwater, Florida

MYRON HAYDEN, D. BLOOMBERG and S. B. UPCHURCH

University of South Florida

Tampa, Florida

HGS-87-15

SINKHOLES RESULT IN MILLIONS of dollars of damage each year. The Standard Penetration Testing (SPT) is widely used to identify potential sinkhole occurrences. Although widely used, the data obtained from SPT are of limited value and difficult to apply to sinkhole mapping. A need exists for a quick, cost-effective method to delineate potential sinkholes and evaluate associated soil properties. The Electric Friction Cone Penetrometer (CPT) provides an excellent method for the preliminary evaluation of sinkholes.

Two known sinkholes with varying surficial expression were subjected to a detailed subsurface profiling by both SPT and CPT. The study was conducted on the campus of the University of South Florida, Tampa, an area of intense karstification.

The sinkholes were first investigated using conventional rotary wash borings coupled with SPT taken at 5 ft intervals. The samples obtained were visually classified in the field and later tested in the laboratory for grain size, Atterberg limits, and pH. An evaluation of the SPT data for the sinkholes investigated failed to produce any consistent indicators of potential sinkhole stratigraphy. When SPT values were coupled with the geotechnical properties of the soil some general trends were observed. However, these trends were inconclusive.

The data obtained from CPT were more detailed and informative than SPT data for evaluating soil properties conducive to the formation of sinkholes. This improvement results from the force measurements made along the sleeve of the cone. The friction ratio (ratio of sleeve resistance to tip resistance) was found to be a good indicator of the soil stratigraphy and its properties. By smoothing the friction ratio data, general stratigraphy and changes in soil properties were readily identified.

Near the main conduit of the sinkholes investigated, the loss of cohesive material resulted in a decrease in the sleeve friction

without a consistent reduction in tip resistance. The combination of these two factors resulted in friction ratios of almost zero around the conduits. According to previous research, very low friction ratios coupled with little cohesive material is an indicator of a potentially liquefiable soil. The occurrence of a liquefiable soil coupled with the loss of cohesive material may be a key to the identification of zones conducive to the formation of sinkholes.

INTRODUCTION

The formation of sinkholes cause millions of dollars in damage each year in the United States. Considerable research has been conducted to correlate soil stratigraphy with the formation of sinkholes; however, the exploration techniques used to delineate the stratigraphy are usually insufficient to identify the stratigraphy associated with sinkhole prone areas.

The Standard Penetration Test (SPT) is the most widely used method of identifying and evaluating stratigraphy associated with the formation of sinkholes. Because of the nature of the SPT, the data obtained are of limited value and difficult to apply to sinkhole mapping.

Based on the results of this study the Electric Friction Cone Penetrometer Test (CPT) is believed to be an exploration method superior to the SPT for the preliminary evaluation of sinkholes. The CPT was determined to be more sensitive to minor lithologic variability, in addition to providing a continuous subsurface profile with less disturbance.

STUDY AREA

The sinkholes studied were located on the campus of the University of South Florida in northern Hillsborough County, Florida. The terrain is underlain by paleokarst limestone which is known to contain many cavities. The

predominant types of sinkholes in this area have been classified as cover-collapse and cover-subsidence (1,4).

Underlying the site to a depth of six meters is a noncohesive fine sand which grades silty with depth. Below the sand is approximately four meters of dense clayey silt

with layers of sand interbedded in the clayey silt. Underlying the clayey silt is the paleokarst limestone. The surficial water table was located in this study at a depth of about 3 m which is within the upper sands. The limestone contains an artesian aquifer which has a phreatic surface below the surficial water table.

INVESTIGATION

The sinkholes examined in this study had been active in the past with well defined surficial expressions. The surface contours of the two sinkholes examined in this study are shown in Figures 1 and 2. As can be seen in these two Figures the CPT and SPT locations were aligned perpendicular to the axis of the sinkhole. Due to vegetation and topography one limb of Sinkhole 2 was inaccessible for testing. Measurements made on Sinkhole 3 during 1986 indicated that it is actively subsiding at a rate of 10 mm/month.

The sinkholes were initially investigated using conventional rotary wash borings coupled with SPT taken at 1.5 m (5 ft) intervals. The samples from the SPT were visually classified, sealed, and transported to the laboratory for water content, Atterberg limits, grain-size, specific gravity, pH, and organic content determinations. In addition all samples were classified according to the Unified classification system.

After the conventional rotary borings were made, the CPT exploration program was conducted. The tip resistance (Q_c), sleeve friction (F_s), friction ratio ($F_f = F_s/Q_c$), and inclinometer readings were recorded every 0.05 m (2 in) thus providing virtually continuous data acquisition and subsurface profiling (4). Prior to analysis the raw friction ratio values were smoothed using a smoothing routine to enhance the soil lithology (2).

RESULTS

Sinkhole 2 was identified as a cover-collapse sinkhole since the conduit was located during the SPT. As shown in Figure 3 the stratigraphy clearly dips toward Boring 2A which was believed to be in close proximity to the conduit. As can be seen in the Figure,

the N values were much lower in Boring 2A than in the surrounding soils.

Another criterion often used to identify potential sinkholes is the loss of drilling fluid. At this site all four borings

exhibited significant loss of drilling fluid. Had the SPT not been in the proximity of the conduit, the topographic feature may not have been positively identified as a sinkhole.

Sinkhole 3 appeared to be a sinkhole because of the distinct surface depression; however, a conduit was not encountered. As shown in Figure 4 the stratigraphy determined from rotary wash samples and SPT could not be positively identified as a sinkhole. The soil profile was similar in all four borings. The ground surface dipped gently toward Boring 3A which was made at the lowest point and presumed to be the conduit.

Thickening of noncohesive material and dipping strata following the topography are not conclusive evidence to positively identify a depression as a sinkhole. Without locating the conduit, conclusive identification of a sinkhole cannot be certain with only SPT data.

Sinkhole conduits typically contain a long column of predominantly noncohesive sand in a metastable state. Conduit materials of little cohesion produce very low F_s values with variable Q_c data. A decrease in F_s without a subsequent reduction in Q_c will result in a reduction in R_f . The R_f values of noncohesive conduit materials will be almost zero. According to Robertson and Campanella (3), very low friction ratios for noncohesive soils indicate a material which is potentially liquefiable. Unstable, noncohesive material is the key to identifying zones conducive to the formation of sinkholes.

The soil stratigraphy shown in Figure 5 is very similar to that shown in Figure 3 for the SPT. The conduit has been identified as being located in the proximity of Cone Penetration CPT2-4. The strata lines dip toward CPT2-5 with most of the clayey silt unit missing.

The stratigraphy shown in Figure 6 is different from that depicted in Figure 4. The profile in Figure 6 is more complete than Figure 4 and closer to known conditions. As can be seen in Figure 6, the conduit is close to Cone Penetration CPT3-9 which was only about 2 m from Boring 3A. The low R_f for the

sand is indicative of a loose potentially liquefiable sand.

The cohesive property of the soil is a key to identifying zones conducive to the formation of sinkholes. For this reason, CPT data are more informative than SPT in developing the soil stratigraphy and soil properties indicative of potential sinkhole formation.

Unless the conduit can be located, conclusive sinkhole potential cannot be certain with only SPT. CPT is more cost-effective than SPT and regardless of whether the conduit is encountered, CPT provides more information than SPT in locating potential sinkhole-prone areas.

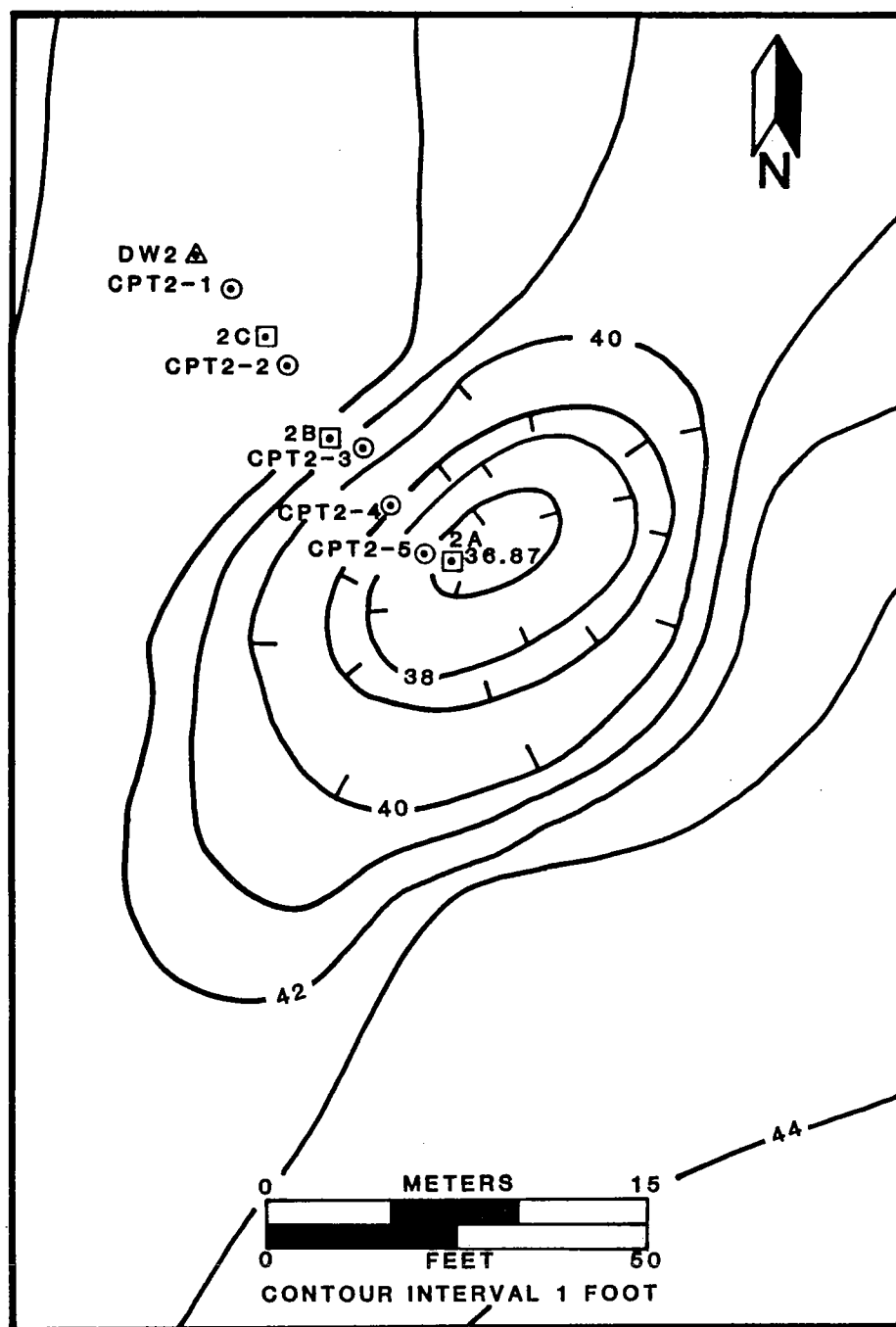


FIGURE 1. CPT and SPT locations at Sinkhole 2
(Datum - NGVD; Circle - Cone Penetration
Test; Square - Standard Penetration Test).

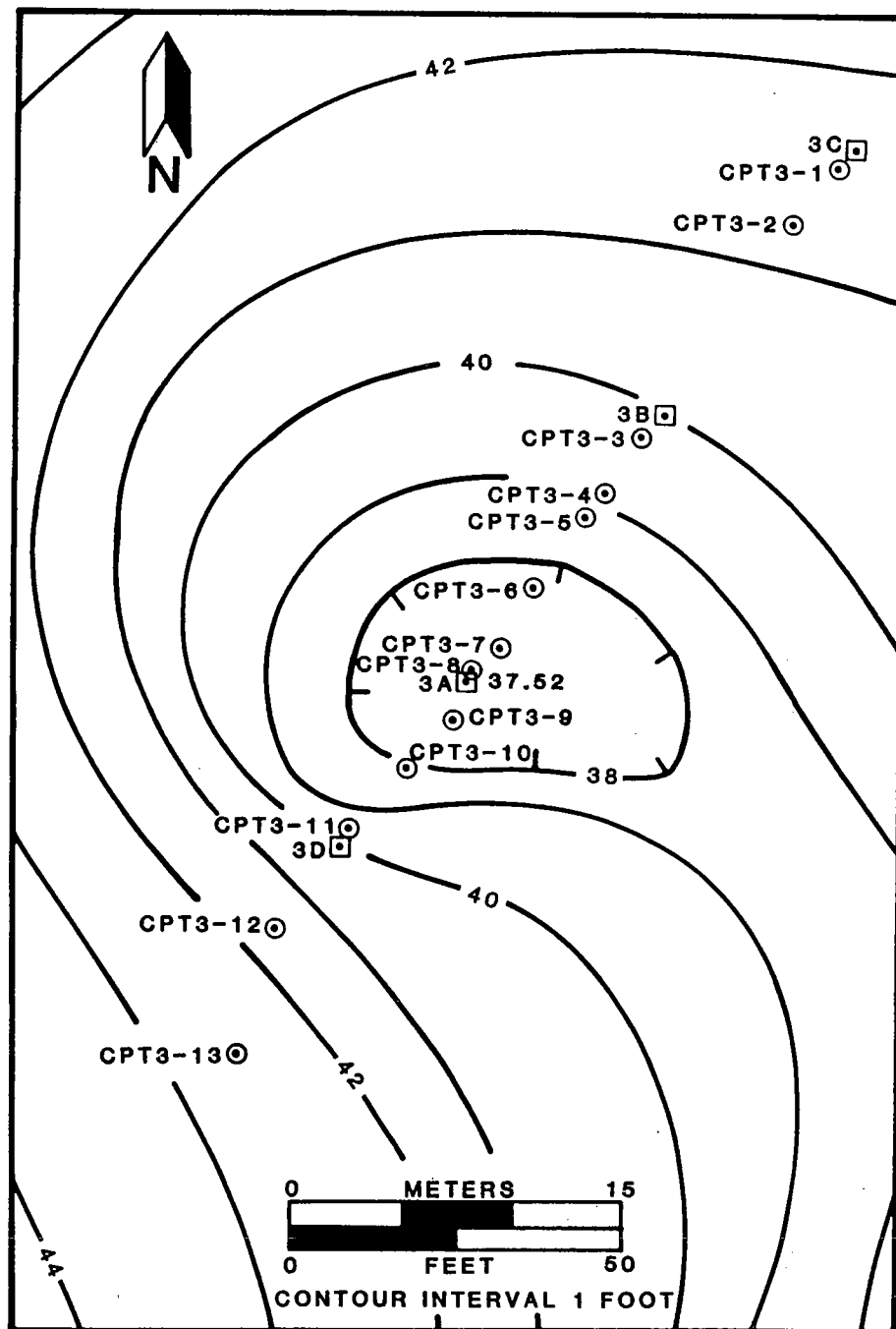


FIGURE 2. CPT and SPT locations at Sinkhole 3
(Datum - NGVD; Circle - Cone Penetration
Test, Square - Standard Penetration Test).

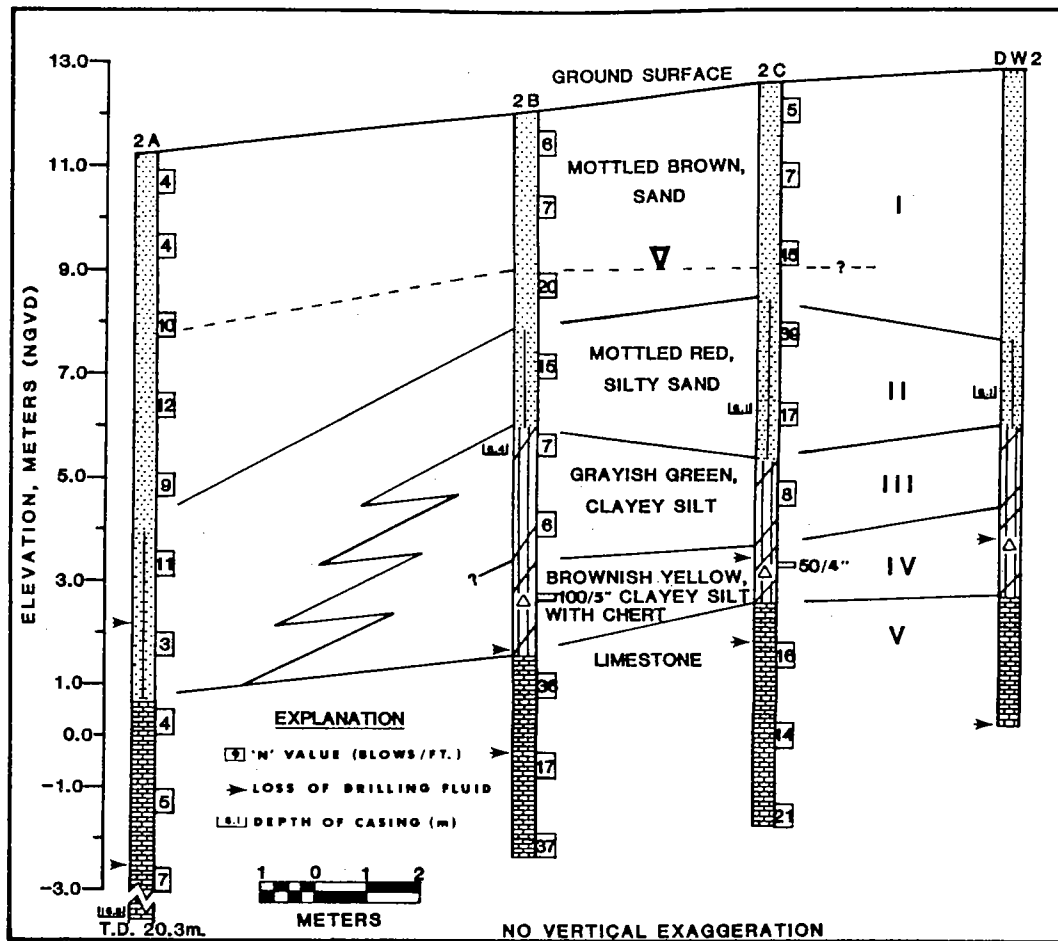


FIGURE 3. Sinkhole 2 cross section showing SPT borings.

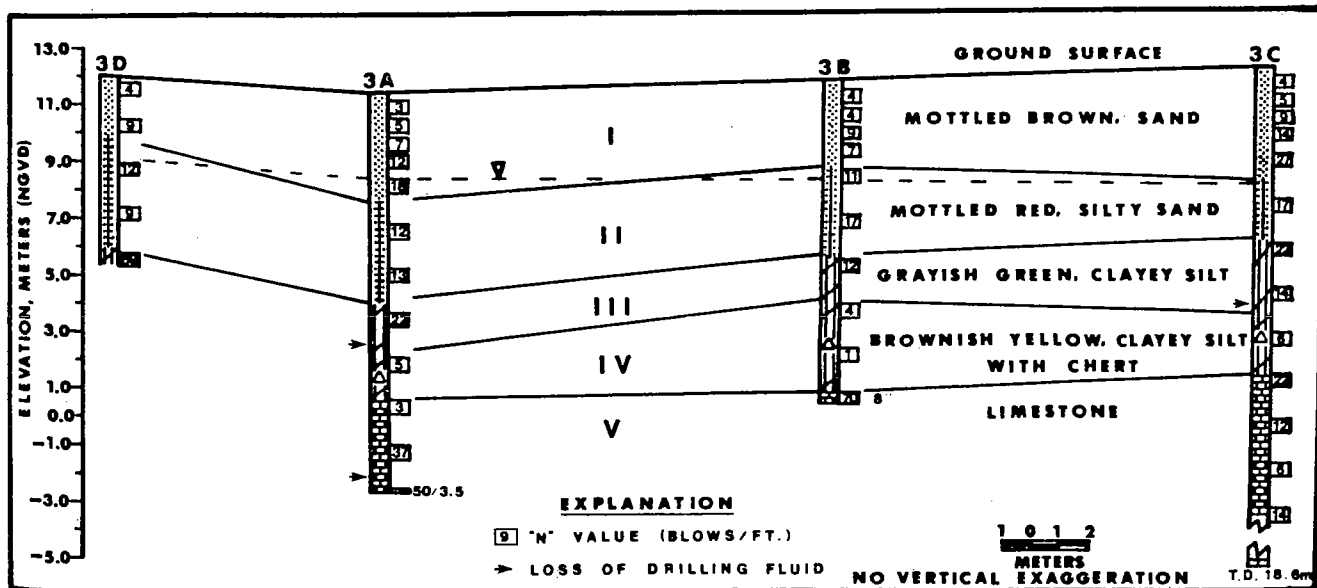


FIGURE 4. Sinkhole 3 cross section showing SPT borings.

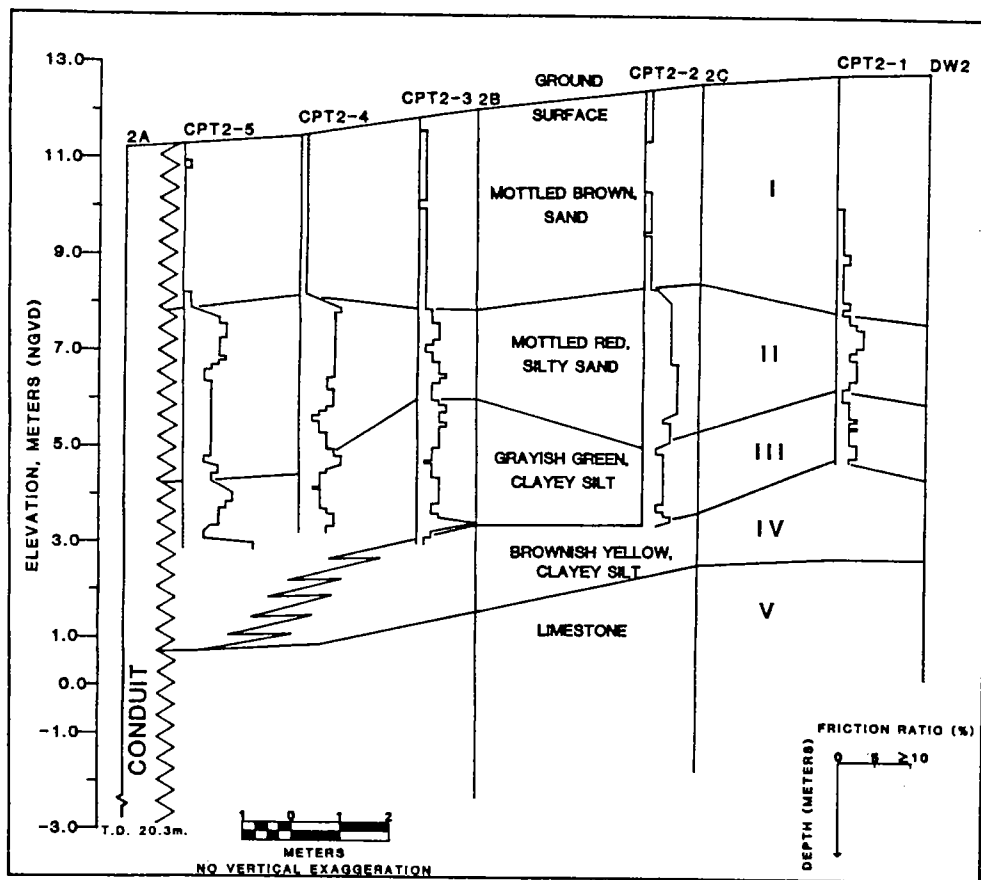


FIGURE 5. Sinkhole 2 cross section showing CPT soundings, SPT locations, and the conduit in relationship to surrounding sink fill.

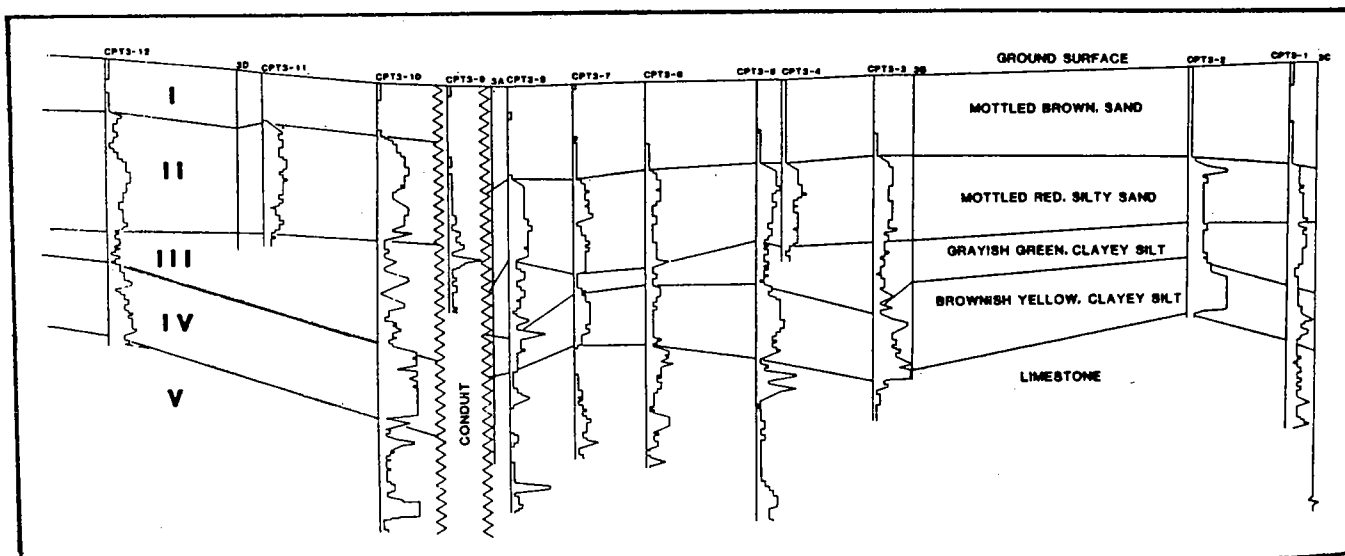


FIGURE 6. Sinkhole 3 cross section showing CPT soundings, SPT locations, and the conduit in relationship to surrounding sink fill.

CONCLUSIONS

Two sinkholes were subjected to a detailed subsurface profiling by both SPT and CPT. Although SPT is widely used to identify potential sinkholes, the information gained is difficult to apply to sinkhole mapping. CPT is an excellent method for preliminary evaluation of sinkholes. It is sensitive to minor lithologic variability and is superior to SPT as a cost-effective technique for determining general soil conditions and lithostratigraphy of sinkholes.

The conduit soils examined in this study indicated there was a downward movement of insoluble sediments with a subsequent formation of a long column of predominantly noncohesive sand. Lack of cohesion in the conduit soils caused a decrease in F_s without a subsequent reduction in Q_c . The combination of these two factors resulted in R_f values of almost zero indicating a potentially liquefiable soil. An occurrence of liquefiable soils and the loss of cohesive materials are the keys to identifying zones conducive to the formation of cover-collapse or cover-subsidence sinkholes.

REFERENCES

1. Bloomberg, D.L., S.B. Upchurch, M.L. Hayden, and R.C. Williams, "Cone-Penetrometer Exploration of Known Sinkholes," Proceedings of the Second MultiDisciplinary Conference of Sinkholes and the Environmental Impacts of Karst, Orlando, FL, Feb. 1987, pp 145-151.
2. Beck, B.F., and W.C. Sinclair, "Sinkholes in Florida: An Introduction," Report 85-86-4, The Florida Sinkhole Research Institute, Orlando, FL. 1986.
3. Robertson, P.K., and R.G. Campanella, Guidelines For Use and Interpretation of the Electronic Cone Penetration Test, Hogentogler and Co., Inc., Maryland, 1984.
4. Sinclair, W.C., and J.M. Stewart, "Sinkhole Type, Development, and Distribution in Florida," Map Series 110, Florida Bureau of Geology, 1985.

A Cut Slope Design for a Major Urban Highway in the Pittsburgh Area

JAMES M. SHEAHAN
HDR-Richardson Gordon, Inc.
Pittsburgh, Pennsylvania

HGS-87-16

INTRODUCTION

IN 1984, THE PENNSYLVANIA DEPARTMENT OF TRANSPORTATION entered into a design contract for the widening and reconstruction of a mile long section of State Route 28 located in Shaler Township, Allegheny County, Pennsylvania. This is between interchanges at Millvale and Etna, just outside the Pittsburgh city limits (Fig. 1). Initially constructed in about 1926, it currently carries traffic between the city and areas to the north or northeast. The daily traffic flow of about 50,000 vehicles consists mostly of commuter traffic during peak periods although off peak periods are also heavy. The roadway section consists of 4, undivided, 11-foot wide lanes between curbs. The alignment is tightly fitted between mainline Chessie tracks on the south, and a steep hillside on the north (Fig. 2). This stretch of roadway is well known for its accidents and traffic snarling breakdowns. It has also been plagued with rockfalls and landslides which have at times reduced traffic to one lane in each direction. This paper describes the process used in developing the slope design for the reconstruction project.

PROJECT OBJECTIVES

The principal design objectives for this project were:

1. To provide a safer roadway, with shoulders where possible.
2. To minimize maintenance of slopes or allow maintenance with little or no impact and interruption of traffic.
3. To provide an economical design with the first two objectives in mind.

STUDY SCOPE

The study included the following:

1. A review of available geologic data and aerial photography
2. A site reconnaissance
3. Test borings
4. Laboratory testing
5. A review of slope performance and maintenance practices on similar projects

PROJECT GEOLOGY

The site is located on the eastern flank of the Kellersburg Anticline which dips toward the McMurtry Syncline (Fig. 3) at about 10 feet per 1000. Geologic units encountered are from the Conemaugh Series, between the Connellsville sandstone which caps the hilltop and the Saltsburg sandstone which is at or below the base of the existing slopes (Fig. 4). Included among

these units are several formations which are well known for their contributions to landslides and rockfalls in the Pittsburgh area, notably the Pittsburgh "red bed" claystones and the Birmingham shales. A geologic profile was developed from all information obtained in the study and is shown in Fig. 5. Developmental processes for existing slopes were examined, and key strata such as the Ames Limestone and the Duquesne carbonaceous shale were identified as marker beds to delineate transitions from one major formation to another. A general description of formations on the project, given in descending order includes:

CONNELLSVILLE AND MORGANTOWN SANDSTONES - Fairly massive, medium to fine grained sandstones observed in exposures at the highest elevations on the slope. Interbedded shale layers may have allowed undercutting and rockfalls. No borings encountered this rock since it generally falls above the area being influenced by this design. The Clarksburg formation which usually separates these two sandstones was not found in outcrops.

WELLERSBURG CLAYSTONE - A medium hard, multi-colored claystone believed to be this unit was encountered in some borings. The character of this rock in borings varies from a claystone to a siltshale or clayshale. This unit may also contribute to undercutting of the overlying sandstones.

BIRMINGHAM SHALES - This unit lies between the Wellersburg and Duquesne formations. It is 40 to 50 feet thick and is primarily silty or sandy shale in the project area. Although individual beds may be a few feet thick, it is generally thinly bedded and heavily jointed. Major joint sets appear to be more perpendicular to the slope, but relief joints parallel to the valley are expected to be numerous. This

formation outcrops in the hillside over most of the project.

DUQUESNE COAL, SHALE and LIMESTONE - This unit lies at the base of the Birmingham shale. Its location is conspicuous on the project as a carbonaceous shale, underlain by a limestone and/or a claystone. The unit is generally 10 to 15 feet in total thickness and is especially important because it typically weathers and undercuts the Birmingham formation.

AMES LIMESTONE - This is a key marker bed due to its continuity throughout the project and its position just above the Pittsburgh "red beds." The interval of 20 to 30 feet between the Duquesne unit and the Ames is principally shaley although some claystone and sandstone were encountered.

PITTSBURGH "RED BED" CLAYSTONE - This formation is a very soft to medium hard, multi-colored claystone which is heavily jointed and slickensided. Some areas appear to be shaley, but it is principally a claystone. It is generally 25 to 35 feet thick and is notorious for its propensity to deteriorate and weather into clays which are prone to landslides.

BAKERSTOWN COAL - This stratum does not appear to be continuous over the site. In one boring, it seems to be represented by a carbonaceous shale about 45 feet below the Ames limestone. Another boring encountered a thin carbonaceous seam about 50 feet below the Ames. It was generally absent otherwise.

SALTSBURG SANDSTONE - This formation is divided into at least two parts. The upper section is located between the red beds and the Bakerstown Coal. It varies between a sandstone and a silty or sandy shale with some claystone. A second section is beneath the Bakerstown. Little information was obtained in this zone, but its quality appears to vary significantly between siltshales, sandy shales and sandstone with some claystones. A lower, more massive sandstone section was encountered in borings toward the Etna end of the project.

EXISTING SLOPE CONDITIONS

The development of the existing slopes in the project area has been heavily influenced by the intense weathering of weak siltshales, clayshales and claystones. More resistant strata which are interbedded with these formations are undercut until they fall due to the weight of the overhang or because a parallel joint is reached. The Birmingham formation is usually susceptible to this undercutting because it is so heavily jointed. Falls from this formation create talus which frequently breaks down and mixes with underlying clays in more mature slopes. The mature colluvium derived from this weathering process is therefore a mixture of clay and shale or rock fragments. The original excavation for SR 28, made in the 1920's, cut into the hillside and aggravated this natural balance which probably existed, causing continuous maintenance since then.

Conditions in the existing slopes can be represented by five typical sections.

STATION 194 TO 200 (Fig. 6) - This area was graded in the 1960's to accommodate a ramp for the Millvale interchange. A bench was designed about 15 feet above grade. Weak claystone from the Pittsburgh red beds has weathered in the lower section and small rocks from overlying shales, occasionally roll onto the sidewalk or pavement. Cut slopes above the Ames are graded at a 3/4:1.

STATION 200 TO 210 (Fig. 7) - The Ames limestone is about 40 to 55 feet above grade through this area placing the red beds near the base of the slope. The original excavation for SR 28 removed most of the overburden leaving weak claystone and shales exposed at the base of a near vertical slope. Talus up to about 20 feet thick has accumulated at the base of the slope in some areas. Slides have occurred between station 200 and 202 in past years, spilling onto the highway. As weathering continues and the Birmingham shales are undercut, similar rockfalls are anticipated. Several homes and an electrical transmission tower are located at the top of the slope.

STATION 210 TO 218 (Fig. 8) - Conditions in this area are similar to the previous except that it contains a relatively flat plateau 60 to 80 feet wide on which several homes have been built. Overburden is as much as 30 feet in thickness and groundwater levels are high. There are no signs of past landslide activity in the area.

STATION 218 TO 232 (Fig. 9) - The Ames limestone is 60 to 80 feet above grade in this area. Formations of slightly more resistant rock are found at the base of the slope adjacent to SR 28. Weathering of the red beds as well as the Duquesne formation and undercutting of the Birmingham shales has created masses of colluvial soil and talus 10 to 20 feet thick which are perched atop the more resistant formations. Two major slides have occurred in the area in the past 5 years as portions of the colluvial material has fallen onto the roadway. The slopes between stations 227 and 232 are flatter and appear to be generally more stable although there are signs of creep. Several homes are located near the top of the slope between stations 222 and 227.

STATION 232 TO 242 (Fig. 10) - A significant transition occurs in this area. The depth of overburden increases, reaching a maximum of about 50 feet at station 242. Soils are red-brown silty clays containing shale fragments and boulders. Groundwater levels are erratic and isolated wet zones are typical. Several spring boxes in the area appear to pick up water flowing through the Birmingham formation at higher elevations. This area represents a typical fully developed colluvial slope.

REVIEW OF SLOPE PERFORMANCE ON SIMILAR PROJECTS

As part of this study, a review was made of slope performance on projects having cuts in the

same or similar formations. Four of these are described here:

SR 28, ETNA TO SHARPSBURG (Fig. 11) - This cut is about 0.5 miles north of the project on the other side of the Etna interchange. It was completed over 20 years ago. It illustrates the use of a maintenance bench at a higher elevation above the roadway. This and other similar benches in the area collect fall material regularly and are periodically cleaned, but the cleaning process is costly because equipment must remove and carry debris to trucks below. If the fall area is not maintained, the higher bench may act as a chute which propels material onto the roadway.

SR 28, RIDC TO HARMARVILLE (Fig. 12) - This section of SR 28 is approximately 7 miles northeast of the project. Cuts in this area are several hundred feet high and slopes have been completed for at least 10 years. The cut most closely observed contains a deposit of red claystone, believed to be the Pittsburgh red beds. The Ames limestone which overlies this claystone is located 70 feet above grade. About 20 feet above this is the base of the Birmingham formation. A wide, flat drop zone was constructed at the base of the slope and double sided guiderail was installed at the shoulder. Slopes below the limestone vary between 1-1/4:1 and 1-1/2:1, while slopes above are near vertical. The red beds have continuously weathered while numerous rocks of varying sizes have fallen from the slope. Most collect in the drop area but some have impacted the guiderail or have occasionally reached to the roadway. Periodic maintenance is required to clean up collected material.

SR 28, HARWICK TO CREIGHTON (Fig. 13) - This cut is located about 12 miles northeast of the project. Slopes were completed within the past 5 years. A steep roadway grade has exposed what are believed to be the same sequence of formations encountered on the project. A limestone, believed to be the Ames, is underlain by 25 feet of red claystone while the base of the Birmingham shale is 30 feet above the limestone. By survey, it appears that a 1:1 slope was designed on this cut. The standard safety zone and shoulder width have been provided without the use of guiderail.

Both sides of the cut have weathered although the east slope has progressed much more quickly, probably due to the presence of more water in the slope. Weathering is intensive in the claystone and undercutting has reached joints in the limestone causing blocks to fall. Smaller fragments have also raveled from the higher shale units. Most, but not all, of the fall debris is retained within the safety area at the base of the cut. To provide additional protection, maintenance forces have created a berm at the base of the cut using weathered material.

I-376, PARKWAY CENTRAL (Fig. 14) - This slope is located close to the heart of Pittsburgh and separates the Boulevard of the Allies from the Parkway Central which is about 100 feet

below. The Birmingham shales stand near vertical for 50 feet just beneath the Boulevard while the underlying shales and claystones slope at about 1:1 to the Parkway. The Ames limestone is located 15 to 20 feet above the Parkway. In the past, differential weathering of the Duquesne formation has undercut the Birmingham formation causing rockfalls. Some have been relatively large and have spilled onto the Parkway. Since continued undercutting also threatened the Boulevard, measures were taken to stabilize the Duquesne unit with anchored, concrete wall facing. Overhangs were scaled, and a limited drop area was created at the base of the slope. Wire mesh was applied to some areas of the shale. This work has been completed for almost 2 years and has performed well to date.

SLOPE DESIGN METHODOLOGY ON PROJECT

A key requirement for design of the widened roadway section was holding the curbline adjacent to the railroad. This created a significant impact on the existing hillside. Cut slopes in soil were not considered feasible in the area to station 232; therefore, a rock cut design which removed all overburden below the Birmingham formation was used. In fully developed soil slopes beyond station 232, widening did not impact slopes as significantly and cuts were kept to less than about five feet in depth.

The entire zone of rock from the Duquesne unit through the Pittsburgh claystone and the immediately underlying units is considered highly susceptible to weathering. The overlying Birmingham formation, although slightly more resistant to weathering, will also break down due to the closely spaced joints, thin bedding layers and fracturing. Flatter slopes of at least 1-1/2:1 would create a stable slope with little or no falls, but this was considered impractical due to the tremendous quantity of earthwork required and the impact on homes and property at the top of the slope. A cut slope of 1:1 was ultimately selected for all rock since it is anticipated that the rate of weathering could be reduced to manageable levels. Where possible, this 1:1 slope was set to daylight at the base of the Birmingham unit. The existing slope in the Birmingham unit averages 1:1, and the design allows for removal of overhangs during construction.

A drop area was recommended at the base of the cut slopes. Design of this drop area was based on a concept developed in the State of Washington and has therefore been referred to as the Washington Ditch (Fig. 15). This ditch is intended to not only provide a catch area but also to contain rock that might otherwise escape to the roadway. The design was modified slightly to use a minimum 4 foot deep ditch which increases to 6 feet for slope heights over 50 feet high. A guiderail with a channel (or rub rail) was placed at the edge of shoulder to provide additional protection against possible escape of rockfalls to the shoulder. When rock is exposed in the base of the ditch, a 12-inch

undercut, backfilled with suitable material, will be provided as a cushion for rockfalls.

APPLICATION TO THE PROJECT

Figure 16 illustrates the typical section used on the project. Application of this section to roadway design required some variations. Figures 17 through 21 illustrate some representative sections. The ramp at Millvale (Station 194 to 199) will not be excavated at all. Between stations 199 and 204, a wall will be constructed at the base of the slope. It will be high enough to cover the red bed formation and provide a drop zone behind the wall. Initial designs had considered a grade level ditch with the wall placed further up the 1:1 cut slope but access for construction would have been very difficult. Figure 18 represents sections between Station 204 and 212 and illustrates the application of the 1:1 cut slope, daylighting at the Duquesne carbonaceous shale layer, with the design drop area. The area from station 212 to 218 is represented in Figure 19. All soil will be removed and an intermediate bench has been designed to minimize rock excavation. Where used, intermediate benches are accessible, but maintenance would be optional. From Station 218 to 222 the cut is similar to Figure 18 except the width of the drop area is sometimes greater than design minimum so the 1:1 cut slope meets the Duquesne unit. The cut between stations 222 and 226 requires a wall to prevent taking homes at the top of the hillside. The wall will be placed near the base of the Birmingham shale (Fig. 20).

Figure 21 illustrates the use of an intermediate bench in the area between stations 226 and 233. A transition soil cut slope of 2 1/2:1 is used thereafter to the end of the cut at station 234.

Construction is expected to begin on the project in 1987 with an estimated total cost, including walls, of \$18 million.

REFERENCES

- (1) Hamel, J. V., "Geology and Slope Stability in Western Pennsylvania," ASCE Spring Meeting, Pittsburgh, PA, April 24-28, 1978
- (2) Johnson, M. E., "Pittsburgh Quadrangle," Atlas of Pennsylvania No. 27, Pennsylvania Geological Survey, 4th series, 1929
- (3) Piteau, D. R. and Peckover, F. L., "Engineering of Rock Slopes," Chapter 9 of Transportation Research Board Special Report 176, "Landslides, Analysis and Control," 1978, Pages 192-228
- (4) Ritchie, A. M., "Evaluation of Rockfall and Its Control," Highway Research Board, Highway Research Record No. 17, 1963, Pages 13-28

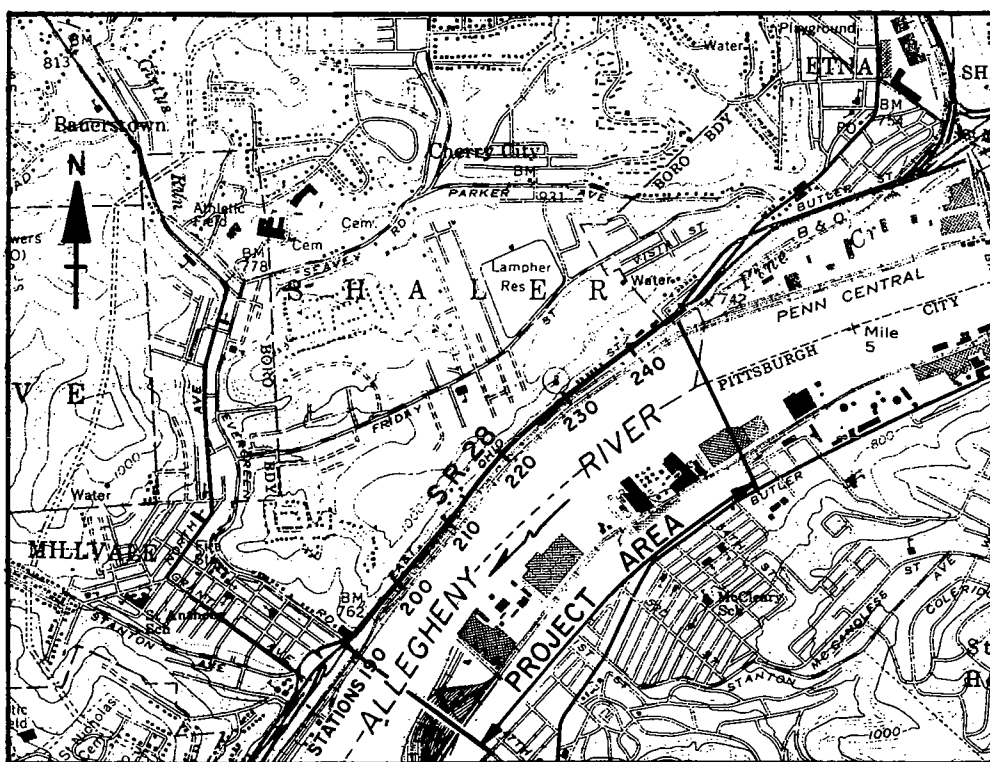


FIG. 1 - PROJECT PLAN, SR 28
(PGH. EAST USGS MAP - PHOTO REVISED 1969)

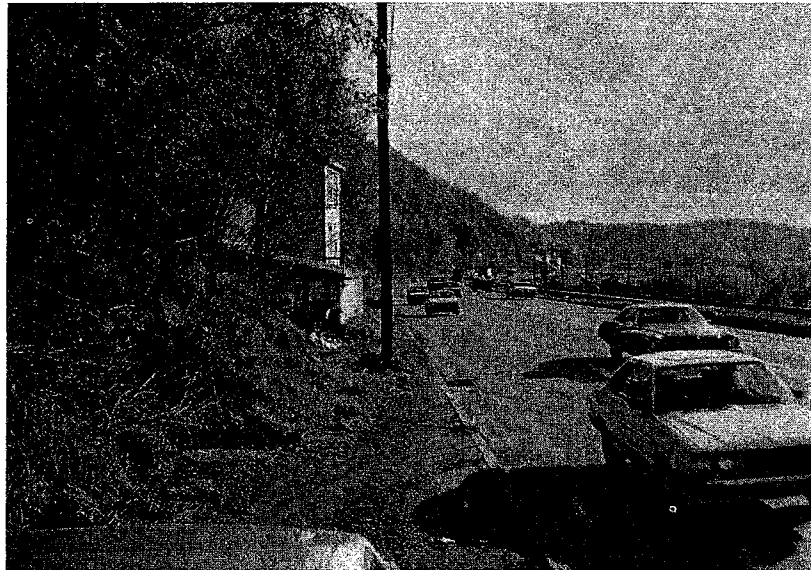


FIG. 2 - SR 28, PROJECT LOOKING TOWARD ETNA

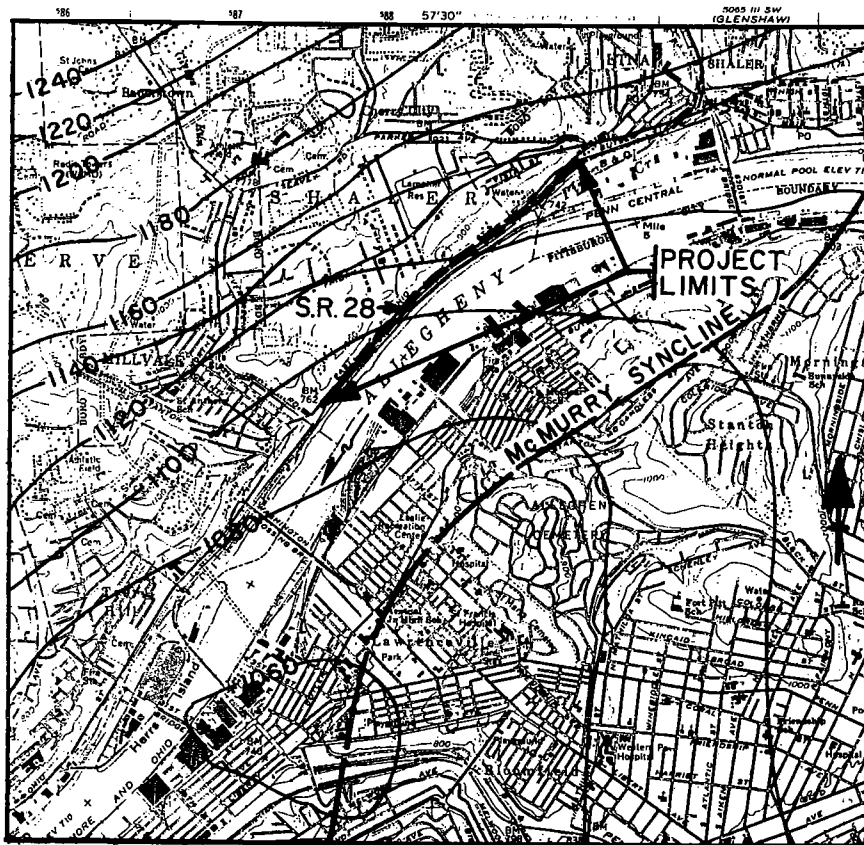


FIG. 3 - GEOLOGIC STRUCTURE MAP OF PROJECT AREA. CONTOURS SHOWN ARE ON PITTSBURGH COAL SEAM WHICH IS ABSENT IN THE AREA (i.e., ELEVATION ABOVE EXISTING GROUND)

PENNSYLVANIAN PERIOD		
SERIES	FORMATION OR UNIT	THICKNESS
MONONGAHELA	PITTSBURGH COAL	
CONEMAUGH	LOWER PGH. SANDSTONE	630±
	LITTLE PGH. COAL	
	LOWER PGH. LS & SH	
	CONNELLSVILLE SS	
	CLARKSBURG COAL,	
	CLAY & SS	
	MORGANTOWN SS	
	WELLERSBURG COAL &	
	CLAY	
	BIRMINGHAM SH & SS	
ALLEGHENY	DUQUESNE COAL, CLAY	
	& LS	
	AMES LS	
	HARLEM COAL	
	PGH. RED BEDS	
	SALTSBURG SS	
	BAKERSTOWN COAL	
	WOODS RUN LS	
	PINE CREEK LS	
	BRUSH CREEK LS & COAL	
ALLEGHENY	MAHONING SS, COAL,	
	SH & CLAY	
ALLEGHENY	UPPER FREEPORT COAL	

FIG. 4 - GENERALIZED GEOLOGIC COLUMN ALLEGHENY COUNTY

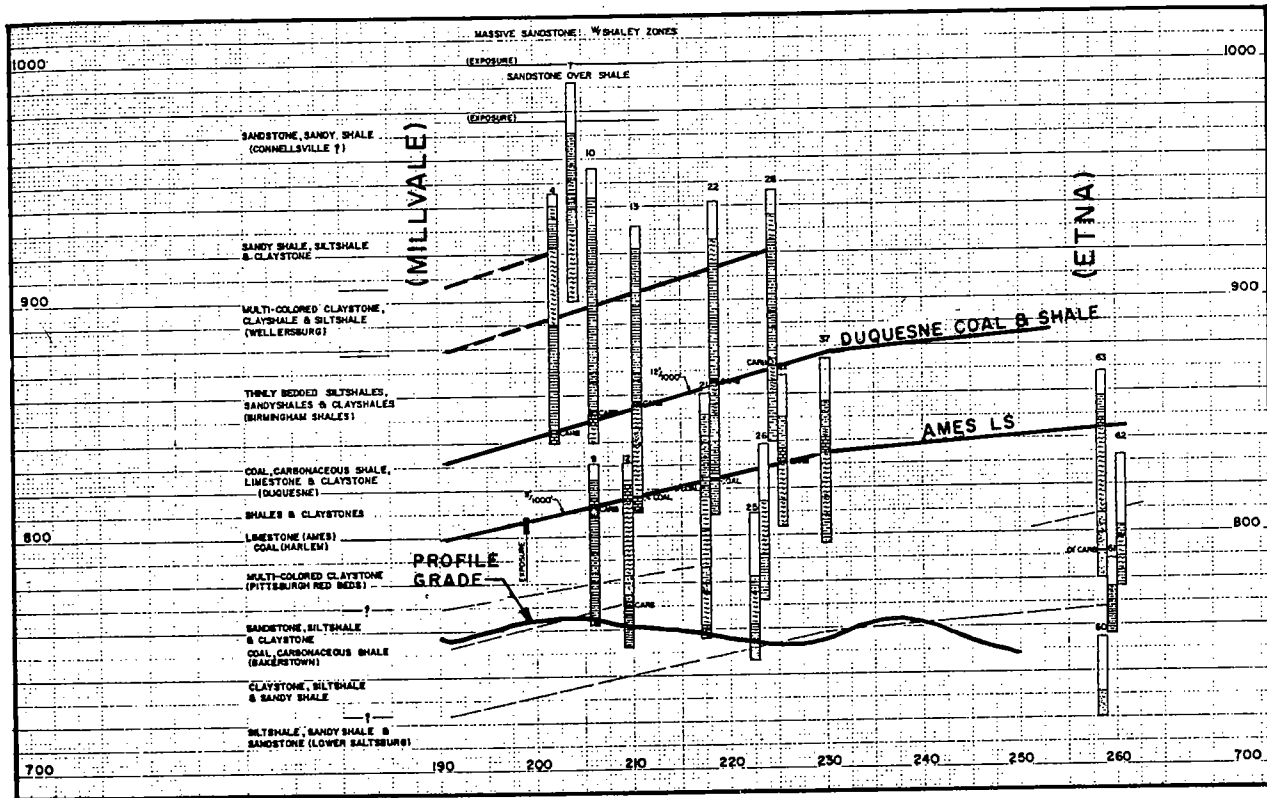


FIG. 5 - PROJECT GEOLOGIC PROFILE (WITH SELECTED BORINGS)

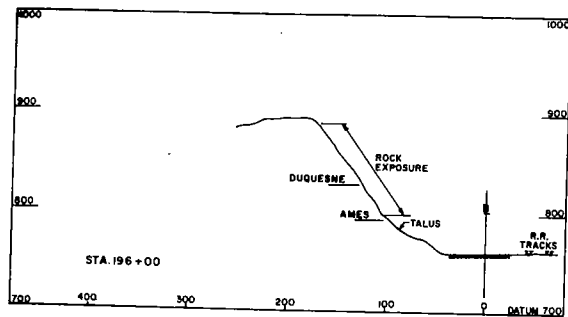


FIG. 6

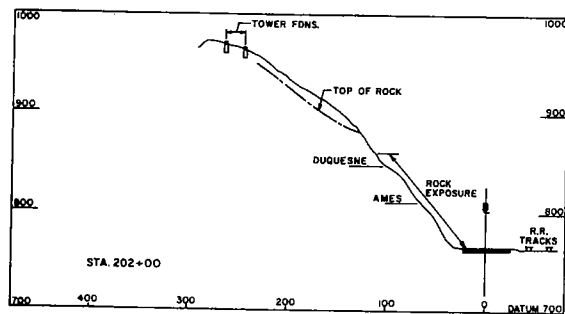


FIG. 7

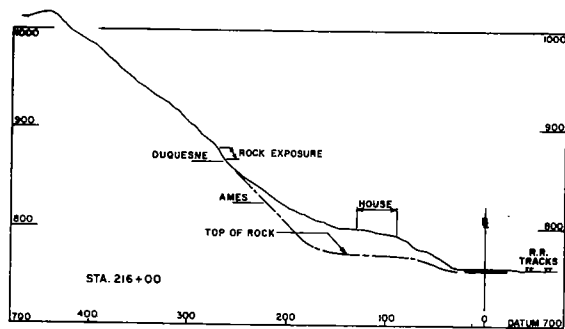


FIG. 8

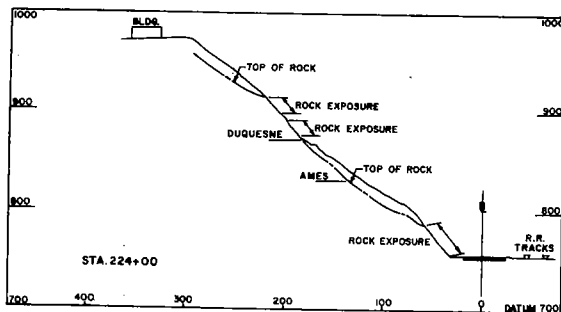


FIG. 9

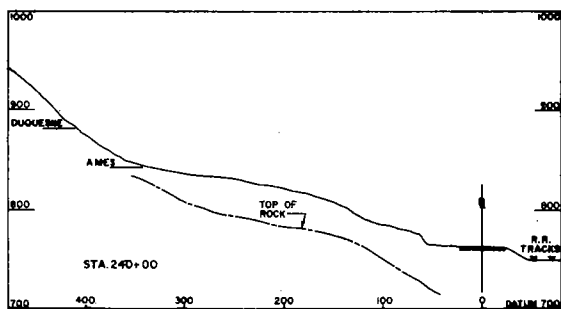


FIG. 10

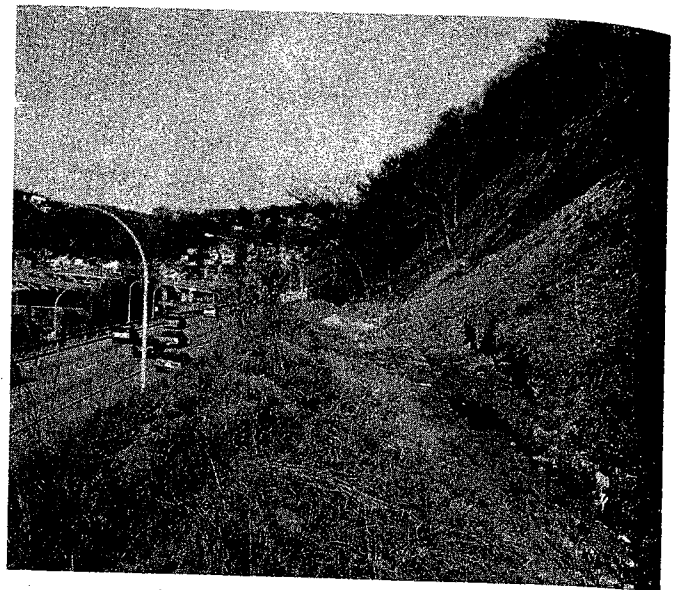


FIG. 11 - ETNA TO SHARPSBURG

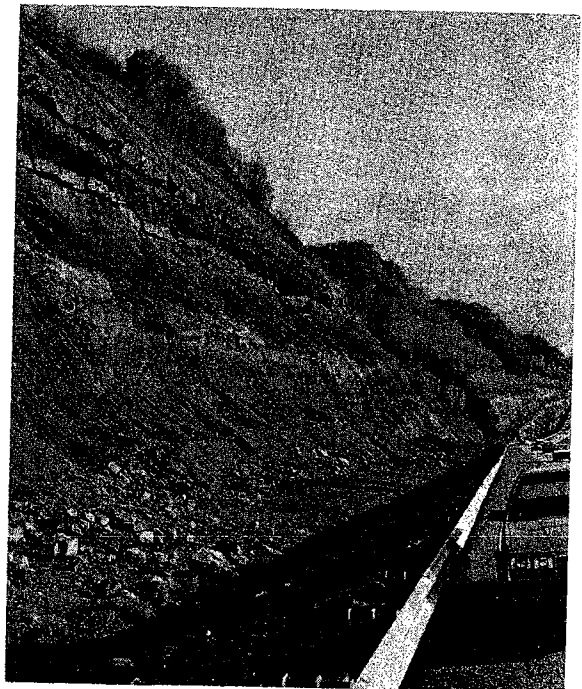
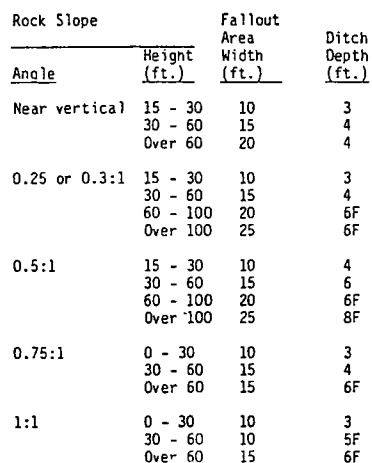
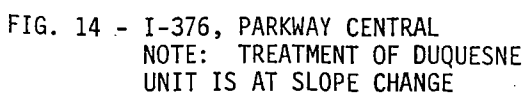
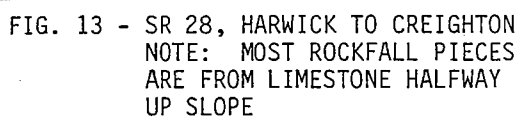


FIG. 12 - SR 28, RIDC TO HARMARVILLE



F = Ditch Depth "D" may be 4' if catch fence is used.

FIG. 15 - WASHINGTON DITCH CRITERIA

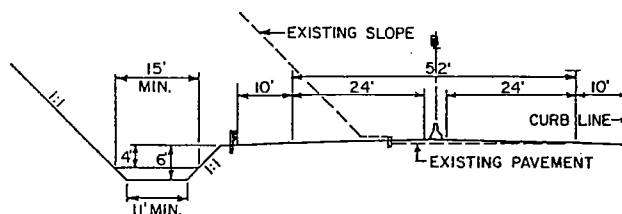


FIG. 16 - TYPICAL CUT SECTION WITH
MODIFIED WASHINGTON DITCH

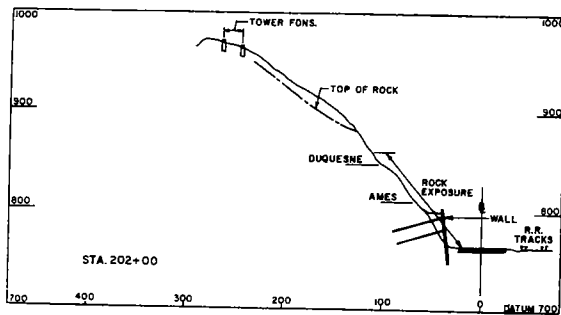


FIG. 17
STATION 202 WITH WALL

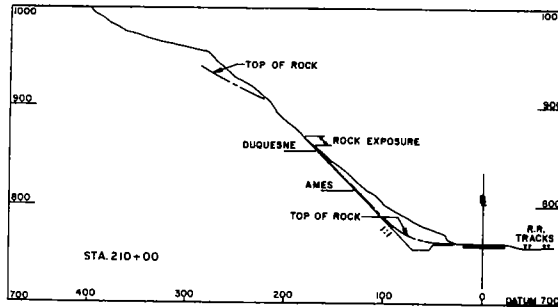


FIG. 18
STATION 210 WITH CUT
SLOPE AND DITCH

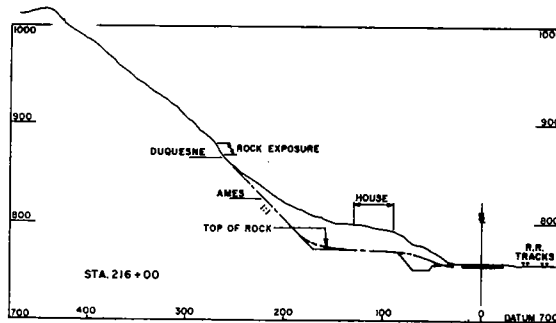


FIG. 19
STATION 216 WITH CUT
SLOPE, BENCH AND
WASHINGTON DITCH

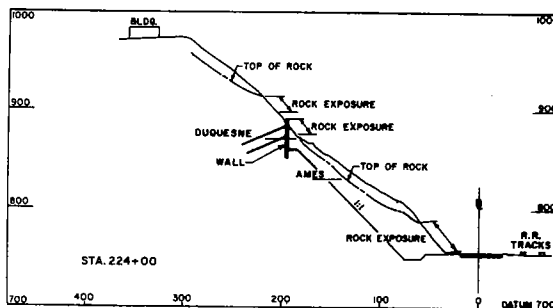


FIG. 20
STATION 224 WITH CUT
SLOPE, WALL AND
WASHINGTON DITCH

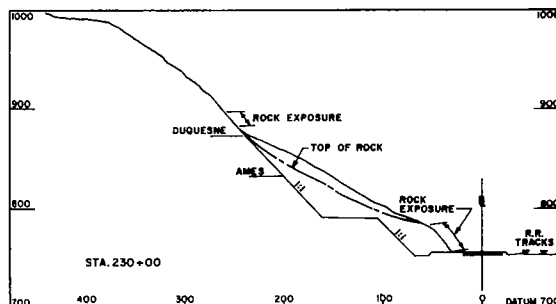


FIG. 21 -
SECTION 230 WITH
INTERMEDIATE BENCH

Karst Vs. Highway Ditchlines in East Tennessee

HARRY L. MOORE

Tennessee Department of Transportation
Knoxville, Tennessee

HGS-87-17

ABSTRACT

THE CONSTRUCTION AND MAINTENANCE of highways in karst areas of East Tennessee has resulted in the occurrence and treatment of induced sinkholes along the ditchlines of numerous highways.

Due to the geologic conditions prevalent in the Ridge and Valley province, "belts" of karst areas are frequently crossed by transportation corridors. Roadways constructed in these karst areas have repeatedly resulted in the development of sinkholes along the ditchlines. The collapse features occur where the pinnacle-soil interface is breached by the ditchline excavation and the resultant ditchline is "untreated".

Treatment of the sinkholes usually involves "plugging" of the collapse with chunk rock backfill and treating the ditchline with an impervious material. Geotechnical input into the transportation planning, design, and construction has resulted in the treatment of the ditchline (at the design phase) in areas where highways are constructed across terrain prone to karst development.

INTRODUCTION

East Tennessee being underlain by broad belts of folded carbonate rock contains large areas of karst. The construction of highways in the East Tennessee region has resulted in numerous karst encounters of the induced kind. Roads and bridges that are built across "belts" of active karst increase the incidence of induced sinkholes and subsidence.

The greatest number of karst problems experienced along highways in East Tennessee involves subsidence and collapse of the ditchlines (Figure 1). The occurrence of these karst

problems are referred to as being "induced" as opposed to being naturally developed (Newton, 1981).

Karst problems along Tennessee highways have been described by Royster (1984), and Moore (1981, 1984, 1987). In addition Sowers (1976), Newton (1976), Foote, et al (1979), and Amari and Moore (1985) have detailed possible geotechnical solutions to karst problems in the Ridge and Valley Province from Alabama to Pennsylvania.

The present study involves the analysis of 72 karst related subsidence and collapse problems experienced along highways in East Tennessee over the last ten years. The data collected in this study indicates that of the 72 sinkholes researched 85% were "induced", while 15% were considered "natural" (Figure 2). The most important result of this study was the revelation that 74% of the karst problems occurred in roadway ditchlines. The remaining 26% occurred in roadway subgrades and in areas unrelated to highway facilities (fields, yards, woods) (11% and 15% respectively).

The majority (93%) of the ditchline problems studied occurred along untreated roadway ditches. Untreated ditches are defined in this study as being standard roadway drainage ditches which are constructed without the benefit of pavement or other

impervious materials.

The major emphasis of this study is to describe the problems of "induced" sinkhole collapse and subsidence along untreated roadway ditchlines in East Tennessee.

KARST AREAS OF EAST TENNESSEE

East Tennessee is divided into three physiographic provinces: Blue Ridge, Ridge and Valley, and the Cumberland Plateau section of the Appalachian Highlands Plateau (Figure 3). The preponderance of sinkhole related problems in East Tennessee are located within the Ridge and Valley province where the sedimentary carbonate strata are complexly folded and faulted. Areas of karst have developed in elongated "belts" corresponding to the regional strike of the bedrock.

In some instances these "belts" are very long and narrow especially where strata of the Conasauga and Chickamauga groups are located. Very large broad areas of karst are developed where the subsurface consist of slightly dipping, thick units of carbonate rock (particularly the Knox Group).

Along the Blue Ridge province, karst areas are confined to isolated coves or valleys where erosion down through the Great Smoky thrust sheet has exposed Cambrian and Ordovician age carbonates. Examples of these would be Cades Cove (in the Great Smoky Mountain National Park) and Tuckaleechee Cove at Townsend, Tennessee.

Karst areas in the Ridge and Valley province are randomly spaced throughout with a higher concentration of sinkholes being located in the middle and northwestern portions. This corresponds to the abundant outcrop areas of carbonate rock types found in East Tennessee.

The karst areas located on the Cumberland Plateau are generally restricted to the Plateau escarpments

(east and west) and to the Grassy Cove and Sequatchee Valley areas. Numerous carbonate strata are exposed along the escarpments where vertical relief and extensive weathering have contributed to the development of sinkholes and caves.

SINKHOLE DEVELOPMENT IN ROADWAY DITCHES

The treatment of ditchline in karst areas is a relatively new concept. In most highway design situations when roadway ditches have a gradient of 3% or less then those ditches are designed to be left barren or sodded. Only when the ditchline gradient exceeds 3% are the ditches designed to be paved or treated with an impervious material (i. e. concrete, polyethelyene, geofabrics).

In areas of karst the problem with untreated ditches often results when run-off accumulates in the ditchline and percolation into the underlying soil/rock interface develops. As the surface water migrates downward, clay particles are taken from the soil/rock interface and carried into existing solution channels in the top of rock. Over a variable period of time a cavity develops in the soil mass (above top of rock) which enlarges from the top of bedrock toward the surface. When the soil cavity enlarges to a point where the overlying soil cannot "bridge" the cavity, then the soil mass collapses into the underlying cavity and results in a surface depression or sinkhole (Figure 4). The mode of induced sinkhole failure has been previously described by Donaldson (1963), Jennings, et al (1965), Moore (1981) and Newton (1984).

Most of the collapse features analyzed in this study developed openings from 1 to 5 meters (3 to 15 feet) in diameter and up to 6.7 meters (20 feet) deep. Characteristically the induced sinkholes expose pinnacles of bedrock within the throat or walls of the failure.

Paved or treated ditchlines were found to be associated with only 6.9% of the induced collapses. Results of the study indicate that the majority of the ditchline induced sinkholes (93.1%) developed along untreated ditches (Figure 5). The

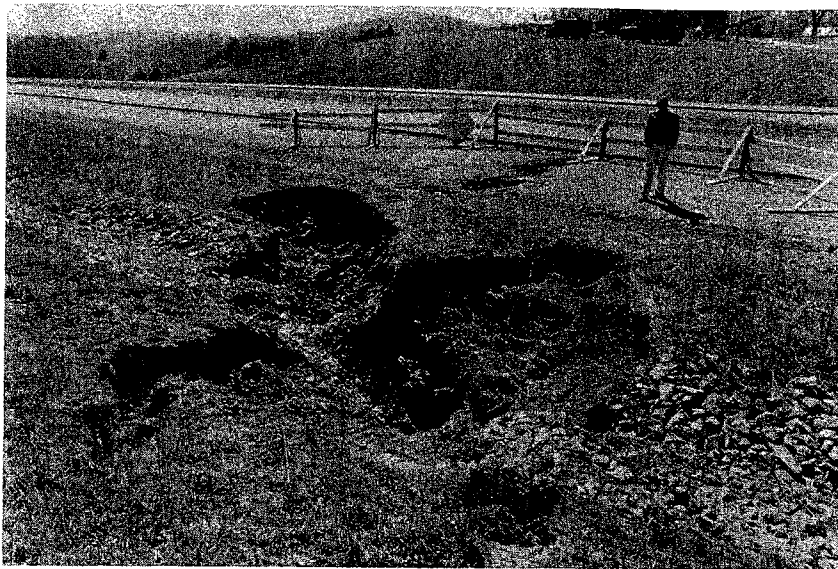


Figure 1. This ditchline collapse occurred in February, 1987, after heavy rains; location is on newly 4-laned U. S. 421 near Bristol Caverns, Sullivan County.

TYPE OF SINKHOLE	NATURAL INDUCED
LOCATION OF SINKHOLE	YARDS, FIELDS, WOODS ROADWAY, DITCHLINES ROADWAY SUBGRADES
TYPE OF DITCHLINE	TREATED UNTREATED
	25 50 75 100 % OF OCCURRENCE

Figure 2. Chart based on 72 case studies

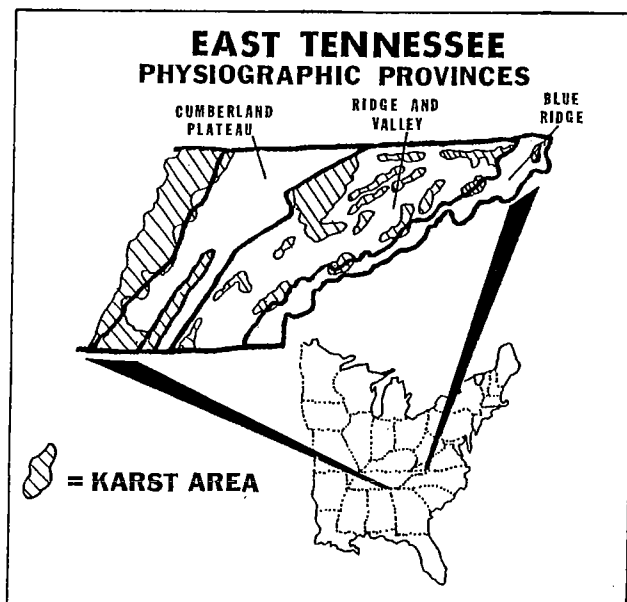


Figure 3.
Location of Study Area.



Figure 4. Induced sink-hole along untreated ditch-line; note bedrock in throat of collapse.

ability of surface water to infiltrate soil or bedrock beneath the ditchline contributes excessively to the overall cause of the problem. Disregarding this conceptual mode of failure will only lead to additional problems.

Occasionally a soil-filled cavity will be eroded out by surface water which can result in a cavity being formed in the ditchline. However, the data obtained from this study indicates that this is not a common occurrence.

Another rare cause of ditchline collapse is the structural failure of bedrock into an underlying solution cavity. Results of the study indicate that none of the 72 induced collapses fit this mode of failure.

TREATMENT

The treatment of karst related subsidence and collapse features that involve engineering structures can involve methods as varied as the sinkholes themselves. Most treatment concepts usually involve a method of "bridging" to provide adequate stability. In addition, providing appropriate drainage design to accommodate surface and subsurface flow is also a necessity.

The bridging remedial concept has been described and utilized by a number of workers in the geotechnical profession. Those concepts applicable to this study have been discussed by Foose (1976), Moore (1976, 1981, and 1984), and Sowers (1984).

The present study defines the bridge concept simply as a method which utilizes "chunk rock" to backfill the induced collapse (Figure 6). The interlocking action of the "chunk rock" backfill and the placement of the backfill on the top of rock effectively bridges the solution cavity. Additional details involved in the bridging concept

require further explanation.

The following elucidates the remedial concept for treating ditchline collapse problems by utilizing the chunk rock backfill bridging concept:

*The induced collapse is excavated down to in-place bedrock or pinnacles (in some cases this is not feasible requiring alternative methods of treatment).

*Clean chunk rock is backfilled into the excavated collapse (a geofabric may be utilized along the bottom of the collapse with placement of the rock backfill on top of the fabric).

*The chunk rock backfill should be clean non-degradable limestone or sandstone free from shale or clay; at least 50% (by volume) of the rock should be 0.3 to 1 meter (1 to 3.3 feet) in the longest dimension with no more than 15% (by volume) passing the 5 cm (2 inch) screen; the maximum size of chunk rock shall be 1 meter (3.3 feet).

*The chunk rock backfill is to be "choked off" at the top by using crusher-run material, geofabrics, or concrete.

*The ditchline should be paved, preferably with concrete; polyethylene sheeting or other impervious geofabric may be utilized beneath the paved ditch.

The source of surface water and direction of groundwater flow must be determined in some situations. This may require the use of geotechnical instrumentation such as standpipe piezometers and dye tracings. Sinkholes and caverns (located outside of the highway right-of-way but adjacent to the problem area) may need to be investigated as possible contributors to the problem.

CONCLUSIONS

The avoidance of karst areas by



Figure 5. An unpaved ditchline resulted in this collapse along S. R. 73 in Maryville, Tennessee.

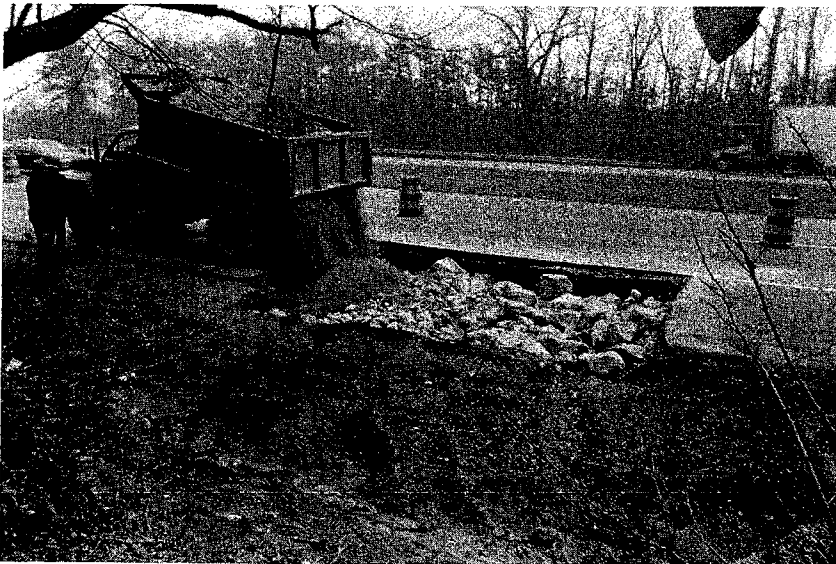


Figure 6. Chunk rock backfill remedial concept used for repairing sinkhole collapse along highways.

transportation facilities is recognized as a highly desired alternative. However, expanding population centers with the required new highway construction necessitates the use of non-desirable land. Proper investigation and design is mandated to utilize karst areas in man's engineering pursuits. Even with the best of planning and design, problems may result which lead to the development of subsidence and induced sinkholes.

The present study revealed that 74% of the karst problems experienced along East Tennessee highways were located in roadway ditchlines. In addition, it was shown that 93% of the ditchline collapse problems involved untreated ditches.

Induced sinkhole collapse and subsidence along untreated ditchlines have resulted in numerous innovative methods of treatment. The "chunk rock" backfill method utilized by the Tennessee Department of Transportation has resulted in proven success at low cost.

The most effective way of preventing karst related problems along roadways is to provide for treatment of all ditchlines with an impervious material (such as concrete). Areas of potential karst type subsidence and collapse can be identified during the location and design phases of engineering projects. The result of proper planning and design of roadway ditchlines in karst areas is fewer remedial projects and a more efficient highway facility.

REFERENCES

Amari, Dominick and Moore, Harry L. (1985), "A Solution to

the Solution Problem", Proceedings of the 35th Annual Highway Geology Symposium, Clarksville, Ind., May (in press).

Donaldson, C. W., (1963), "Sinkholes and Subsidence Caused by Sub-surface Erosion", Proceedings 3rd Regional Conference for Africa on Soil Mechanics and Foundation Engineering, pp. 123-125.

Foose, Richard M., and Humphreville, James A., (1979), "Engineering Geological Approaches to Foundations in the Karst Terrain of the Hershey Valley", Bull. of the Association of Engineering Geologists, Volume XVI, Number 3, Summer, pp. 335-381.

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

Moore, Harry L., (1976), "The Piping Phenomonon", Proceedings of the 25th Annual Highway Geology Symposium, Orlando, Fla., May, pp. 112-131.

Moore, Harry L., (1981), "Karst Problems Along Tennessee Highways: An Overview, Proceedings of the 31st Annual Highway Geology Symposium, Austin, Texas, pp. 1-28.

Moore, Harry L., (1984), Geotechnical Considerations in the Location, Design, and Construction of Highways in Karst Terrain - The Pellissippi Parkway Extension, Knox-Blount Counties, Tennessee", in Sinkholes: Their Geology Engineering & Environmental

- Impact. (Ed. by Barry F. Beck). Proceedings of the 1st Multidisciplinary Conference on Sinkholes, Orlando, Fla., October, pp. 385-389.
- Moore, Harry L., (1987), Sinkhole Development Along "Untreated Highway Ditchlines in East Tennessee, in Karst Hydrogeology: Engineering and Environmental Applications, (Ed. by Barry Beck and William L. Wilson); proceedings of the 2nd Multidisciplinary Conference on Sinkholes, Orlando, Florida, February, pp. 115-119.
- Newton, J. G., (1976), "Induced and Natural Sinkholes in Alabama -- A Continuing Problem Along Highway Corridors", Transportation Research Record 612, pp. 9-16.
- Newton, J. G., (1981), Induced Sinkholes: An Engineering Problem", ASCE Journal of the Irrigation and Drainage Division, Volume IR2, June, pp. 175-185.
- Newton, J. G., (1984), "Review of Induced Sinkhole Development" in Sinkholes: Their Geology Engineering, and Environmental Impact (Ed. by Barry F. Beck); "Proceedings of the 1st Multidisciplinary Conference on Sinkholes, Orlando, Fla., Oct. pp. 3-9.
- Royster, D. L., (1984), "The Use of Sinkholes for Drainage", Proceedings of the 63rd Annual Transportation Research Board, Washington, D. C., January, pp. 1-26.
- Sowers, George F., (1976), Mechanisms of Subsidence Due to Underground Openings", Transportation Research Record 612, pp. 2-8.
- Sowers, George F., (1984), "Correction and Protection in Limestone Terraine" in Sinkholes: Their Geology Engineering and Environmental Impact (Ed. by Barry F. Beck); Proceedings of the 1st Multidisciplinary Conference on Sinkholes, Orlando, Fla., October, pp. 373-378.

Cures for Slope Failures in Arkansas

SAM I. THORNTON

Civil Engineering, University of Arkansas,
Fayetteville, Arkansas

W. E. KIRKPATRICK

Arkansas Highway and Transportation Dept.
Little Rock, Arkansas

HGS-87-18

SUMMARY

SEVENTY HIGHWAY EMBANKMENT FAILURES were observed in one year by a district maintenance superintendent. Slopes failed from classical deep curved slides and shallow surface mud flows. Reconstruction cost of failed slopes approaches \$100,000 per year.

Several repair methods were tried in an evaluation project including: Geo-grid (Tensar); Replacement with upgraded borrow; Lime processing; Welded wire; Geotextile fabric; and Replacement with Fly ash/Bottom ash mixture. The most effective method was replacement of a portion of the embankment with a granular low plasticity material. The Geo-grid (Tensar) is potentially an economic repair but is still being monitored.

Lime-Fly ash injection was also tried but rejected when the embankment failed.

THE EMBANKMENTS

In order to interchange traffic without grade crossings, many embankments were built along I-40 and I-55 in Arkansas. Because the terrain is level, there is a typical configuration for the embankments. Fill heights are about 20 feet above natural ground. Side slopes are 3 horizontal to 1 vertical from the beginning up to the bridge, then wrap around to a 2 horizontal to 1 vertical under the bridge.

GEOLOGY - Most of the embankments are located in the fertile bottom land of the Mississippi River flood plain on an alluvial deposit over 100 feet thick. The soils are recent deposits, brought in by the Mississippi and Ohio rivers in the last 11,000 years. Unconsolidated sediments which contain many minerals from 24 states make up the alluvium. Most of the minerals are only slightly weathered. A wide range of texture

exists because of the variation in distance from the river to the embankments (1).

SOILS - The embankments were constructed from native surface soils (borrow pits). Soils from the study area (Crittenden County) are of the Sharkey and Alligator series which contain clays from the montmorillonite family. These are poorly drained soils formed in thick beds in old slack water areas.

Representative samples were taken from the embankments at 34 sites. Depths ranged from 4 to 29 feet.

Unified soil classifications are predominantly CH with some CL. AASHTO classification is A-7. Soil particles are in the clay and silt size range (below .075 mm).

Liquid limits ranged from 47 to 91. Plastic indexes ranged from 28 to 56. The average values for the liquid limit and plastic index were 66 and 42 respectively. Moisture contents ranged from 25% to 42%. The average moisture content was 32%.

The soils were compacted in lifts to 95% of standard compaction (AASHTO T99) at or near optimum moisture content.

FAILURES

Slope failures began to occur within a year after completion of embankment construction. The failures usually occur near the bridge end, both on the side slopes and under the bridge.

Failures vary from shallow, pocket type "mud flows", to full height failures that extend for a considerable length of the approach embankment. The failure plane never penetrates the natural ground level, nor extends under the roadway paving structure. Side slope failures down the approach are almost always the mud flow type.

EXTENT - The number of failures on the Interstate embankments is unknown but large. Seventy (70) slides were observed by the District One superintendent in one year. District One expenditures on embankment reconstruction is near \$100,000 per year when the slides are very active.

MECHANISM - The embankments develop vertical cracks, measured to depths over 12 feet, during dry weather cycles. Cracking in clay soils is estimated to equal twice the cohesion divided by the unit weight (2). Soils in the study area have cohesions of 400 to 600 pounds per square foot and unit weights near 110 pounds per cubic foot (3). Based on the estimate, cracks would be 7 to 10 feet deep.

Embankment cracks are open to channel rainfall and runoff deep within the fill. Runoff water sometimes fills the cracks with granular material (sand) which, in turn, allows water into the fill even in the wet season. Hydrostatic pressure, when the cracks are filled with water, is a factor that contributes to failure.

Cracks are discontinuities which do not have shear strength. The failure soil mass may be defined by a circular surface from the toe to the intersection of the crack. The failure surface is then vertical along the crack to the top of the slide.

Soil samples taken immediately after a slope failure have moisture contents 15 to 20% greater than the Plastic Limit.

CORRECTIVE METHODS

PREVIOUS METHODS - The Arkansas Highway and Transportation Department have considerable experience in the repair of embankment slope failures.

Temporary Correction - Due to shortage of time or funds, slides may be repaired by air drying the failed mass then pushing the soil back into place. The slope is then shaped and seeded. Cost of this temporary measure is \$2.00 to \$3.00 per cubic yard of material moved, depending on the size of the slide. Slopes repaired by the push up method may stand from a week to a year. Failure usually recurs during the first rainy season.

Replacement Method - Failed material may be removed and replaced with select material. Granular, non plastic, fill material is compacted in lifts in the slide area. Replacement by the AHTD maintenance forces costs from \$6.55 to \$8.16 per cubic yard. By contract, the cost was \$8.55. Replacement has been very successful, but a haul of 40 miles or more makes the method expensive.

Sand Mix - One part river sand may be mixed with four parts slide material and the mixture placed in lifts to grade. Lateral underdrains may be installed to keep the slopes dry. Sand mix costs \$2.50 to \$8.96 per cubic yard depending on the percent sand used and the haul distance. Some slopes repaired by the sand mix method show no movement after two years, others have been termed relatively successful.

Slurry Injection - Stabilization of the slopes by injection with a lime and fly ash slurry was tried but was unsuccessful (4). A slurry of 3 pounds of fly ash and one pound of lime per gallon of water was used to pressure inject a total of 28,778 square feet of surface area. In all 168 tons of fly ash and 56 tons of hydrated lime were used at a total estimated cost of \$17,000. The estimated cost of replacement with select material was \$15,857. The west slope of the south embankment was left untreated as a control section. Evaluation included slope inclinometer data, moisture measurements, and visual inspections. The report on the failure (5) stated, "As both the control slope and the injected slope have failed, monitoring has been terminated. Our conclusion is that this method of treatment is not successful". An increase in moisture content due to the injection could have been a contributing factor. Samples taken in the failure zone had a moisture content of 44 percent.

PROPOSED METHODS - A variety of slope failure corrective methods are being tried in a research project funded by the Arkansas Highway and Transportation Department (TRC Project No. 84). The following corrective methods were chosen for monitoring and evaluation: Geogrid (Tensar); Replacement with upgraded borrow; Lime processing; Welded wire; Geotextile fabrics; and Replacement with Fly ash/Bottom ash mixture. Replacement with a crushed stone/cement mixture was planned but not installed.

Geogrid - Geogrid (Tensar) is a high strength plastic reinforcement which increases the strength of the soil mass. The geogrid will be placed in two feet thick layers 40 feet wide in the embankment face. A total of 118,500 square yards of geogrid was placed in a new approach ramp with a maximum fill height of 30 feet. In addition, approximately 8,000 square yards were used to repair an existing failed slope. The bid price of the geogrid, including material and placement costs, was \$2.90.

Upgraded Borrow - A section of upgraded borrow was included for comparison. Two sites using approximately 25,000 cubic yards each were used. The bid price was \$6.40 per cubic yard.

Lime Processing - Quicklime was used to stabilize a section of embankment which had been cut to a 1:1 slope. Treated material was placed in 6 to 8 inch lifts and compacted with sheepsfoot rollers. The estimated cost of lime processing is \$10.00 per cubic yard.

Welded Wire - Steel wire reinforcement (chain link fence) was used on one approach embankment which had to be raised 1.87 feet. The wire was placed in layers similar to the installation of geogrid.

Geotextile Fabric - Two geotextile fabrics were installed in two locations. One fabric was woven with a 200 pound grab strength and a 400 psi burst strength. The other was nonwoven with a 100 pound grab strength and a 200 psi burst strength. The fabric was placed in two foot layers with a two foot overlap at the face.

Fly Ash/Bottom Ash - A mixture of 50% fly ash and 50% bottom ash was used in one installation. The mixture was placed in layers from the ground surface with sufficient water for hydration. A soil layer was placed on top to support a grass cover.

COSTS - A comparison of costs and relative success of the various corrective methods is under evaluation. The following comparison is based on information available to date (Table 1).

TABLE 1
Cost Comparison

Method	Cost/CU YD	Expected Life
Push Up	\$ 2.50	Temporary
Replacement	\$ 7.20	Permanent
Sand Mix	\$ 5.25	Several Years
Slurry Inject	\$ 5.00	A Few Months
Geogrid	\$ 4.80	Permanent
Lime Process	\$10.00	Permanent

REFERENCES

1. Kirkpatrick, W.E., 1985, "Corrective Measures For Slope Embankment Failures", Thesis at Univ. of Arkansas, Fayetteville, Ark., p. 1
2. Perloff William F., and William Baron, 1976, "Soil Mechanics, Principles and Applications", John Wiley & Sons, New York
3. LeFevre, Walter E., Project No. 28, 1975, "Development of Design Criteria for Fill Slopes in Eastern Arkansas", College of Engineering, Univ. of Arkansas, Fayetteville and Arkansas State Highway Dept.
4. Arkansas Highway and Transportation Department, 1984, "Demonstration Project No. 59: Lime-Fly Ash Slope Stabilization; West Memphis, Arkansas", Federal Highway Administration, Contract no. DTFH71-83-59-AR-06
5. Arkansas Highway and Transportation Department, Feb. 1985, "Update of Research on East Arkansas Slides", Research Informer No. 84, Little Rock, AR

Anchored Solutions for Unstable Topography

CHARLES H. LEICHNER III
Earth Support Systems
Leesburg, Virginia

HGS-87-19

INTRODUCTION

SEVERAL TYPES OF ANCHORED solutions are available for use in unstable topography. Tiedback retaining walls are being used with increasing regularity in cut situations and have become widely accepted as an economical support system. Soil nailing is another anchored solution that may be used in areas susceptible to slope movements. This stabilization technique is becoming more prevalent in highway construction. The primary form of unstable topography considered in this paper is the landslide. A landslide can be defined as a downward and outward movement of slope-forming materials which includes natural rock, soils, artificial fills or combinations of these materials. It is currently more popular to use the term "slope movement" as opposed to "landslide" when referring to this phenomenon.

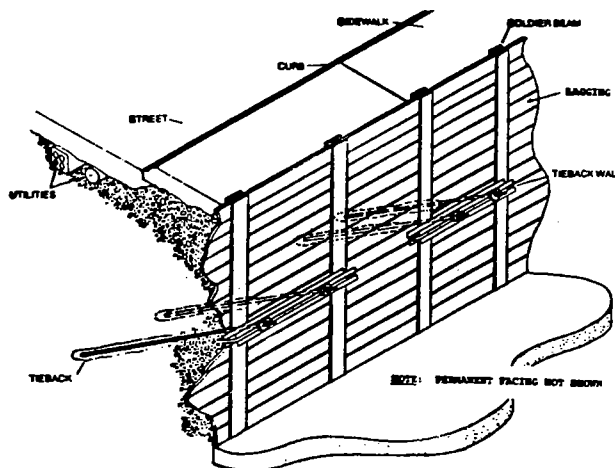
Anchored solutions are applied to individual slope failures or potential failures where the magnitude of loading is not excessive for structural applications. These are not as spectacular or as costly as large deep seated slides, but are more widespread. Post-tensioned tiebacks with tiedback walls are used for slides with failure surfaces up to 100 feet from the face of the wall. Soil nailing applications are used in the case of shallow slides or potential slides. This paper will briefly review tiedback and soil nailing walls, and discuss their similarities and differences. Permanent facing details will be addressed due to their impact on long-term performance of the walls. Finally, the future of anchored solutions for unstable topography will be discussed.

TIEDBACK WALLS

A typical tiedback wall system is shown in Figure 1 and is comprised of soldier beams, lagging, tiebacks and facing (permanent structures). The soldier beams may consist of H-piles, wide flange shapes, reinforced caissons or sheet piling. Depending on the soil conditions, the piles may be driven or set in drilled holes and filled with lean concrete. Lagging is provided between soldier beams to prevent the soil from raveling and is installed as the

FIGURE 1

TYPICAL TIEDBACK WALL



excavation proceeds from the top down. Space is left between lagging boards so that drainage is promoted. Tiebacks are installed after the excavation and lagging is complete down to a predetermined elevation. Tiebacks may be pressure injected, hollow stem auger, regrowable, rock tiebacks or some other type based on the geotechnical site conditions. The process of excavating, lagging and installing tiebacks is continued until the desired subgrade is reached. In permanent walls, adequate facing and drainage provisions should be provided to ensure satisfactory long-term performance. Tiedback walls have been used for landslide correction, permanent highway walls, temporary earth support and various other applications. In the case of a landslide, a "top down" method of construction such as a tiedback wall, is often the only way to build a wall. A conventional retaining wall requires excavation in order to build from the bottom up. Frequently, this excavation removes base support and causes a slide.

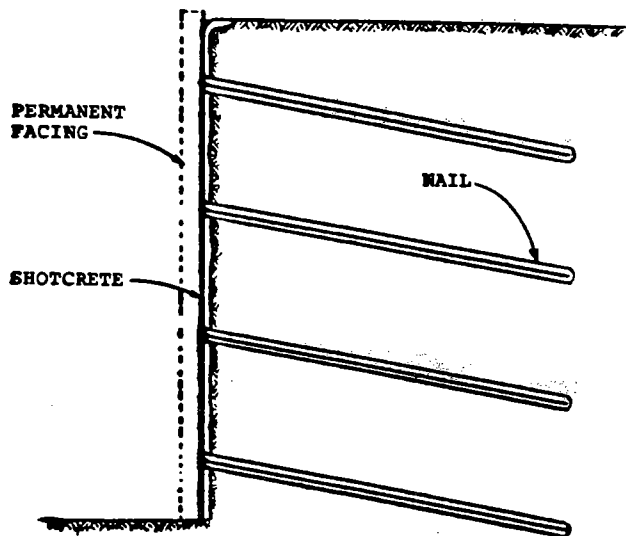
SOIL NAILING

Soil nailing is another anchored solution

for unstable topography. In this method, the native soil adjacent to the excavation is strengthened so that it can stand unsupported at depths which would normally require sheeting. Soil nailing is similar to mechanically stabilized embankments. The difference is that soil nailing is constructed from the top down and mechanically stabilized embankments are constructed from the bottom up. Soil nailing is also similar to rock bolting and the Australian tunneling method. Prior to the middle 1970's, soil nailing was used for temporary structures. Since the middle 1970's, permanent soil nailing structures have become feasible due to the technology of drainage materials such as drainage mats and wicks. Soil nailing walls are comprised of an array of reinforcing members that are drilled and grouted into the soil mass (refer to Figure 2).

FIGURE 2

TYPICAL SOIL NAILING WALL



Excavation begins at the top of the wall and proceeds in lifts of approximately five feet. After each excavation lift is completed, shotcrete is applied to the exposed soil slope. The reinforcing members or "nails" are then installed for the lift which has just been shotcreted. The process of excavation, shotcreting and installing nails is continued until the desired subgrade is reached. Adequate provisions for drainage must be provided so that a buildup of hydrostatic pressure behind the shotcrete does not occur. Due to past performance problems of shotcreted facings and their susceptibility to ground water problems, Earth Support Systems considers a shotcreted facing adequate for temporary soil nailing walls only. In permanent applications, a precast concrete or cast-in-place concrete facing with adequate drainage provisions should be provided.

Soil nailing has been used for permanent highway walls, temporary earth support and remedial work including repair of Reinforced Earth walls. It is particularly suited to sites with access problems, limited right of way and where it is difficult to install soldier beams such as mixed rock and soil conditions. Soil nailing should not be used where the ground cannot stand unsupported for at least five feet, in the presence of large amounts of water or for slides with deep seated failures.

COMPARISON OF TIEDBACK AND SOIL NAILING WALLS

Some similarities exist between tiedback walls and soil nailing walls. Both of these methods of earth support are used in cut situations and are built from the top down. Tiedbacks and soil nails are forms of ground anchors. Several types of tiedbacks and nails are available for use depending on the soil conditions encountered at the site. Testing is required of both systems to ensure that the assumptions made for design are valid. Both tiedback walls and soil nailing walls have been used in temporary and permanent applications. In order to ensure satisfactory long-term performance, adequate drainage provisions must be provided with both types of walls to prevent a buildup of hydrostatic pressure from acting on them. In permanent applications, tiedback walls and soil nailing walls should have a facing that is aesthetically pleasing and of sufficient structural capacity to last the design life of the wall.

Although tiedback walls and soil nailing walls are similar in many ways, there are significant differences between the way in which the two types perform. Therefore, they should be considered entirely different systems. In other words, a soil nailing wall is not a tiedback wall.

A tiedback wall uses anchors which are post-tensioned. The tieback load is transferred to the pile which in turn pushes against the soil. An apparent earth pressure loading diagram is used which corresponds to the ground response to the tieback load. A tieback design load is much larger than a soil nail design load due to the larger wall area the tieback supports and the magnitude of the earth pressure. Tieback testing consists of performance testing a percentage of the tiebacks and proof testing the remainder. These load incremented tests are required to assure that each tieback has adequate capacity and the load transfer is taking place beyond the theoretical failure surface. Creep testing is performed when tiebacks are installed in creep susceptible soils. A significant right-of-way acquisition or easement is necessary for tiedback walls to allow for the length of tiebacks.

Contrary to tiedback walls, soil nailing walls use anchors that are not prestressed. The reinforcing anchors are installed in an

array that knits the entire soil mass together so that it acts as a gravity wall. Soil nail design loads are much smaller than tieback design loads. The load acting on the face of the wall is very small due to the composite mass of earth and grouted nails. The design for each nail is simply the load required to maintain stability of the soil mass. Testing of soil nails consists of loading the nail to the design load multiplied by the appropriate safety factor and holding this load for a specified period of time. A load incremented test such as the performance test or proof test for tiebacks is not appropriate for soil nails because load from the soil is transferred to the nails by skin friction along the length of the nail. There is no unbonded length for soil nails and consequently, no need to check the elastic elongation of the anchor. Soil nails are particularly effective in mixed soil and rock excavation cuts and can be installed with much less right of way or easement than that required for tiebacks.

FACING CONSIDERATIONS FOR PERMANENT STRUCTURES

Permanent facing systems for anchored structures should be compatible with the construction sequence of the structure and provide the required structural capacity for the life of the structure. In addition, the permanent facing should be aesthetically pleasing. Adequate drainage provisions must be provided with all systems in order to prevent a buildup of hydrostatic pressure from acting on the facing.

Consider permanent tiedback walls. Although many facings have been used with tiedback walls, it is the author's belief that precast concrete is the best alternative. Therefore, this discussion will be limited to precast concrete facings. This type of facing has been commonly accepted with alternate walls used in conjunction with highway construction. The reasons for acceptance include economy, quality control and appearance. In the past, precast concrete facings have included lagging planks and a variety of sizes and shapes of panels. Many permanent tiedback walls have used precast concrete planks or panels between soldier beam flanges to act as both lagging and permanent facing. The problem with this detail is that since tiedback walls are built from the top down, the planks or panels are installed prior to stressing the anchors. When the anchors are stressed, the soldier beams are pushed back against the soil. This movement and associated earth pressure causes the planks or panels to bow outward and crack. Structurally and aesthetically, performance such as this is clearly undesirable. Another aesthetic shortcoming of this facing system is that the pile flanges and tiedback anchor heads are exposed.

Improved performance and appearance can be achieved by using timber lagging and providing precast facing panels to conceal the anchor

heads, soldier beams and lagging boards. This system allows the panels to be installed from the bottom to the top after the anchors have been stressed and movement associated with stressing has occurred. Problems associated with this system in the past have been related to panel size. Excessive cracking has occurred when large panels have been used causing the system to become too stiff and restricting the breathing ability of the wall. One solution to this problem is to provide small precast panels that provide flexibility and allow the wall to breathe.

The majority of permanent soil nailing walls have utilized shotcrete as the final facing material. However, some soil nailing walls have used cast-in-place concrete or precast concrete facing panels. Control of groundwater is extremely important for soil nailing walls and provisions for drainage require serious consideration. In the past, the poor performance of shotcrete as a facing has been attributed to inadequate provisions for drainage. Hydrostatic pressure buildup behind the shotcreted face has led to cracking and spalling of the shotcrete. This is unacceptable in terms of structural capacity and overall appearance. Previous experience with various drainage materials and shotcrete have led to the conclusion that shotcrete is acceptable in temporary applications only. Precast concrete facing panels or cast-in-place concrete should be used for permanent soil nailing walls with adequate provisions for drainage.

FUTURE OF ANCHORED STRUCTURES

The future of anchored structures appears to be very promising. Tiedback walls are being used with increasing regularity and will continue to be an economical solution for unstable topography. Soil nailing is becoming more prevalent in highway construction applications. As more transportation departments become comfortable with soil nailing, its use should increase dramatically. Future anchored applications will use combinations of tiebacks and soil nails, tiebacks and mechanically stabilized embankments, and soil nails and mechanically stabilized embankments. In particular, research for the Federal Highway Administration is currently taking place on an anchored system that is comprised of tiebacks and soil nails. Model studies have been performed and full scale testing will be taking place in the near future. This could yield results which may lead to implementation of this system in highway construction applications. New types of anchored structures will be developed that continue to expand the use of tieback and soil nailing concepts. One way to encourage development of innovative cost effective systems is to allow the use of alternate designs by design/build contractors.

CONCLUSION

Several types of anchored solutions are available for use in unstable topography. Tied-back retaining walls and soil nailing walls are being used with greater frequency in highway construction. In the case of landslides, "top down" methods such as tiedback and soil nailing walls are often the only way to build a wall without causing a slide. Although tiedback walls and soil nailing walls fall in the category of anchored structures, they perform in very different ways. Tiebacks are post-tensioned and apply a force to the earth through soldier beams. Soil nails, on the other hand, are not post-tensioned and simply reinforce the soil so that it acts as one unified mass. Facings for permanently anchored structures need to be compatible with the method of construction. Adequate provisions for drainage are required for satisfactory long-term performance of the structure. Precast facing panels should be used for structural, economy, quality control and aesthetic reasons, and panel size should be restricted in order to provide the flexibility necessary for the structure to breathe. The use of alternate designs by design/build contractors should continue to be allowed in order to develop innovative solutions for unstable topography.

Assessment of Landslide Potential Along Route 3 In Southern Illinois

S. M. ORBOCK MILLER, W. A. CANAVAN, and R. C. KOCHER

Department of Geology
Southern Illinois University
Carbondale, Illinois

HGS-87-20

INTRODUCTION

INVESTIGATIONS OF SLOPE STABILITY are becoming increasingly important in engineering and geologic hazard assessments (Brunsden and Prior, 1984; Costa and Baker, 1981). Recognition of unstable slopes within an area is a major step toward minimizing potential loss of property and life. Many techniques have been developed to synthesize and illustrate the complex interaction of geologic and environmental variables which control slope stability (U.S. Geological Survey, 1982; Hansen, 1984). Detailed site investigations of geotechnical properties are invaluable in slope stability assessments (Moser and Hohensin, 1983; Cararra, 1983; Terranova, 1987; Federal Highway Administration, 1985).

Significant benefits can be achieved, however, with the application of rapid, simple, and inexpensive geomorphic investigations aimed toward the preparation of maps showing relative landslide potential for a region. These kinds of studies have been successful in regional assessments of slope stability in a wide variety of physiographic settings (e.g., DeGraff, 1978; Peatross, 1986) and may be equally well-suited for use by highway departments in planning the locations of roads and for mitigation purposes along existing highways. Our purpose was to assess the relative stability of slopes along a reach of highway in southern Illinois using existing data and a minimum of additional geomorphic field research.

Published data from numerous county, state, and federal agencies were used in combination with a field reconnaissance map of geomorphic features to prepare a map of landslide potential for a selected reach of Route 3 in southwestern Illinois (Fig. 1). Route 3 is located along the base of bedrock bluffs and approximates the margin of the Mississippi River floodplain in northwestern Jackson County. This highway has been frequently damaged by landslides during the past few decades (Table 1). The bluffs are composed of nearly flat-lying sedimentary rocks capped by several meters of Wisconsin loess. Floodplain sediments are characterized by channel gravel and sand, overlain by fine-grained sand and silt. Local relief averages 90 to 110 m along these slopes covered by oak-hickory forest (Fig. 2a). The climate of the area is humid-temperate, with average annual precipitation of 112 cm.

Detailed field observations were made in the vicinity of a major recurring landslide near Jones Ridge Road which was the site of a massive landslide in early 1986 (Fig. 2b). Detailed maps illustrating the various stages of the preparation of the landslide potential map will be shown for this area (Fig. 3 - boxed area).

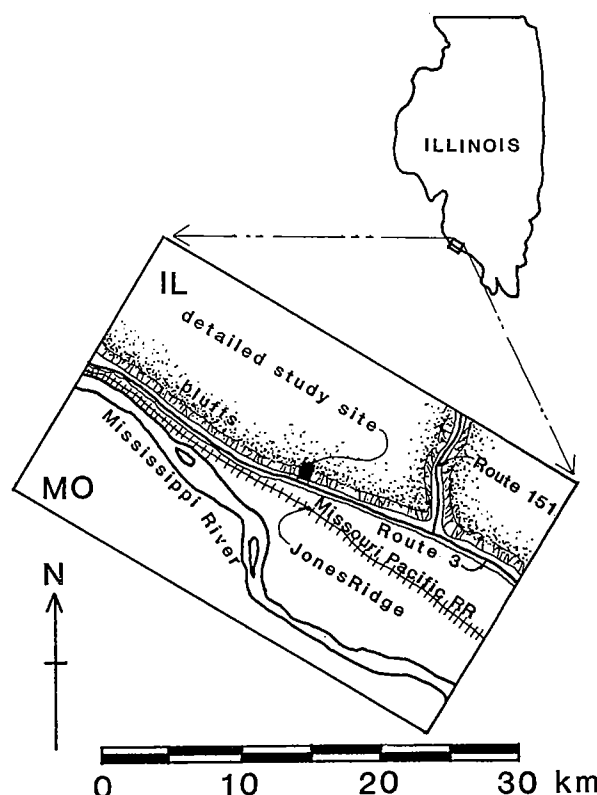


Figure 1. Index map showing location of Route 3 and the study area.

STRATIGRAPHY AND LANDSLIDE GEOMETRY

The bedrock exposed along Route 3 is composed of sedimentary layers of alternating limestone, sandstone, and shale of Mississippian age. The interbedded and laterally variable nature of these units plays an important role in the potential for landsliding along the highway (Fig. 4). The layered sedimentary rocks possess contrasting physical and hydrogeological characteristics which result in differential weathering. The recesses in the bedrock and the position of scarps are associated with contacts between formations and/or textural variations within the same unit. Increased pore pressure from water collecting at the interface between units appears to have been an important cause of slope instability in the study area.

Table 1. Index of Recent Landslides in the Study Area

Date/ Slide	Location	Number of Landslides	Remedial Measures	Recurrence
1(1986)	junction of routes 151 and 3	3	none	no
2(1986)	east of Logan Hollow Road	2	none	no
3(1986)	near slide #2	1	none	no
4(1986)	800 m east of Reeds Creek	1	none	no
5(1986)	650 m northwest Jones Ridge Road	2	toe of slide removed	no
6(1959)	northwest of Reeds Creek	1	toe of slide removed, bench and rock buttress used to prevent further sliding	yes (1986)
7(1962)	Route 151 to Talbot Hollow	2	toe of slide removed, bench installed to catch debris	yes (1986)
8(1962)	south of Logan Road	2	toe of slide removed, bench installed to catch debris	yes (1986)

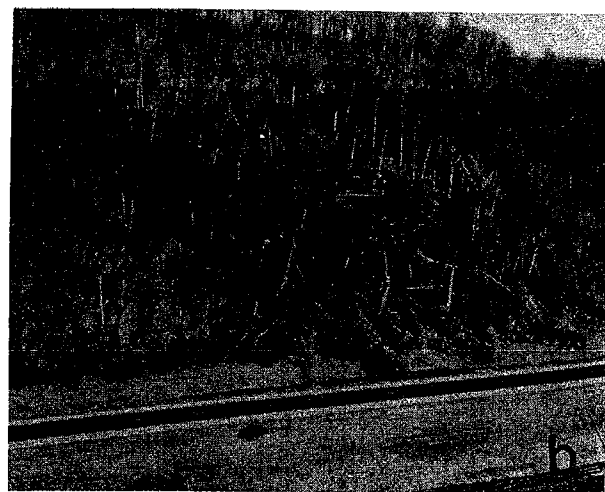
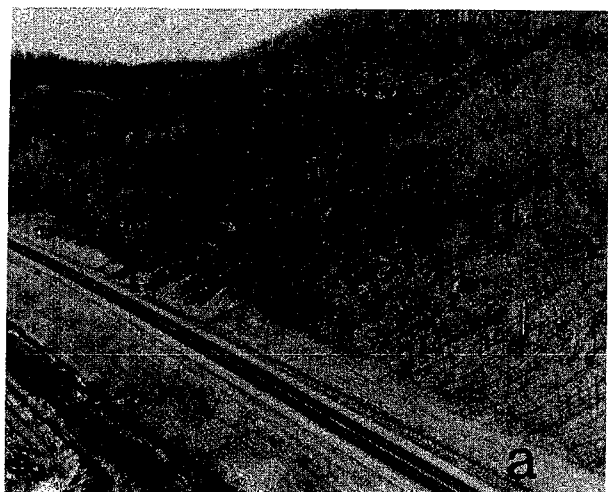


Figure 2. Oblique aerial photographs taken about a month after the 1986 landslide. a) View northwest along Route 3. Floodplain is visible at lower left. b) 1986 landslide area. Degonia Sandstone can be traced from headscarp into the stable area (courtesy of Jackson County Highway Department).

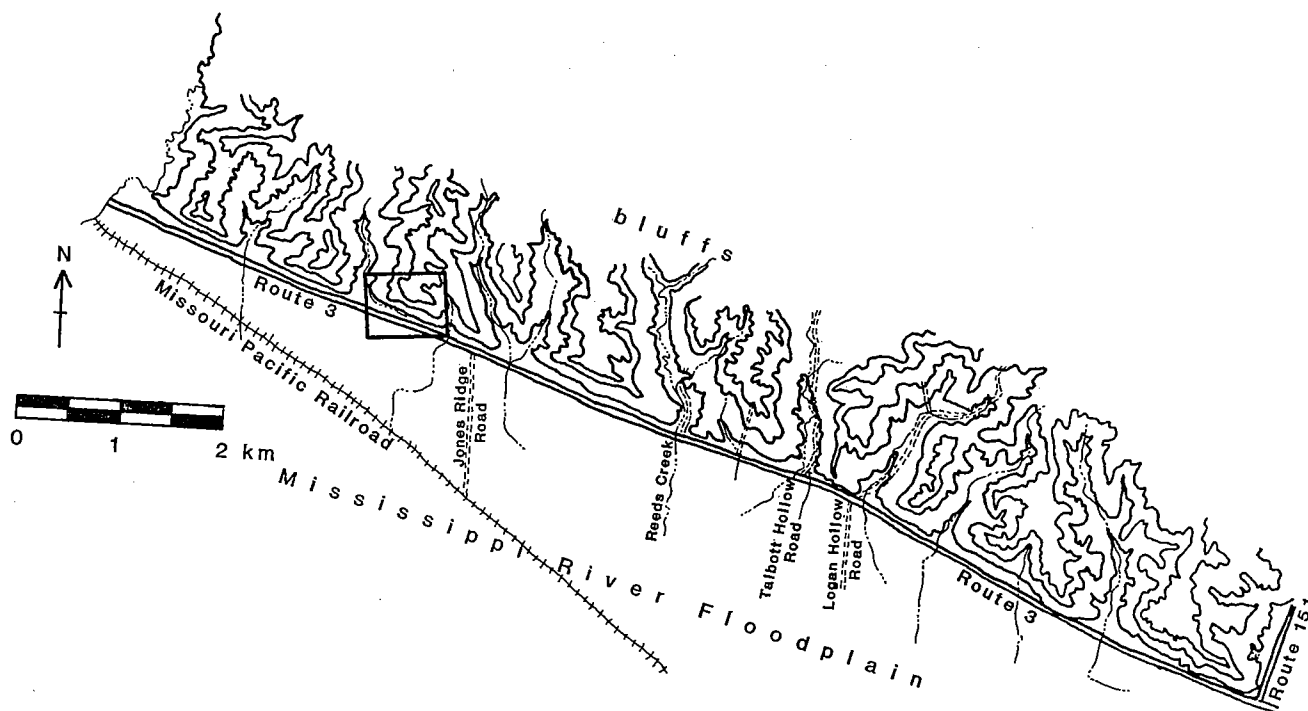


Figure 3. Generalized topographic map of study area.

Four major formations outcrop along Route 3 in the study area. Their differential resistance to weathering and erosion is responsible for the stepped nature of hillslopes along the highway (Swann, 1963). Four distinct slopes separated by flat benches can be recognized in the unfailed reaches of the bluffs (see Fig. 2a). Bedrock exposures are frequent throughout the study area. Four major scarps were mapped within the 1986 landslide (Fig. 5). These scarps can be traced laterally to areas of steep slopes between the benches in the stable areas proximal to the landslide. The upper few meters of the Menard Limestone contain the basal scarp which occurs near the elevation of the highway. It is generally obscured by colluvium within the slide area.

Two major scarps higher in the slide area are associated with lithologic changes within the Clore Formation (Fig. 6). The highest scarp on the 1986 slide occurs at the contact of the Clore and overlying Degonia Sandstone. Shale beds within the upper Clore are highly weathered here. The permeable Degonia Sandstone is the major cliff-former along the Mississippi River valley throughout the region and was not involved in the slide. However, it is the source of many ancient rockfalls mapped along the bluffs.

MAP ANALYSIS

Factors affecting slope stability are often spatially variable within a region. Maps, depicting different measures of stability, facilitate the analysis of landslide potential. For this study, three factors affecting slope stability were evaluated: 1) the distribution of geomorphic features, 2) soils, and, 3) slope. These parameters were assigned an arbitrary ranking and combined to produce a map showing relative landslide potential.

GEOMORPHIC MAP - Geomorphic mapping of an area can provide invaluable information regarding past and potentially unstable zones through the recognition of deposits and features associated with former mass wasting events. Criteria used in geomorphic mapping of mass wasting features have been summarized by Hansen (1984). For this study, a geomorphic map was prepared from field reconnaissance of features which were ranked into unstable, potentially unstable, and stable categories. Areas containing landslides, scarps, hummocky topography, debris lobes, abrupt changes in vegetative density, sag ponds, and uprooted or toppled trees were designated as unstable. Areas with abundant gullies, slightly tilted or bent trees, and rockfalls were designated as potentially unstable. Stable designations were assigned to areas lacking in geomorphic evidence of former mass movements and to areas located a significant distance from the bluffs (Fig. 7a). Recent mass movements have been dominated by rotational slumps. Most of the rockfalls appear to be prehistoric. They may be associated with colder Pleistocene climates or perhaps were generated by seismic disturbances from the New Madrid fault zone in Missouri. Earthflows have also been common in historic times, but are typically small or emanate from the base of larger slump deposits.

SOILS MAP - County soil reports published by the Soil Conservation Service (U.S. Department of Agriculture) contain readily available information on soil characteristics which are pertinent to the analysis of landslide potential. The prominent soils that occur in the study area include the Alford, Haymond, and Neotoma-Wellston series (SCS, 1979). The Haymond silt loam is developed within silty alluvium and occurs on floodplain, terrace, and alluvial fan deposits in low lying areas. This soil was given a stable rating. The Alford silt loam is developed in loess and occurs on the stable portions of ridgetops and hillslopes in the

Generalized Geologic Column

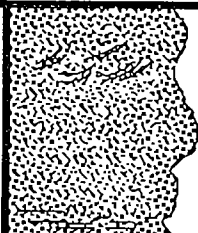
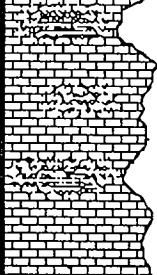
Formation		Lithologic Description
<i>Degonia</i>		(23-30m) Sandstone mixed locally with thin shale beds. Upper part contains large-scale cross-bedding. Lower part is thinly-bedded and contains more shale. Massive cliff former along river. Permeable.
<i>Clore</i>		(12-18m) Interbedded limestone and thin-bedded shale. Shales are slick and are commonly major slide planes.
<i>Palestine</i>		(12-18m) Thin-bedded shaley sandstone. Contains interbedded shale. Formation is highly weathered in outcrop.
<i>Menard</i>		(21-26m) Limestone unit with minor thin shale beds. Unit weathers to hummocky surface in outcrop. Bedding planes are typically uneven. Abundant fossils. Level of Route 3 occurs in the upper few meters of this unit.
<i>Waltersburg</i>		(5-10m) Silty shale, thin sands.
<i>Vienna</i>		(5-10m) Siliceous limestone, chert-rich zones are common.
<i>Tar Springs</i>		(6-12m) Sandstones of variable thickness and texture. Siltstone beds are common.

Figure 4. Generalized stratigraphic column of rocks outcropping along Route 3. The highway, at the 1986 landslide site, occurs at the level of the upper Menard Formation.

uplands. With depth, the clay content of the Alford increases, which may locally inhibit infiltration. This soil was assigned a moderate stability rating. The Neotoma-Wellston soils are developed within colluvium and are characterized by an average cobble content of 35%. They are found on convex ridges and steep portions of hillslopes. The colluvial parent material indicates past unstable activity in these areas. The Neotoma-Wellston soil was ranked as the most unstable because of its poor sorting and thin nature. Figure 7b shows the soils map based on this stability ranking. Evaluation of the physical, chemical, and engineering properties of these soils, also provided in the soils report (SCS, 1979), supports this classification.

SLOPE MAP - A slope map was prepared from topographic maps (Fig. 7c). Slope categories were based on the spacing of contour lines within the study area and subsequent field checking. Because slope stability tends to decrease with an increase in slope angle, the slopes were ranked as increasingly unstable in the following categories: 0-10%, 11-20%, 21-30%, 31-40%, and >40%. Alternatively, evaluation of soils maps (SCS, 1979) can provide a quick estimate of expected slope categories.

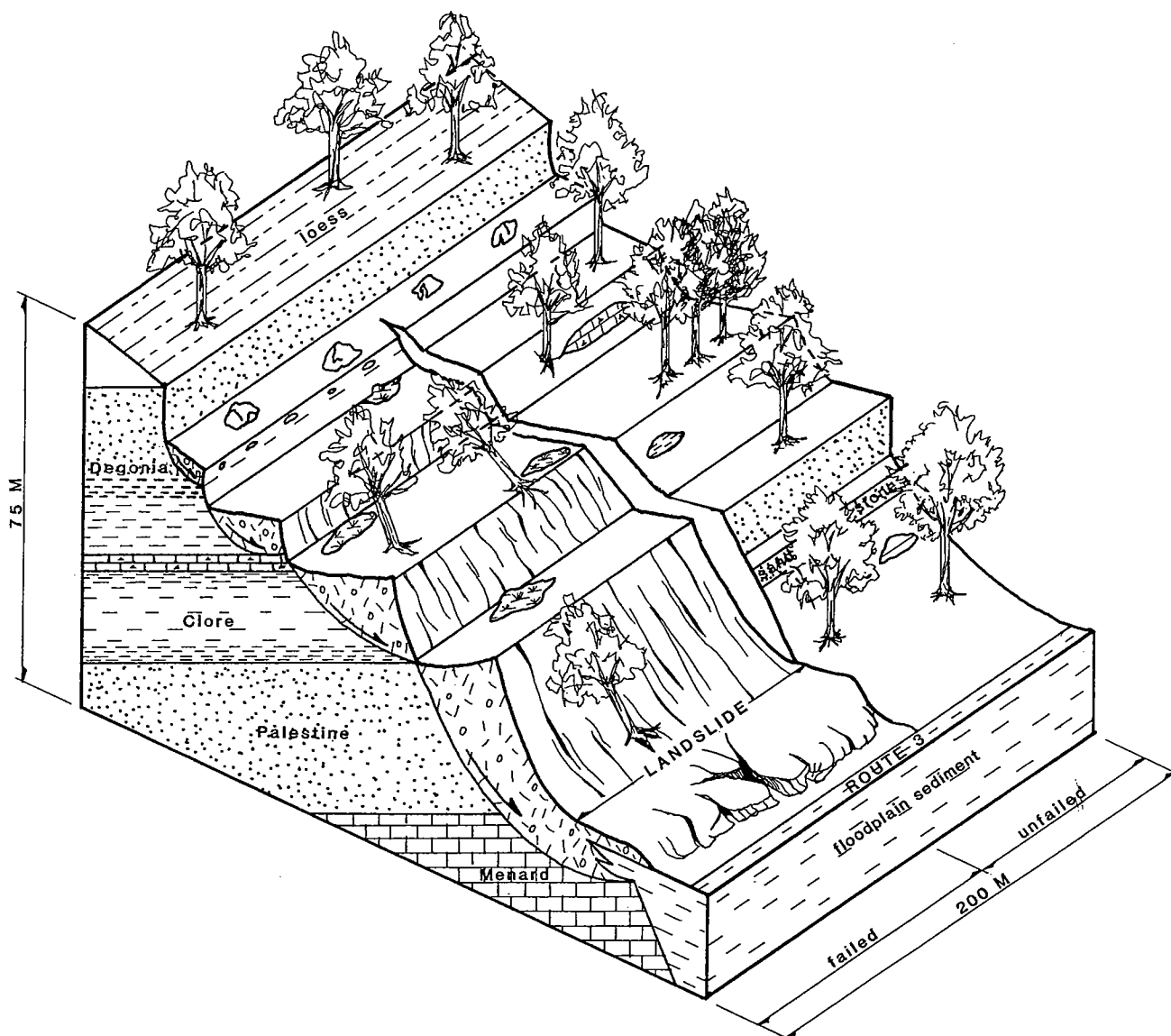


Figure 5. Schematic block diagram showing multiple rotational slumps involved in the 1986 landslide. Location of major slide planes is controlled by stratigraphy. Most slide planes occur at the contact between permeable sandstones and underlying impermeable shales.

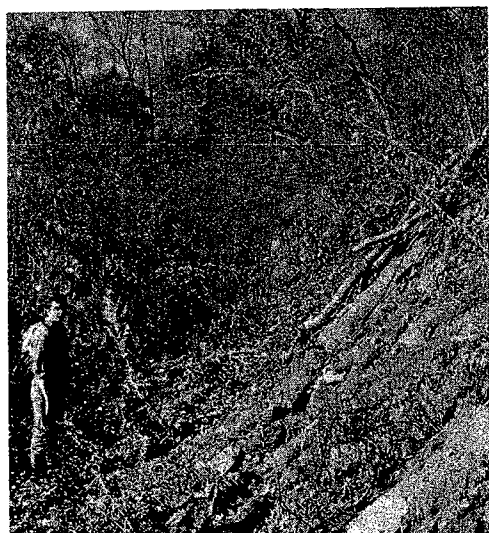


Figure 6. Typical scarp within the Clore Formation. Scarp is about 3 m high and is rilled

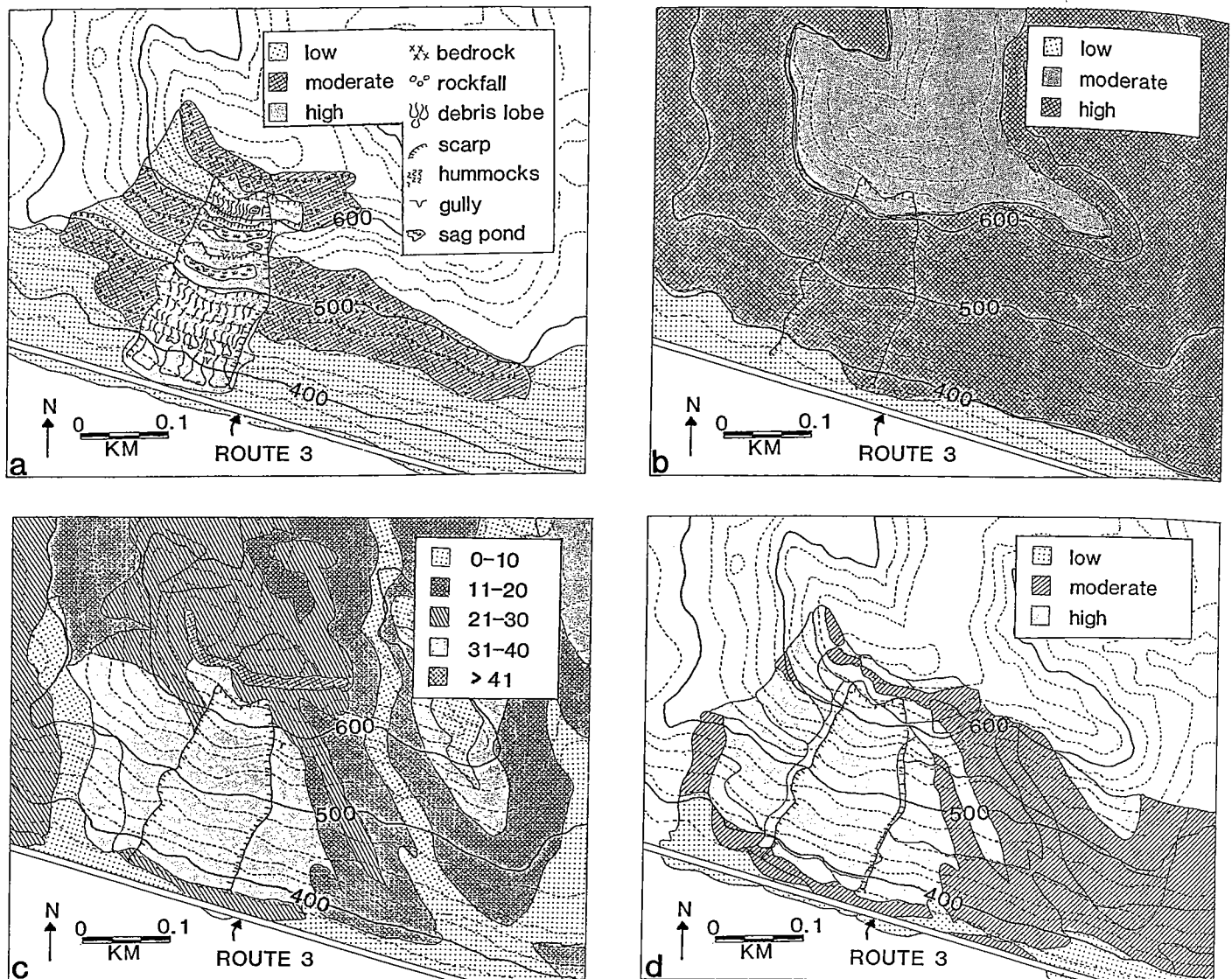


Figure 7. Examples of detailed hazard maps for the vicinity of the 1986 landslide (boxed area in Fig. 3). a) geomorphic map, b) soils map, c) slope map, d) landslide potential map.

LANDSLIDE HISTORY MAP - Landslide activity along Route 3 has been documented by the Jackson and Randolph County highway departments (Table 1). Certain areas along Route 3 have experienced recurring landslides with recurrence intervals ranging from 20 to 25 years. Remedial measures have had moderate success, depending upon their location. The hazard risk and economics determine, in part, whether remedial measures are taken. Areas of documented landslide activity mapped by the county were superimposed on the landslide potential map (Fig. 7d).

LANDSLIDE POTENTIAL MAP

Production of a landslide potential map for the study area shown in Figure 3 consisted of three phases: 1) creation of the slope, soils, geomorphic, and landslide activity maps; 2) assignment of rankings for each map according to landslide potential; and 3) overlaying these maps and combining the rankings create the landslide potential map values (Fig. 7d).

The combination of rankings resulted in the following landslide potential categories: 1) slight (values 1-3); 2) moderate (values 4-7); and 3) high (values 8-11). Superposition of the documented landslide occurrence map on the potential landslide map indicates that slides occur in the high to moderate landslide potential categories (Fig. 7d). This ranking procedure was extended for the entire study area to produce the regional landslide potential map shown in Figure 8.

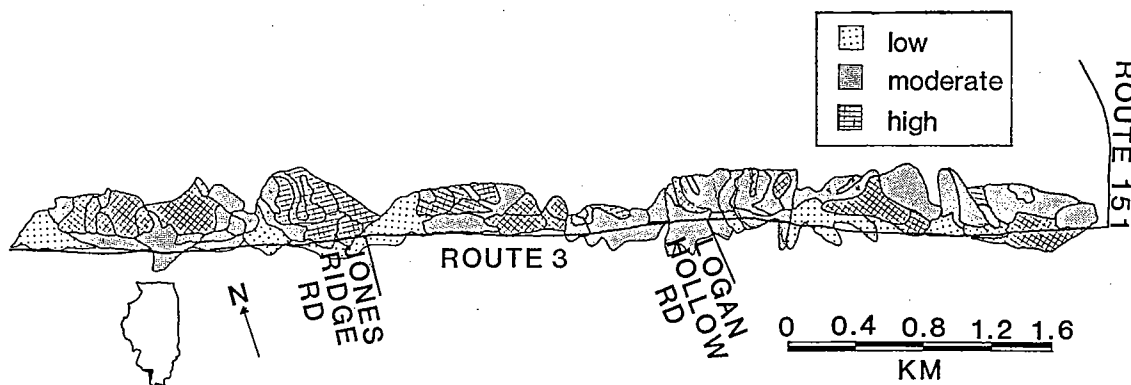


Figure 8. Landslide potential map for the entire study reach along Route 3 (topography can be seen in Fig. 3). Boxed area is the area shown in detail in Figure 7.

SUMMARY

Map analysis, assessment of landslide history, and geomorphic field mapping of past landslide deposits was conducted for selected reaches along Route 3 in Jackson County, Illinois. A landslide potential map was produced from data obtained from published reports, topographic slope analysis, and reconnaissance geomorphic mapping. The coincidence of highest potential hazard ranking with sites of observed landslide activity outside the detailed study area indicates that the methodology is reliable. This study was conducted rapidly and with minimal cost and can be accomplished with a minimum of basic geomorphological training. Similar maps may result in significant savings in highway construction and maintenance planning.

ACKNOWLEDGMENTS

Julie Brown and Brian Bussey contributed greatly to the early phases of this investigation. We thank the Jackson and Randolph County highway departments (Illinois Department of Transportation) for the liberal use of their documents, photos, and time.

REFERENCES

- Brunsden, D., and D.B. Prior, "Slope Instability", 620p, John Wiley, New York (1984)
- Costa, J.E., and V.R. Baker, "Surficial Geology: Building with the Earth", 498pp., John Wiley, New York (1981)
- United States Geological Survey, Circular 880, 49p. (1982)
- Hansen, A., in "Slope Instability", p. 523-602, John Wiley, New York (1984)
- Moser, M. and Hohensin, F., Engineering Geology, 19, 403-426, (1983)
- Cararra, A., Mathematical Geology, 15, 403-426, (1983)
- Terranova, T.F., 113pp, M.S. Thesis, Southern Illinois Univ., Carbondale, (1987)
- Federal Highway Administration, "PCSTABL4", 100p., Dept. Civil Engineering, Purdue Univ. (1985)
- Jackson County Highway Department, Illinois Department of Transportation, 30p. (1986)
- DeGraff, J.V., Environmental Geology, 2, 203-214, (1978)
- Peatross, J.L., 150p., M.S. Thesis, Univ. of Virginia, Charlottesville (1984)
- Swann, D.H., Illinois State Geological Survey, Report of Investigation 216, 91p. (1963)
- Soil Survey of Jackson County, Illinois, U.S. Dept. Agr., Soil Conservation Serv., 192p. (1979)

The Use of Geosynthetics to Support Roadways Over Sinkhole Prone Areas

R. BONAPARTE

GeoServices, Inc. Consulting Engineers, Boynton Beach, Florida

R. R. BERG

The Tensar Corporation, Morrow, Georgia

S. T. BUTCHKO

The Tensar Corporation, New York, New York

HGS-87-21

ABSTRACT

THE USE OF GEOSYNTHETICS to reinforce road embankments and other earth structures built over areas prone to sinkhole development or other types of voids is described. Geosynthetics are increasingly being used beneath embankments to provide a flexible safety net which prevents catastrophic collapse of an embankment into a void. Tensioned membrane and soil arching theories applicable to the design of geosynthetic safety nets are examined. Design procedures and simple design charts are presented for selection of reinforcement type and number of layers. A procedure for computation of the required reinforcement anchorage length is also presented. Geosynthetic properties relevant to design are reviewed. Recommendations are provided on testing methods and interpretation to evaluate reinforcement load-strain-time properties for design. The case history of a multi-layer geosynthetic safety net to span a 12 m wide sinkhole in Lehigh County, Pennsylvania is presented.

Introduction

Voids can occur under an existing roadway or other earth structure due to a variety of causes including cavities resulting from solution activity or collapse of abandoned underground mine works, thawing of subsurface ice lenses (thermokarsts) and differential foundation settlements. Whatever the cause, loss of subgrade support can result in settlement, distortion, and cracking of the overlying pavement structure, loss of serviceability, or worse, total collapse.

Tensile reinforcing elements can be used to support roadway embankments and other earth structures over voids (1,2,4). The problem of design of an earth structure over a void can be

subdivided into two categories: (i) design to resist complete collapse into the void while accepting loss of serviceability; and (ii) design to limit deformations so as to maintain serviceability of the structure over the void. In the first category, the reinforcement can be considered to be acting as a safety net and the reinforcement function is only needed until the void is filled and the structure repaired. In the second category, the reinforcement function is permanent and the reinforced soil structure must behave as a beam or slab with sufficient bending stiffness to limit deformations to an acceptably small level. This paper addresses the use of polymeric materials, termed geosynthetics, as inclusions in soil structures to create a temporary safety net which prevents structural collapse into a void. Because polymeric materials used for soil reinforcement (polyester, polyethylene, and polypropylene) have relatively low tensile moduli compared to steel or concrete, they cannot be used to create "rigid" reinforced soil beams or slabs. An example of the use of steel strip reinforcement to create a reinforced soil slab can be found in (11).

Properties of Geosynthetics

Geosynthetics refer to man made materials used in geotechnical and related engineering applications. Descriptions of the ranges of geosynthetic materials and functions can be found in (5,6). Due to their high tensile strengths and moduli compared to other polymer materials, the two types of geosynthetics most often used in reinforced soil structures are geogrids and woven geotextiles.

Geogrids are reinforcing products consisting of discrete longitudinal ribs that are connected to transverse bars. The spacings of the ribs and bars are large enough to allow earth

Reprinted from the Proceedings of the Second Multidisciplinary Conference on Sinkholes and the Environmental Impacts of Karst, Orlando, Florida, 9-11 February 1987

fill to penetrate the grid apertures. This penetration of soil particles into the plane of the reinforcement results in compaction-induced and load-induced soil-reinforcement "interlocking". There are two main types of geogrids (2): those manufactured by heating and drawing a perforated polymer sheet in one or two perpendicular directions; and, those manufactured by overlapping perpendicular, or nearly perpendicular, strands and then welding the strands at their junctions. Geogrid thicknesses range from about 1.5 mm up to about 6 mm. Geogrid masses per unit area range from about 150 g/m² up to about 1500 g/m².

Geotextiles are defined by the American Society for Testing and Materials (ASTM) as "any permeable textile material used with foundation, soil, rock, earth, or any other geotechnical engineering-related material, as an integral part of a man-made project, structure or system". Woven geotextiles "are composed of two sets of filaments or yarns systematically interlaced to form a planar structure" (5). The two sets of filaments or yarns are typically perpendicular. For a given mass per unit area and polymer type, woven geotextiles tend to be stronger than nonwoven geotextiles and are therefore more commonly used in reinforcement applications. Woven geotextile thicknesses range from about 0.5 mm to 5 mm, and in some special cases, up to 10 mm. Woven geotextile masses per unit area range from about 100 g/m² up to about 1500 g/m².

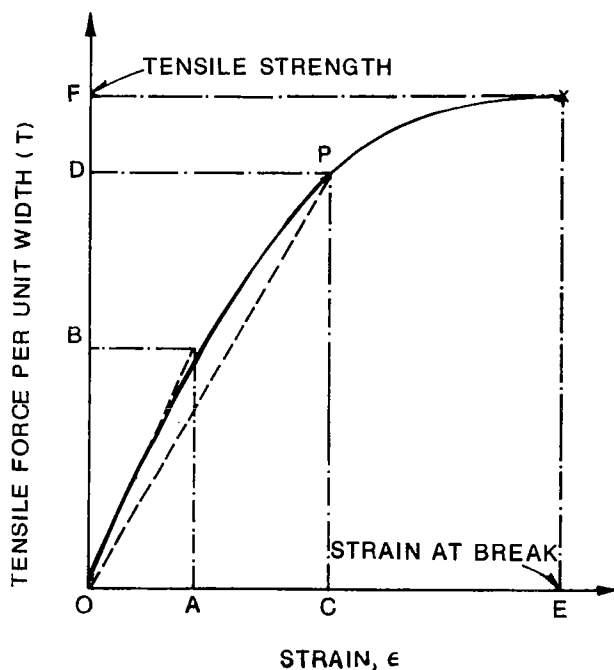


Figure 1: Tensile force per unit width versus strain and definition of terms.

Tensile Properties for Design

Tensile properties of geosynthetic materials relevant to reinforcement design are illustrated in Fig. 1 and include:

- tensile strength - the maximum resistance to deformation when subjected to tension by an external force, kN/m (Point F in Fig. 1);
- tensile resistance - the force per unit width when subjected to a specific strain, kN/m (Point D is the tensile resistance for tensile strain C in Fig. 1);
- secant tensile stiffness - the ratio of the change in tensile force per unit width to a change in strain between the origin and any other point on a force per unit width strain curve, kN/m (equal to D divided by C in Fig. 1); and
- strain at rupture (Point E in Fig. 1).

These tensile properties can be evaluated using the ASTM wide strip test method (D4595-86). The dimensions of the wide strip test specimen are 200 mm wide by 100 mm long. The test is performed at 21°C and at a tensile strain rate of 10% per minute. It must be noted that the short load duration associated with this relatively rapid strain rate does not provide results which reflect the visco-elastic strength loss exhibited by polymeric reinforcing

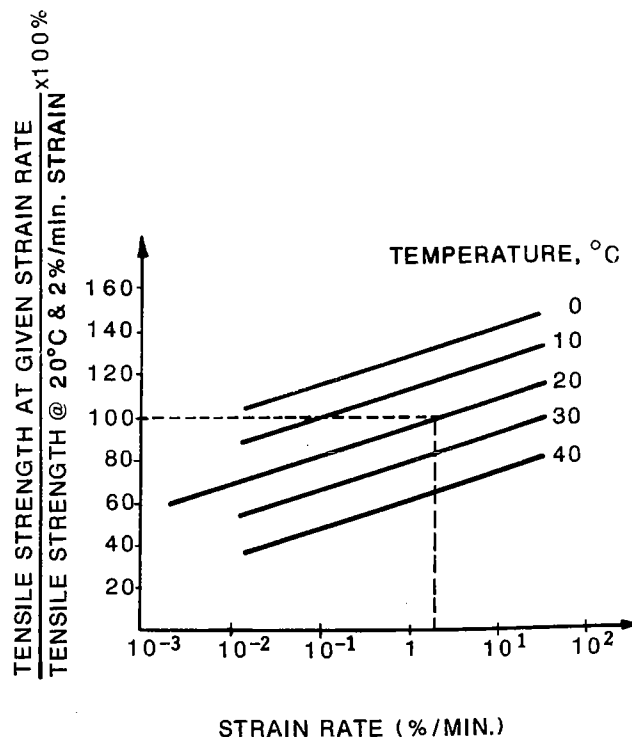


Figure 2: Influence of strain-rate on tensile strength of polypropylene geogrid (McGown, et al., 1984).

materials under sustained loading. To account for this strength loss in permanent load applications, the constant-rate-of strain test is often replaced with a constant-load creep test. Creep tests are performed for durations of from 1000 to 10,000 hrs and the test results are extrapolated to the design life of the earth structure. Since the role of a geosynthetic safety net is a temporary one, long-duration creep tests are not necessary. A simpler approach for temporary reinforcement applications is to carry out a series of wide strip tests at a range of strain rates (Fig. 2) and to conservatively extrapolate these results to the required design life of the safety net. In the absence of a range of wide strip test data, the authors' suggest the use of an allowable design load equal to 20% of the standard wide strip tensile strength. A minimum factor of safety of 1.2 to 1.3 is typically applied to the allowable design load.

Reinforcement Direction

An important design consideration in selecting polymeric reinforcement as a safety net is whether one or two dimensional strength is needed. Geogrids and geotextiles usually have different longitudinal (machine direction) and transverse (cross-machine direction) strengths. Reinforcement may be needed in one direction only, as would be the case for a long, slender void such as a trench. In this instance, the reinforcement should be placed with its strongest direction perpendicular to the long direction of the void. Design is based on an assumption of plane strain deformations. Reinforcing materials with anisotropic strength characteristics can be efficiently used for this case and there are no requirements to provide a strong seam or connection between adjacent strips of reinforcement in the direction perpendicular to the length of the void.

The use of reinforcement with highly anisotropic strength characteristics becomes less efficient when the reinforcement must span a circular or irregularly shaped void. In this case, it may be more efficient to select a reinforcing material with isotropic (or nearly isotropic) strength properties. If more than one width of reinforcement is required, the overlap or joint between adjacent strips must be able to transmit tensile stresses. In this case, geotextile seams should be sewn and not overlapped. Ideally, sewn seams should be as strong as the geotextile itself. Practically, sewn seams rarely exceed two-thirds of the strength of the geotextile itself. Seam strengths, therefore, should meet reinforcement design strength requirements. Adjacent geogrid strips do not need to be mechanically joined if their percent open areas are large (greater than about 60 to 70%) and if their aperture size permits interlock by the fill material (2). In this case, geogrids need only be overlapped. Overlap widths are based on direct shear or pullout test results

with a minimum overlap of 0.6 m being typical. For applications requiring mechanical joining of adjacent geogrid strips, polymer dowel bars, braid and metal hog rings have been used. However, actual overlap length and mechanical connection requirements for geogrids are specific for different geogrid products.

Soil-Reinforcement Interface Shear Behavior

Evaluation of soil-reinforcement interface shear behavior is necessary in order to determine the required embedment length of the reinforcement beyond the edges of the void. The embedment length must be sufficient to prevent slippage of the embedded portion of reinforcement. A minimum factor of safety of 1.5 is typically applied to the pullout length required to prevent slip under the design load.

Soil-reinforcement interface shear behavior can be evaluated using either direct shear tests or pullout tests. For design purposes, results from both direct shear and pullout tests are usually presented as plots of maximum apparent interface shear stress versus normal stress. The maximum apparent interface shear stress is calculated from pullout tests results by dividing the pullout resistance by twice the embedded surface area of the reinforcement. For cohesionless soils, these plots can be further interpreted in terms of a soil-reinforcement interface friction coefficient, $\mu \tan \phi$, where ϕ is the internal angle of friction of the soil in contact with the reinforcement and μ is termed the coefficient of soil-reinforcement interaction.

For geotextiles, which rely mainly on interface friction, μ is always less than one. In the absence of test data, μ can be assumed to be equal to 0.67 if the geotextile is in contact with granular soil and 0.50 if the geotextile is in contact with cohesive soil. Geogrid coefficients of interaction may be significantly greater than 1.0 if the geogrid develops resistance to pullout by passive resistance of soil against the transverse bars of the grid. This passive resistance may not be significant in cohesive soils and/or if the bonds at the junctions between the grid's longitudinal ribs and transverse bars are weak. In these cases, the coefficient of interaction will be lower. If the junctions between a grid's longitudinal ribs and transverse bars are very weak, the grid may respond to pullout forces as a series of longitudinal strands with virtually no resistance provided by the transverse bars. In this case, the required embedment length will be larger than those for either geotextiles or geogrids with strong junctions.

The pullout capacity of geosynthetic soil reinforcement can be estimated from:

$$T_p = \sigma_v L (\mu_t \tan \phi_t + \mu_b \tan \phi_b) \quad (1)$$

where: T_p = pullout capacity of reinforcement per unit width (kN/m); σ_v = vertical stress at elevation of reinforcement (kN/m²); L = embedded length of reinforcement (m); μ_t = coefficient of interaction along top side of reinforcement; ϕ_t = angle of internal friction for soil on top of reinforcement; μ_b = coefficient of interaction along bottom side of reinforcement; ϕ_b = angle of internal friction for soil under the reinforcement.

Design

Tensioned Membrane Theory - An assumption can be made that reinforcement spanning a void behaves as a membrane able to transmit tensile stresses but not shear stresses. If it is further assumed that the soil-reinforcement interface above the void is frictionless, the applied stresses will be normal to the membrane and, for plane-strain conditions, the shape of the membrane will be a circular arc. The tensile resistance, T (kN/m), in a layer of reinforcement spanning an infinitely long void of width b (m), can be calculated for the above conditions by solving the following two equations (4):

$$T = K \left[\frac{2f(\epsilon)}{(180/\pi)} \sin^{-1} \left(\frac{1}{2f(\epsilon)} \right) - 1 \right] \quad (2)$$

$$f(\epsilon) = 1/4 \left[\frac{2y}{b} + \frac{b}{2y} \right] \quad (3)$$

where: K = secant tensile stiffness of the reinforcement (kN/m) and y is the maximum vertical reinforcement deflection (m). The strain in the reinforcement, ϵ , is obtained by dividing the reinforcement tensile resistance by the secant tensile stiffness. Equation 2 can also be used to calculate an approximate tensile resistance in reinforcement spanning a circular void of diameter D (m) by dividing the right hand side of the equation by two (4). Equations 2 and 3 have been solved for a range variables (Fig. 3). The calculations to obtain Fig. 3 assumed that the strain in the reinforcement was uniform throughout the void and zero in the embedment zone. A more accurate approach would account for strain in the embedment zone, as suggested by Kinney (9).

Arching - A simple extension to the tensioned membrane theory described above considers the reduction in vertical stress due to the effect of soil arching over a depression. Arching refers to the transfer of stress from a yielding soil mass onto adjacent nonyielding parts of a soil mass. The mechanism of arching in a soil mass has been described in detail by Terzaghi (12). In the case of a void under a portion of an embankment, arching would reduce the stress on geosynthetic reinforcement spanning a void by transferring part of the stress to adjoining stable portions of the embankment.

Terzaghi (12) presented an approximate method for calculating the vertical stress on a horizontal plane at the base of a soil mass due to yielding of a section of the base. Terzaghi assumed that yielding produced vertical shear surfaces extending from the edges of the yielding base up to the top of the soil mass. Restricting this discussion to h/b less than 2.5, where h (m) is the soil thickness and b (m) is the width of the void, and ignoring the influence of surcharge loads, the vertical stress, p' (kPa), under plane-strain yielding conditions is given by:

$$p' = \frac{b\gamma}{2k\tan\phi} \left[1 - e^{-(k\tan\phi)(\frac{2h}{b})} \right] \quad (4)$$

where: γ = unit weight of soil (kN/m³); ϕ = angle of internal friction of soil; and k = ratio of horizontal to vertical stress along the vertical shear surfaces.

Kezdi (8) used assumptions similar to those by Terzaghi to derive an equation for the vertical stress, p' (kPa), on a circular horizontal yielding surface of diameter D (m):

$$p' = \frac{D\gamma}{4k\tan\phi} \left[1 - e^{-(k\tan\phi)(\frac{4h}{D})} \right] \quad (5)$$

Various references provide recommendations regarding the appropriate value for the earth pressure coefficient k . Most recently, Handy (7) suggested the following expression:

$$k = 1.06 (\cos^2\theta + k_a \sin^2\theta) \quad (6)$$

where:

$$k_a = \tan^2 (45 - \phi/2) \quad (7)$$

and:

$$\theta = 45^\circ + \phi/2 \quad (8)$$

The value of k from Equation 6 has been used with Equations 4 and 5 to compute vertical stresses for a range of conditions. The results from these computations are shown in Fig. 4 for a cohesionless soil with $\phi = 30^\circ$. It can be seen that for a given depth of fill, the vertical stress on the reinforcement decreases with decreasing void width or diameter. From a design standpoint, the magnitude of this decrease is significant.

Soil arching can be destroyed by vibrations, percolating ground water and other external disturbances. The use of arching in design of a safety net should consider the potential for these factors to reduce the effectiveness of the arch. Continuously graded fills with angular particles, slightly cohesive soils, and

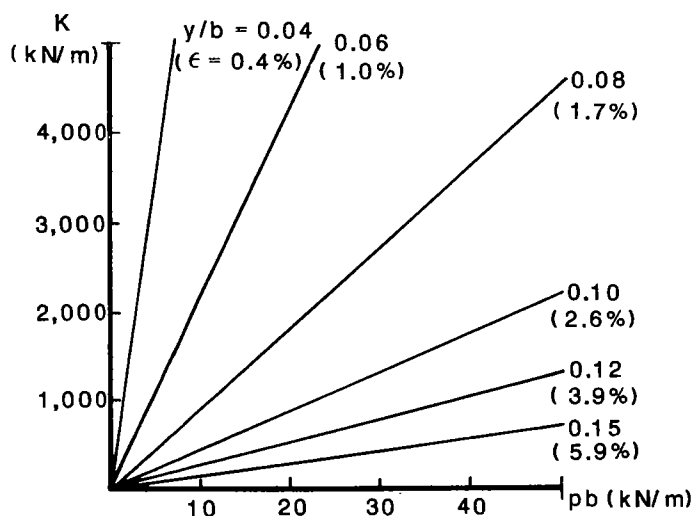
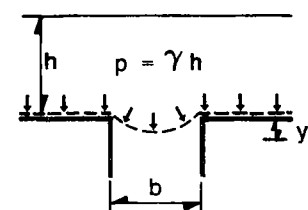


Figure 3: Required secant tensile stiffness for a tensioned membrane spanning an infinitely long void of width b (m) (Chart from Bonaparte and Christopher, 1981; based on equations from Giroud, 1981).

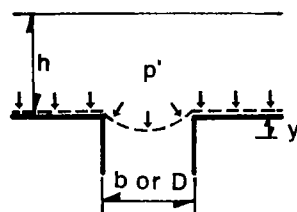
thick fill layers will develop more stable arches than will fills with uniform round particles and thin fill layers. Since the role of a safety net is temporary it may be acceptable to consider arching for design calculations for most practical applications.

Other Theories - Fluet, et al. (3) have suggested that it might be possible to use elastic beam theory to analyze the response of a soil-reinforcement system overlying a void. Elastic plate theory could also be considered. However, in order to use these approaches, it is necessary to determine section properties for the soil-reinforcement system. This can only realistically be done by empirically back-calculating section properties from large scale tests. To date, this has not been attempted.

Tests

Data exists on two different sets of tests involving the use of geosynthetics to support earth structures over voids.

In the first series of tests, Fluet, et al. (3) constructed two 1.5 m high reinforced embankments over a 4.7 m wide, 0.8 m deep void.



SOIL

$$\phi' = 30^\circ$$

$$c' = 0 \text{ psf}$$

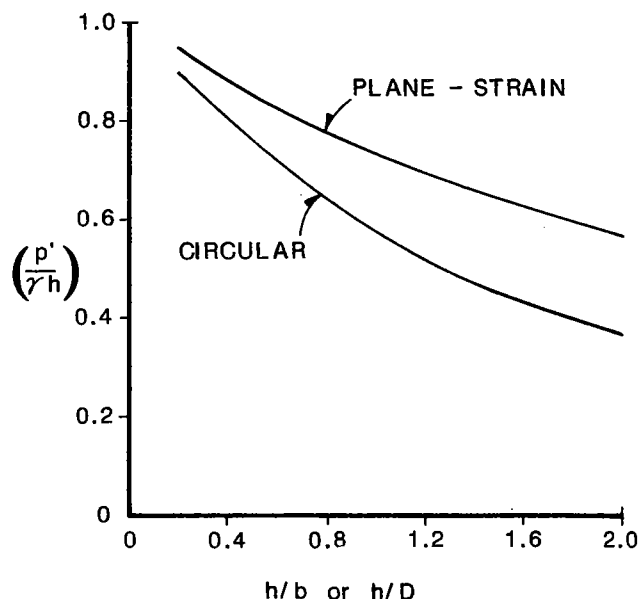


Figure 4: Theoretical reduction in vertical stress due to cohesionless soil arching over an infinitely long void of width b (m) or a circular void of diameter D (m).

In the first test, the embankment fill was placed in lifts over a layer of geogrid reinforcement. The geogrid spanned the void and was anchored on both sides. The geogrid deformed incrementally into the void with each succeeding lift of fill. The deformed shape of the reinforcement over the void indicated that it was acting as a tensioned membrane. The maximum vertical deflection of the embankment and strain in the reinforcement are in agreement with values calculated by the authors using Fig. 3. This test would model construction of an embankment over an existing void or localized weak zone, or the case where the width of the void is large compared to the thickness of the embankment. In these situations, soil arching would be small, as indicated in Fig. 4.

In the second Fluet, et al. test, a pneumatically inflatable support system consisting of air bags was placed in the 4.7 m wide void. The embankment was supported during construction using the air bags. Once construction was complete, the pressure in the air bags was decreased in increments and the response of the embankment-reinforcement system was observed. Interestingly, the vertical deformation of the embankment in this second test was significantly

less than the deformation predicted using tensioned membrane theory, even with arching. In this test, interlocking of the fill and high tensile modulus geogrid at the base of the embankment resulted in localized confinement of the fill particles and an improved ability of the system to span the void. It appears that particle confinement was enhanced by the fact that the reinforcing geogrid had an aperture dimension on the order of 40 mm and the embankment fill was a well-graded gravel with maximum particle size also on the order of 40 mm. This combination of reinforcement opening size and fill particle size probably resulted in the maximum possible amount of soil-geogrid interlocking.

In the second series of tests, Kinney (9) constructed an approximately 1-m thick road embankment over 1-m wide and 2-m wide trenches. Single layers of different types of geosynthetic reinforcement supported the embankment over the trenches. Embankment deformations during construction were recorded and were found to be in approximate agreement with those predicted using tensioned membrane theory. However, as in the case of Fluet et al.'s first test, the embankment was built over an existing void and thus soil arching and/or soil-reinforcement interlocking were minimized. After construction, Kinney trafficked the embankment with 200 passes of a loaded dump truck. Kinney observed no "visual creep" of the reinforcement and measured traffic induced elastic deflections into the 2-m wide trench on the order of 1 mm to 2 mm.

In conclusion, based on the two sets of tests described above, it appears that tensioned membrane theory provides a lower-bound, conservative estimate of soil-reinforcement behavior. This conservatism is greatest for conditions which allow soil arching, and when soil reinforcement interlocking provides a confined zone at the base of the embankment.

Vera Cruz Road Case History

On December 19, 1983 a large sinkhole developed along Vera Cruz Road in upper Saucon Township, Lehigh County, Pennsylvania. Within three days, an adjacent underground roadway bridge collapsed into the sinkhole. The underground bridge was supported by four 12-m long steel girders and was constructed as part of an earlier sinkhole repair scheme. The initial dimensions of the sinkhole were 60 m by 30 m (Figs. 5 and 6). Its initial depth ranged from 11 m to 18 m. The sinkhole slightly increased in size between December, 1983 and June, 1984, and then remained stable (13).

Sinkhole development was not a new phenomenon to the residents living adjacent to Vera Cruz Road. Sinkhole activity in this area was first documented in the early 1960's. Major sinkhole activity along the road alignment was

first encountered in 1971. By the late 1970's, sink development along Vera Cruz Road had become a perennial problem (13).

The local residents were concerned about the interruption of local traffic, water service, and emergency services resulting from the roadway collapse (13). A task force of government agencies and local residents was established to develop a solution to the Vera Cruz Road sinkhole problem. A detailed geologic and geotechnical study was initiated. Results from this study (13) indicated that the sink activity was probably the result of seepage of perched ground water and surface runoff down to and through jointed dolomitic bedrock. The water percolated through the near surface clayey silts and silty clay soils downward into the fractured bedrock causing softening and erosion of the joint materials, erosion of the near surface soils, and eventually, sinkhole formation. Sinkhole potential was increased at this site by human activities which included increased surface water runoff due to development activities, rerouting of a surface stream, and a 60 to 100 m drop in the water table caused by ground-water pumping for an underground zinc mine, which operated between 1953 and October 1983. More than 70% of some 125 documented sinkhole collapses in the Saucon Valley since 1965 occurred where ground-water elevations were 60 m or more below existing grade (13).

Vera Cruz Road remained closed until sink stabilization and roadway reconstruction were completed in the Spring of 1986. Repair work consisted of site drainage improvements, stabilization of the main sink area, and construction of a roadway support system. Only the roadway support system is addressed herein.

A nonconventional approach was taken in the design and construction of the roadway support system. A reinforced soil mattress was constructed at the base of the roadway embankment to support the roadway in the event new sinks develop. The reinforced soil mattress was designed to support the pavement over small openings, while maintaining road serviceability. However, it was also designed for the more critical case of providing a safety net over the main sink area, should it reopen to its former dimensions.

Although reinforced soil mattresses have been used for sinkhole repair (11), this is the first project utilizing a geosynthetic as the soil reinforcing element. The mattress was 1.2-m high and 35-m long (Fig. 7) and consisted of crushed stone gravel interbedded with thirteen layers of high density polyethylene geogrid. The geogrid utilized was a uniaxially drawn material with a longitudinal wide strip index strength of approximately 70 kN/m and a mass per unit area of about 920 g/m² (TENSAR SR2 Geogrid). A relatively impervious fill was

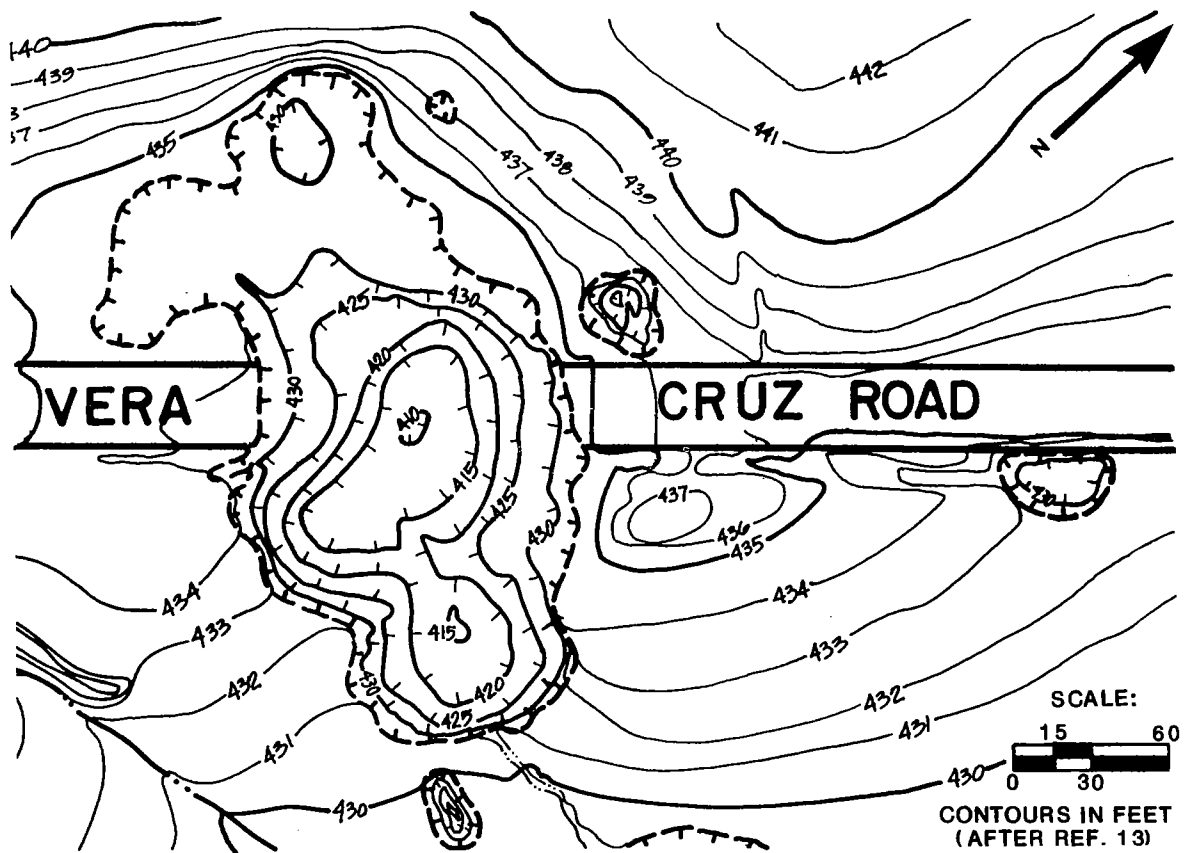


Figure 5: Plan View - Vera Cruz Road sinkhole.



Figure 6: Aerial Photograph - Vera Cruz Road sinkhole.

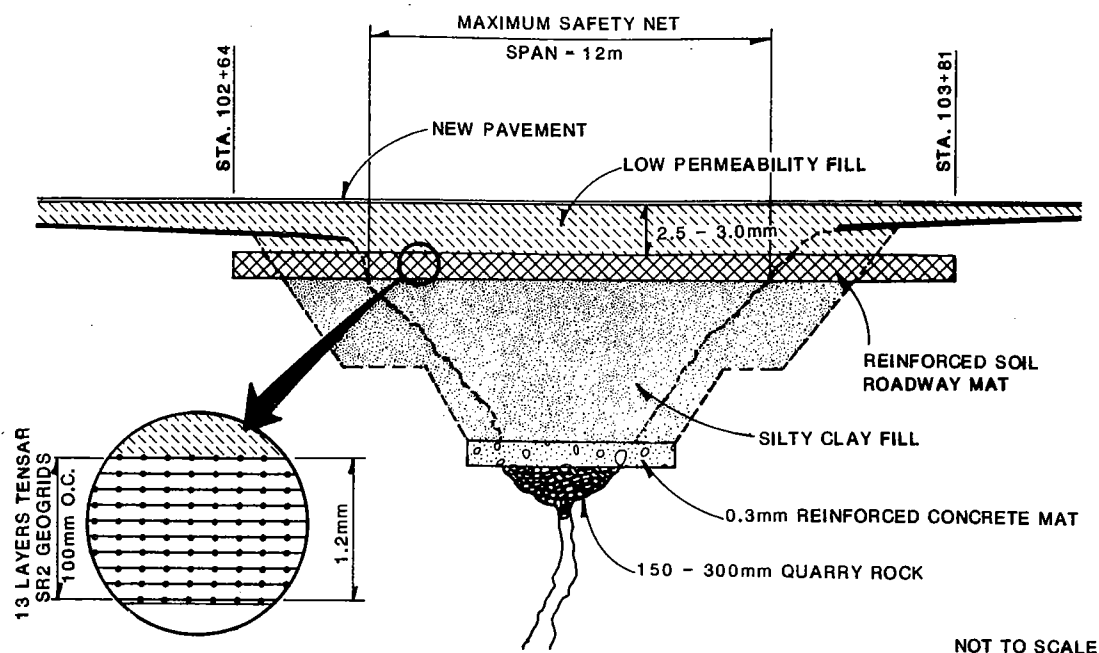


Figure 7: Geosynthetic reinforced soil 'safety net' constructed to support Vera Cruz Road over sinkhole.

placed above the reinforced soil mattress to minimize surface-water percolation through the sinkhole repair zone. The mattress was designed to span a 12-m wide sinkhole, at a depth of approximately 4 m beneath the top of pavement. This depth was dictated by the 3-m minimum cover above the mattress required to provide clearance for utility lines and remedial drainage piping. The specified quantity of reinforcing material was large due to the depth of fill over the mattress (4 m) and the large mattress span (12 m). This quantity decreases significantly as the applied loads due to the depth of overburden decrease, and as the span length decreases.

The geogrid reinforced mattress is designed to deflect and thereby provide a warning should it lose its underlying support. The mattress is further designed, using tensioned membrane theory, to resist collapse and act as a safety net should a 12 m long sink open beneath it. Based on an assumption that the 12-m sink was to appear instantaneously, the visco-elastic response of the geogrid to the applied loads would result in time-dependent deflection of the roadway. Calculations using tensioned membrane theory and time-dependent geogrid moduli predict vertical road deflections at the center of the 12-m sink of 3.1 m, 3.8 m, and 4.3 m at 1 hour, 10 hours, and 100 hours, respectively.

Acknowledgements

The authors would like to thank the engineers at Valley Foundation Consultants, Inc. and the Pennsylvania Department of Transportation who designed the Vera Cruz Road project, for permission to use the Vera Cruz case study and for their stimulating input; J. P. Giroud for suggesting that they consider the effect of soil arching on the tensioned membrane forces in the reinforcement; J. F. Beech and J. P. Giroud for reviewing a draft of the paper; Doris Campbell and Sandy Waddell for typing the paper; and Rick Komada for drafting the figures.

References

1. Bonaparte R. and Christopher, B.R., 1987. Design and Construction of Reinforced Embankments Over Weak Foundations: Symposium on Reinforced Layered Systems, Transportation Research Board Meeting, Washington D.C., January. (To be published in Transportation Research Board Record.)
2. Bonaparte R., Holtz, R.D. and Giroud, J.P., 1985. Soil Reinforcement Design Using Geotextiles: ASTM Symposium Geotextile Testing and the Design Engineer, Los Angeles, July, 127p. (to be published in ASTM Special Technical Publication 592.)

3. Fluet, J.E., Christopher, B.R. and Slaters, A.R., 1986. Geosynthetic Stress-Strain Response Under Embankment Loading Conditions: Proceedings of the Third International Conference on Geotextiles, Vienna, Vol. 1, pp. 175-180.

4. Giroud, J.P., 1981. Designing with Geotextiles: Matériaux et Constructions, Vol. 14, No. 82, pp. 257-272. (Reprinted in: Giroud, J.P., 1984: Geotextiles and Geomembranes, Definitions, Properties and Design, Industrial Fabrics Association International, St. Paul, Minnesota, 404;.)

5. Giroud, J.P., 1984. Geotextiles and Geomembranes: Geotextiles and Geomembranes Journal, Vol. 1, No. 1, pp. 5-40.

6. Giroud, J.P., 1986. From Geotextiles to Geosynthetics: A Revolution in Geotechnical Engineering: Proceedings of the Third International Conference on Geotextiles, Vienna, Vol. 1, pp. 1-18.

7. Handy, R.L., 1985. The Arch in Soil Arching: Journal of the Geotechnical Engineering Division, ASCE, Vol. III, No. 3, March, pp. 302-318.

8. Kezdi, A., 1975. Lateral Earth Pressure: Foundation Engineering Handbook, Van Nostrand Reinhold Company, NY, pp. 197-220. (Edited by H.F. Winterkorn and H.Y. Fang.)

9. Kinney, T.C., 1986. Tensile Reinforcement of Road Embankments on Polygonal Ground by Geotextiles and Related Materials: Interim Report to State of Alaska, Department of Transportation and Public Facilities, Fairbanks, Alaska, March, 25p.

10. McGown, A., Andrawes, K.Z., Yeo, K.C. and DuBois, D.D., 1984. The Load-Strain-Time Behavior of TENSAR Geogrids: Proceedings of the Symposium on Polymer Grid Reinforcement in Civil Engineering, Institution of Civil Engineers, London, pp. 11-17.

11. Steiner, R.S., 1975. Reinforced Earth Bridges Highway Sinkhole: Civil Engineering, ASCE, July, pp. 54-56.

12. Terzaghi, K., 1943. Theoretical Soil Mechanics, John Wiley & Sons, Inc., NY, 510 p.

13. Valley Foundation Consultants, Inc., 1984. The Vera Cruz Road Sinkhole Investigation, A Report to Upper Saucon Township, Lehigh Township, Pennsylvania Department of Transportation, and Pennsylvania Department of Environmental Resources. Bethlehem, Pennsylvania.

Ground Magnetic Surveys in Appalachian Valley Karst

STEVEN J. STOKOWSKI, JR.
Consulting Geologist/Petrographer
Ashland, Massachusetts

HGS-87-22

ABSTRACT

GROUND MAGNETIC STUDIES can help locate soil-filled solution crevices and filled sinkholes. The magnetic anomalies which correlate with these difficult-to-detect features exist because of the contrast between the carbonate bedrock and iron-rich terra-rossa soil. High reading densities along multiple traverses and corrections for time variations are required. Anomalies over lineaments and filled sinkholes in the Newala Fm. (Knox Group) carbonate rocks correlate with the karst.

INTRODUCTION

This paper reports ground magnetic surveys undertaken to help solve a quarry flooding problem in the southern Appalachian Valley. The project was an integrated program of geologic, hydrologic, and geophysical surveys including acoustic emission, ground magnetic, and spontaneous potential measurements. The bulk of this work was reported elsewhere (Stokowski, 1987) and will only be mentioned as necessary for understanding the ground magnetic surveys.

GEOLOGY

The ground magnetic surveys were performed on the karst surface of the Newala Fm. The Newala is an Ordovician-age, 168 m (550 ft) sequence of gray interbedded limestone and subordinate dolomite in the Knox Group of geologic formations. The Knox Group in the study area is deformed into a dome that is crosscut by a number of anticlines, synclines, monoclines, and lineaments (Figure 1). The Newala Fm. karst is influenced by the fracturing associated with these structural features.

The Newala Fm. in the study area has a soil-covered fluviokarst with

about 37 m (120 ft) of relief. Karstification is extensive and extends to approximately 100 m (330 ft) below the surface or about 70 m (220 ft) below the river. Large-scale karst features include bowl-shaped sinkholes, funnel-shaped sinkholes in areas of fresh collapse, parallel trenches, and solution crevices.

The solution crevices are commonly a maximum of 0.3 m (1 ft) wide at limestone beds and pinch at dolomite beds; they may be 10's of meters deep. They are joint-controlled and often follow the trend of the lineaments. The crevices are often enlarged to approximately a meter wide below the soil/rock contact and filled with terra-rossa soil.

Two lineaments of great significance to the quarry flooding problem are shown on Figure 1. They and the area in-between were covered with ground magnetic traverses. The first, or northern lineament, is about 100 m (300 ft) wide and oriented WSW-ENE. It is defined by the location of a river-bank cavity whose water was dye-traced to the quarry, an associated ENE jog in the river, and by two reactivated and one fresh sinkhole in the field about 460 m (1500 ft) to the NE. The second, or southern, lineament is about 150 m (500 ft) wide, and is oriented SW-NE. It is defined by several old bowl-shaped sinkholes visible on air photos, three reactivated sinkholes in fields to the south and southeast of the quarry, and one fresh sinkhole to the east of the quarry. Karst trenches also occur along the trend of this lineament.

GROUND MAGNETICS

Ground magnetic studies are a little-used technique for detecting terra-rossa filled karst features (McDowell, 1975; Miller et al, 1984; Stokowski, 1987). The magnetometer

detects the magnetic contrast between the carbonate bedrock and the iron-rich terra rossa soil. Anomalies correlate with deeper soil profiles.

The equipment used in this study was a Geometrics Model G856 recording proton magnetometer. During the survey the magnetometer was positioned 2.4 m (8 ft) above each station. Data collection was in a grid pattern at 3-m (10 ft) intervals along traverses. These traverses were oriented parallel and perpendicular to lineaments in the Newala Fm. (Knox Group) carbonate rocks. Sets of mutually parallel traverses, separated by 15 to 30 m (50 to 100 ft), began with a base station reading so that diurnal variations could be removed.

Computer processing and plotting in an inverted mode produced pseudo-profiles of the soil/rock interface for convenient analysis (Figures 2-5). The surveys in Figures 2 and 5 overlie the lineaments outlined in Figure 1. Variations of about 2 gamma were resolvable, and anomalies of 5 to 10 gamma above the background could be correlated across traverses.

Analysis of the pseudo-profiles was a two-step process. The first step defined the location of the terra-rossa filled crevices. This task was simple because the crevices could be modeled as vertical rectangular prisms with a depth and length much greater than the width. This model yields a vertical projection of the anomaly (Breiner, 1973) in the magnetic field inclination of the study area (Figure 6). One need only mark the locations of the pseudo-profile lows (the magnetic highs) and then connect the dots into linear crevice trends. The second step was to contour and then interpret the remainder of the magnetic data. This interpretation was not as easy as the previous interpretation for the crevice model because the remaining anomalies are caused by filled sinkholes. These sinkholes could be modeled as a bowl or inverted cone magnetic source that is much larger than the filled crevices. This model yields a magnetic anomaly (Figure 6) in the magnetic field inclination of the study area that is offset from the actual location of the feature.

Figure 7 contains the interpretation of the magnetic surveys. Indicated are the trends of the terra-rossa filled solution crevices and the locations of areas of deeper and shallower soil. The orientations of the terra-rossa-filled

crevices compare favorably with the lineament locations derived from air photos and the deepest soil profiles determined by drilling. Several of the interpreted crevices were drilled and proved to be mud-filled crevices, in one case more than 36.6 m (120 ft) deep, and in contrast to the general soil depth of approximately 5-10 m (15-30 ft).

CONCLUSIONS

Ground magnetic studies are a newly developed and useful technique for defining the location of soil-filled solution crevices and filled sinkholes. The technique works because the magnetometer detects the contrast between the carbonate bedrock and iron-rich terra-rossa soil. Anomalies correlate with deeper soil profiles. Computer processing of the magnetic data is necessary to remove diurnal variations. Plotting in an inverted mode produced pseudo-profiles of the soil/rock interface for convenient analysis. Variations of about 2 gamma were resolvable, and anomalies of 5 to 10 gamma above the background could be correlated across traverses. These anomalies were drilled in the lineaments and proved to be terra-rossa-filled solution crevices, in one case more than 36.6 m (120 ft) deep. Complex anomalies in sinkhole areas away from the lineaments correlated with the surface depressions but were often multiple or displaced to one side of the sinkhole.

REFERENCES

- Stokowski, S.J. Jr., Locating Groundwater Conduits in Carbonate Rocks, in Beck et al (eds.), Karst Hydrogeology: Engineering and Environmental Applications, A.A. Balkema Publishers (Boston), 185-196 (1987)
- McDowell, P.W., Detection of clay-filled sink-holes in the chalk by geophysical methods, Quart. J. Eng. Geology 8 303-310 (1975)
- Miller, C.H., J.R. Ege, J.K. Odum, and J.J. Golub, Jr., Preliminary seismic-velocity and magnetic studies of a carbonate rock - sink-hole area in Shelby County, Alabama, U.S. Geol. Survey Open File Rept. 84-409 (1984)
- Breiner, S., Applications manual for portable magnetometers, Geometrics, Palo Alto, CA, 58 p (1973)

APPENDIX

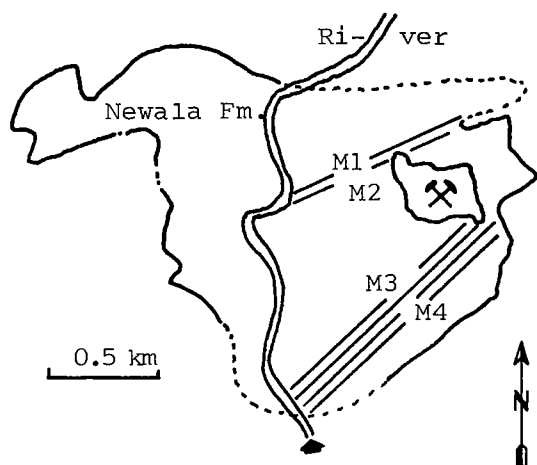


Figure 1. Map illustrating outcrop of Newala Fm. dome, lineaments, quarry, river, and location of magnetic surveys.

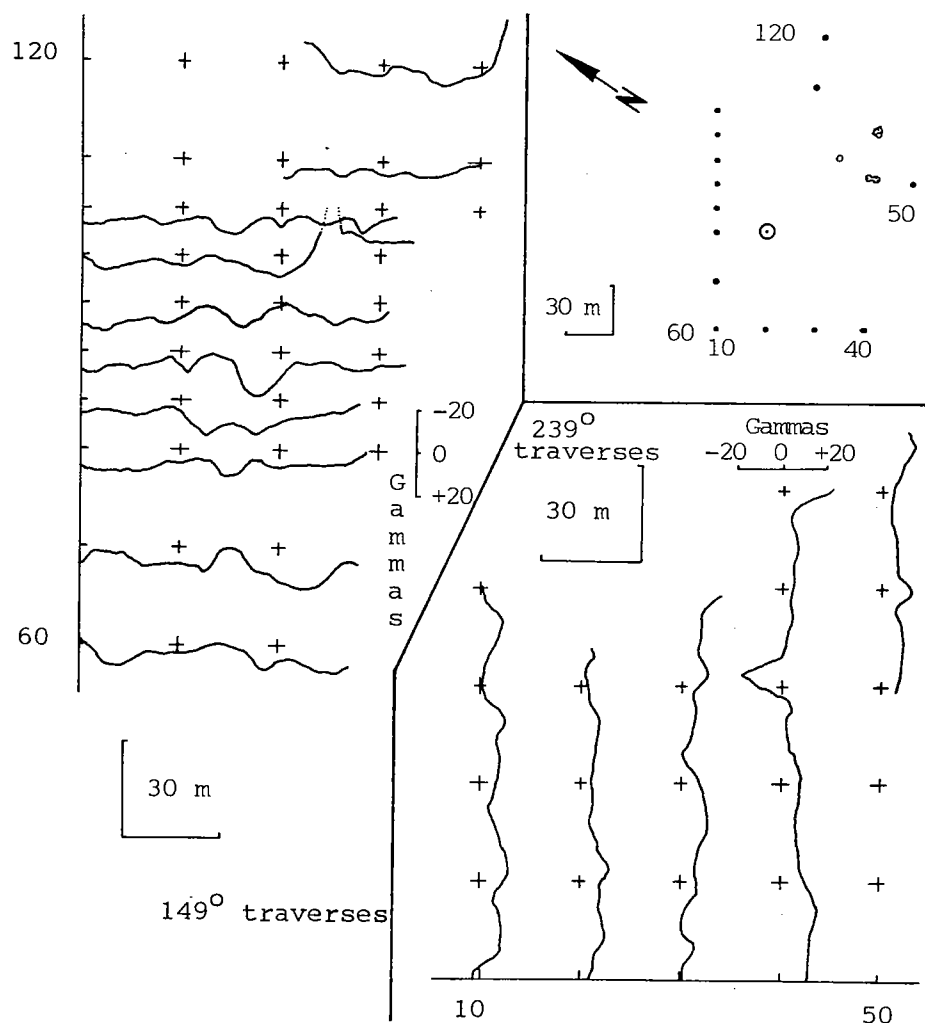


Figure 2. Ground magnetic survey M1 (see Figure 1). Data plotted in inverse mode to provide pseudo-profiles of soil/rock interface. Position of 149 degree traverses relative to 239 degree traverses diagrammed in upper right corner. All plots oriented properly with respect to N arrow.

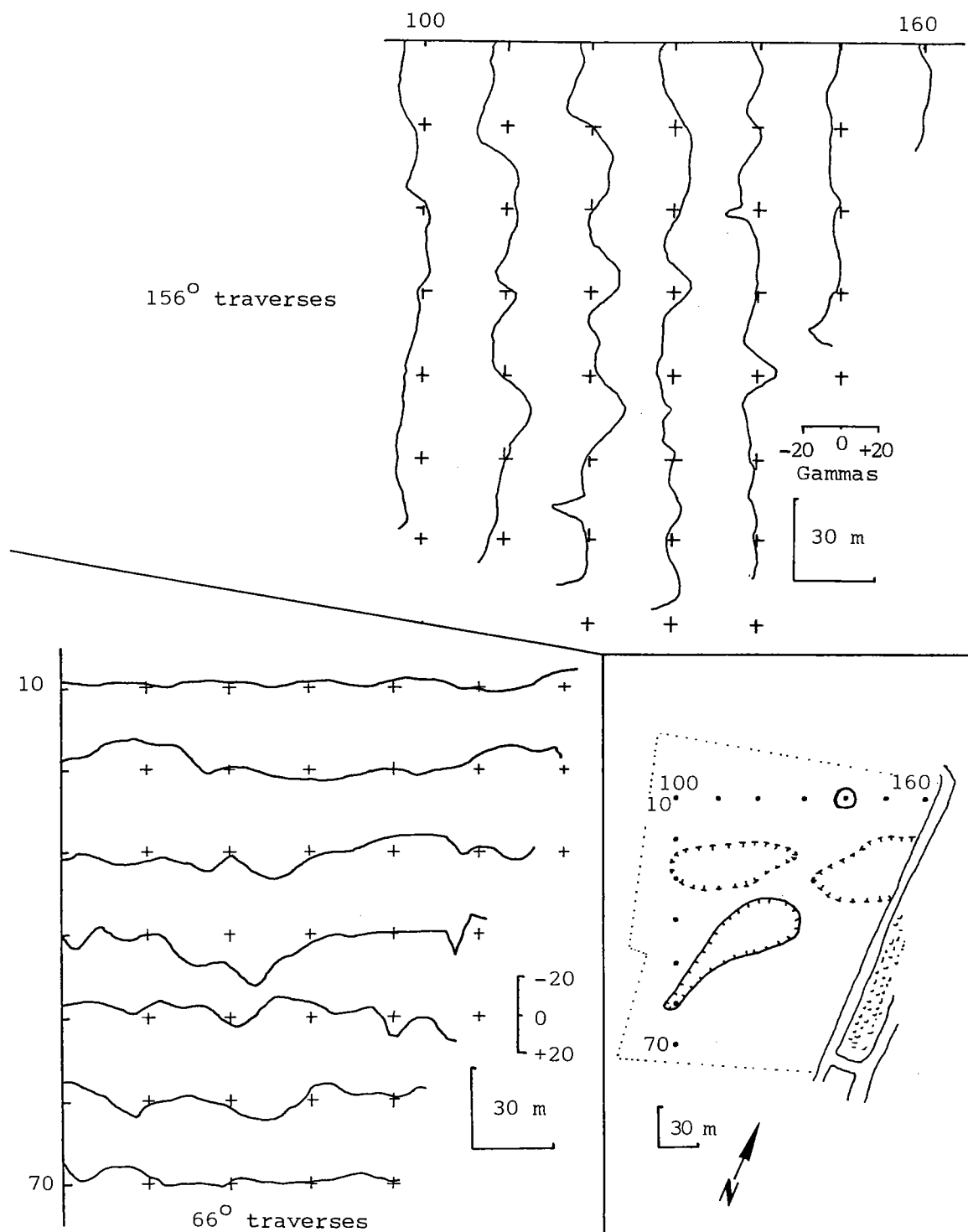


Figure 3. Ground magnetic survey M2 (see Figure 1). Data plotted in inverse mode to provide pseudo-profiles of soil/rock interface. Position of 66 degree traverses relative to 156 degree traverses diagrammed in lower right corner. All plots oriented properly with respect to North arrow at lower right.

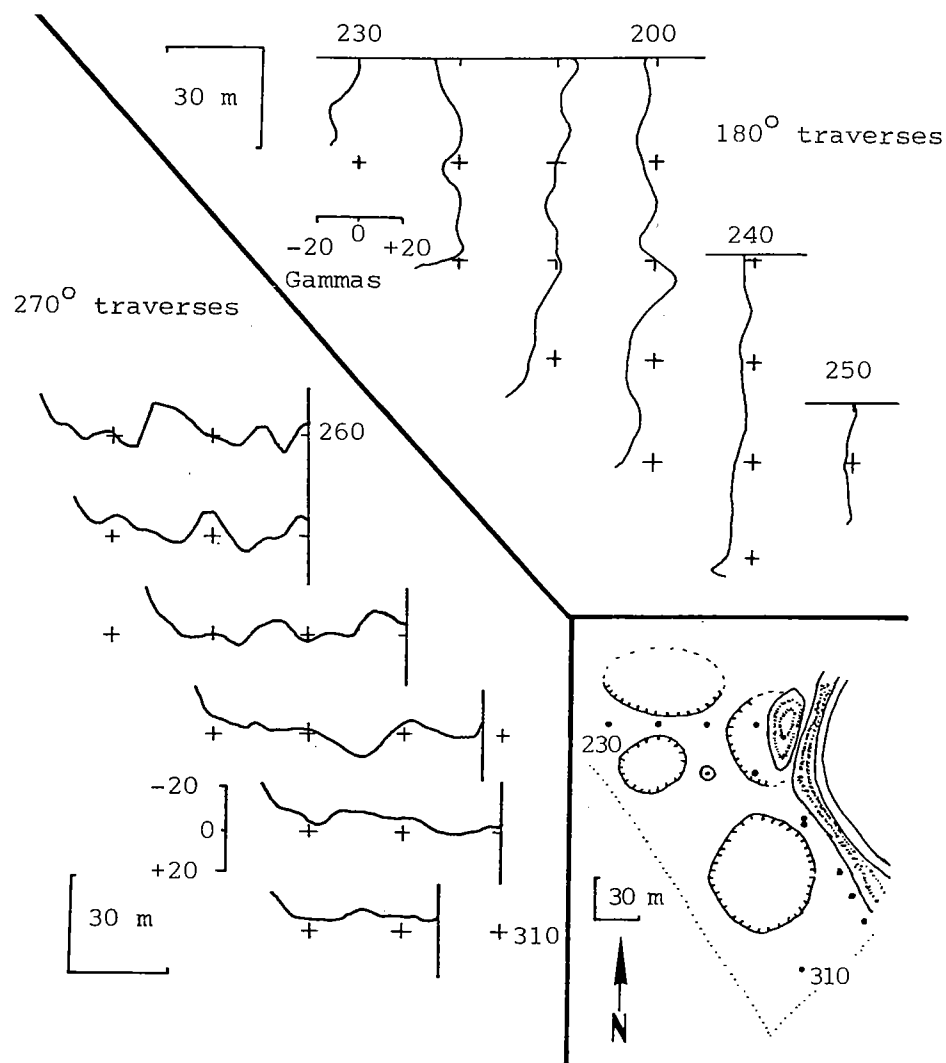


Figure 4. Ground magnetic survey M3 (see Figure 1). Data plotted in inverse mode to provide pseudo-profiles of soil/rock interface. Position of 180 degree traverses relative to 270 degree traverses diagrammed in lower right corner. All plots oriented properly with respect to North arrow at lower right.

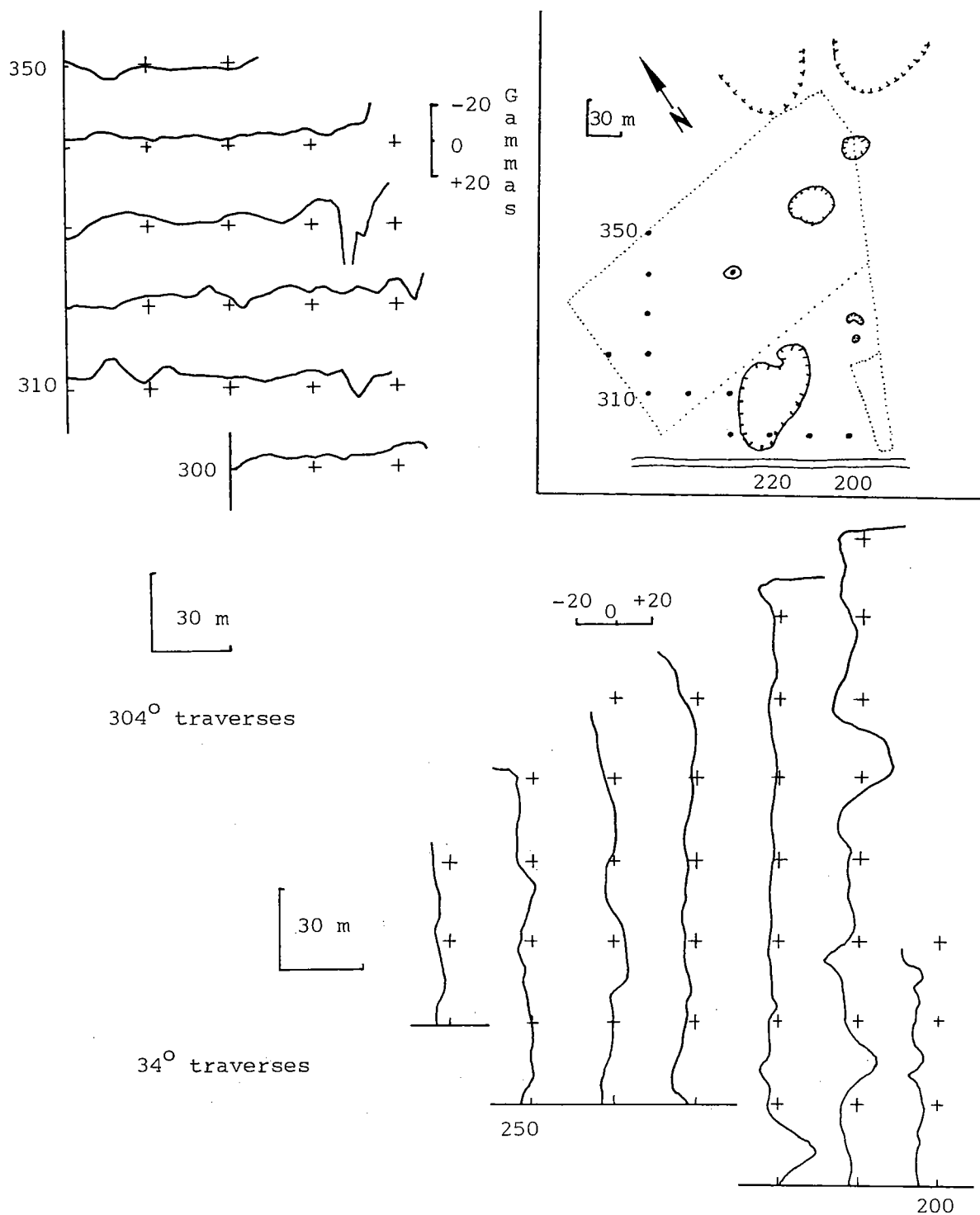


Figure 5. Ground magnetic survey M4 (see Figure 1). Data plotted in inverse mode to provide pseudo-profiles of soil/rock interface. Position of 34 degree traverses relative to 304 degree traverses diagrammed in upper right corner. All plots oriented properly with respect to North arrow at upper right.

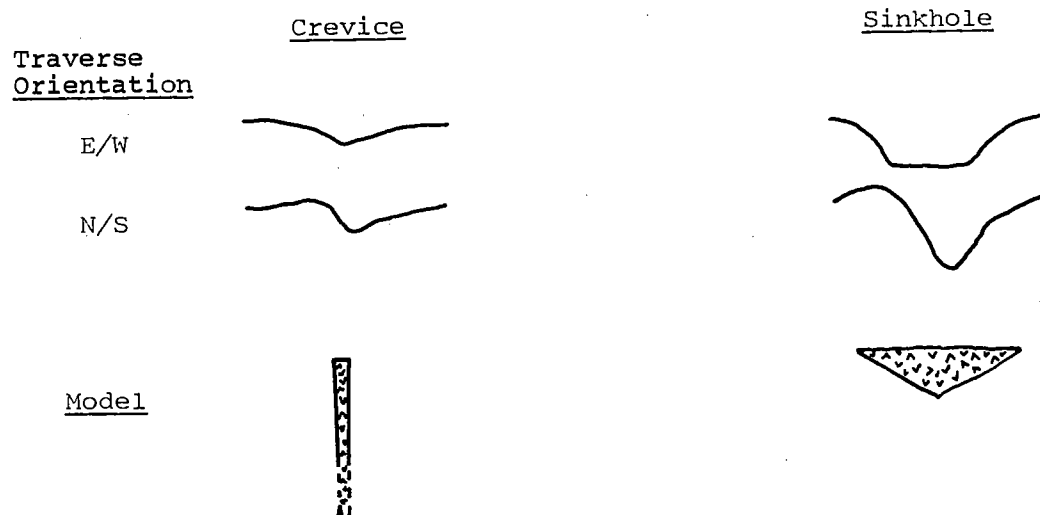


Figure 6. Diagrammatic, inverted cross-sections of magnetic anomalies from models of terra-rossa filled solution crevices and filled sinkholes.

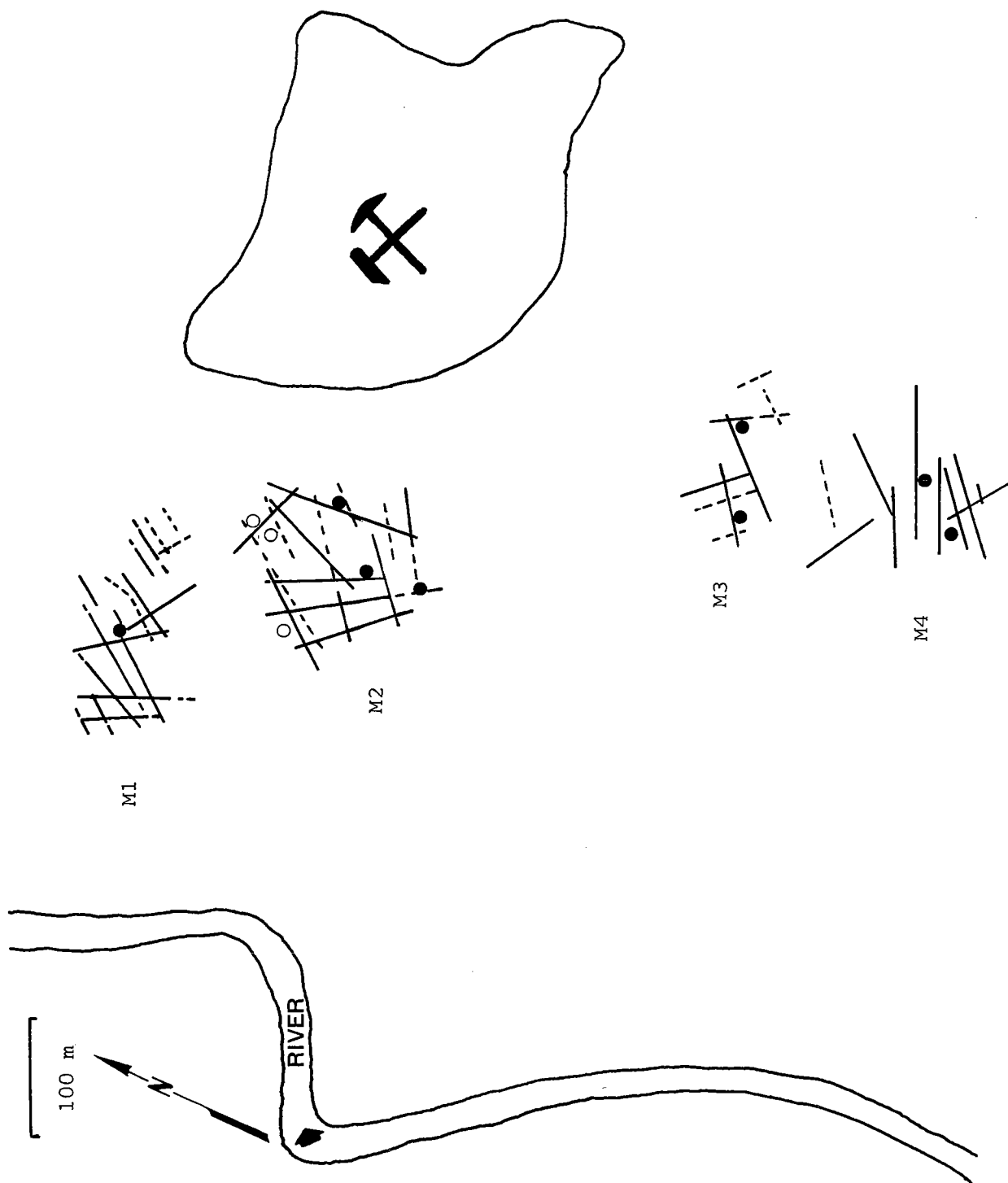


Figure 7. Geomorphic interpretation of magnetic data in Figures 2 - 5. The lines represent trends of terra-rossa filled solution crevices. The solid dots represent areas of deep soil. The open dots represent near-surface rock.

Rock Slope Modifications Above the North Portal of the Mt. Washington Tunnel

EDWARD P. VOYTKO
VINCENT A. SCOVAZZO
NEIL K. COPE
STS D'Appolonia Ltd.
Monroeville, Pennsylvania

HGS-87-23

INTRODUCTION

THE MT. WASHINGTON TRANSIT TUNNEL was constructed in the early 1900's to provide access for trolleys to Pittsburgh through Mt. Washington from the southern suburbs. The tunnel, owned by the Port Authority of Allegheny County (PAT), has recently been rehabilitated for use in Pittsburgh's reconstructed Light Rail Transit (LRT) System.

The face of Mt. Washington above the north portal of the tunnel is composed of horizontal layers of shale, limestone, claystone and sandstone. Differential weathering of the various rock types resulted in undercutting of the more resistant beds, forming overhangs. Primary joint sets in the rock mass are essentially vertical and intersect to form "columns" of rock. Infiltration and periodic freezing and expansion of water within the joints tilted several rock "columns". The existence of overhangs and leaning rock columns led to a concern about major rock falls or topples that could damage the portal structure, or otherwise disrupt or endanger tunnel traffic.

This paper describes the assessment of the deteriorated rock face above the north portal of the tunnel, and the subsequent design and construction of modifications to improve the slope stability. The assessment included evaluations of the topography, geology and criticality of existing hazards. The slope modifications implemented involved removal of an unstable overhang and "column", and stabilization of a large overhang with rock bolts and a concrete buttress. In addition, shotcrete was applied to the slope and drains were installed to reduce future differential weathering and slope deterioration. Other important aspects of the project included implementation of measures to protect the portal structure and tunnel traffic during slope repairs, and the sequencing of activities for safety and ease of construction.

SITE LOCATION AND TOPOGRAPHY

The north portal of the Mt. Washington tunnel is located south of the Monongahela River near the intersection of Carson and Smithfield

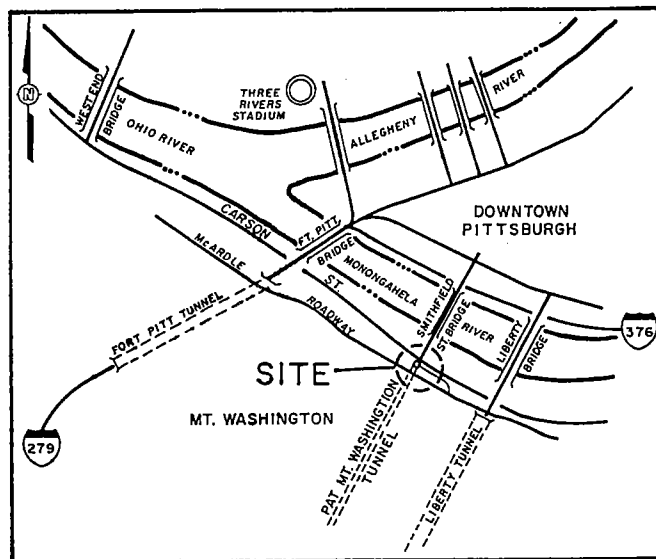


Fig. 1 Site Location Map

Streets (Figure 1). The steep north slope of Mt. Washington was created through dissection of an ancient peneplain by the Monongahela River. The rock slope addressed in this paper is approximately 150 feet high and is located above and to the south and west of the portal and below and north of McCardle Roadway.

The topography at the site was mapped by means of a close-range photogrammetric survey and is shown in Figure 2. The roadway inside the portal structure is at approximate Elevation 746⁽¹⁾ and the portal roof is at Elevation 785. The steep rock slope above the portal is a result of both natural weathering processes and the original excavation of the tunnel. The slope immediately above the portal up to Elevation 880 averages about one horizontal to four vertical, while the slope from Elevation 880 up to the McCardle Roadway (approximate Elevation 930) is typically about one horizontal to one vertical.

(1) All elevations in feet referenced to USGS Sea Level Datum.

GEOLOGY

The geologic conditions at the site were evaluated based on a field reconnaissance and mapping performed as part of this study, in conjunction with geotechnical information gathered and published previously for the Greater Pittsburgh area in general, and for sites adjacent and in the vicinity of the north portal (Refs. 1 through 11).

STRUCTURE AND STRATIGRAPHY - The site is located in the Allegheny Plateau physiographic province, where the geologic structure is characterized by broad, open folds. The site lies between two subtle structures, the Nineveh Syncline to the west and the Castle Shannon Anticline to the east (Ref. 1). Regionally, the rock strata are nearly horizontal, dipping to the south or southwest at about 1 degree or less. On a local basis and at the site, the rock strata generally dip to the southwest at about 2 degrees.

Bedrock at the site is of the Pennsylvanian System, Conemaugh Group. The Conemaugh Group characteristically contains the following (Refs. 2 and 3):

1. Weak claystone units or red beds;
2. Thin sequences of thinly bedded limestones, most of which are irregular and local in occurrence;
3. A few minable coal seams;
4. An abundance of thick sandstone units, most of which are channel sandstones.

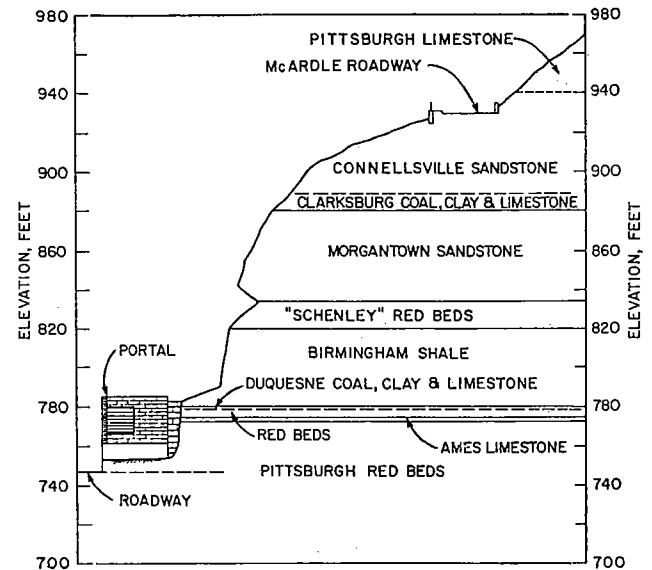


Fig. 3 Generalized Geologic Section

The predominant stratigraphic units occurring at the site are shown in Figure 3. The strata exposed on the slope between the McArdle Roadway (Elevation 930) and the base of the portal structure (Elevation 746) prior to and during construction consisted of the following:

1. Approximately 45 feet of hard brown sandstone between Elevation 835 and 880 (Morgantown sandstone);
2. Approximately 15 feet of soft red to gray claystone between Elevations 820 and 835 (Schenley red beds);
3. Approximately 40 feet of medium hard dark gray shale between Elevation 780 and 820 (Birmingham shale);
4. A thin seam of dark gray to black carbonaceous shale at Elevation 780 (Probably the Duquesne coal, clay and limestone horizon); and
5. Approximately 35 feet of soft red claystone below Elevation 780 (red beds and Pittsburgh red beds) containing a one- to two-foot thick bed of hard gray limestone (Ames limestone) at Elevation 775.

JOINTING - A site reconnaissance was conducted to map the orientation, spacing and condition of joints in the rock mass. Two nearly orthogonal joint systems were identified at the site. One joint set was well defined and exhibited smooth straight joints striking between $N42^{\circ}W$ and $N54^{\circ}W$ (nearly parallel to the rock face) and dipping from 78° to 83° to the southwest. The spacing of joints in this set varied

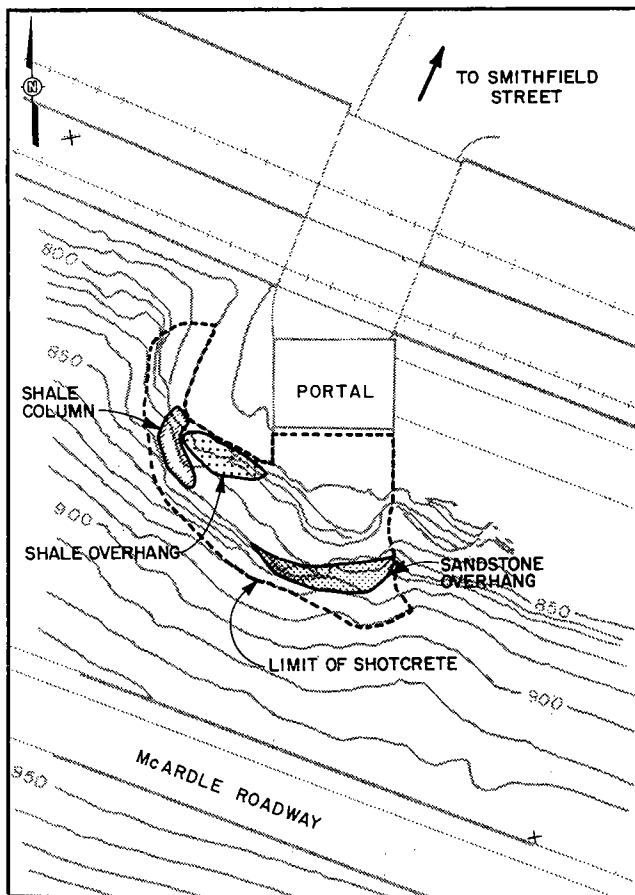


Fig. 2 Site Plan Showing Existing Hazards and Limits of Shotcrete

between 4 and 5 feet. The second joint set was less well defined and exhibited rough, wavy joints striking between N55°E and N67°E (nearly perpendicular to the rock face) and dipping from 65° southeast through vertical to 60° southwest. The joint spacing in this second set was approximately 14 feet. The northwest trending joint set was observed to produce water, while the second joint set was comparatively dry.

WEATHERING - Differential weathering between weak claystones (red beds) and relatively resistant sandstones and shales results in undercutting of the more resistant rock beds and the formation of overhangs. Undercutting of the steeply jointed sandstone and shale, along with ground water movement, precipitation and freeze-thaw action in the intersecting joints eventually results in rockfalls. The rate of retreat of the Schenley red beds was estimated at approximately 0.4 feet per year at the nearby Wabash trolley tunnel which was refurbished in 1975 (Ref. 4). This rate of degradation is consistent with rates of 0.2 to 0.6 feet per year observed in the Schenley red beds on the south side of Mt. Washington adjacent State Route 51 between 1930 and 1959, during which time no perceptible retreat of the overlying Morgantown sandstone was reported (Ref. 5).

ASSESSMENT OF EXISTING HAZARDS

Three hazards were identified on the rock slope above and adjacent the portal: an overhang of massive Morgantown sandstone; a second overhang in the Birmingham shale; and a leaning "column" of rock, also in the Birmingham shale (Figure 2).

SANDSTONE OVERHANG - The sandstone overhang was located immediately over and approximately 50 feet above the portal roof. The overhang was approximately 60-feet long and had a maximum undercut depth of about 13 feet. This overhang was formed by degradation and sloughing of the Schenley red beds from beneath the Morgantown sandstone. It appeared that further retreat of the red beds would soon undercut another joint in the sandstone, dramatically increasing the risk of a rockfall. Because the processes of degradation of the red beds and expansion of joints in the sandstone are gradual and nonuniform, and because the exact support mechanism for each block of sandstone could not be accurately defined, the time before a rockfall would occur could not be predicted. It was apparent, though, that a rockfall from this overhang could occur at any time and could involve large blocks of rock capable of causing substantial damage to the portal structure and danger to tunnel traffic.

SHALE OVERHANG - The shale overhang was located immediately to the west of the portal and just above the portal roof level. The arch-shaped overhang, which was approximately 30 feet long and extended out as much as 10 feet from the underlying rock face, was formed by degradation and sloughing of the red beds from beneath the

Birmingham shale, followed by unraveling of the shale. Some minor falls of shale from this overhang were observed during the field reconnaissance and mapping. At the time, it was observed that the undercut had retreated to a major joint, producing a very unstable condition and probable imminent failure. As with the sandstone overhang, however, the time to final collapse of the shale overhang could not be predicted. It was anticipated that a sudden collapse of the shale overhang could result in damage to the portal structure, but that the potential for personal injury, property damage, and disruption to tunnel traffic would not be as significant as with the sandstone overhang due to the relative location of the overhangs with respect to the portal.

SHALE COLUMN - The shale column was approximately 5 feet by 10 feet in plan by 40 feet high with a base level with the portal roof. The column, located about 30 feet west of the portal, was leaning toward the portal as a result of the natural joint orientation and the freeze-thaw action of water and saturated soil in the two joints defining the column. The leaning may also have been aggravated by and unraveling of the column at its base. It was apparent that continued joint expansion would eventually lead to collapse of the column, although the timing of such an event could not be predicted. It was believed that the potential for personal injury, property damage and disruption to tunnel traffic would be similar to that related to collapse of the shale overhang.

DESIGN AND CONSTRUCTION OF SLOPE MODIFICATIONS

Two approaches were considered to mitigate the existing and future slope hazards. One approach was to protect the portal with walls, fences and/or other structures which could absorb the impact of falling rock. The second approach was to increase the stability of the rock slope and reduce the potential for rockfalls that could impact the portal or endanger traffic. The latter approach was chosen for two reasons:

1. A protective structure would only protect a finite area, such as the portal, and would not necessarily protect traffic in the vicinity of the portal. Conversely, slope stabilization measures would reduce the potential for danger to traffic as well as damage to the portal.
2. The protective structure would only protect the portal against a certain design event (i.e., a certain size of rock fall), while slope stabilization would reduce the risk of any size rockfall.

Subsequent to the selection of slope stabilization as the means for reducing existing and future hazards, the following objectives were established for the design of slope modifications:

1. Elimination of existing slope hazards to protect the portal and tunnel traffic against immediate danger.
2. Reduction in the rate of future degradation and differential weathering to lessen the potential for development of additional overhangs.
3. Control of surface runoff and seepage from joints in the rock mass to lessen future erosion and joint separation.
4. Removal of talus resting on the portal roof and against the portal walls to prevent damage due to possible excessive loads for which the portal may not have been designed.

The slope modifications implemented included stabilization of the sandstone overhang, removal of the shale overhang, removal of the shale column, and other modifications (such as shotcrete application and drainage installation) as described in the following paragraphs.

SANDSTONE OVERHANG SUPPORT - Considering the steep slope between the overhang and the McArdle Roadway, and the location of the overhang directly above the portal, support, rather than removal of the overhang was chosen as the means for improving slope stability. Support of the overhang was selected instead of excavation because of access difficulties, and because overhang removal would require excavation of large hard-to-handle blocks of sandstone and could hasten the future undercutting of the McArdle Roadway.

Support of the sandstone overhang was accomplished by first rock bolting the overhang to rock behind the face, and then constructing a concrete buttress in the undercut (Figure 4).

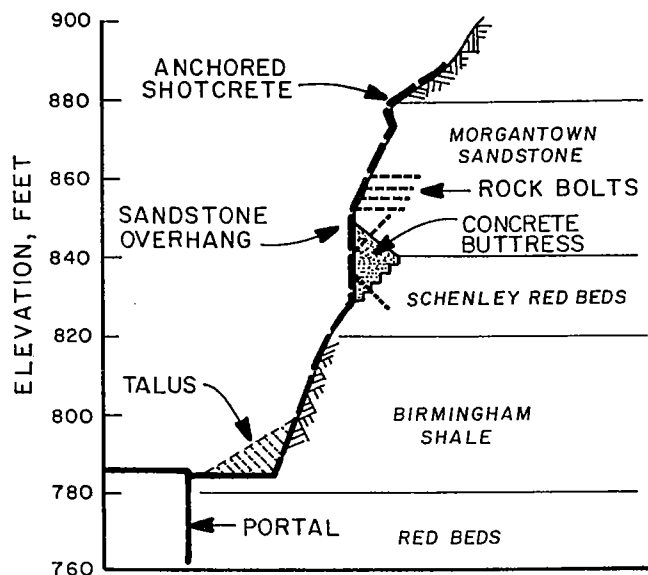


Fig. 4 Section Showing Rock Bolts and Buttress at Sandstone Overhang

Post-tensioned rock bolting of the overhang was completed first to increase the stability of the overhang before work was performed below. Fifty rock bolts were installed at selected locations in the lower 10 to 25 feet of the Morgantown sandstone at an orientation of N35°E, which is nearly perpendicular to the most prominent joint system. These bolts were solid Grade 70 steel rebar bolts 12 feet eight inches long and 7/8-inch in diameter, and were fitted with expansion shells and 6-inch x 6-inch bearing plates. Each bolt was torqued to set the expansion shell anchorage and was subsequently grouted over its full length with a high bond, non-shrink crack-resistant cement grout. Before grouting, each bolt was post-tensioned to a torque of 290 ft-lbs and eight of the 50 bolts were load tested in-place using a hydraulic jack to confirm the bolt design capacity of 23,000 pounds. The bolts provided a minimum factor of safety of 1.5 against short term failure of the overhang.

After the overhang was bolted to increase short term stability, the area under the overhang was cleaned of loose rock and fallen material by hand and with a compressed air jet to allow good bonding with the concrete buttress. The shale and soft rock under the overhang was rough-cut in steps (Figure 4) to remove weathered rock and increase the shear resistance along the shale-concrete buttress interface. Twenty-eight rock bolts were then installed as shown in Figure 4 to support the buttress formwork and the completed buttress. This included twenty-one 7/8-inch diameter bolts for the buttress, and seven additional 3/4-inch diameter bolts for the formwork. The buttress, in conjunction with the initial bolting, increased the minimum long-term factor of safety against failure of the overhang to 2.5.

The 21 buttress bolts were initially designed based on an estimated buttress geometry, and were all assigned an equal design load of 23,000 pounds. The design loads for bolts was reevaluated using measured cross-sections after preparation of the area for buttress construction. Required design loads ranged from 16,000 to 23,000 pounds. The required minimum design capacity and test load for each bolt in the buttress was then established at 110 to 120 percent of the final design load.

The bolts were installed to project from the rock face, typically 6 to 7 feet, to accommodate the buttress and formwork. The embedded portion of each bolt was initially anchored into rock using a mechanical expansion shell and/or by grouting the embedded length. The mechanical anchorage was effective for the upper bolts which were embedded in sandstone, but insufficient to resist test loads on the lower bolts which were embedded in shale and claystone. Due to the poor performance of the mechanical anchorages on the lower bolts, all lower bolts except two having very low design capacities were subjected to load testing. Any bolt failing initial testing was replaced with a new bolt grouted throughout the rock embedment and tested to the design capacity.

The rock bolt projections were encased in two inch diameter steel sheaths countersunk into the rock face to protect the bolts during concrete placement. The bulkhead formwork was erected and supported by the bolts until the concrete placement was completed. The bulkhead concrete was placed in three lifts which were keyed together with 3-inch deep by 7-1/2 inch wide continuous keys. Between concrete pours, each of the twenty-eight rock bolts was relaxed and re-tightened to dissipate pressure developed against the formwork by the fluid concrete. The 60 foot-long bulkhead was provided with one vertical expansion joint at its midpoint to accommodate temperature changes, and a drainage system composed of 4-inch diameter pipes to convey seepage from the rock to the face of the bulkhead. After the final concrete pour was permitted to set, the formwork was removed and each bolt was retensioned to its design load.

SHALE OVERHANG REMOVAL - The shale overhang (Figure 5) was removed in small controllable pieces using a hydraulic percussion chisel (jackhammer) mounted on a small backhoe. The backhoe was secured to a platform which was suspended from the boom of a 165 ton crane positioned on Sycamore Street adjacent the portal. Excavation rather than support of this overhang was the selected stabilization technique because removal did not further jeopardize the stability of the slope. The overhang was removed to an existing predominant joint located behind the overhang to provide a relatively smooth final rock face.

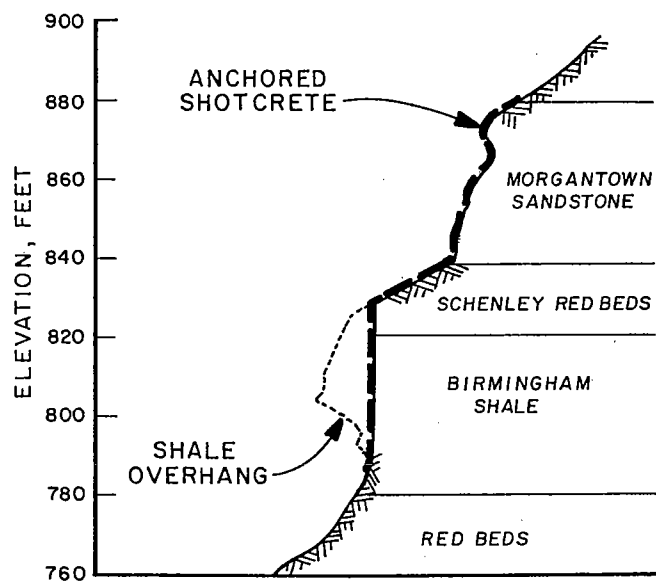


Fig. 5 Section Showing Shale Overhang Excavation

The overhang excavation was periodically interrupted to permit removal of excavated rock from the underlying area using a clam shell with the crane. This effort prevented excessive material accumulation against the portal and provided a clear area for fall of additional excavated material.

SHALE "COLUMN" REMOVAL - The leaning "column" of shale (Figure 6) offered a distinct problem because it was anticipated that the column could fall during its removal as a result of vibration and impacts associated with the excavation operations. If permitted to topple in an uncontrolled manner, the column would impact and possibly damage the portal. To provide some control in the event of failure of the column, a 2-inch x 2-inch, W1.4 x W1.4 welded wire fabric was placed around the "column," and anchored on either side of the column using 4-foot long No. 4 Grade 40 deformed steel reinforcing bars grouted into predrilled holes in the adjacent rock. The bars were generally placed every two feet around the mesh to secure it to competent rock. In addition, a total of six 3/8-inch diameter steel cables were placed across the column at three levels and attached to the fabric anchors. After the fabric was installed, the column was to be removed in layers, starting from the top down. The mesh was to be peeled back intermittently to permit removal of small, easily handled pieces of shale.

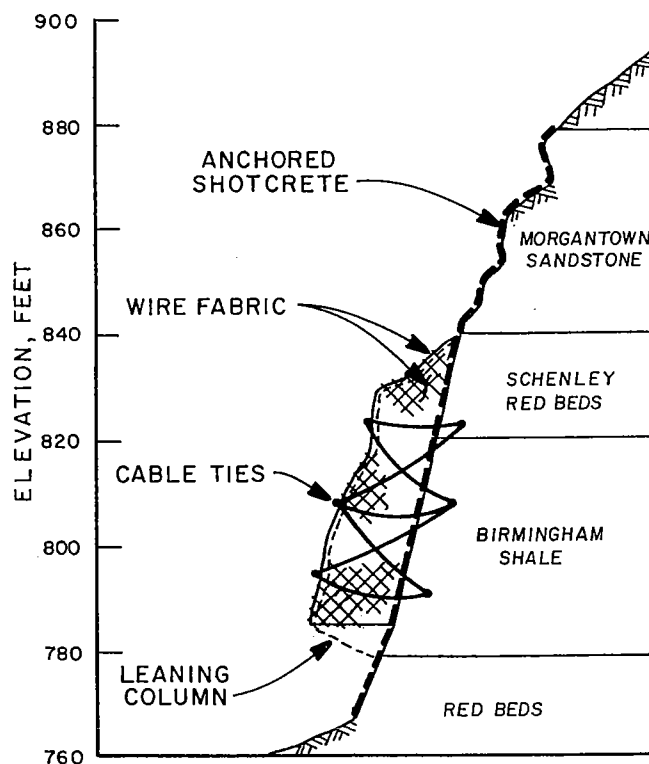


Fig. 6 Section Showing Leaning Shale Column Excavation

The shale column did, in fact, fail before removal operations were initiated. The cables and wire fabric were effective in restraining the column sufficiently to prevent toppling, and the column crumbled and slid into the area adjacent the portal without damage to the structure.

OTHER MODIFICATIONS - After the buttress construction and shale removal were completed, the rock slope was cleared and grubbed, scaled

using mechanical means (barring, prying and raking) and cleaned using compressed air jets to remove loose rock and soil. The working area was then covered with 3-inch x 3-inch, W1.4 x W1.4 welded wire fabric which was anchored with 4-foot long No. 4 Grade 40 deformed steel reinforcing bars spaced at 4-feet center-to-center. The bars were grouted into rock in 3/4-inch diameter pre-drilled holes using non-shrink grout. After the fabric was anchored, a 4-inch minimum thickness of shotcrete was sprayed over the rock face and fabric. The shotcrete coating was equipped with 2-inch diameter weep pipes which were spaced about 15 feet apart, to relieve hydrostatic pressures and direct ground water to the shotcrete surface.

Loose and fallen material was removed from beside and above the portal as the work proceeded. This included removal of material already resting on top of and against the portal structure as a result of deterioration and sloughing of material from the slope. The previously existing talus was removed to reduce earth pressures on the portal and allow for storage on site of falls during construction. Upon completion of the modifications described above, 6-inch to 2-foot equivalent diameter stone was placed and grouted at the toe of the slope west of the portal to protect the nonresistant red beds from further deterioration.

PROTECTION OF EXISTING FACILITIES

To protect the portal structure and minimize hazards to the public and construction personnel during implementation of slope modifications, the following four measures were proposed:

1. Removal of existing talus resting against and on top of the portal structure;
2. Placement of welded wire fabric over the leaning shale column and anchoring the fabric to solid rock on either side of the column;
3. Installation of a protective net south of and above the portal structure (Figure 7);
4. Construction of a protective platform to the west of the portal structure (Figure 7) (subsequently replaced by a berm).

Talus removal was completed to reduce loads on the top and walls of the portal and to provide space for additional fall material, particularly during initial construction operations. As discussed previously, the welded wire fabric was placed over the shale column to prevent an uncontrolled topple of the column during excavation.

The protective net (Figure 8) was constructed to catch small (2 foot equivalent diameter) slabs or blocks of rock falling from the face during bolting of the sandstone overhang and construction of the buttress. The net was 60-feet long by 9-feet wide and was positioned approximately 35 feet above the portal roof, oriented at about 45 degrees from vertical to prevent falling material from rolling or bouncing onto the portal.

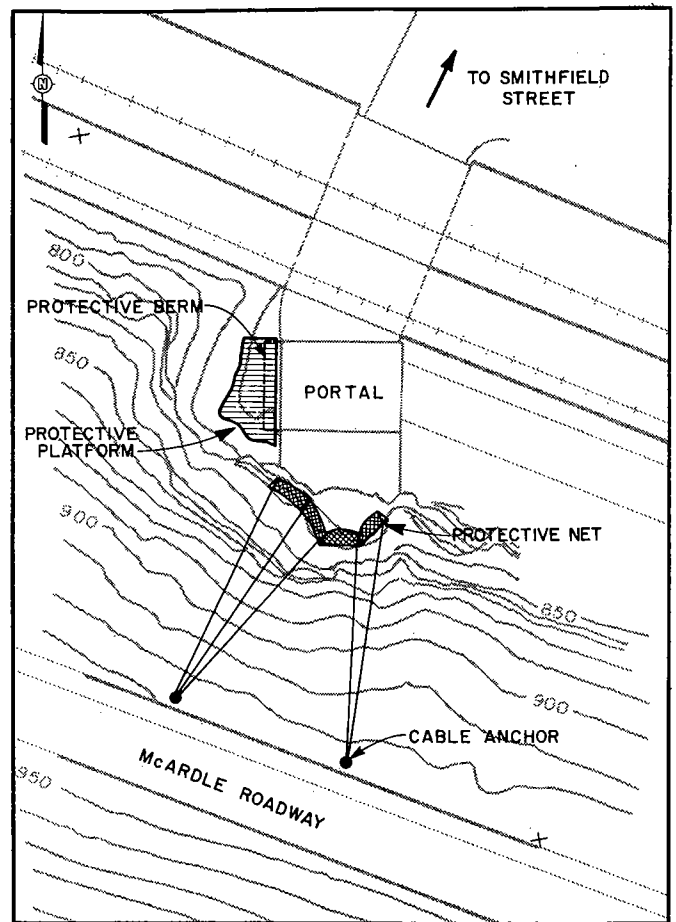


Fig. 7 Site Plan Showing Locations of Temporary Measures for Protection of Existing Facilities During Construction

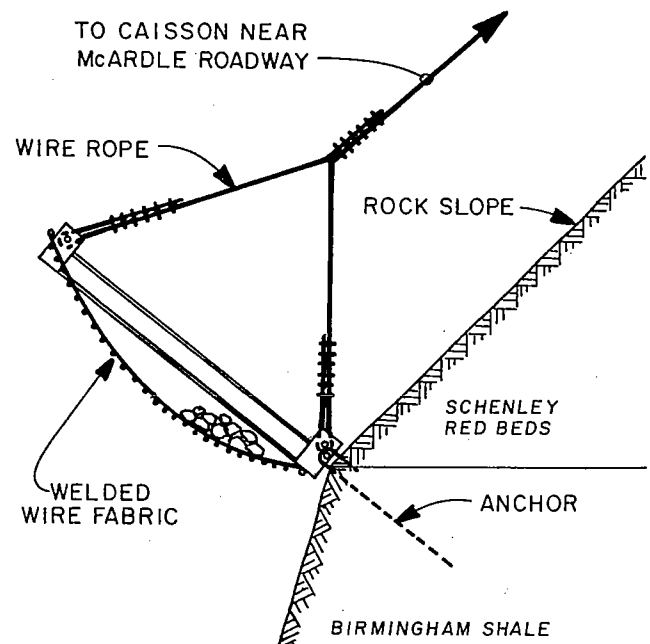


Fig. 8 Section of Protective Net Above Portal

The net was constructed of two layers of 4-inch x 4-inch welded wire fabric spanning five 9-foot long W6x12 steel beams which were spaced 15 feet apart and tied together with three one-inch diameter wire ropes. The net was supported from above by two 30-inch diameter, 17 feet deep caissons which were constructed of concrete encased W21x68 steel beams. Each of the W6x12 steel beams in the net was supported by a 1-inch diameter wire rope suspended from one of the caissons, located near McArdle Roadway. The net was effective in catching a substantial amount of fall material during construction, and was periodically cleaned to maintain capacity.

The construction of a temporary 40-foot long by 10- to 20-foot wide steel and timber protective platform (Figure 9) was originally specified for the area west of the portal. The primary purpose of the platform was to prevent excavated or fall material from impacting or otherwise loading the west wall of the portal during removal of the shale overhang and shale column. The area was first prepared for platform construction by removing loose talus with a clam-shell bucket using the crane located on Sycamore Street.

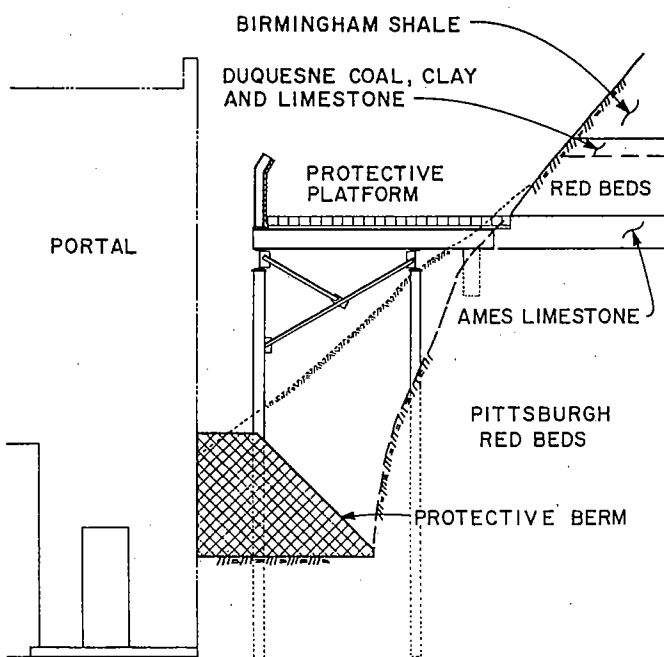


Fig. 9 Section of Protective Platform and Replacement Protective Soil Berm

Before the platform construction was initiated, however, the shale column failed. For this reason, and because the volume of talus excavated from the west side of the portal was greater than originally anticipated, the need for the protective platform was reevaluated. It was concluded that the platform could be eliminated and replaced with a more economical protective soil (or talus) berm against the portal wall.

The platform was therefore eliminated and the talus was excavated such that an approximately 8-foot high berm was left in place against the portal. Upon the completion of the major slope modifications, and prior to the placement of grouted stone slope protection at the toe of slope west of the portal, the berm was excavated and removed from the site.

SEQUENCE OF CONSTRUCTION

Slope modifications and associated work required for short-term slope stability during construction included placement of the protective net above the portal, construction of the protective platform (subsequently replaced by a protective soil berm) west of the portal structure to catch small rockfalls and reduce the impact of large rockfalls during construction, and rock bolting of the sandstone overhang. Modifications required for long-term slope stability included excavation of talus, construction of a concrete buttress to support the sandstone overhang, and shotcrete application and improvements to surface and subsurface drainage to prevent future differential weathering and deterioration of the slope.

The sequence of construction activities was governed first by general safety and facility protection considerations and second by ease of construction. With the intentions that no construction personnel be permitted to work below unsupported, unstable rock, and that the portal structure and traffic be protected from falls resulting from work on unstable rock, the originally specified construction sequence was as follows:

1. Establish access from McArdle Roadway and construct the protective net;
2. Scale the rock face from McArdle Roadway down to the sandstone overhang;
3. Scale and rock bolt the sandstone overhang;
4. Construct the concrete buttress and drainage system;
5. Construct the protective platform on the west side of the portal and secure and anchor welded wire fabric over the shale column;
6. Remove the shale overhang and shale column;
7. Scale and clean the slope and remove all remaining talus from above and beside the portal; and
8. Install reinforced shotcrete slope protection and drainage.

Construction of the protective platform and securing of the shale column, followed by removal of the shale overhang and column, were permitted to be undertaken concurrently with buttress construction.

Two significant changes in this sequence occurred due to failure of the shale column during preparation for construction of the protective platform.

1. The protective platform was eliminated.

2. Substantial talus excavation was completed in the area to the west of the portal prior to removal of the shale overhang. This excavation was made to form the protective soil berm which replaced the platform.

SUMMARY

A deteriorated, near-vertical, 150-foot high rock slope above the portal of an operating transit tunnel was evaluated relative to the geologic environment and processes responsible for deterioration of the slope. Subsequently, modifications to stabilize the slope were designed and constructed.

Differential weathering between soft claystone and relatively resistant shale and sandstone, along with separation of blocks of resistant rock along joints resulted in the formation of slope hazards consisting of (1) a massive sandstone overhang, (2) a shale overhang, and (3) a leaning "column" of shale. Further slope deterioration and aggravation of these hazards could be expected to result in failures that could disrupt the operation of the portal structure and pose danger to the public.

A slope modification program was designed and implemented to increase the stability of the slope and reduce the potential for future slope deterioration so as to protect the portal from falling rock debris. The principal components of the slope modification program consisted of: (1) construction of a soil berm and protective net to catch small- to medium-sized rock falls during implementation of slope modifications; (2) construction of a concrete buttress and installation of post-tensioned rock bolts to support the sandstone overhang; (3) removal of the shale overhang; (4) removal of a leaning shale "column" using anchored wire fabric to permit excavation of the shale in a controlled manner; and (5) scaling the slope followed by application of anchored, reinforced shotcrete and drainage to reduce future slope deterioration.

ACKNOWLEDGMENTS

The authors wish to acknowledge the cooperation and support of the Port Authority of Allegheny County, Pennsylvania, the owner of the Pittsburgh Light Rail Transit (LRT) system project, of which this work was part, and Parsons Brinckerhoff - Gibbs & Hill Consulting Engineers, the General Architectural and Engineering Consultant on the LRT project.

REFERENCES

1. Wagner, W.R., L. Heyman, J.L. Craft, W.E. Edmunds, and J.A. Harper, 1975, Greater Pittsburgh Region Structure Contour Map, Pennsylvania Topographic and Geologic Survey Map No. 43.
2. Wagner, W.R., L. Heyman, R.E. Gray, D.J. Belz, R. Lund, A.S. Cate, and C.D. Edgerton, 1970, Geology of the Pittsburgh Area, Pennsylvania Geological Survey, 4th Series, 144 pp.
3. Ackenheil, A.C., and Associates, 1968, Mining and Physiographic Study Allegheny County, Pennsylvania.
4. GeoMechanics, Inc., January 1982, "Subsurface Exploration and Geotechnical Investigation, Stage 1 - Light Rail Transit System, East Sycamore Street Relocation and Bridge," prepared for the Mackin Engineering Company, Pittsburgh, Pennsylvania.
5. Philbrick, S.S., 1959, "Engineering Geology of the Pittsburgh Area (Pennsylvania)," Geological Society of America, Pittsburgh, Pennsylvania, 1959, Field Trip Guidebook 6, pp. 189-203.
6. Pullman Swindell, Division of Pullman, Inc., March 1983, "Soil and Geologic Area Investigation and Feasibility Study for Improvements of William and East Streets," prepared for the City of Pittsburgh, Department of Public Works, Bureau of Engineering.
7. Ackenheil and Associates Consulting Engineers, GeoSystems, Inc., January 1981, "Soils and Geologic Study, Reconstruction of the P.J. McArdle Roadway," Mt. Washington, Pittsburgh, Pennsylvania, prepared for Kozel Engineering Company, Inc., Pittsburgh, Pennsylvania.
8. Ackenheil and Associates Consulting Engineers, GeoSystems, Inc., July 1980, "Proposed McArdle Roadway Bridge," Mt. Washington, Pittsburgh, Pennsylvania, prepared for Kozel Engineering Company, Inc., Pittsburgh, Pennsylvania.
9. D'Appolonia Consulting Engineers, Inc., March 1983, "Evaluation of Rock Fall and Slope Reconstruction," State Route 51 and Woodruff Street, Pittsburgh, Pennsylvania, prepared for the City of Pittsburgh, Department of Public Works, Bureau of Engineering.
10. D'Appolonia Consulting Engineers, Inc., May 1983, "Letter Report, Stabilization of Rock Cut Slope between East Sycamore Street and McArdle Roadway," prepared for the City of Pittsburgh, Department of Public Works, Bureau of Engineering.
11. D'Appolonia Consulting Engineers, Inc., April 1982, "Assessment of Subsurface Conditions, North Portal of Mt. Washington Tunnel to Panhandle Bridge, and Arlington Avenue Connection," Stage I LRT, Port Authority of Allegheny County, Geotechnical Report Volume VIII, Part I, prepared for Parsons Brinckerhoff-Gibbs & Hill.

The Effect on Highways of Surface Subsidence Resulting From Longwall Coal Mining

GLENN A. HAZEN, SHAD M. SARGAND

Department of Civil Engineering

Ohio University

Athens, Ohio

S. WILLIAM DUDLEY

Maintenance Bureau of Ohio

Columbus, Ohio

HGS-87-24

INTRODUCTION

LONGWALL MINING IS THE PREFERRED method for removing coal from the ground since it provides for a high extraction rate and increased productivity when compared to room-and-pillar type of mining. Thus the coal tonnage removed each year by longwall mining methods has increased dramatically, Ref. (1). A high extraction rate has immediate effects on ground such as surface subsidence and large horizontal strain levels which can cause damage to surface structures. The high productivity of the longwall mining method means increased efficiency for coal operators but also results in large areas being mined in just a few years.

BACKGROUND

Land subsidence over underground coal mines has been recognized as a serious problem in the Appalachian coal regions and elsewhere. The problem used to be precluded by avoiding previously mined regions, if possible, and by buying mineral rights before road construction was begun in unmined regions. Recent renewed interest in coal as a replacement energy source for imported oil has made it possible to sell coal in quantities which may result in substantial damage to roadways as the result of immediate surface subsidence.

Because longwall mining is new to the United States and totally removes coal in large rectangular panels, there is a need to understand normal subsidence related phenomena. Moreover, the design of major highways, bridge structures, and retaining walls require knowledge of mine design, coal seam characteristics, and stratigraphy to establish deformation response resulting from the redistribution of in situ loads. Advance planning is invaluable when one is considering surface construction on an area where there is to be underground mining. To aid when planning roadway maintenance, mine maps are sometimes available for older areas

and are always available when coal is mined under state law or permanent regulations of the Surface Mining Control and Reclamation Act of 1977. These maps provide information about: (1) areas which were mined by the room-and-pillar method of mining, (2) areas which were mined by the longwall method of mining, (3) information to calculate the percent of coal extracted, and (4) location and size of coal support pillars remaining when mining was completed. Also useful for comparisons, but somewhat less detailed than the mining permit applications, are environmental impact statements written in response to the National Environmental Protection Act of 1969.

Investigations into empirical subsidence predictions, computer methods of analysis, fissuration of strata and model studies were well represented in a literature review of fifty-six publications over the last thirteen years. Many analyses are theoretical in nature due, at least in part, to limited subsidence data available. "Routine monitoring of surface subsidence was not a requirement and made little sense for many underground coal mining areas, Ref. (1)." In addition while longwall mining is relatively new to this country, there has been substantial experience with longwall mining in Great Britain, France, Germany, and Poland. The evidence indicates, however, that longwall subsidence differs substantially from data published in Europe. Limited investigations conducted by the Bureau of Mines and Department of Energy in the United States show large variations depending on local mining procedure and stratigraphy.

As an energy source, coal will continue to provide a major contribution to total energy consumed in Ohio, although there is a constant trade off between allocating resources for coal production and setting aside funds for environmental development. Thus, with abundant oil now available, the Department of Energy has

curtailed its coal research program which included a subsidence investigation over a longwall installed by North American Coal Company's Quarto No. 4 Mine in Belmont County, Ohio. Information from Quarto No. 4 Mine study was available to this project. It was used to determine the approximate stratigraphy in the area of S.R. 556.

The de-emphasis of environmental concerns by the Federal Government has left the Bureau of Mines with only small subsidence programs at Pittsburgh and Denver research centers. Nevertheless, the Office of Surface Mining regulations has required state agencies to correct and regulate mining as it affects the environment. The Ohio Department of Reclamation regulates mining and approves mining permits for longwall mining in Ohio. However, the state transportation departments must rely on their own design information to set bond when a highway or highway structure is undermined. And it must employ reasonable maintenance and construction techniques over previously mined regions. Any additional maintenance or building cost is usually borne by the constructor and only rather exceptionally by the mine operator. In Europe, experience has shown the usefulness of adopting simple design criteria, Ref. (2), in regions with mineable coal resources.

Since the longwall mining method is new to Southeastern Ohio there are little data recorded on surface effects and a real concern with the magnitude of damage to highways, culverts, and bridges. To investigate the possible effects of surface movements on highways and highway structures this research project was conducted in two phases. Phase one was to review the literature for predictive models and qualitative information. Phase two was to measure subsidence and strains and to examine undermined highway and highway structures for damage in light of the bonding requirements of Ohio Law for mine operators who excavate near highway right-of-ways.

LITERATURE REVIEW - There have been a number of recent theoretical and experimental investigations into numerical and physical simulations of mine overburden. Probably the most extensive research has been done at Sandia National Laboratories. An example is the work of Sutherland and others, Ref. (3). The theoretical part of this investigation uses a finite element simulation in which strata failure, bulking of rock rubble, and recompaction of the rubble are incorporated. Comparisons are drawn to a laboratory study of subsidence. The experimental aspect of this investigation used scale subsidence models which were

tested in a laboratory centrifuge. Since subsidence over a longwall panel is the result of the weight of overburden, the centrifuge is an ideal laboratory device to simulate gravity forces. Results of tests performed on a layered media show the strata breaks progressively upward as the weight is increased. Strata fractures from an inverted "v" as they move to the surface. Beam-like failure of strata emphasize the importance of interface elements in any numerical simulation. Yet it appears that further work needs to be done on the determination of layer thickness as equal layering does not model field experience and directly affects failure strength.

Empirical methods are employed in the European coal fields to model subsidence characteristics, Ref. (4). These methods can be divided into graphical and mathematical methods where the mathematical methods use either a profile or an influence function. The National Coal Board (NCB) of Great Britain has compiled a large number of field measurements into graphical form. Results are published in the Subsidence Engineer's Handbook, Ref. (5), and are widely quoted in the United States.

When matched against measurements taken in the Illinois Coal Basin, the maximum subsidence prediction by the graphical method is consistent with field observation, but the subsidence profile does not compare well. The NCB method predicts a wider trough, and consequently a lower strain level. Mathematical profile functions show excellent agreement with the subsidence trough. But strains predicted by the profile method were larger than measured values. The gage lengths of measured deformations are critical to strain measurements, and so one can expect a large variation between measured and predicted values.

Influence functions are used primarily where irregular areas are mined and surface subsidence must be obtained by adding incremental areas with an appropriate influence function. Since the longwall panel has a simple geometrical shape the use of influence functions was unwarranted for this investigation.

Empirical methods of Europe have not compared well to measured values of subsidence in the United States. The lack of agreement in the case of the NCB graphical method is probably due to presence of mostly shales in the overburden of the Great Britain coal fields whereas the United States coal fields occur beneath both shale and sandstone, Ref. (4). One approach to better prediction would be to compile graphical data on Longwalls in the

Appalachian Basin. This has been done for seven longwall panels in four mines, Ref. (6). Another approach is to adjust a profile shape to local conditions. Proceeding along this approach a profile function to describe the subsidence trough has been modified by Peng, Ref. (7), for six longwalls, and the maximum subsidence is calculated according to the strength of mine overburden. Others have also included the overburden strength in calculations of the predicted subsidence factor. Tandanand and Powell, Ref. (8), of the United States Bureau of Mines have made calculations which show that from seventeen longwalls in the Appalachian Coal Basin the subsidence factor is approximately linear with respect to the total percentage of sandstone and limestone.

PREDICTIVE MODELS - There are three methods for predicting subsidence characteristics over a longwall. These are the graphical method, the profile method, and a finite element method. All of these methods can be used to obtain a subsidence profile once the width and depth of the extraction is known. In addition, local conditions, such as the rock types and composition of the overburden, enter into refinements to the predicted profile.

The graphical method was developed by the NCB (National Coal Board) of Great Britain and published in the Subsidence Engineers' Handbook, Ref. (5). It is based on subsidence readings taken over many longwall panels, and while local conditions will affect the predictions, results have been found to be conservative when applied to mines in the United States, Ref. (4). As an example, Fig. 1 illustrates a typical subsidence

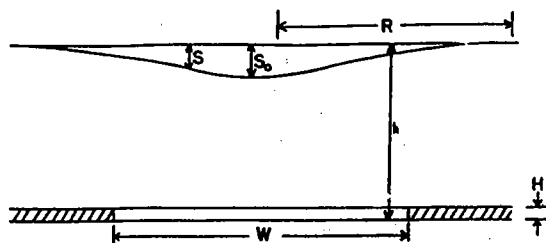


FIG. 1 TROUGH SUBSIDENCE FOR PANEL WIDTH LESS OR EQUAL TO CRITICAL VALUE

profile encountered during mining of a panel having a width less or equal to a critical value. The latter is the value in which the maximum possible subsidence is reached. When the panel is larger than

the critical value the trough subsidence profile will appear flattened as shown in Fig. 2.

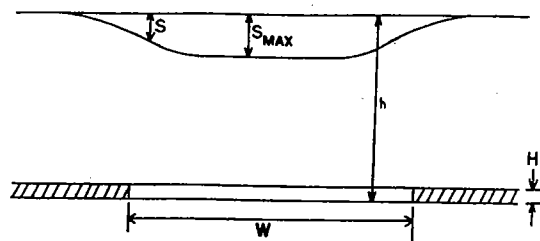


FIG. 2 TROUGH SUBSIDENCE FOR PANEL WIDTH LARGER THAN THE CRITICAL VALUE

On the other hand the profile method is based on an algebraic expression which fits the transitional zone from unsubsided surface to fully subsided surface. The expression which is felt to be the most appropriate is

$$S(x) = S_0 e^{-az^b}$$

where S_0 is the maximum subsidence on the cross section, a and b are site specific constants which can be derived by curve fitting known subsidence profiles. The site specific constants a , b do not have any significant meaning but rather are numbers that must be known before accurate predictions can be made for subsidence, strain, and curvature. The expression was applied to six longwall panels in the Appalachian region by Peng, Ref. (9), to obtain average values for a and b of 8.97 and 2.03, respectively. The dimensionless distance z equals x/R where x is the horizontal distance from the point of maximum subsidence and R is the distance across subsidence transition region. For a fully developed subsidence trough equals $0.74 H$ where values for subcritical areas are given by Peng.

SUBSIDENCE MEASUREMENT - Three sections of roadway were investigated. The most complete investigation was conducted along S.R. 689 in Meigs County, Ohio from mileage posts 0.11 to 0.26. Along this section of highway survey pins were installed every 25 ft. Reinforcing rods, 5 ft long, were driven into the ground. A 10 in. by 6 in. diameter concrete top was added to the steel rods spaced every 100 ft. The untopped rods (pins) were found to be as stable as the topped rods (monuments), and thus no differentiation was observed when analyzing readings. Leveling was accomplished with a Kern tilting level with a standard rod

graduated to 0.01 ft. All leveling measurements closed to maintain an accuracy in ft to within 0.1 miles of the circuit. Changes in horizontal distances between survey pins were measured with a steel survey tape to within an accuracy of 0.03 ft. Standard third order surveying equipment was used for all measurements. A bench mark was established outside the mined area as shown in Fig. 3. The longwall panel, its advance labeled with

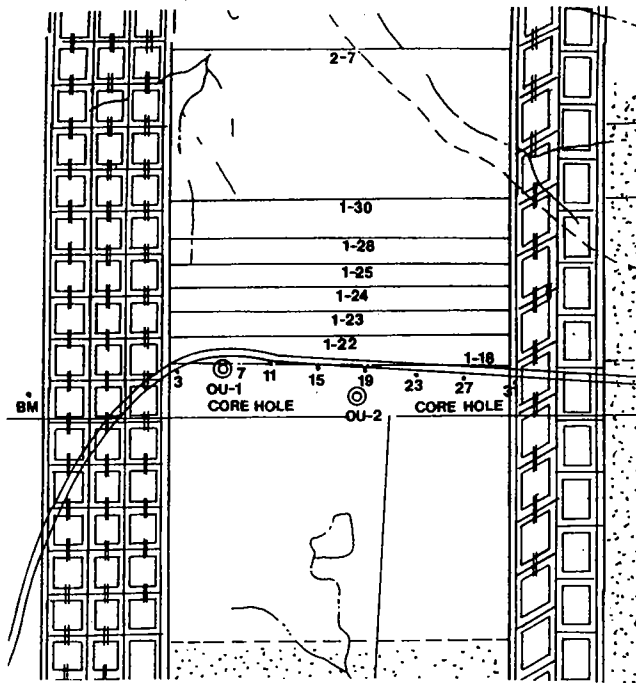


FIG. 3 LONGWALL PANEL WITH THE POSITION OF THE FACE LABELED

respect to subsidence and survey dates, is shown. Survey pins are labeled along the roadway. The longwall panel was 700 ft across, and advanced at the rate of

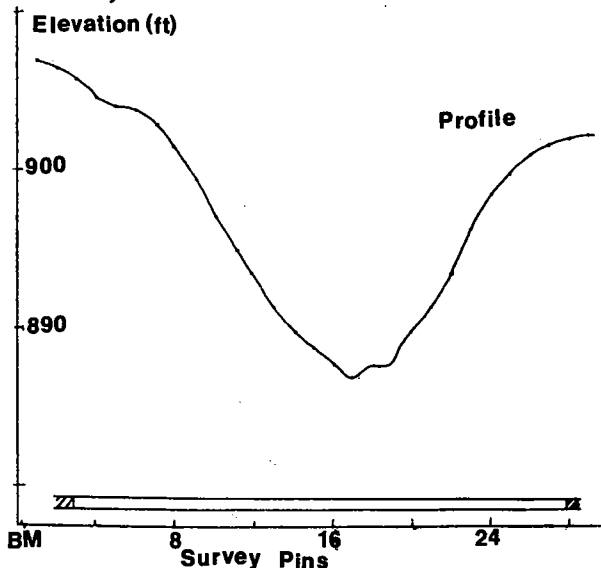


FIG. 4 ORIGINAL ROAD PROFILE OF S.R. 689 FROM 0.26 MILEAGE POSTS

approximately 50 ft per day, perpendicular with respect to S.R. 689. Five feet of coal was removed. The average depth of coal was 380 ft. The original ground profile is shown in Fig. 4. The local terrain is hilly where the longwall advanced up the hollow.

Fig. 5 shows the results of several subsidence measurements. There

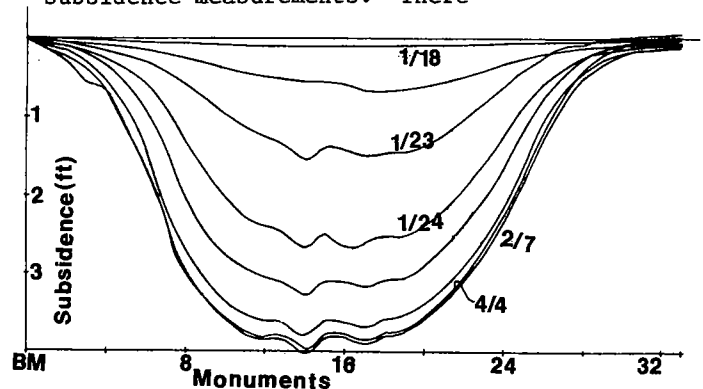


FIG. 5 LONGWALL MINING SUBSIDENCE OF S.R. 689, MEIGS COUNTY, OHIO FROM JANUARY, 1985 TO APRIL, 1985

was very little subsidence as the longwall passed beneath the survey line on January 18. The rate of settlement increased as the longwall progressed for about 150 ft, and then the rate of subsidence decreased until settlement was almost complete when the longwall had progressed 380 ft past the monument line. The last measurements were recorded on April 4. The center region of the subsidence profile is discontinuous and exhibits compressive features. The smooth shape and change in curvature along the sides indicates a tensile region. It is interesting to note just how localized the subsidence pattern is with respect to the longwall since very little settlement occurred outside the mined area.

COMPARISON OF FIELD DATA WITH PREDICTION

SUBSIDENCE - Two empirical methods are commonly used to predict maximum subsidence and shape of the subsidence trough when coal is removed with a longwall mining procedure. The National Coal Board of Great Britain has developed monographs based on years of experience and numerous measurements. The accuracy level, when applied to a new location, is expected to be within ten percent. National Coal Board data represents the largest group of data on longwall mining. Results have been compared for the locations under investigation. Fig. 6 shows that NCB predicts a subsidence trough that is deeper and wider than the one measured. The data are not symmetric because of a previously mined panel located to the right. The effect of

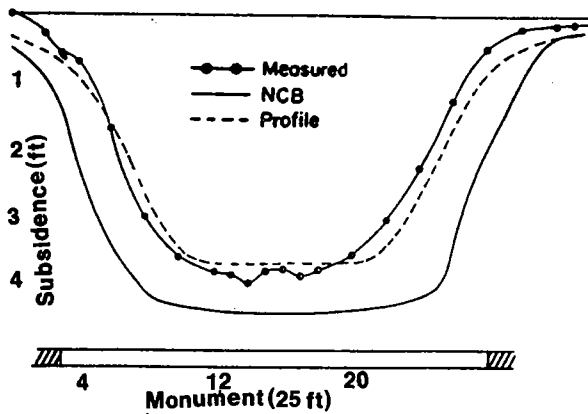


FIG. 6 SUBSIDENCE COMPARISON WITH PREDICTION ON S.R. 689 BETWEEN 0.11 AND 0.26 MILEAGE POSTS

the gob area was to result in additional subsidence along that side of the panel and over the entry between panels.

The profile method shows that the prediction of maximum subsidence is excellent, but the trough is again wider than field data indicate. Since the structural response of overburden (which determines the profile shape) and maximum subsidence could be further refined to apply to a particular mine, agreement can be further improved by calculation of profile equation coefficients S_0 , a , and b .

STRAIN - Strain is a direct measure of loading on surface structures and, consequently, a measure of damage which may result to highways and highway structures. In the cases investigated, highways were asphalt laid over a gravel base. Two concrete culverts with concrete entrys were situated above the longwall panels mined on S.R. 556. Both of these culverts were located in the compression zone near their respective panel centers.

Examination of the strains along S.R. 689 in Fig. 7 shows the discontinuous

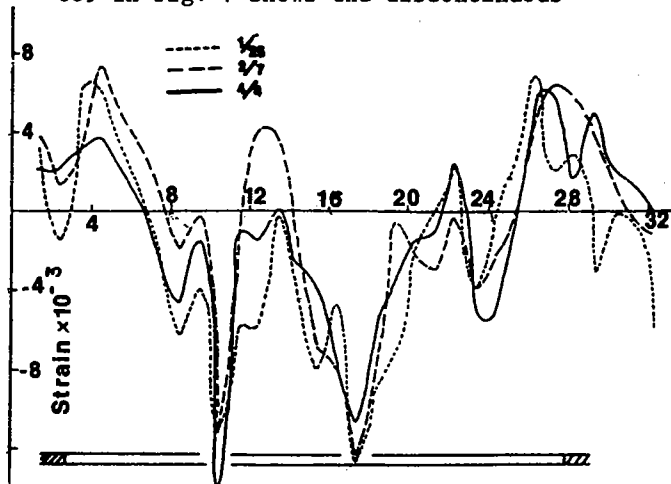


FIG. 7 STRAIN LEVELS MEASURED DURING LONGWALL MINING BENEATH S.R. 689 FROM 0.11 TO 0.26 MILEAGE POSTS

nature of subsidence with strain values in the range of + 7.2 in./in. to -14 in./in. Tensile cracks perpendicular to the road were recorded at panel edges. Fig. 8., upper photo, illustrates

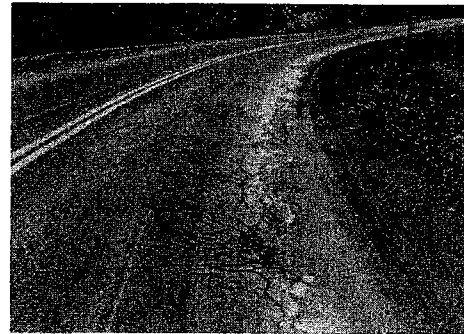


FIG. 8 UPPER VIEW IS OF BERM REPAIR ALONG S.R. 556 WITH A SOFTNESS IN PAVEMENT NOTED; LOWER VIEW IS OF TEMPORARY CRACK WHICH OPENED ALONG S.R. 689

some alligator and block cracking that were caused by longwall mining along S.R. 556 and aggravated by the winter weather. Approximately twenty percent of the cracks existed before mining took place. Cracks along S.R. 556 were minor which may be the result of the greater depth of mining. Along S.R. 689 one crack measured 3.5 in. across and displaced 2 in. in the direction parallel to the longwall (Fig. 8, lower photo). This crack opened at core hole OU-1 on January 17 before measureable subsidence took place. When the survey crew leveled monuments on January 22, the crack had closed, and it remained closed during the observation period. Compression movements were also recorded in the center of the panel, but no compressive damage was noted.

Comparison of strains, predicted by the NCB graphical method and the profile method, to measured values in Fig. 9 shows that the trend is the same in both cases with tension present at the panel edges and compression at the panel center. However the magnitudes are substantially different. Compressive strains calculated using the NCB method are much less than measured values. The

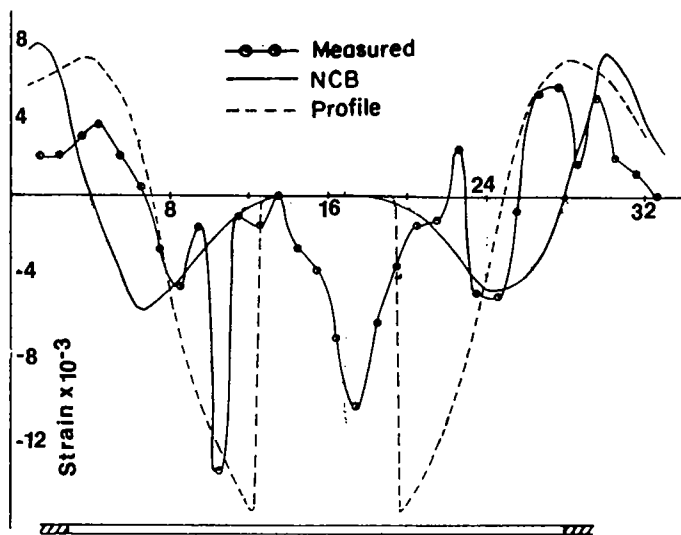


FIG. 9 COMPARISON OF MEASURED STRAIN TO PREDICTED VALUES FOR S.R. 689 FROM 0.11 TO 0.26 MILEAGE POSTS

erratic nature of the measured strain is probably due to the influence of the terrain on unstable soils.

STRATIGRAPHY

Core hole labeled OU-1 was located 100 ft from the edge of the 700 ft panel (see Fig. 3) in the compressive region predicted by the NCB graphical method. Core hole labeled OU-2 was located in the center of the longwall panel where final subsidence damage was predicted to be minimum since NCB graphical method calculates zero surface strain. The coal overburden was found to consist of interbeds of laminated sandstone, shale, and claystone with a four foot layer of limestone at 371 ft depth. The residual soil is typically 25 ft thick consisting of soil with sandstone fragments. Several thin seams of coal were encountered during coring. These tended to be brittle and highly fractured on removal from the core barrel. The bedrock in this area is of Pennsylvanian age. The lime content in the recovered shale corings was negligible with only an occasional fossil present. The layers having been deposited in a regular, but nonuniform manner tend to pinch out and reappear at other locations. The lithologic composition was 34.6% sandstone, 31.6% shale, 3.3% coal, and 1.8% limestone. The number of major layers were 8 sandstone, 11 shale, and 5 coal.

The laminated sandstone is a light grey, homogeneous rock that cores very easily with 15 ft continuous cores occasionally removed. Shales deposit in thin layers and weather very easily. Recovery of shale cores was difficult as the core barrel tended to slip on a water

softened shale surface.

VISUAL INSPECTION - A video camera probe, 1-1/2 in. in diameter, was lowered in all core holes. Special features such as cracking, resulting from subsidence were recorded on video tape. Core holes OU-1 and OU-2 were inspected both before and after the mining took place. The core holes were closed by the longwall mining sufficiently that the 1-1/2 in. probe could not be dropped into either core hole. Thus to examine features of subsidence damage, the core holes were drilled open again. Located in the tension zone, 100 ft from the edge of the 700 ft panel, OU-1 had to be completely drilled. The core hole in the center of the panel, OU-2, was more intact. When drilling OU-2, to reopen the core hole after the longwall had passed, the drill cut easily and dropped fast to indicate that the core hole had remained substantially undamaged.

Visual inspection of OU-1 showed that there were parallel layer cracks at rock interfaces and zones of crushed material, such as the thin seams of coal. High angle cracks were common in homogeneous portions of thick layers. Visual inspection of OU-2 showed parallel tension cracks to frequently occur. A mud coating was noticeable on viewing after mining which indicated a deposit of mud particles from surface water entering the core hole. The water level in OU-2 was only 10 ft from the surface which tended to obscure observations. Observations were recorded only at shallow depths as muddy water leaking into the Geoprobe under pressure obstructed the camera view.

An additional hole, OU-3, was cored into the center of adjacent longwall panel after subsidence took place. Corings removed from OU-3 were less fractured than expected with only the black shale strata, immediately above the excavated coal seam, found in a rubble state. High angle fractures were found in the remaining overburden. Therefore, based on video and core observations, it is concluded that the overburden rock failed with high angle cracks extending, in an irregular pattern, from the entry to the surface. The failed materials in the overburden settled into the excavated zone in continuous segments and not in small broken fragments as first believed. This failure mechanism is illustrated in Fig. 10.

CONCLUSIONS

The examination of subsidence measurements from this investigation gives subsidence profiles over three longwall panels which were mined beneath state highways in southeastern Ohio. Maximum

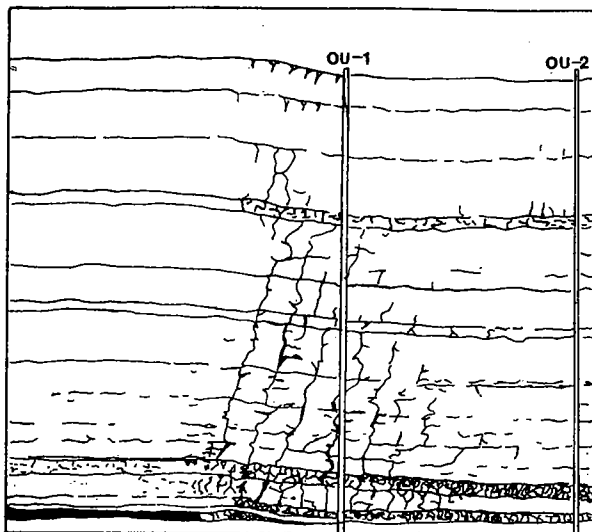


FIG. 10 FAILURE MECHANISM OF ROCK ABOVE A LONGWALL MINE

subsidence were 4.0 ft, 3.3 ft, and 3.1 ft for S.R. 689, S.R. 556 upper panel, and S.R. 556 lower panel. The rate of subsidence decreased to zero when the longwall mining face had proceeded to 600 ft, 650 ft, and 610 ft, respectively, beyond the roadways. The average depths of mining were 380 ft, 630 ft, and 533 ft, respectively. A crack appeared on S.R. 689 before the longwall face passed beneath it. But surface subsidence was not measurable until the longwall face had passed beyond the roadway. Ten percent additional subsidence was recorded on S.R. 556 over the upper panel when the lower panel was mined.

Comparison of the results from this investigation with an exponential profile method, modified by Peng to the Appalachian coal region, shows reasonable agreement between predicted and measured values of subsidence. Such a prediction would be important when examining drainage effects, induced bending strains in structures, and differential settlement of structures. The largest surface curvature was measured in the direction of mining on the lower panel on S.R. 556. However, the curvature perpendicular to the longwall face, is only temporary.

Maximum compressive strains measured tangent to the roadway were -14.0 and -5.7 micro in./in. for S.R. 689 and S.R. 556, respectively. Base lengths on which deformations were measured were 25 ft on S.R. 589 and 100 ft on S.R. 556, respectively. Because of the rapid variation in surface movement, measurements taken on a 25 ft gage length are felt to provide a more reasonable estimate of horizontal strains experienced by any surface structure.

Geoprobe examination of core holes before and after mining revealed no large voids or rubble zones in the upper regions

which were accessible to the Geoprobe. The upper thin coal layers were observed to be highly fractured to confirm the results anticipated from examination of corings. High angle fractures appear to be the primary overburden failure mechanism. This was duplicated in laboratory triaxial specimen tests conducted on cores of both sandstone and shale, Ref. (9).

DAMAGE PREDICTION AND PREVENTION - In general the mining laws of Ohio have been established to allow a coal company to remove the coal it owns while maintaining highways and highway structures. Thus, according to section 4153.11, an operator or the mining company is permitted to remove coal located beneath state highways, but is responsible for any damages which results from mining. This section of the law states that the coal company shall deliver a bond and that no permits shall be issued prior to receiving payments of such bond.

As an outline procedure for setting a bond, the Ohio Department of Transportation may determine the highway distance in which a 0.002 in./in. or more tensile strain and a 0.003 in./in. or more compressive strain occurs. The cost of resurfacing pavement and shoulders will then dictate the cost of such bond.

For the case in which a bridge is located in the subsidence zone, the type of bridge must be considered. If the bridge has supports which allow vertical and horizontal movements, then the damages should be minor. However, if the bridge requires rigid supports, strain and deformation produced by mining would determine the extent of damage and a cost estimate must be done on case by case basis.

If it is determined that coal must remain to protect against displacements or strains, then the distance at which mining should cease can be deduced from the measurements taken along S.R. 689. For example, a large surface crack (see Fig. 9) was recorded along S.R. 689 on January 17. This would indicate the highway to be located at about 50 ft ahead of the longwall face. At this position, the maximum strain recorded along the highway indicates a strain level of 0.004 in./in. If 0.002 in./in. is taken as a maximum permissible tensile strain of the pavement, then, based upon the strain reading at this location, the longwall face should stop advancing 70 ft prior to reaching the spot directly under the pavement if damage has to be prevented. In such a case the State of Ohio would be required to negotiate for the value of the coal. However, for the usual case it will be more economical to maintain the roadway or bridge.

It must be noted that the distance estimate is dependent upon the condition at each location and should take into account such factors as: 1) the stratigraphy of the area which includes depth of the overburden, height of the coal seam, and the inclination of the overburden materials as well as the coal seam, 2) the width (length) of the longwall panel which will be mined under the highway, and 3) the dimensions of pavements, shoulders, bridges or structures to be protected.

It is recommended that other longwall mining operations be monitored for long-term as well as short-term subsidence and strain effects. This would help establish strain and displacement values at which structural damage could be expected. Further research would also clarify several economic issues such as the cost of coal left in place to prevent subsidence compared to the cost of repairing subsidence damage.

The primary benefit of this project is to provide the Ohio Department of Transportation with a quantitative measure of subsidence resulting from longwall mining. This is the first public study of this type which has been conducted in the Meigs County area, and so it will be a valuable contribution to the development of a subsidence model. Moreover, the visual inspection of surface features should be of value when assessing initial versus long-term surface movement.

ACKNOWLEDGEMENTS

The authors appreciate the cooperation and financial support of the Federal highway Administration and the Ohio Department of Transportation under Grant No. 14376(0).

REFERENCES

1. Von Schonfeldt H., F.D. Wright, and K. F. Unrug, Min. Cong. J., 66(5) 41-45 (1980).
2. Wardell K., Min. Cong. J. 55(1) 36-42 (1969).
3. Sutherland H. J., K. W. Schuler, and S. E. Benzley, In Situ, 7 87-113 (1983).
4. Munson D. E. and W. F. Eichfeld, Sandia Lab. Res. Report SAND80-0537 (1980).
5. National Coal Board, "Subsidence Engineer's Handbook," Hobart House, Grosvenor Square, London (1975).

6. Peng S. S. and S. L. Cheng, Coal Min. and Proc. 18(5) 84-95 (1981).

7. Peng S. S. and D. W. Chen, Dept. of Min. Engr., West Virg. Univ., TR81-1, (1981).

8. Tandanand S. and L. R. Powell, U.S. Bur. of Min. RI 8630 (1982).

9. Hazen, G. A. and S. M. Sargand, Ohio Dept. of Trans., FHWA/OH-86/006 (1985).

A Rock Slide on Interstate 81, Northeastern Pennsylvania

J. PETER WILSHUSEN and JON D. INNERS

Pennsylvania Geological Survey

Harrisburg, Pennsylvania

DUANE D. BRAUN

Bloomsburg University

Bloomsburg, Pennsylvania

HGS-87-25

INTRODUCTION

AT 12:15 AM ON MARCH 20, 1986, a massive rock slide blocked all three south-bound lanes of Interstate Route 81 approximately 2 miles south-east of Nanticoke, Luzerne County, northeastern Pennsylvania (Figure 1). The failure occurred in a deep cut on the dip slope of Penobscot Mountain, one of several spectacular rock cuts excavated in the mid-1960's to allow passage of I-81 through the southeastern rim of the Wyoming-Lackawanna basin. Because cuts on both north- and south-bound lanes expose rock layers that dip directly into the roadway, this portion of I-81 had long been suspected of being particularly susceptible to rock slides. For this reason, Pennsylvania Department of Transportation (PennDOT) geotechnical engineers had visually monitored its behavior on several occasions. The cuts appeared to be stable, however, and only a few falls of individual rocks took place in the twenty years between construction of the highway and the failure of last March.

Luckily, the cut slope in question failed catastrophically at a time of minimal traffic volume on the highway. Although one car ran up over the rock pile an instant after the slide had stopped moving, no one was injured. By early in the afternoon of the same day, PennDOT maintenance crews had cleared enough of the debris to permit southward traffic flow in the outer two lanes. The third lane was opened several days later.

GEOLOGIC SETTING

The mountainous southeastern rim of the Wyoming-Lackawanna basin in the vicinity of Nanticoke is formed by clastic sedimentary rocks ranging from the Upper Devonian Duncannon Member of the Catskill Formation to the Lower Pennsylvanian Pottsville Formation (Figure 2). Penobscot Mountain, the higher of two northeast-southwest trending ridges, is underlain by the Duncannon Member on its southeastern, or scarp, slope and the Lower Mississippian Pocono Formation on its northwestern, or dip, slope. The cut-slope failure of March 20, 1986, took place in the latter unit.

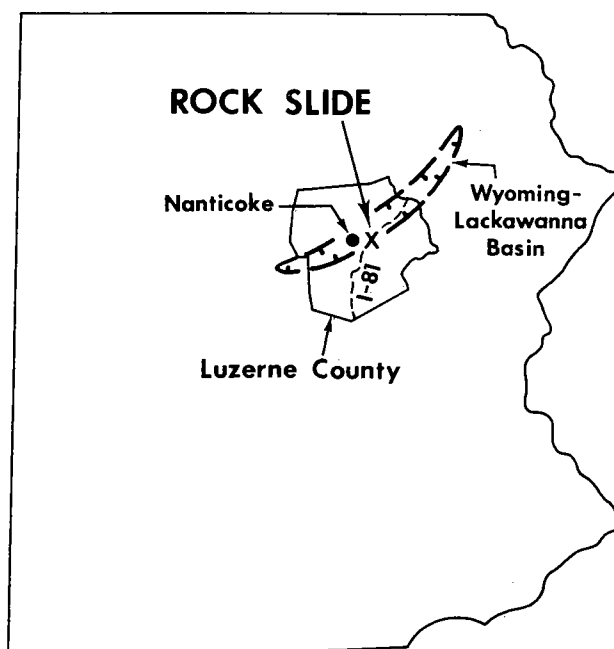


Figure 1. Location map.

The Pocono Formation at the site of the rock slide is medium to thick-bedded (mostly 1- to 2-foot beds), trough-crossbedded, medium-gray (N5), medium- to coarse-grained, micaceous quartzose sandstone, containing thin, discontinuous beds of medium-light-gray (N3) mudstone. These rocks represent the deposits of a great braided-river system that developed along the margin of the Appalachian highlands approximately 360 million years ago (Inners, 1978).

Structurally, the site lies on the southeast limb of the Wyoming-Lackawanna synclinorium, a complex, canoe-shaped downfold that contains the numerous anthracite deposits of the Nanticoke-Wilkes-Barre-Scranton area. Bedding planes dip at moderate angles (30 to 50°) to the northwest, and conspicuous strike and dip joints are mostly subvertical. Spacing of the joints varies from less than 0.5 ft to 10 ft, averaging 1 to 2 ft.

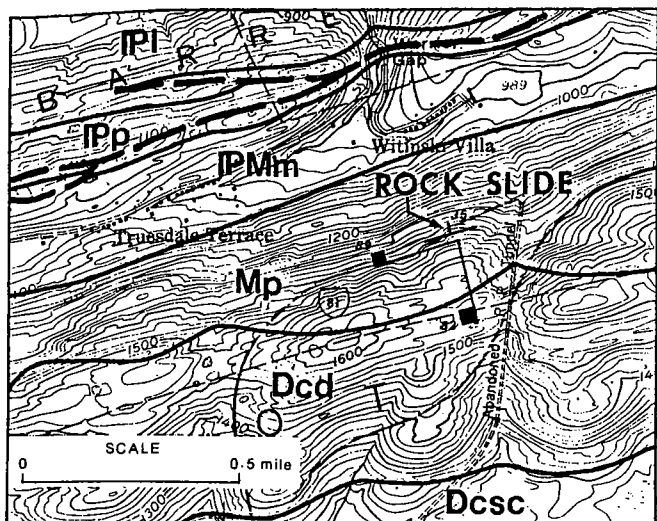


Figure 2. Geologic map of a portion of the Wilkes-Barre West quadrangle, showing the site of the I-81 rock slide. (IPl = Llewellyn Fm.; Pp = Pottsville Fm.; IPMm = Mauch Chunk Fm.; Mp = Pocono Fm.; Dcd = Duncannon Member of Catskill Fm.; Dcsc = Sherman Creek Member of Catskill Fm.). Geology after Sevon, in Berg and Dodge (1981), and Bergin (1976).

The bedding planes and joints are geologic discontinuities which greatly facilitate the percolation of vadose water and the circulation of groundwater through the rock mass.

The area lies 10 to 15 miles northeast of the Late Wisconsin terminal moraine (Crowl and Sevon, 1980) and was last subject to the rigors of continental glaciation about 15,000 years ago. Preserved glacial deposits are very sparse on the mountain slopes, however, and none were observed locally.

THE ROCK SLIDE OF MARCH 20, 1986

At failure, the slide mass was approximately 185 ft wide, 40 ft long, and 20 to 40 ft high and had a total volume of approximately 7000 yd³ (Figure 3). The limiting discontinuities of the failure consisted of a curved, subvertical strike joint (head scarp), subvertical dip joints (side scarps), and several undulatory bedding planes (slide surface) (Figure 4). Shear failure took place mainly within weathered mudstone lenses that partially fill several adjacent northwest-plunging crossbed troughs. At the extreme northeast end, however, slippage occurred on a 1- to 2-inch band of friable sandstone that had been intensely leached and oxidized by the intermittent movement of perched groundwater. The average size of blocks in the slide - 1 to 2 ft on a side - reflected the typical spacing of bedding planes and strike and dip joints; one huge block - more than 10 feet square - remained intact at the extreme northeast end (Figure 5).



Figure 3. Aerial view of I-81 rock slide as it appeared on March 21, 1986, one day after cut-slope failure. Note the long headwall scarp and the blocky nature of the slide mass. (Photo courtesy of Pennsylvania Department of Transportation).

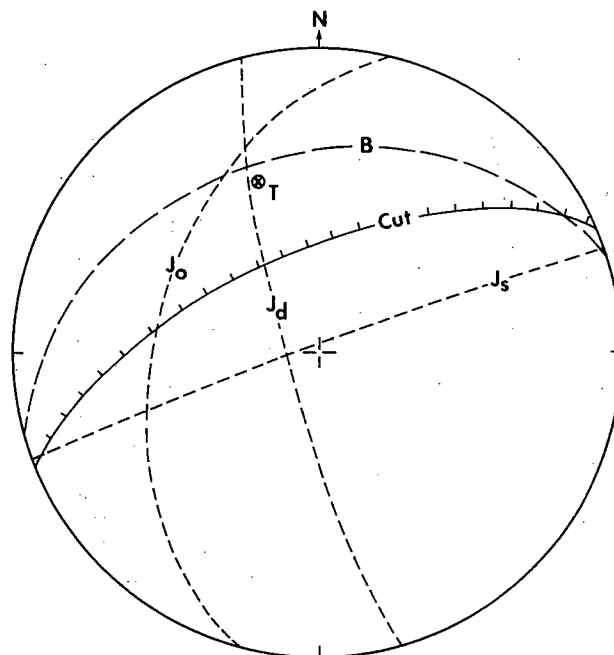


Figure 4. Stereogram (equal-area lower hemisphere projection) of discontinuity planes, etc., in the Pocono sandstone at the site of the I-81 rock slide. Bedding: B = N73E/35°NW (failure surface); joints: Js = N70E/89°NW (head scarp), Jd = N15W/82°SW (side scarps), Jo = N15E/46°NW; axis of crossbed trough: T = N20W at 40°. The cut face shown (N66E at 1/2 to 1, or 63°) is the design cut slope prior to failure.

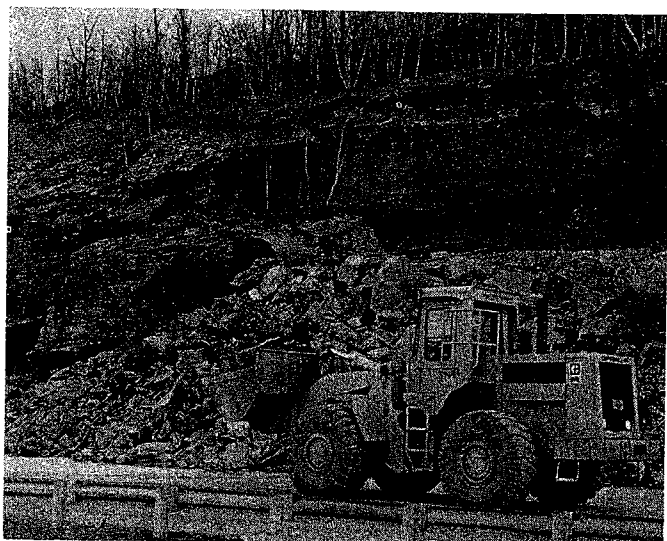


Figure 5. View of the northeast end of the rock slide, showing the large intact block adjacent to the side scarp. Note the subvertical head scarp.

The trigger for the rock slide may have been two periods of rainfall five days (3.1 in within 24 hours) and 10 hours (0.4 in within 3 hours) before failure (Figure 6; Inners et al., 1987). Excess pore-water pressure in the mudstone lenses probably reduced shear strength below a critical level, leading to abrupt slippage. The 5-day lag between the major rainfall event and failure is apparently related to the time it took for the precipitation to infiltrate from the surface and upslope of the incipient slide and generate maximum pore-water pressure within the potential slide surface and the headwall tension crack (Freeze and Cherry, 1979; see Figure 6). Wilson and Dietrich (1985) observed a similar delayed response (about 4 days) in the maximum pore-water pressures registered in piezometers in an incipient slide mass on a slope in the California Coast Ranges.

Repair of the I-81 rock slide caused an extensive traffic detour, took-up several days of unplanned-for work time, and cost approximately \$53,000.

SIGNIFICANCE

This case history is not unusual; most of us have seen, and many have worked with, cut slopes in rock or soil that show the potential for failure. However, transportation routes still need to be put through questionable terrain and excavations still need to be made.

A great many factors are included in route selection, engineering design, and construction practices with geology (and related groundwater conditions) being only part of the whole. Consideration of land availability, budgets, and construction materials is certainly necessary, but no more so than geology. The catchy statement by Will Durant that "Civilization exists by geological consent...subject to change without notice" illustrates nicely that geological con-

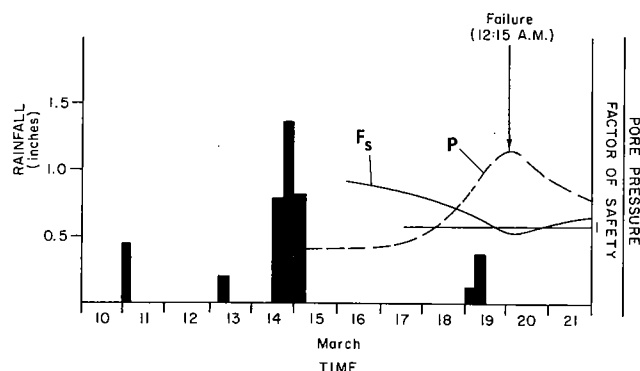


Figure 6. Composite of histogram of rainfall at Wilkes-Barre Scranton Airport, Avoca, PA, in mid-March, 1986 (data from the National Oceanic and Atmospheric Administration) and graphs of the ideal temporal relationship of pore pressure and factor of safety to a period of rainfall (from Freeze and Cherry, 1979). Note the lag time between peak rainfall and slope failure at presumed maximum pore pressure within a slope.

ditions are among the most important factors to be considered in project planning and development.

What is available to contribute to the lasting success of a project? We may have a consultant's report with drill hole data to estimate quantities of common and rock excavation. But beyond that there is a demonstrated need for designers to evaluate geologic and groundwater conditions and plan accordingly.

Available information dealing with bedrock and surficial geologic conditions, groundwater data, and mineral resources is contained in geological survey publications, open-file reports, and project geologists' heads. Examples are atlases and general geology, environmental geology, mineral resource, and groundwater reports. Time spent by Project Managers assimilating and evaluating information from these studies and sources can enhance project design and help cut costs. Services from outside experts are essential to successful design, but their incorporation into overall project planning and development requires an understanding by engineers of the total geological framework.

REFERENCES

- Berg, T. M. and C. M. Dodge, Pa. Geol. Surv., 4th ser., Map 61 (1981)
- Bergin, M. J., U. S. Geol. Surv. Misc. Inves. Ser. Map I-838 (1976)
- Crowl, G. W. and W. D. Sevon, Pa. Geol. Surv., 4th ser., Gen. Geol. Rept. 71 (1980)
- Freeze, R. A. and J. A. Cherry, "Groundwater," p. 472-475, Prentice-Hall, Englewood Cliffs, New Jersey (1979)

Inners, J. D., Pa. Geol. Surv., 4th ser.,
Atlas 174c (1978)
Inners, J. D., D. D. Braun, and J. P.
Wilshusen, Geol. Soc. Amer. Abs. with Prog.
19, 21 (1987)
Wilson, C. J. and W. E. Dietrich, EOS 66, 898
(1985)

Roadway Relocation Through Abandoned Municipal Dumps: A Case Study Near Hershey, Pennsylvania

BERNARD MARKUNAS

Gannett Fleming Geotechnical Engineers, Inc.
Harrisburg, Pennsylvania

HGS-87-26

INTRODUCTION

SUBSURFACE INVESTIGATIONS for the proposed relocation and widening of Route 39 (Hershey Park Drive) in Derry Township, Pennsylvania disclosed the presence of two municipal dumps beneath the highway alignment. The dumps had not been identified by the project's Design Location Study and the waste material in the dumps created the possibility of significant geotechnical and environmental difficulties during and after construction. Additionally, the potential presence of hazardous or toxic materials in the dumps created legal concerns regarding long-term liability for any groundwater contamination that might be associated with the waste materials.

A drilling and sampling program was conducted in the Spring of 1986 to characterize the waste material in the dumps identified during the roadway investigation. Additional drilling and sampling was implemented to evaluate the local surface water and groundwater quality. This work was necessary to determine if subsurface contamination was present at the dumps, because such contamination could affect: (1) the acceptance of the dump material at licensed sanitary landfills or (2) the Department of Transportation's liability if the dump material were to remain in place under the proposed right-of-way.

SITE CONDITIONS

Much of the project area is underlain by Ordovician carbonates (Hershey-Myerstown, Annville, Ontelaunee, and Epler Formations) typical of the Great Valley. The Hershey Valley is bounded by topographic highs formed by the shales and siltstones of the Martinsburg Formation to the north, and by the sandstones, siltstones, and shales of the Gettysburg Formation to the south. Numerous thrust faults are located in the area and the bedrock is generally quite deformed, with relatively tight fold geometry and overturned bedding commonplace.

DESIGN ALTERNATIVES

The presence of dump material under the proposed roadway embankments could result in undesirable roadway settlements if no remedial treatment was undertaken. Settlements would include: primary consolidation, and long-term secondary consolidation due to embankment loading, and long-term consolidation due to decomposition of organic material. Settlement calculations indicated that a maximum of approximately 1.6 feet of short-term settlement would take place during and after construction, and that approximately 0.5 feet of long-term, postconstruction settlement would occur. Additionally, significant differential settlements were anticipated because of the extremely heterogeneous nature of the waste material. Evidence of these extreme differential settlements (in excess of 2.1 feet) can be found in the hummocky topography of the proposed highway alignment.

Five remedial treatment alternatives were evaluated for the two dump areas: (1) removal of all waste material from beneath the roadway limits and disposal at a sanitary landfill; (2) removal of the waste material and relocation of this material to the dump area outside of the highway right-of-way; (3) leave the waste material in place and support the highway on stone columns; (4) leave the waste material in place and use impact densification to overconsolidate the area; (5) leave the waste material in place and build the highway without any special treatment. Characterization of the waste material became a necessity before any of these schemes could be adequately evaluated.

FIELD INVESTIGATION

The subsurface investigation was divided into two phases, Phase I in which samples of the soil, solid waste and leachate within the dumps were collected along with samples of the leachate coming from the dumps; and Phase II was conducted outside the limits of the dumps. No Phase II monitoring wells were installed in

the dumps because of the potential for creating a pathway for contaminants to enter the local groundwater system.

SITE SAFETY - Safety of field personnel was a major consideration when conducting exploratory investigations on site. Although the dumps were thought to contain only municipal waste, precautions were taken to protect the workers on site in the event that hazardous materials were encountered. All personnel involved with the waste sampling wore chemically resistant boots, coveralls, and gloves. Air quality was monitored during the Phase I exploratory investigation with a photo-ionization device (PID) and a combustible gas indicator (CGI). Air purifying respirators were kept on hand in the event that a respiratory hazard was detected.

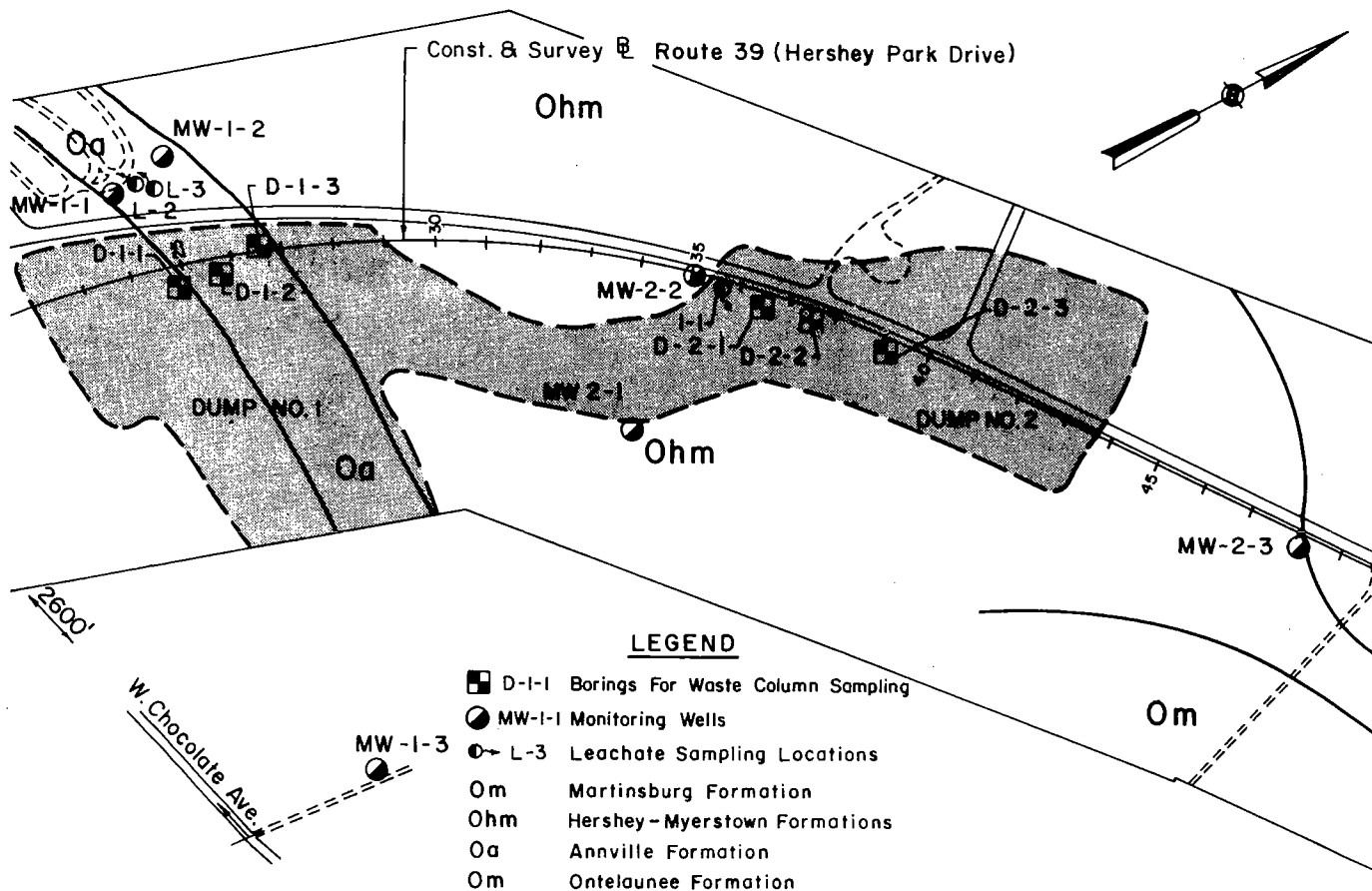
No measurements made in the work space with the air monitoring instrumentation during the Phase I investigation indicated levels substantially above background. Therefore, air monitoring was not conducted during the Phase II investigation.

PHASE I INVESTIGATION - The boring and leachate sampling locations for the Phase I investigation are shown on Figure 1. Six boring locations were chosen to provide representative samples of the two identified dump areas.

The borings were continuously sampled using 3-inch diameter split-spoons driven with a 300-pound hammer. Termination depths were established in the field, generally using the criterion of obtaining one sample of native soil below the fill. To prevent cross contamination, the sample spoons were scrubbed with water and detergent and then rinsed with distilled water before each sample was obtained. Additionally, no wash water was used to complete these holes, facilitating the location and sampling of perched water (i.e., leachate) within the boring, as well as limiting the possibility of cross contamination of samples. Where leachate was encountered in the borings, it was sampled with a PTFE bailer. Upon completion, a bentonite and Portland cement grout plug was placed in the bottom of each boring and the remainder of the hole was backfilled to limit the possibility of groundwater contamination.

The soil and waste samples were screened in the field with a PID before shipment to the laboratory. Most of these PID measurements were at background levels, although measurements of a few samples produced readings of 0.2 to 0.6 PPM (measured as benzene). Following the PID screening, the solid waste samples were placed in new glass jars with PTFE lined lids and then put in a

FIGURE - I
SITE PLAN



cooler on ice for transport to the laboratory. Individual solid waste samples for each boring were composited in the laboratory before analysis, to make a composite sample for each of the two dump areas.

The boring program revealed a layer of fill of variable thickness, overlain by a relatively thin soil cap. The cover thickness ranged from approximately 1 to 4 feet and the fill thickness ranged from approximately 1 to 22 feet. The principal constituents of the fill appeared to be ashes, cinders, wood, and bricks, with lesser amounts of municipal waste including paper, plastic, metal cans and glass.

PHASE II INVESTIGATION - The objective of the second phase of the exploratory program was to characterize the groundwater quality of the area. Six monitoring well locations were established at the mutual agreement of the Consultant, PennDOT and PennDER. Well locations are shown on Figure 1. Two of these wells were located upgradient of the dumps to be sampling points for background data and the other wells were located downgradient of the dumps so that the effect of the dumps on local groundwater quality could be determined.

At each of the well locations an air rotary drill rig was used to advance the 6-inch borehole. In some instances, bedrock and overburden were incompetent and the borings would not stay open without temporary casing. In these situations the holes were reamed to 8-inch diameter and a nominal 6-inch diameter steel casing was installed. Remaining drilling and well construction was completed through the temporary casing. The upgradient wells were drilled to depths of approximately 50 feet and the downgradient wells were drilled to depths ranging from 29 to 70 feet. No samples of the overburden or rock were collected but a gross visual classification of these materials was made from inspection of the cuttings. After a water-bearing zone had been penetrated, the wells were developed by air-lifting, using the drill rods and the drill rig's compressor. This procedure was continued in each well until there was no visible reduction in turbidity through time.

Threaded, flush-joint, 2-inch diameter PVC well screens and casing were then installed in the holes. The screens were packed with silicious 2B gravel to a point above the top of the screen and the remainder of the hole was backfilled with granular bentonite to approximately 3 feet from the ground surface. A steel protective casing with a locking cap was then installed and set in cement. The drilling equipment was steam cleaned at the completion of each hole to limit the possibility of cross contamination. The wells were allowed to equilibrate for approximately 2 weeks before groundwater samples were collected.

Groundwater levels were measured with an

electric depth sounder in all of the wells prior to purging and sampling any of the wells. The wells were purged using a stainless steel submersible pump. The exterior of the pump was decontaminated and the interior of the pump and the hose was flushed after each use to avoid cross contamination between wells. Water samples were bailed from the wells using PTFE bailers after three well volumes of water had been purged. A separate bailer was used for each well. Samples were preserved in accordance with EPA methods and placed on ice in coolers for transport to the laboratory. Chain-of-Custody documentation was maintained throughout the sampling and testing.

ANALYTICAL INVESTIGATION

TEST METHODS - Solid waste samples were subjected to all of the chemical and physical analyses prescribed to characterize waste for the Pennsylvania Department of Environmental Resources (PennDER) Module 1 (Request for Approval to Treat, Store, or Dispose of a Hazardous or Residual Waste Stream). All solid waste samples were tested in accordance with EPA SW-846, Test Methods for Evaluating Solid Waste. The analyses required for Module 1 provide a comprehensive characterization of the physical and chemical properties of the waste material and include those parameters [Extraction Procedure (EP) Toxicity, reactivity, corrosivity, and ignitability] necessary to determine whether a waste is hazardous. Analyses of the solid waste samples were completed first, and then these samples were leached and this extract was analyzed for any constituent or parameter found in concentration greater than 0.001 mg/kg in the total analysis of the solid material.

Leachate and groundwater samples were analyzed for priority pollutant compounds using EPA Methods 601, 602, 608, and 625. Analyses for toxic metals and cyanide were conducted according to Standard Methods for the Examination of water and Wastewater (Greenberg, Connors, and Jenkins, 1980). Priority pollutant analysis was chosen because it is relatively comprehensive and the specific nature of the components of the dump waste were unknown. It should be noted that the water samples were not filtered prior to analysis for toxic metals, consequently, the results presented are for total analytes rather than for dissolved analytes.

RESULTS - Table 1 summarizes significant parameters from the physical analyses of the dump waste material and chemical analyses of EP extracts of this waste, as well as the Maximum Contaminant Levels (MCLs) of contaminants characteristic of EP Toxicity as set forth in the EPA regulations promulgated under Section 3001 of the Resource Conservation and Recovery ACT (RCRA). A number of contaminants were found in concentrations at least two

TABLE 1
ANALYTICAL SUMMARY: DUMP WASTE & EP EXTRACTS

SAMPLING LOCATION		D1-1	D1-3	D2-1	D2-2	D2-3
CONTAMINANT	MCL					
Arsenic	5.0	0.0013	0.0052	0.0024	0.0057	0.0039
Barium	100.0	2.73	2.00	4.09	3.27	2.46
Cadmium	1.0	0.0045	0.004	0.004	0.0075	0.004
Chromium	5.0	0.0500	0.050	0.050	0.050	0.050
Lead	5.0	0.084	0.025	0.29	0.065	0.025
Mercury	0.2	0.0005	0.0005	0.0006	0.0005	0.0005
Selenium	1.0	0.005	0.005	0.005	0.005	0.005
Silver	5.0	0.010	0.010	0.010	0.010	0.010
Endrin	0.02	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Lindane	0.4	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Methoxychlor	10.0	<0.020	<0.020	<0.020	<0.020	<0.020
Toxaphene	0.5	<0.050	<0.050	<0.050	<0.050	<0.050
2,4-D	10.0	<0.010	<0.010	<0.010	<0.010	<0.010
2,4,5-TP Silvex	1.0	<0.005	<0.005	<0.005	<0.005	<0.005
Reactivity	NR	NR	NR	NR	NR	NR
Corrosivity		NC	NC	NC	NC	NC
Ignitability		DNI	DNI	DNI	DNI	DNI

NOTES: MCL = Maximum Concentration Level
All values in MG/L

NR = Non-Reactive
NC = Non-Corrosive
DNI = Did Not Ignite

orders of magnitude less than MCLs established by RCRA.

Tables 2 and 3 summarize significant parameters from the chemical analyses of the groundwater and leachate samples, as well as the EPA public water supply MCLs as set forth in the National Primary and Secondary Drinking Water Regulations. Review of the analytical results for the water samples revealed that concentrations of virtually all of the organic priority pollutant compounds were below the detection limits of the instruments performing the analyses. The only organic compounds found above the detection limits were toluene (2 PPB), bis (2-ethylhexyl) phthalate (40 PPB), di-n-butyl phthalate (20 PPB), and butyl benzyl phthalate (11 PPB). Generally levels of inorganic compounds were also quite low, though several compounds, particularly chromium, cadmium, lead, arsenic, and mercury were found in levels exceeding the National Drinking Water Standards. It should be noted that there was a problem with spike recovery when the composite perched water (leachate) samples from the dumps were analyzed for selenium. Apparently something in the matrix was inhibiting recovery and the detection limit had to be raised (the detection limit for selenium for these two samples was 0.05 PPB though the MCL for selenium is 0.01 PPB).

DISCUSSION - Generally, a waste is considered hazardous if: (1) it is or contains a waste listed in Appendix VIII of 40 CFR §§ 261; (2) it demonstrates any of the following hazardous characteristics: Ignitability, Corrosivity, Reactivity, EP Toxicity; or (3) it is otherwise capable of causing envi-

ronmental or health damage if disposed of improperly. The Extraction Procedure for Toxicity identifies a waste whose constituents have a tendency to leach or migrate when disposed of in an improperly designed sanitary landfill. A solid waste exhibits the characteristic of EP Toxicity if the extract obtained from a representative sample of the waste is analyzed and is found to exceed the threshold levels shown in Table 1. A waste is considered hazardous if an extract contains toxic contaminants in concentrations 100 times greater than the safe drinking water MCLs.

Analysis of the solid waste from the dumps indicated that none of the samples tested were hazardous and that none of the contaminants present even approached the MCL. Based upon this investigation, there were not indications that the physical and chemical characteristics of the waste material in the dumps necessitated its disposal in a landfill for hazardous wastes had it been decided to remove and transport the dump material.

Analyses of leachate from both the borings and outbreaks indicated that a number of inorganic and organic contaminants were present, but in such low concentrations that if anything, these leachate samples were atypical because of the low concentrations of most contaminants. The toluene detected, was probably the result of the presence of common household solvents, lubricants, gasoline, etc. in the landfill. Though some contaminants do exceed the MCLs for drinking water, the leachate from the dumps is not a source of public water and contaminants in these concentrations should not be a cause for concern.

TABLE 2
ANALYTICAL SUMMARY: GROUNDWATER

SAMPLE TYPE		GROUNDWATER					
SAMPLING LOCATION		MW 1-1	MW 1-2	MW 1-3	MW 2-1	MW 2-2	MW 2-3
CONTAMINANT	MCL						
Cyanide (total)		<0.03	<0.03	<0.03	<0.0001	<0.0001	<0.03
Phenol		0.0131	0.0107	<0.0100	0.0117	0.0723	0.0188
Antimony		0.0005	<0.0005	<0.0005	0.0120	<0.0005	0.019
Arsenic	0.05	0.07	0.003	0.023	0.018	0.010	<0.001
Beryllium		<0.020	<0.020	<0.020	<0.010	<0.010	<0.020
Cadmium	0.01	0.0055	0.017	0.005	0.006	<0.005	<0.005
Chromium (total)	0.05	0.057	<0.025	0.051	<0.025	<0.025	<0.025
Copper	1.0	0.14	0.039	0.14	0.67	0.16	0.054
Lead	0.05	0.088	<0.025	0.11	0.065	<0.025	<0.025
Mercury	0.002	0.0035	0.0036	0.002	<0.0005	<0.0005	<0.0005
Nickel		0.098	<0.050	0.074	<0.030	<0.030	<0.050
Selenium	0.01	<0.003	<0.003	<0.003	<0.025	<0.025	<0.003
Silver	0.05	<0.025	<0.025	<0.025	<0.020	<0.020	<0.025
Thallium		0.003	<0.003	<0.0039	<0.001	<0.001	0.013
Zinc	5.0	0.34	0.03	0.47	0.085	0.013	0.019
Organics D.L.		20(*)	NONE	NONE	NONE	11(**)	40(***)

NOTES: MCL = Maximum Contaminant Level
D.L. = Detection Limit
* = di-n-butyl phthalate
** = butyl benzyl phthalate
*** = bis (2-ethylhexyl) phthalate

All values in MG/L (except as noted)
Only those priority pollutant organic compounds found in concentrations > detection limits of instruments are included in this tabulation

TABLE 3
ANALYTICAL SUMMARY: LEACHATE

SAMPLE TYPE		LEACHATE IN BORINGS		LEACHATE IN OUTBREAKS		
SAMPLING LOCATION		DUMP 1	DUMP 2	L-1	L-2	L-3
CONTAMINANT	MCL					
Cyanide (total)		<0.0005	0.007	0.004	0.005	<0.0005
Phenol		0.1	0.105	0.112	0.092	0.107
Antimony		<0.040	<0.040	<0.002	<0.002	<0.002
Arsenic	0.05	0.32	0.88	0.17	0.15	0.12
Beryllium		0.013	0.029	<0.005	<0.005	<0.005
Cadmium	0.01	0.008	0.014	0.005	<0.005	<0.005
Chromium (total)	0.05	0.24	0.45	<0.025	<0.025	<0.025
Copper	1.0	0.27	0.71	0.019	<0.020	<0.020
Lead	0.05	0.65	1.36	0.052	<0.025	<0.025
Mercury	0.002	0.0013	0.0094	0.0008	0.0005	0.0007
Nickel		0.2	0.47	<0.025	<0.025	<0.025
Selenium	0.01	<0.050	<0.050	<0.005	<0.005	<0.005
Silver	0.05	<0.010	<0.010	<0.010	<0.010	<0.010
Thallium		<0.010	<0.010	<0.002	<0.002	<0.002
Zinc	5.0	1.76	2.24	0.079	0.054	0.033
Organics D.L.		NONE	1.4(*)	NONE	1.3(*)	NONE

NOTES: MCL = Maximum Contaminant Level
D.L. = Detection Limit
* = Toluene
All values in MG/L (except as noted)

Only those priority pollutant organic compounds found in concentrations > detection limits of instruments are included in this tabulation.

Additionally, no known public water supplies depend upon the groundwater in the vicinity of the dumps.

Although there are many possible sources for groundwater contamination, the large quantities of ash present in the dumps may be responsible for at least some of the trace metals present in the groundwater. Ashes produced by the combustion of coal frequently have significant levels of trace metals, through the presence of these trace metals in the upgradient wells suggests that the dumps are not the only possible source of groundwater contamination. The three phthalate compounds detected in the groundwater are probably the result of the degradation of plastic containers in the dumps. Another possible source might be the PVC well screen and casing, though the screen manufacturer claims that the screen and pipe are not a possible source of contaminants.

Even though there are indications that there is some groundwater contamination in the area, it appears that the dumps are not the sole source of this contamination because each of the upgradient wells had detectable levels of organic or metal contaminants. Analysis of groundwater from MW 1-3 indicated that background levels of chromium and lead were above the MCL and mercury was at the MCL. The groundwater sample from the other upgradient well (MW 2-3) had detectable levels of bis (2-ethylhexyl) phthalate.

CONCLUSIONS

As might have been expected after inspecting samples of the waste material, groundwater in the vicinity of the dumps, and leachate in the dumps and in seepage outbreaks, the waste contained more inorganic contaminants than organic contaminants. Much of the waste encountered appeared to be ash, a material that frequently contains a number of trace metals and little organic matter. Organic contaminants in most municipal landfills are usually the result of disposal of solvents, cleaning fluids and degreasers, gasoline, used motor oil, and other lubricants. Traces of toluene were found in leachate sampled in the dumps and in the leachate seeping out of the dumps. This compound is one of the most common volatile organics in industry and it is virtually impossible to identify the source. The phthalate compounds are frequently used in the manufacture of plastics and it is quite probable that these compounds are the result of degradation of plastic containers in the dumps. The presence of bis (2-ethylhexyl) phthalate in MW 2-3 is problematic because this well is hydraulically upgradient from the dumps.

Physical and chemical analyses of the solid waste indicate that the waste material was not hazardous. Though some of the EP

Toxicity metals were present, they were not present in sufficient quantity to be considered hazardous. None of the waste samples ignited, were reactive or corrosive. Therefore, the physical and chemical characteristics of this material were not a major determinant in the ultimate disposition of the waste.

No data produced by this investigation indicated that the dump materials were toxic or hazardous and there was no evidence of significant damage to the local groundwater. Consequently, economic and environmental considerations supported a recommendation of removal of the dump material from beneath the highway alignment and disposal adjacent to the right-of-way, with environmental monitoring during and after construction.

REFERENCES

A. E. Greenberg, J. J. Connors, and D. Jenkins, "Standard Methods for the Examination of Water and Wastewater", pp. 141-245, 312-331, American Public Health Association, Washington, DC (1980).

Highway Construction in the Lake Bed Deposits, Southwestern Indiana

TERRY R. WEST

Purdue University, Dept. of Earth and Atmospheric Sciences
West Lafayette, Indiana

HGS-87-28

INTRODUCTION

PLEISTOCENE AGED LAKE BED deposits overlie Pennsylvanian bedrock in thirteen or more counties in southwestern Indiana. These lake beds continue to cause problems in strip mining operations of the Pennsylvanian coals in this portion of Indiana.

The lake beds consist of interfingering, unconsolidated sediments resulting from glacial action but generally deposited beyond the glacial ice contact boundary. The beds were deposited in standing water adjacent to upland areas composed primarily of Pennsylvanian bedrock. Outwash, dune sand and loess deposits are directly associated with the lake beds, and in some areas till is also involved. The lacustrine deposits themselves consist of layers of silt and clay plus thin sands, organics and marl. Overall stratigraphy of the system is extremely complex.

Lacustrine clays pose problems for coal strip mines in both the high wall and in the cast over piles. Research performed at Purdue University by Greengold (1981), and Oschman (1984), have examined these problems in coal mining areas of Indiana and Illinois, respectively. In the future, underground coal mining will prevail in Indiana as strip-pable coals are exhausted. Subsidence of the thick deposits of lake bed clays lying above these mines will yield another environmental concern.

In the construction of Interstate 164 to the east of Evansville, similar lake beds were encountered in southwestern Indiana. Review of the exploration and testing data from this highway project provides an opportunity to obtain additional engineering data on these deposits. This information is examined later in this discussion.

OTHER ENGINEERING STUDIES

1981 STUDY - In 1981 a study was completed at Purdue University (Greengold, 1981) involving coal strip mining and lake bed clays. The study area is located near Petersburg, Indiana, (Pike County) approximately 30 miles north of Evansville.

For this coal mine area, topography of the bedrock surface is quite variable. Sub-surface investigations indicated the bedrock elevation varies in excess of 50 feet for a site of about 4 square miles. The land surface of the site prior to strip mining also had a relief of nearly 100 feet. Because of this combination the thickness of the unconsolidated materials varied from 0 to 105 feet.

Unconsolidated materials consist of loess, glacial-lacustrine sands, silts and clays; and glacial till which collectively yield an extreme range in properties.

Stratigraphy is dominated by organic-bearing lacustrine clays and loose, fine to medium, saturated sands. Sands and clays range from small pockets to others greater than 50 feet thick and hundreds of feet across. There are two lake bed deposits on the four-square mile site. The northern unit with an areal extent of about 1/4 of a square mile is Wisconsin in age. The southern unit is of Illinoian age and is about one square mile in extent. Bedrock is the Pennsylvanian aged Dugger Formation, here a gray sandy shale interbedded with yellow and gray sandstone and some limestone. The coal seam being mined is the Indiana Number V seam or Springfield Coal which is about 6 feet thick at this location.

Samples were obtained in borings by standard penetration tests and by Shelby tube sampling. Split spoon samples were taken at 5 foot intervals. Twelve borings were accomplished. Two layers of unconsolidated materials are indicated; the upper layer 18 to 23 feet thick showed N values ranging from 8 and 22 blows per foot. This layer presumably included loess, lake silts, alluvium and glacial till. The lower layer includes glacial till, plus gray lacustrine clays and sands. Blow counts in the lower unit ranged from zero in the loose saturated sands to 91 blows per foot in the very stiff glacial till.

Shear strengths for the gray lacustrine clay ranged from 0.08 to 1.37 tons/ft² with an average of 0.6, based on 15 unconfined compression tests and 16 shear vane tests.

The loess had a range of shear strength from 0.08 to 1.44 tons/ft² and an average of 0.7, based on four unconfined tests and four vane tests. The yellow clays or "lake silts" averaged 0.08 tons/ft² for two shear vane tests of 0.04 and 0.11 tons/ft². In a previous study of the coal mine area, values ranging from 0.42 to 2.03 were obtained for the silty yellow clays. In that earlier study shear strengths of the glacial till ranged from 2.51 to 5.25 tons/ft² with dry unit weights between 94 and 101 lbs/ft³.

In the study by Greengold (1981) the sensitivity of the gray clay averaged 1.7 based on 15 shear vane tests. Values ranged from 1.2 to 2.5. Clays with sensitivities < 2 are considered to be insensitive (Bowles, 1979, p. 377). The loess had an average sensitivity of 1.5 (2 tests) whereas the lake silts averaged 1.6 (2 tests).

Units weights for the loess (in situ or wet density) ranged from 106 to 133 pcf, averaging 124 pcf for four samples. The lake silts (calcareous, pebbly yellow clays) ranged from 120 to 135 pcf, averaging 125 pcf for two samples. The gray lake clays varied from 113 to 144 pcf with an average of 139 pcf for 13 samples.

Water content of the gray clay ranged from 17 to 46% with an average of about 26% for 29 samples. In a general way shear strength decreased with increasing water content but there was not a strong correlation between them.

Again with respect to the gray lacustrine clay the shear strength did not increase with depth (or with increasing overburden pressure). Instead the lowest values of shear strength were found near the base of the gray lacustrine clay. This indicates a potential slope stability problem when strip mining through these materials.

For this strip mine in Pike County, Indiana slope stability in the spoil piles formed by the large drag line is also a concern. The unconsolidated material ranges from 0 to 105 feet thick as stated previous and the rock overburden above the coal seam ranges upward to 160 feet thick. Where soils comprise more than 40% of the total overburden section, spoil pile instabilities arise as rock volume is insufficient to retain the soil. The rock spoil is piled into two parallel rows with the soil placed in the V-notch between them. When the soil comprises more than 40% of the total overburden it overflows the V-notch volume. In such cases, the extra soil must be stripped away prior to the dragline operation and moved to the high wall side of the cut using large trucks.

In the study by Greengold (1981) several recommendations were made to reduce slope stability problems in the highwall. These included: dewatering sands where applicable, benching in thick soils to reduce slope

failures and reorienting the pit to prevent the concentration of soft material in certain sections of the excavation.

CURRENT RESEARCH AT PURDUE UNIVERSITY - At the present time another study is underway involving lacustrine clays and strip mining of coal. The Prides Creek-East Pit, owned by Solar Sources, Inc. is located immediately south of Petersburg, Indiana in Pike County. A thick section of lacustrine clay was encountered by the coal company during strip mining operations near the western boundary of their property.

A research project supported by the Indiana Mining and Minerals Resources Research Institute (IMMRRRI) is being conducted by the engineering geology group, Department of Earth and Atmospheric Sciences, Purdue University.

A boring involving continuous split spoon sampling was made through the lacustrine section in summer, 1986. A stratigraphic section for this boring was developed from the drilling logs and compared to the natural gamma ray for the boring. This was used to supply a signature for the deposit.

In Table 1 results are presented for the laboratory analyses completed to date on the Prides Creek site.

It is clear that the strength of this lacustrine deposit is quite low. These materials occur below a 12 foot thick layer of loess which is located at the surface. Table 1 indicates that the N values range between 3 and 5 blows/ft for the lacustrine section.

The lacustrine clays are reasonably plastic with plasticity indices ranging from 4 to 51. The clay content ranges as high as 52%. The lacustrine soils are designated as ML, CL, CH and OH according to the Unified Soil Classification.

Work is continuing on these soil samples. Unit weights, natural moisture contents, and unconfined compression strength tests are also being determined on samples obtained using Shelby tube sampling. Some of these results are presented in Table 2.

INTERSTATE HIGHWAY I-164 STUDY

Interstate Highway 164 is currently under construction east of Evansville, Indiana. When completed it will extend for about 13 miles connecting Indiana State Road 66 to Interstate 64. With almost a north-south alignment, it will serve as a bypass around the east side of Evansville (Atlas Soils Report, 1985).

I-164 is located close to the north-south boundary between Warrick County on the east and Vanderburgh County on the west. These two counties are immediately north of the Ohio River in southwestern Indiana.

TABLE 1
Soil Sample Results, Prides Creek-East Pit

Description	Approx. Depth (ft)	N value blows/ft	Liquid Limit(%)	Plastic Limit(%)	Plasticity Index	% Silt (>.004mm)	% Clay (>.004mm)	Unified Soil Classification
Gray w/tan mottling slightly clayey silt with organics	6	6	-	18	-	71	25	-
Dark gray soft moist silty clay w/some small calcareous pieces	13	3	88	37	51	43	43	OH
Dark gray soft moist silty clay w/small white calcareous pieces	17	2	50	36	14	72	17	MH
Dark gray moist to wet silty clay w/ a few small calcareous pieces	22	2	51	26	25	76	7	CH
Dark gray wet soft clay to silty clay w/ a few small calcareous pieces	26	4	32	28	4	83	9	ML
Dark gray clayey silt w/ a few small calcareous pieces	30	5	24	17	7	72	23	CL
Dark gray moist to wet clayey silt w/ trace of small calcareous pieces and trace oragnics	30.5	2	33	16	17	48	52	CL
Dark gray moist clay w/ a few small calcareous pieces	33	3	39	30	9	78	11	ML
Dark gray soft silty clay	37	2	35	27	8	77	11	ML

TABLE 2.
Comparison of results, three locations of lacustrine clay deposits.

Study Designation	N value Blows/ft	Shear Strength	Average Shear Strength	w%	w%	PI	PI	Sensitivity		Natural Unit Wt.		Dry Unit Wt.	
		Range T/ft ²	Range T/ft ²	Range	Average	Range	Average	Range	Avg.	Range pcf	Avg. pcf	Range pcf	Avg. pcf
1981 Study	8-22	0.08-1.37	0.6	16-53	26.4	-	-	1.2-2.6	1.7	106-144	129	76-122	103
Current Study	3-5	0.16-0.60	0.4	21-67	41.1	4-51	17	-	-	99-122	114	59-100	82
I-164 Study	7-30	0.07-1.50	0.56	16-50	25.8	7-21	12	-	-	117-132	125	88-109	99

The site lies south of the maximum extent of Illinoian glacial ice in the unglaciated portion of Indiana. Glacial fluvial deposits in the form of lake bed clays and sandy outwash occur in southern Vanderburgh and Warrick Counties. In the extreme southern part of the counties near the Ohio River, alluvial sand prevails as stream deposits and adjacent terraces. Lacustrine deposits are found northward for several miles until residual soils formed by the weathering of Pennsylvanian shales and siltstones prevail. The lacustrine deposits of gray clay and sand are found up to 50 feet thick in eastern Vanderburgh County.

To obtain additional details on the engineering properties of lacustrine clays, a 4.7 mile section of the design plans for I-164 were evaluated. For this section of road, 37 road centerline borings were taken and 36 structure borings were completed. The road borings ranged in depth from 6 to 36 feet deep whereas the structure borings, located at bridge overpass structures, were typically 30 to 80 feet deep. Many of these extended to bedrock.

In this area the upper 10 feet or so was typically a brown silty loam, i.e. loess. This wind deposited silty material is prevalent on the eastern side of the Wabash River in Indiana. The gray silty clay, lacustrine materials occurred below the loess. The gray clays had standard penetration values (N values) ranging from 7 to 30 blows per foot with typical values about 15 to 20. Moisture contents ranged from 16 to 50% with most values around 26%. The plasticity index for the gray clays was about 12, with liquid limits and plastic limits about 32 and 20, respectively. Wet unit weights ranged from 117 to 132 lb/ft³ with natural moisture contents for these samples about 25%.

Unconfined compression tests had been run on 17 soil samples. Shear strengths ranged from .066 to 1.50 tons/ft² with an average of 0.56 tons/ft². Rimac values had been made on the split spoon samples following drilling. For 374 sample tests an average of 0.935 T/ft² was obtained for these tests. Comparing the two answers, 0.935 is 1.67 times greater than the 0.56 value obtained from unconfined compression tests. The Rimac values tend to be high for such comparisons.

Eleven triaxial compression tests had been run on the samples. The cohesion values ranged from .25 to 1.3 tons/ft² and the ϕ angles from 0 to 3.8 degrees.

COMPARISON OF TEST RESULTS

A comparison can be made for the gray lacustrine clays for the three sites discussed above. This is summarized in Table 2.

From this comparison it is suggested that the lacustrine deposits for the current study area are weaker than those of the other two sites. In the current study area the soils are more plastic and contain a larger percentage of clay minerals. The other two sites seem to be more typical of a larger area of lake bed deposits.

Laboratory analysis is continuing on the soils for the current study area. As this work continues a greater amount of information will be obtained for these weak soils of southwestern Indiana.

REFERENCES CITED

- Atlas Soils Inc., 1985, Soils Report for I-164, Project No. I-164-1(2)9, Vanderburgh County, Indiana, STA 840 + 00 to 1000 + 95, for the Indiana Department of Highways.
- Bowles, J.E., 1979, Physical and Geotechnical Properties of Soils, McGraw Hill, Inc.
- Greengold, G.E., 1981, Engineering Geology of An Operating Area Strip Mine, Pike County, Indiana, Unpublished M.S. Thesis, Purdue University.
- Oschman, K.P., 1984, Slope Stability for a Strip Mine Box Cut, Williamson County, Illinois, Unpublished M.S. Thesis, Purdue University.