49th. HIGHWAY GEOLOGY SYMPOSIUM

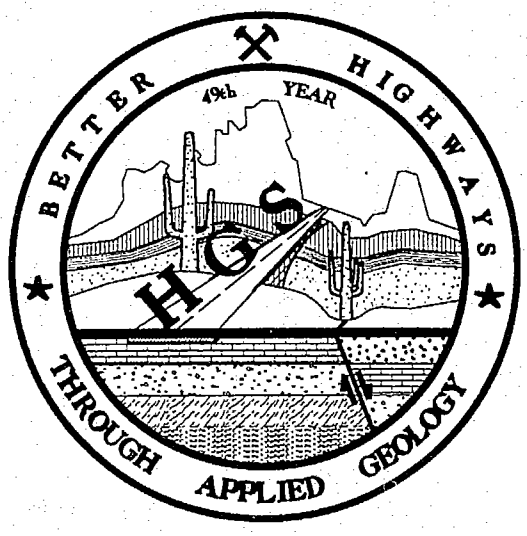
PROCEEDINGS

&

FIELD TRIP GUIDE

Prescott, Arizona
September 10-14, 1998

Compiled By Nicholas M. Priznar



Arizona Department of Transportation,
Materials Group
Arizona Geological Survey
Northern Arizona University, Department of Geology

HIGHWAY GEOLOGY SYMPOSIUM

HISTORY ORGANIZATION AND FUNCTION

Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium (HGS) was organized in 1950, and held its first meeting on February 16, in Richmond, Virginia. From this start, a total of 48 consecutive annual meetings have been held in 31 different states. The meetings were held east of the Mississippi River between 1950 and 1962 with Virginia, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as host states.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona. Since then, it has rotated from east to west almost every year. Following meetings in Texas and Missouri in 1963 and 1964, the annual Symposium moved to different locations as follows:

Year	HGS Location	Year	HGS Location
1965	Lexington, KY	1966	Ames, IA
1967	Lafayette, IN	1968	Morgantown, WV
1969	Urbana, IL	1970	Lawrence, KS
1971	Norman, OK	1972	Old Point Comfort, VA
1973	Sheridan, WY	1974	Raleigh, NC
1975	Coeur d'Alene, ID	1976	Orlando, FL
1977	Rapid City, SD	1978	Annapolis, MD
1979	Portland, OR	1980	Austin, TX
1981	Gatlinburg, TN	1982	Vail, CO
1983	Stone Mountain, GA	1984	San Jose, CA
1985	Clarksville, IN	1986	Helena, MT
1987	Pittsburgh, PA	1988	Park City, UT
1989	Montgomery, AL	1990	Albuquerque, NM
1991	Albany, NY	1992	Fayetteville, AR
1993	Tampa, FL	1994	Portland, OR
1995	Charleston, WV	1996	Cody, WY
1997	Knoxville, TN		

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 to three-year terms, with their elections principally determined by participation in and contribution to the Symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. Officers except for the treasurer may only succeed themselves for one additional term.

A number of standing committees conducts the affairs of the organization. The lack of rigid requirements and the relatively relaxed overall functioning of the organization is what attracts many of the participants.

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

The symposia are generally for two and one-half days, with a day-and-a half for technical papers and a full day for the field trip. The symposium usually begins 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 150 to 200 miles, provide six to eight scheduled stops, and require about eight hours. Occasionally, cultural stops are scheduled around geological and geotechnical points of interest. To cite examples; in Wyoming the group viewed landslides in the Big Horn Mountains, and Florida's trip include a tour of Cape Canaveral and the NASA space installation. The Idaho and South Dakota trips viewed mining activities, while 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 while the Central Mineral Region was visited in Texas.

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 provided stops in the San Francisco Bay area that illustrated the planning, construction and maintenance of transportation systems. In 1985, the one-day trip illustrated new highway construction procedures in the greater Louisville area. The 1986 field trip was through the Rockies on new interstate construction in the Boulder Batholith; the trip highlight was a stop at the Berkeley Pit in Butte, Montana, an open pit copper mine.

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

HGS
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Area Geologist
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1997
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HIGHWAY GEOLOGY SYMPOSIUM

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HIGHWAY GEOLOGY SYMPOSIUM

MEDALLION AWARD WINNERS

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

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

*Deceased

HIGHWAY GEOLOGY SYMPOSIUM

EMERITUS MEMBERS OF THE STEERING COMMITTEE

Emeritus Status is granted by the Steering Committee.

R. F. Baker*
David Bingham
Virgil E. Burgat*
Robert G. Charboneau*
Hugh Chase*
A. C. Dodson*
Walter F. Fredericksen
Joseph Gutierrez
John Lemish
George S. Meadors, Jr.*
Willard McCasland
David Mitchell
W. T. Parrot*
Paul Price*
David L. Royster*
Bill Sherman
Mitchell Smith
Berke Thompson
Burrell Whitlow*
Ed J. Zeigler

*Deceased

HIGHWAY GEOLOGY SYMPOSIUM

FUTURE SCHEDULE

YEAR	ANNUAL	LOCATION	COORDINATORS
1999	50th	**Golden Anniversary**[1] Virginia	Mr. William McKay Virginia DOT (804) 737-7731
2000	51st	Washington	Prof. Robert Holtz Univ. of Washington & Steven Lowell Washington DOT
2001	52nd	Maine	Prof. Dana Humphrey Univ. of Maine
2002	53rd	California	John Duffy CALTRANS (805) 549-3663
2003	54th	Massachusetts	Prof. William Highter Univ. of Massachusetts
2004	55th	Minnesota	Chuck Howe Minnesota NDOT (612) 779-5602
2005	56th	Vermont	Tom Eliassen Vermont DOT (802) 828-2561
2006	57th	British Columbia[2]	Duncan Wyllie Golder Assoc. (604) 298-6623

1. The 50th anniversary of the Highway Geology Symposium will be celebrated in Virginia, which not by coincidence is the location of the first annual symposium.

2. A symposium in Canada will require a revision of the By-Laws.

49th. Highway Geology Symposium

EXHIBITORS

Indoor Exhibitors

AGRA Earth & Environmental David Perterson Phoenix, AZ	**Alliance Environmental, Inc.** Greg Zekoff Mesa, AZ
Arizona Bureau of Mines and Mineral Technology Ken Phillips Phoenix, AZ	**Arizona Geological Society** Cori Hoag Tucson, AZ
B.A.T. Construction, Ltd. Jim La Greca Kamloops, BC	**Drilling Services/Boyles Bros.** Kleigh Hirschi Phoenix, AZ
Brugg Cable Products Robt. Thommen Santa Fe, N.M.	**Central Mine Equipment** *Rick Hutchings* *Earth City, MO*
Crux Subsurface, Inc. Nick Salisbury Spokane, WA	**Foremost Mobil** David Brazee Indianapolix, IN
Geokon, Inc. Michael McDonough Lebanon, NH	**Geomation Inc.** John McMullen Golden, CO
Hilifiker Retaining, Walls, Inc. Beverly Bishop Eureka, CA	**Hi-Tech Rockfall Construction** Howard Ingram Forest Grove, OR
KANE GeoTech, Inc. Wm. Kane Stockton, CA	**Maccaferri Gabions, Inc.** Steve Sullivan Phoenix, AZ

Michael Baker Jr., Inc. Chris Ruppen Beaver, PA	**Mills Machine Co.** Chuck Mills Shawnee, OK
Monotube Pile Corp. Steve Udenlhoven Canton, OH	**Pennsylvania Drilling Co.** Tom Sturges McKees Rocks, PA
P.E. Lamoreaux & Asso., Inc. Brad Stephenson Oak Ridge, TN	**Rio Grande Fence** Brent Terry/Dan Journeaux Johnson City, TX
Roctest, Inc. John Bessette Plattburg, NY	**R.S.T. Instruments Ltd.** Jason Lutty Coquitlam, BC
SIMCO Richard Clarke Osceola, TN	**Slope Indicator Co.** Jon Nagler/Larry Lewis Bothell, WA
Synthetics Industries John Cannon Chattanooga, TN	**Williams Form Engineering** Kevin Heinert Grand Rapids, MI
DBM Contractors, Inc. Tom Armour Federal Way, WA	**Arizona Geological Survey** Larry Fellows Tucson, AZ
Bill Johnson Equipment Company Ginger Johnson Phoenix, AZ	

Outdoor Exhibitors

Alliance Environmental Gregg Zekoff Mesa, AZ	**Drilling Services/Boyles Bros.** Kleigh Hirshci Phoenix, AZ
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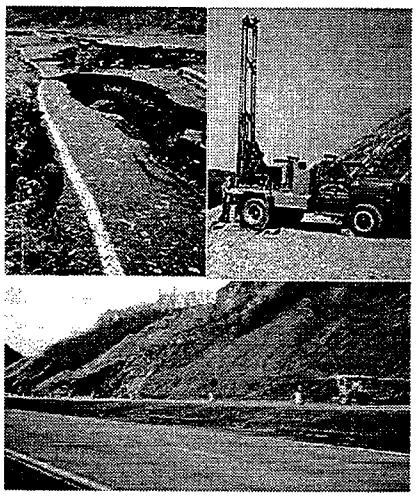

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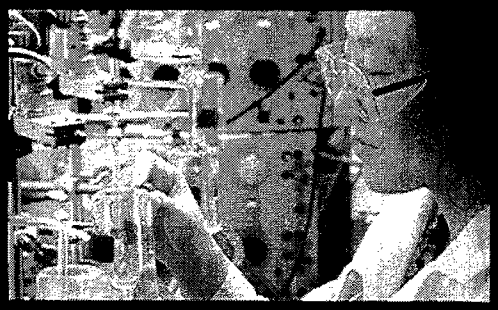

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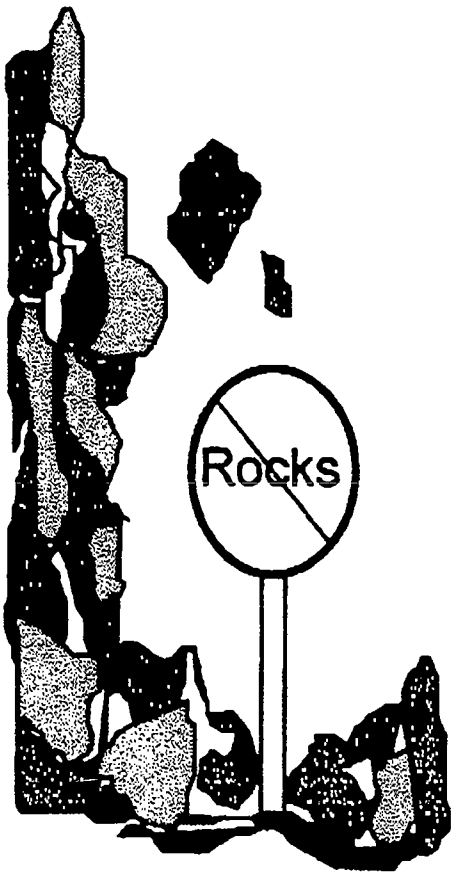

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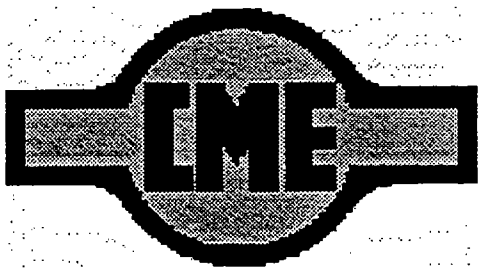

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Larry D. Fellows
Director and State Geologist
Arizona Geological Survey

Welcome to Prescott and the 49th Highway Geology Symposium! Prescott is one of my favorite Arizona towns. The geology here is fascinating and the history is quite significant to the Arizona Geological Survey. In 1863 Arizona Territory was spun off from New Mexico Territory and Prescott became the capital. This was an active mining area at that time and Federal troops were brought in to protect the miners from the Indians. The Prescott area was connected to the southern part of the Territory by a primitive system of trails and crude roads. As you noticed on your way here, some of the highways are still being improved, primarily because of the substantial population growth in Phoenix, Prescott, and Prescott Valley.

In 1880 the Territorial Legislature, which met right here in Prescott, established the Office of the Territorial Geologist, the parent organization of the Arizona Geological Survey. The Office was intermittently occupied until Statehood in 1912. Three years after statehood the State Legislature created the Arizona Bureau of Mines, which continued, essentially unchanged, functions of the Territorial Geologist.

Geologically, there are two Arizonas: Colorado Plateau Arizona and the Basin and Range Arizona. The northern part of the State is within the Colorado Plateau Province, which also includes parts of Utah, Colorado, and New Mexico (Four Corners area). Sedimentary rocks of Paleozoic age crop out in much of the Arizona portion of the Plateau. These rocks are nearly flat lying and have not been substantially folded or faulted. When you look into the Grand Canyon, as many of you will Monday, you'll be looking into a gigantic gash cut into the Plateau by the Colorado River.
The southern part of the State, extending from just north of Phoenix to the Mexico border, is within the Basin and Range province, which includes portions of Mexico and the states of Texas, New Mexico, California, Nevada, Utah, Wyoming, Oregon, and Idaho. Rocks in this province have been cut by a series of normal faults, movement along which resulted in alternating mountain and basin blocks.
The boundary between the Basin and Range and the Colorado Plateau is gradational across a zone that is several tens of miles wide. Many geologists refer to this area as the Transition Zone. Prescott is within the Transition Zone, as are all but one of tomorrow's fieldtrip stops

Tomorrow we will see some exceptional Transition Zone geology. Stop one is on relatively young, flat lying basalt that is unconformable on metamorphosed, vertical Precambrian strata. Then we will observe the fault-bounded Verde basin, which, from about 8 million to 2 million years ago, was a shallow lake that partially filled the valley with sediment. Today the Verde River and its tributaries are eroding the soft lakebed deposits. Next, we drive up out of the Verde Valley onto the Colorado Plateau and stop at the head of Oak Creek Canyon, one of the most scenic parts of Arizona. Paleozoic-age redbeds, sandstones, shales, and limestones, overlain by Tertiary basalt, are exposed in the canyon walls. After driving down Oak Creek Canyon to Sedona, in the heart of scenic "Red Rock Country," and on to Cottonwood, we proceed to the village of Jerome, which is on the side of a mountain that overlooks the Verde Valley. Jerome developed around a world-class deposit of copper, gold, and silver. We'll make two stops at the mine, time permitting, before we return to Prescott. It will be a full day of fascinating geology, history, and scenery.

It has been my pleasure to have worked with Nick Priznar (Arizona Department of Transportation) and others in preparing for this meeting. I hope your participation is memorable.

TABLE OF CONTENTS

Page

History, Organization and Function.....i

HGS Steering Committee Officers.....ii

HGS Steering Committee Membership List.....iii

Emeritus Members of the HGS Steering Committee.....iv

HGS Medallion Award Winners.....viii

Future Highway Geology Symposium Schedule.....ix

Exhibitors.....x

Sponsors.....xii

Welcoming Remarks To HGS: Larry Fellows, Director, Arizona Geological Survey, Tucson, Arizona.....xvii

PROCEEDINGS PAPERS

Technical Session I

Rock Slopes/Rockfall/Tunnels

Design Principles and Methodology for Excavations in Geologic Media
Richard Z.T. Bieniawski, Ph.D., Penn. State University.....1

Final Rock Slope Blasting Techniques: Considerations in Design and Execution
D. Scott Scovira, Michael F. Barber, Energetic Solutions, Dallas, Texas.....14

Case Examples for Rock Stability on Transportation Projects
C.O. Brawner, C.O. Brawner Engineering Ltd., W. Vancouver, BC.....26

Rock Slopes and Bridge Foundations in a Foliated Rock Mass along portions of
State Route 69 near Mayer, Arizona
David E. Peterson, P.G., and Keith H. Dahlen, P.E., AGRA Earth & Environmental, Inc., Phoenix, AZ.....39

Site Characterization and Slope Stability Evaluation for the San Francisco-Oakland Bay Bridge
Timothy J. Beck, California DOT, Sacramento, CA
William F. Kane, *KANE* GeoTech, Inc., Stockton, CA.....52

A Large Wedge Failure Along Interstate 40 at the North Carolina-Tennessee State Line
F.R. Glass, North Carolina DOT, Asheville, TN.....65

Performance Review of Rockfall Mitigation Measures
John D. Duffy, Eng. Geologist, CALTRANS, San Luis Obispo, CA.....76

Rockfall Control Fence, Unaka Springs Road, Unicoi County, TN
Harry Moore, Tennessee DOT, Knoxville, TN.....84

The Performance of Rockfall Mitigation Measures in New York State
Michael P. Vierling, Eng. Geologist, NYDOT, Albany, NY.....94

Tunnel to the 21st CenturyJ.L. Gattis, Ph.D., P.E., and Sam I. Thornton, Ph.D., P.E.,
Mack-Blackwell Transportation Center, Univ. of Arkansas, Fayetteville, AK.....104

Page

(Continue)

Technical Session I
ROCK SLOPES/ROCKFALL/TUNNELS

Tunnel Enlargement and Rehabilitation
Richard Humphries, Golder Associates Inc., Atlanta, GA 113

High Altitude Slope Reconstruction and Restoration on Colorado State Highway 82, at Independence Pass, A Continuing Case Study
Jonathan L. White, Colorado Geological Survey, Denver, CO 124

400,000 Cubic Yard Slope Failure on I-26
Jerome Beard, P.E., NCDOT, Raleigh, NC 138

Technical Session II
Landslides/Retaining Walls/Foundations

Wolf Mountain Landslide Milepost 136.57 Snake River Canyon, Teton County, Wyoming
Mark Falk, P.E., P.G., Wyoming DOT, Cheyenne, WY
G. Michael Hager, P.G., Wyoming DOT, Cheyenne, WY 145

Micropile and Tieback Reinforced Walls for Landslide Stabilization, Snake River Canyon, Wyoming
John Turner, Ph.D., P.E., University of Wyoming, Laramie, WY
Robert Hasenkamp, P.E., Wyoming DOT, Laramie, WY
John Wolosick, Hayward-Baker, Roswell, GA 156

Three Interesting Drill Shaft Foundations Completed in Arizona.
Robert E. Schock, Case Foundation Company, Roselle, IL 169

Statnamic and Static Lateral Load Analysis of a Battered Pile Group vs. A Drilled Shaft Group
Michael J. Wright, Mississippi DOT, Jackson, MS 181

Testing and Application of Flexible Wire Rope Netting Barriers to Retain Mud & Debris Flow
Robert Thommen, V.P., Brugg Cable Products, Inc., Santa Fe, NM
John D. Duffy, Eng. Geologist, CALTRANS, San Luis Obispo, CA 195

Soil Nailing: Geology and Construction Considerations as They Relate to Field Inspection
Barry D. Siel, P.E., FHWA, Lakewood, CO
Harry L. Siebert, Consultant in Applied Earth Science, Indian Hills, CO 203

Call of the Canyon Mechanically Stabilized Earth Structure with Simulated Rock Facing
O.R. MacIntosh, P.E., MacIntosh Engineering & Development Co., Auburn, CA
Douglas E. Alexander, P.E., ADOT, Phoenix, AZ 209

Compaction Grouting of a Deep Embankment Fill Along Interstate 15, Virgin River Gorge, Arizona
Gary E. Taylor, Hayward Baker Inc., Santa Paula, CA
Glen A. Gorski, Hayward Baker Inc., Santa Paula, CA
Douglas E. Alexander, P.E., ADOT, Phoenix, AZ 221

Paleoseismology and Seismic Hazard Assessment in Arizona
Philip A. Pearthree, Ph.D., Arizona Geological Survey, Tucson, AZ 230

Former Manufactured Gas Plants (FMGP); Nemesis of Transportation Planning
Allen W. Hatheway, Ph.D., University of Missouri-Rolla, MO 243

Structural Controls of Devils Kitchen Sinkhole; Sedona, Arizona
Paul A. Lindberg, Geological Consultant, Sedona, AZ 256

Geologic Investigation and Construction of the Spring River Bridge
Robert W. Henthorn, Regional Geologist, Bureau of Materials and Research, Kansas DOT, Topeka, KS 266

Page

continue

Technical Session II
Landslides/Retaining Walls/Foundations

Roadway Construction Considerations through a Young Volcanic Center,
at the south end of Big Sandy Valley, near Wikieup, Arizona
David E. Peterson, AGRA Earth and Environmental, Phoenix, AZ
Lorena M. Rosberg, AGRA Earth and Environmental, Phoenix, AZ
Ronald E. Price P.E., URS Greiner, Inc., Phoenix, AZ 283

Technical Session III
Karst

Caves Discovered During The Construction of a Motorway Over Slovene Karst
Dr. Tadej Slabe, Scientific Research Center of the Slovenian Academy of Sciences and Arts 296

Geotechnical Investigation in Karst: Bent Rock Versus Flat Rock
Joseph A. Fischer, Geoscience Services, Bernardsville, NJ
Joseph J. Fischer, Geoscience Services, Bernardsville, NJ
Richard S. Ottoson, Geoscience Services, Bernardsville, NJ. 307

Assessment, Prediction and Remediation of Karst Conditions on I-70, Frederick, Maryland
Richard C. Benson, Technos, Inc., Miami, FL
Ronald D. Kaufmann, Technos, Inc., Miami, FL
Lynn B. Yuhr, Technos, Inc., Miami, FL
David Martin, Maryland State Highway Administration, MD 313

Ground Penetrating Radar: An Effective Tool for Locating Dry Caves Along a Portion of
State Route 69 Near Mayer, Yavapai County, Arizona
Kenneth M. Euge R.G.(Principal Geologist), Geological Consultants Inc., Phoenix, AZ
Kerry Hennon, R.Gp., Geological Consultants Inc., Phoenix, AZ
Jason C. Williams, G.I.T., Geological Consultants Inc., Phoenix, AZ
Nick Priznar Geologist, ADOT, Phoenix, AZ 326

Design Considerations for Bridge Foundations in Karst Terrain
Bashar S. Qubain, Valley Forge Laboratories, Inc. Devon, PA
Eric J. Seksinsky, Valley Forge Laboratories, Inc. Devon, PA
Jianchao Li, Valley Forge Laboratories, Inc. Devon, PA ..... .339

Technical Session IV
Aggregates/ Pavement/ Earthwork

Prospecting for Local Material Sources
W.R. Meier, Jr., Ph.D., P.E., Western Technologies Inc., Phoenix, AZ
John C. Rosner, Ph.D., P.E., Western Technologies Inc., Phoenix, AZ 350

Sequential Deterioration of Concrete, Congress Street Bridge, Boston, MA
Steve Stokowski, Stone Products Consultants, Ashland, MA 359

How to Avoid Excessive Expansion of Concrete Due to Alkali-Aggregate Reaction
Bryant Mather, Director, Structures Laboratory, Waterways Experiment Station,
Department of the Army, Corps of Engineers, Vicksburg, MS 365

Runaway Truck Ramp Testing Program
Dennis Duffy, Ph.D., Arizona State University, Tempe, AZ
J.P. Zaniewski, Virginia West University, Morgantown, WV 375

Evaluation of Crushed Stone Used for High Traffic, Bituminous Pavement Wearing-Courses in Indiana
Dr. T.R. West, Purdue University, West Lafayette, IN 388

The Effect of Bedrock Depth on Pavement Response
Dar-Hao Chen, Ph.D., P.E., Texas DOT, Austin, TX
John Bilyeu, E.I.T., Texas DOT, Austin, TX
Mike Murphy, P.E., Texas DOT, Austin, TX 399

Page

(Continue)

Technical Session IV
Aggregates/ Pavement/ Earthwork

Influence of Bedrock Type and Depth on Highway Deflections
Jeffrey G. Paine, Ph.D., University of Texas, Austin, TX
Michael R. Murphy, P.E., Texas DOT, Austin, TX 408

Percolation Theory Approach to Estimating Earthwork Factors in Weathered Granites
Michael L. Rucker, AGRA Earth & Environmental, Inc., Phoenix, AZ 422

Recycled Aggregate in Road Construction - An Economic Assessment
David Wilburn, Thomas Goonan, US Geological Survey, Denver, CO 432

Crushed Concrete Substitution for Construction Aggregates - A Materials flow Analysis
Thomas Kelly, US Geological Survey, Denver, CO 444

The Ultimate Roadside Geology: Geological Heritage Sites
Terry S. Maley, US Bureau of Land Management, Boise, ID. 456

Alternate Presentation

Current Rockfall Mitigation Research in Japan
Duncan C. Wyllie, Golder Associates Ltd., Vancouver, Canada ..... (Discussion)

Geology and Environmental Issues within the Grand Canyon
Dr. Stanley Beus, *Professor Emeritus*, Northern Arizona University, Flagstaff, AZ ..... (Discussion)

FIELD TRIP GUIDE
Highway Geology and Road Log Through Arizona Transition Zone
(Prescott Region, Black Canyon Hwy., Verde Valley, and Redrock Country)
Larry D. Fellows, Arizona Geological Survey, Tucson, AZ
Nicholas M. Priznar, Arizona Department of Transportation, Phoenix, AZ
J. Dale Nations, Northern Arizona University, Flagstaff, AZ
Paul A. Lindberg, Consultant Geologist, Sedona, AZ
Beverly A. Everson, Prescott National Forest, Prescott, AZ. ..... (Attached)

Appendix A: Cumulative Index of HGS Proceedings ..... (Attached)

Appendix B: HGS Proceedings Availability List ..... (Attached)

Geologic Map of Arizona, Map 26, 1988
Stephen J. Reynolds, Arizona Geological Survey ..... (In Pocket)

Grand Canyon of Arizona (Geologic Column)
Dr. Stanley Beus, *Professor Emeritus*, Northern Arizona University, Flagstaff, AZ ..... (Back Cover)

DESIGN PRINCIPLES and METHODOLOGY for EXCAVATIONS in GEOLOGIC MEDIA

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Abstract

A good designer needs not only knowledge for designing (technical know-how) but also must have knowledge about designing (appropriate principles and methodology). In the fields of engineering geology and rock mechanics only limited attention has been paid to the design process because design of structures in rock masses presents unique challenges to the designers as a result of the uncertainties inherent in characterization of geologic media. Nevertheless, in recent years, major attention has focused on interaction of geology and design engineering because of the needs of such critical projects as very large highway tunnels and nuclear waste disposal repositories.

The proposed paper discusses the principles and methodologies of engineering design process directed to integrating geologic site characterization activities with design, construction and performance on structures in rock masses. The design process is seen as the use of specific design principles, including geologic uncertainties, within an integrating design methodology. In this respect, it is argued that a design methodology is not just a sequence of flow charts for step-by-step design. A design methodology must incorporate appropriate design principles which can be used to guide decision making for geologic characterization and to evaluate the best engineering design which fulfills the perceived objectives.

A rock mass classification method, known as the Rock Mass Rating (RMR) System, is described demonstrating an approach for integrating geological information with design parameters appropriate for highway tunnel projects.

INTRODUCTION

Designing structures to be situated in geologic media, such as rock formations, is different from designing a conventional structure - a building or a bridge. For example, in a typical engineering design of a structure, the external loads to be applied are first determined and a material is then prescribed, following which the structural geometry is selected. In dealing with geologic media, the designer is faced with complex rock masses, the specific *in situ* properties of which cannot be prescribed.

Unlike other engineering materials, geologic media - such as soil and rock masses - present the designers with unique problems. First of all, rock is a complex material, varying widely in its properties, and in most highway engineering situations not one but a number of rock types will be present. Furthermore, a choice of rock materials is only available if there is a choice of alternate sites for a given project, although it may be possible, to some extent, to reinforce the rock surrounding the excavation. Most of all, however, the designer is confronted with rock as an assembly of blocks of rock material separated by various types of geologic discontinuities, such as fractures and faults. This assemblage constitutes a rock mass, including both a rock material and also geologic features.

To be fully successful, design of structures in geologic media depends critically on the effective interaction of geologic site characterization (which provides input data for engineering parameters) with engineering design process (which incorporates these parameters in design specifications) for the purpose of achieving a safe and economic construction.

DESIGN METHODOLOGY FOR HIGHWAY ENGINEERING

A good engineering designer needs not only knowledge *for* designing (technical knowledge that is used to generate alternative design solutions) but also must have knowledge *about* designing (appropriate principles and systematic methodology to follow). After all, concepts such as "design for manufacture" or "concurrent engineering" are widely used in the manufacturing industry. Yet, in the field of ground engineering, only limited attention has been paid to the design process. Nevertheless, the importance of the design methodology in tunneling has been widely recognized, particularly by the International Tunneling Association.

However, a design methodology is currently available as a systematic decision-making process directed at improving design of structures in rock. Known as the Systems Design Methodology (SDM) for rock engineering, it was developed specifically for excavations in geologic media (Bieniawski, 1992).

Design Principles

A comprehensive design methodology is not just a sequence of flow charts for step-by-step design. To be comprehensive, a design methodology must incorporate design principles which can be used to evaluate designs and to select the optimum one fulfilling the perceived objectives. To obtain a better performance, one requires correct design principles and methodologies to guide

decision-making in design. After all, there are unacceptable design solutions as well as good design solutions, so there must be a fundamental set of principles determining good design practice. Without them, design would be a mysterious creative process; with them it can be a rational and systematic activity. A design methodology must indeed recommend an order of design stages but these must be so structured as to assist in effective decision making and promote design innovation in accordance with the appropriate design principles.

Suh (1990) of MIT developed an axiomatic approach to mechanical engineering design, identifying design axioms which constitute the basic principles for analysis and decision-making, and help the creative process of the design activity. In essence, Suh proposed just two principles of design, each pertinent to its own domain. In the functional domain, we ask: "what do we want to achieve"? In the physical domain, we must provide the actual design solution by answering: "how do we want to achieve it"? The significance of this approach is that it provides two specific principles for distinguishing between a poor design and a good design. Suh's work paved the way for proposing further design principles, specifically for rock engineering, and incorporating them all in a specific design methodology.

There are six principles of design recommended for rock engineering (Bieniawski, 1992), as the principles for developing, evaluating and optimizing alternative designs. Thus, while Suh's two principles are considered necessary, they are not sufficient for rock engineering design because the behavior of rock masses is governed by the geologic environment which imposes special constraints not found in other branches of engineering. These principles are:

#1 *Design Objectives Principle:* Clear design objectives and independent functional requirements completely characterize problem definition for a specific need (Suh, 1990).

#2 *Minimum Uncertainty Principle:* The best design is one which poses the least uncertainty concerning geologic conditions; the geotechnical data obtained from site characterization must be sufficient and traceable to the design objectives as well as to the design solution.

#3 *Simplicity Principle:* The complexity of any design solution can be minimized by subdividing the system design into components and creating the fewest number of these components forming a part of the overall design solution (Suh, 1990).

#4 State-of-the-Art Principle: The best design maximizes technology transfer of the state-of-the-art research and the best industrial practice.

#5 Optimization Principle: The best design is the optimal design which is evolved from quantitative evaluation of alternative designs as well as on quality assurance and cost effectiveness considerations.

#6 Constructability Principle: The best design facilitates the most efficient construction of a rock engineering structure by selecting the most appropriate construction method and sequence, together with a fair construction contract.

Some additional explanations are helpful when working with the above design principles, as follows:

- Independence Principle. Proper problem definition is the most important in design and Design Principle #1 is directed to that purpose. Since the designer can arbitrarily define the functional requirements to meet a certain perceived need, an acceptable set of functional requirements is not necessarily unique. Moreover, corresponding to a set of functional requirements there can be many design solutions. This then provides ample scope for creativity and produces design winners and losers.

- Minimum Uncertainty Principle. This principle was proposed for rock engineering because rock masses cannot be fully characterized for engineering design in a manner that steel or concrete can. Rock masses are complex geologic structures governed by large scale geologic discontinuities and are difficult to test as a full scale prototype. Accordingly, extrapolation of data from small-scale laboratory samples to large-scale in situ features will always involve a degree of uncertainty. In fact, asking: "When are the needs for site characterization met?" and "How much information is enough?" have no consensus of answers in the rock engineering community.

- Simplicity Principle. Once Principles #1 and #2 are satisfied, it should be remembered that the output of the design process is in the form of drawings, specifications, and other relevant knowledge required to create the physical entity. Thus, the best design solution should be as simple as possible, so the design output can be conveyed with minimal effort. Thus, the essence of Principle #3 is: "the simpler, the better."

- State-of-the-Art Principle. In spite of extensive research performed in the field of rock engineering since the First International Congress on Rock Mechanics held in Lisbon in 1966, innovation in rock engineering design has not proceeded as rapidly as in other engineering fields. This is mainly due to the

designers reacting cautiously to change and being reluctant to introduce new products and approaches until they have been proven elsewhere, or simply not being aware of the latest developments. Great strides which have been made in rock mechanics research and in the tunneling industry worldwide must find a way to rock engineering practice by innovative designs featuring state-of-the-art technology.

• Optimization Principle. A good design is one achieved by the use of optimization techniques resulting in an innovative design product. In essence, optimal design is viewed as applying the well known optimization techniques to engineering design. Optimization is crucial in design because most engineering problems do not have a unique solution. Reconsideration of the solution may be necessary in an attempt to approach a feasible compromise between the often conflicting requirements and resources.

• Constructability Principle. In rock engineering design, we can envision three domains: functional domain, physical domain and construction domain. Each of these domains is defined by its own multiparameters or multivariables. During the design stage, the functional requirements (FRs) must be satisfied by choosing a proper set of design components (DCs), whereas during the construction phase, the DCs must be satisfied by selecting an optimum set of construction procedures (CPs). Effective design for constructability requires the optimization of the relationships among the functional, design and construction domains; so, there is a relationship between the functional requirements and the construction procedures.

The *Constructability Principle* is a unique innovation in rock engineering. It is already recognized in mechanical engineering that the concept of "design for manufacturability" has led to highly streamlined production processes. The construction principle deserves a special place in the field of highway engineering, particularly that in so many tunneling and road projects, design and construction are performed by different companies, often lacking integration of the overall effort. Because a selected construction procedure (CP) cannot be fully tested in advance, events such as a tunnel boring machine being immobilized in a drift are not unusual.

Rock mass classifications are particularly relevant to the constructability principle. For example, the tunnel support guidelines inherent in the RMR system include specifications for excavation advance rates appropriate for various rock masses.

Design Methodology

A design methodology for rock engineering can benefit from a structured process featuring a number of design stages but one that would not constitute a 'straight jacket'; rather it should be a flexible framework adaptable to the problem at hand. One should thus visualize design methodology as a checklist (not unlike the one used by pilots before taking off) or a road-map guiding the designer to fulfillment of the problem objectives by evolving the best design option. It is thus a sequence of steps or activities within which a design can unfold logically but it must incorporate specific design principles. They serve to evaluate design alternatives and to select the optimum design solution fulfilling the design and performance objectives. The design methodology serves as a useful reference of where we are, where we ought to be, and what the next step should be within the overall work plan. Accordingly, an effective design methodology for geologic media can include elements of a systematic design process and it can also incorporate the use of engineering heuristics ("rules of thumb").

Figure 1 presents the details of the advocated Systems Design Methodology for rock engineering. This methodology should be seen as a systematic decision-making process aimed to satisfy the perceived needs, identified by independent functional requirements. Effective design solutions are represented by design components which meet the corresponding functional requirements and facilitate selection of efficient construction procedures. The design principles ensure that a good design is produced and offer a basis for developing, comparing, evaluating, and optimizing designs. Team-work and integration are emphasized in all these activities.

THE ROLE OF HIGHWAY GEOLOGISTS

It is clear from the above discussion that highway geologists play the leading role in site characterization for engineering design. This task is particularly difficult because it is a fact of life design engineers are not always specific enough as to their requirements, or the time and money constraints do not always permit a co-ordinated geologist-engineer effort!

In this respect, engineering-geological rock mass classifications have evolved as indispensable aids in highway projects and both geologists and engineers must understand the potential and limitations of the various classification systems. Alas, the debate is still on "when 'we have enough information' is enough for engineering site characterization?"

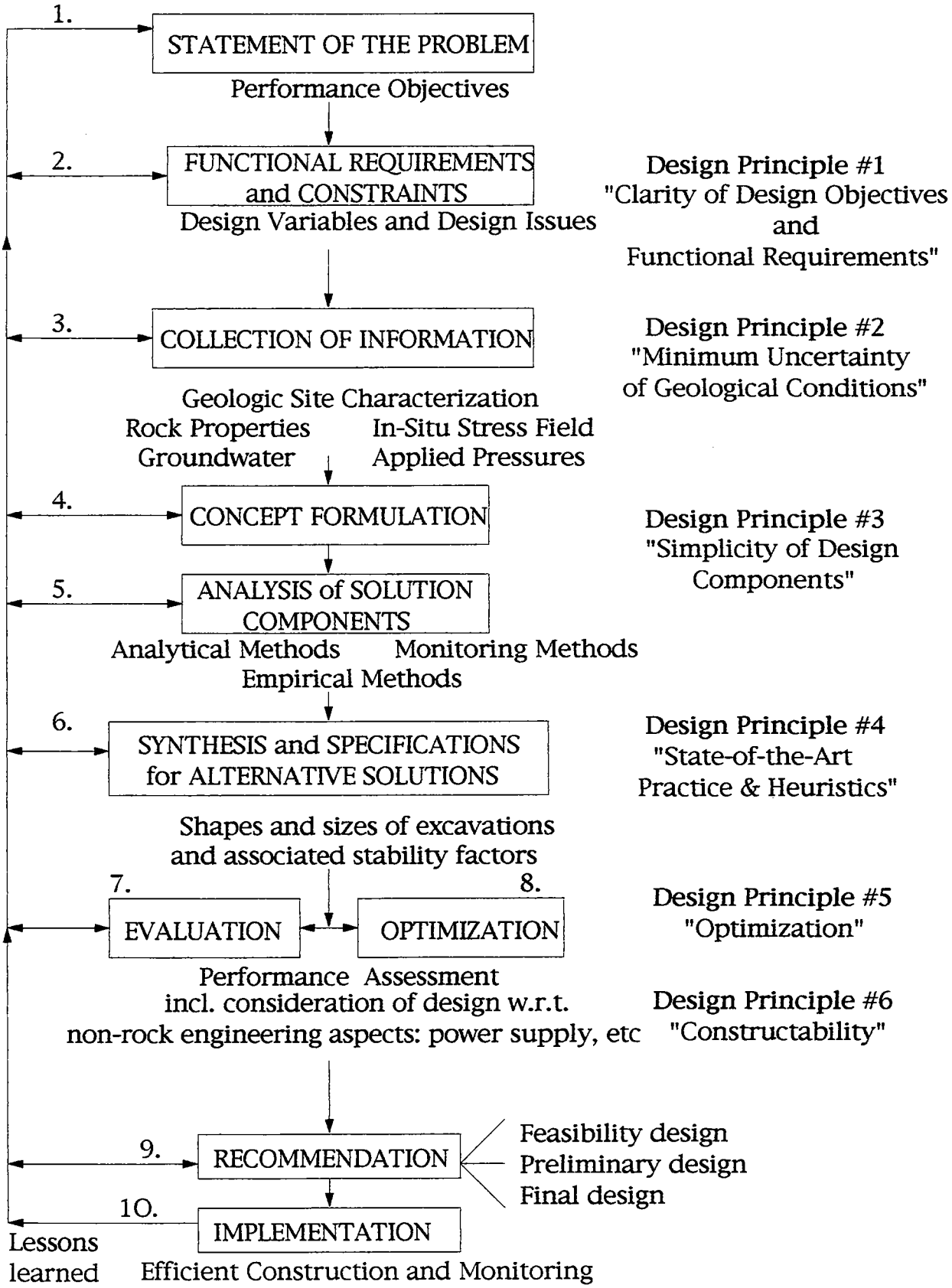

Figure 1 Systems Design Methodology for rock engineering, including the use of design principles (Bieniawski, 1992)

Engineering-Geological Rock Mass Classifications

There are three main benefits of rock mass classifications: (i) improving the quality of site investigations by calling for the minimum input data as classification parameters; (ii) providing quantitative information for design purposes; and (iii) enabling better professional judgment and more effective communication on a highway project. Specifically, the objectives of rock mass classifications are to:

1. Identify the most significant geologic parameters influencing the behavior of a rock mass.

2. Divide a particular rock mass formation into groups of similar behavior, that is, rock mass classes of varying quality.

3. Provide a basis for understanding the characteristics of each rock mass class.

4. Relate the experience of rock conditions at one site to the conditions and experience encountered at others.

5. Derive quantitative data and guidelines for engineering design.

6. Provide common basis for communication between geologists and engineers.

The Rock Mass Rating (RMR) System

Of the many rock mass classification systems in existence, six are best known (see: Bieniawski, 1989): the rock load classification of Terzaghi (1946), the stand-up time classification of Lauffer (1958), the RQD (rock quality designation) of Deere et al (1967), the RSR system of Wickham et al (1972), the Rock Mass Rating system of Bieniawski (1973) and the Q-system of Barton et al (1974). Using the RMR system as an example, here is how a rock mass classification system is used for site characterization on a highway project.

There are six parameters in the RMR system to classify a rock mass:

1. Uniaxial compressive strength of rock material (σ_c).
2. Rock [core] quality designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

Table 1 Rock Mass Rating System (After Bieniawski 1989).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1. - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)						
Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS						
Rating		100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number		I	II	III	IV	V
Description		Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES						
Class number		I	II	III	IV	V
Average stand-up time		20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)		> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)		> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions					
Discontinuity length (persistence)	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m
Rating	6	4	2	1	0
Separation (aperture)	None	< 0.1 mm	0.1 - 1.0 mm	1 - 5 mm	> 5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Ratings	6	5	3	1	0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**			
Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very favourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.
** Modified after Wickham et al (1972).

To apply the RMR system, the rock formation is divided into a number of structural regions such that certain features are more or less uniform within each region. Although rock masses are discontinuous in nature, they may nevertheless be uniform in regions when, for example, the type of rock or discontinuity spacings are the same throughout the region. After the structural regions have been identified, the classification parameters for each structural region are determined from geologic data measured in the field and entered onto the standard RMR input data sheet (US Army Corps of Engineers, 1990).

The RMR System is presented in Table 1. In Section A of Table 1, five parameters are grouped into five ranges of values. Since the parameters are not equally important for the overall classification of a rock mass, importance ratings are allocated to the different value ranges of the parameters, a higher rating indicating better rock mass conditions. The importance ratings are assigned to each parameter according to Table 1. In this respect, the average typical conditions are evaluated for each discontinuity set and the ratings are interpolated.

After the importance ratings of the classification parameters are established, the ratings for the five parameters of Section A in Table 1 are summed to yield the Basic RMR for the structural region under consideration.

The next step is to include the sixth parameter, namely, the influence of strike and dip orientations of discontinuities by adjusting the Basic RMR according to Section B of Table 1. This step is treated separately because the influence of discontinuity orientations depends on the engineering application, such as a tunnel, a slope or a foundation. This parameter reflects the significance of the various discontinuity sets present in a rock mass. The main set, designated usually as no.1, controls the stability of an excavation (e.g. the set whose strike is parallel to the tunnel axis). The summed up ratings of the classification parameters for this discontinuity set will constitute the overall RMR. On the other hand, when no one discontinuity set is dominant, the ratings from each discontinuity set are averaged for the individual classification parameter.

Next, the rock mass as a whole is classified according to Section C of Table 1 which groups the final RMRs into five rock mass classes. Finally, Section D of Table 1 gives the practical meaning of each rock mass class by relating it to specific engineering problems.

Note that for rock slope stability applications, the cohesion and friction estimates in Section D are of particular interest.

THE ROLE OF HIGHWAY ENGINEERS

The engineering output from the RMR System includes the overall rock mass quality as well as guidelines for tunnel support and estimates of the stand-up time of an unsupported tunnel span and of such rock mass properties as strength and deformation. Also, a relationship exists between the standard RMR (for drill-and-blast excavations) and the RMR for TBM (tunnel boring machine) excavated tunnels, as developed by Alber (1993):

$$RMR_{TBM} = 0.8\ RMR + 20.$$

The actual values of RMR are very important, and not just the rock classes, because the ratings are used for estimating the deformation modulus and the rock mass strength as follows* :

In situ Modulus of Deformation: $E_M(GPa) = 10^{\frac{RMR-10}{40}}$ for RMR > 20.

When RMR < 20, $E_M(GPa) = \sqrt{\frac{\sigma_c}{100}}\,10^{\frac{RMR-10}{40}}$

Hoek-Brown Rock Mass Strength Criterion

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \sqrt{m\frac{\sigma_3}{\sigma_c} + s}$$

where $m/m_i = e^{\frac{RMR-100}{28}}$

m_i=f(rock type), e.g. sandstone, m_i=15-17

and $s = e^{\frac{RMR-100}{9}}$

Yudhbir-Bieniawski Rock Mass Strength Criterion

$$\frac{\sigma_1}{\sigma_c} = A + B\left(\frac{\sigma_3}{\sigma_c}\right)^{0.75}$$

where $A = e^{\frac{RMR-100}{13}}$ and B=f(rock type) e.g. sandstone, B=4.5

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* where σ_1 is the major principal stress at failure, σ_3 is the applied minor principal stress, σ_c is the uniaxial compressive strength of the rock material, and *m* is a constant dependent on the properties of the rock mass (for rock material, m=m_i) while s is a constant dependent of the degree of fracturing of the rock mass (s=1 for rock material).

Next, knowing a mass strength criterion and after determining stress distributions around a tunnel or a slope, the zones which need to be reinforeced are delineated and appropriate support is selected and monitored.

SOME THOUGHTS FOR THE FUTURE

During the 1970s, highway tunnel engineers used rock mass classifications as *the main design method.* For practical purposes, analytical methods were not reliable enough, mainly because of uncertainties with the input data. In the 1980s, rock mass classifications graduated to providing an estimate of rock mass properties and rock reinforcement requirements could be assessed analytically, supplemented by rock mass classification predictions, and verified by measurements during construction.

The 1990s saw "pendulum behavior" in the applications of rock mass classifications: either the rating methods were not used at all, as in the New Austrian Tunneling Method, or they were used extensively and decisively, as in the Q-system. The RMR and the Q systems have become entrenched throughout the world as the most utilized classification systems and the correlation between the two of them is well substantiated. However, many tunneling projects have resulted in lengthy arbitrations and costly litigations. In fact, in the last three years alone, of the six major tunneling projects that the author was involved with (four in the USA, two in Europe), all of them involved disputes where rock mass classifications were an issue: five projects featured changed rock conditions during construction and one was related to design procedures.

Based on this experience, these observations are submitted: (1) in site investigations, rock mass classifications are indispensable for quantifying geological descriptions, and there is hardly any controversy about the classification procedures; (2) during the design phase, there is a controversy among tunnel designers as to how highway tunnel support requirements should be established. Some use only descriptive classification methods, as in NATM, some use only rating methods, such as the Q-system, others use the RMR classification to assess rock mass properties and rely more on analytical methods; and (3) during construction, numerical rock mass classifications (i.e. rating methods) are absolutely essential because they become decisive when comparing predicted rock mass conditions based on site investigation, with encountered rock mass conditions during construction. Measurements of deformations during tunnel construction are, of course, also very valuable but they are not sufficient.

CONCLUSION

Engineering-geological rock mass classifications are most effective if used within an overall engineering design process. This leads to alternative designs and comparison of the different design methods. The time has come in highway engineering that a specific design process - that is, a design methodology featuring design principles and based on design science and practice - is recognized as a viable process for effective design, based on clear objectives, alternative designs, and the concept of design-in-constructability.

Furthermore, rock mass classifications on their own should only be used for preliminary, planning purposes and not for final highway tunnel support. This is so because for preliminary design and planning purposes, rock mass classifications are excellent; they quantify rock mass conditions, enable an estimate of rock mass properties, provide the reference basis for expected rock conditions, and enhance effective communication between geologists and engineers. But the designers should not use one method alone for final tunnel support design, because new support types (e.g. steelcrete, swellex bolts) and new excavation methods (different TBM procedures) are being introduced, and the design process calls for comparison of analytical, empirical and observational (i.e. measurements) methods.

Finally, the two main methods of rock mass classifications - the RMR and the Q systems - are absolutely essential for monitoring rock conditions during construction, to enable effective comparison of predicted conditions from site investigation with those encountered during construction. In fact, both these systems must be used for correlation purposes because any departure from the well established correlation between the two systems may be indicative - as was found in one case - of potential unforeseen behavior of the rock mass. For this purpose, a classification by the New Austrian Tunneling Method is deficient: not being a rating method it does not provide a continuous quantification of the encountered conditions.

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Final Rock Slope Blasting Techniques: Considerations in Design and Execution

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Abstract

This paper examines the primary techniques and considerations for the blasting of final rock slopes in today's construction industry. Techniques reviewed include: 1) traditional pre-splitting with decoupled charges, 2) airdeck presplitting, 3) cushion blasting for slopes with natural appearance. Blasting design guidelines are provided for these techniques. Benefits and drawbacks to these techniques are discussed. The paper also addresses several issues in regard to universal slope design and blasting specification without account for geologic controls that often result in unstable slopes.

Introduction

Highway slopes are intended to remain stable over a long period of time. Unexpected slope failures can be costly in terms of human life and property damage or loss. Cleanup of failures can prove to be extremely expensive.

Blasting is detrimental to intact rockmass but has to be carried out in order to excavate hard rock materials. During the blasting process, shock energy and high pressure gas disrupt intact joint contacts. As a result of blasting, some part of the composite resistance forces stabilizing the rockmass is lost.

Blast damage beyond the excavation line is termed "overbreak". Overbreak damage includes the opening of cracks behind the intended excavation limit, the loosing of rock blocks in the newly formed face and along the crest, and extra material blasted beyond the intended excavation limit.

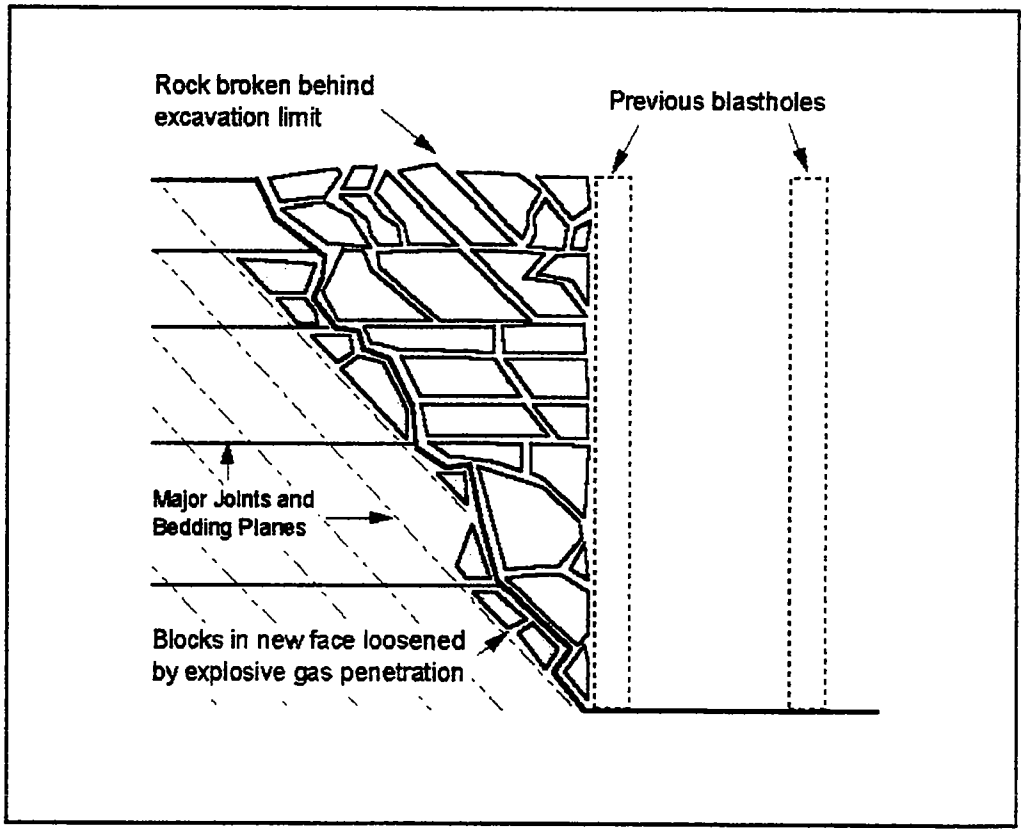

Figure 1: Blasting Overbreak in Final Wall

During cut excavation, blast induced damage to the rockmass can be minimized by exercising controlled blasting methods. The following paper looks at three controlled blasting techniques that are being used today. It also examines several issues regarding universal slope design and blasting specification without account for geologic controls that often result in unstable slopes.

Presplitting with Decoupled Charges

Presplitting with decoupled explosive charges is the most common controlled blasting method for forming final rock slopes along highways today. This type of presplitting requires drilling a row of closely spaced holes along the final excavation limit which is then blasted with decoupled explosive charges.

An explosive charge is said to be "decoupled" when the diameter of the product loaded into the blasthole is smaller then the blasthole diameter itself. A decoupled charge significantly reduces the contact between the explosive column and the blasthole wall. When fired, as compared to a fully coupled explosive charge, a decoupled charge transmits lower energy into the surrounding rock. The objective of firing decoupled charges is to minimize the amount of crushing around the blasthole walls, while still delivering sufficient shock wave energy between holes to put the rock web into tension so that a single predominant crack is developed.

The contribution of accurately drilled blastholes to high quality presplits cannot be over emphasized. Blasthole diameter strongly influences the amount of blasthole deviation in relation to blasthole depth. 2.5 to 3.5 inch diameter drill bits are typically used, or specified, for drilling presplit lines associated with highway work. In general, the smaller the drill bit and the lighter the drill string, the greater the chance for hole deviation with increasing depth. Small diameter bits pushed at high penetration rates tend to deflect and wander along discontinuities. In presplit faces that have been drilled too deeply, it is not uncommon to see extreme hole deviations, including crossovers, and erratic toe line shape.

At the time of blasting, hole deflections out towards the face can be a real problem if undetected. Should insufficient rock burden be left in front of a loaded blasthole, then when fired it can result in dangerous high velocity, long distance flyrock.

An operator exercising careful setup and watchful drilling can keep hole deviations under control. Many drills entering the field today offer electronic aids to assist in fast, proper setup as opposed to glass sight levels used on older equipment. In addition, explosive company technical service divisions and consultants offer 3-D measurement services to assess drillhole accuracy.

Today, there are three primary types of explosives made specifically for decoupled presplit blasting. The most widely used is of these three is paper cartridged nitroglycerin or emulsion explosives. Cartridged products are joined together by couplers as they are loaded in the hole. If a detonating cord downline is needed for reliable initiation, it is wrapped around the cartridge as it is lowered down the hole.

Continous film packed watergel and emulsion explosives traced with detonating cord are gaining acceptance in the field. Continuous products offer faster loading rates as compared to couple joined product.

Under special conditions where final excavation limit tolerances must be tightly controlled and overbreak into the surrounding rock is highly undesirable, such as the footing for a bridge

abutment, then high strength detonating cord (200 or 400 grain per foot) is often loaded into very closely spaced holes. This explosive product is by far the most costly to use on a per loaded foot basis.

The "charging" or "shearing" ratio is the explosive quantity used to treat each square foot of face presplit. Charging ratios vary with rock strength and jointing frequency. The closer the blasthole spacing, the greater the charging ratio. In practice, charging ratios for decoupled presplits drilled with small diameter holes typically range from 0.07 to 0.14 lb/ft^2; with 0.10 lb/ft^2 being a fairly common starting point.

As a guideline, blasthole spacings (in inches) usually run from 8 to 12 times the blasthole diameter (in inches). The initial selection of the blasthole spacing is largely judged on joint frequency. As joints cause interference with the splitting process, holes are typically drilled closer together as joint frequency increases. Rock strength influences hole spacing to a lesser extent. Soft ground tends to transmit shock energy poorly, and as a result, closer spacings are needed to get complete shear between holes. Ultimately, the optimum hole spacing is usually determined by firing and evaluating several blasts.

Holes that are drilled too closely together can result in overbreak, while those drilled too far apart can underbreak and exhibit a scalloped appearance. Presplits that are under charged can fail to develop complete shearing between holes that results in a web of rock which cannot be pulled away from the face by the production blast. Clean up blasting of this web (also called "cling") is normally done with holes drilled and blasted from the toe of the presplit. Carelessly drilled clean up shots can damage the final wall, most often undercutting the wall toe.

As a guideline, there should be no subdrill on presplit lines with multiple benches. If the presplit holes are drilled below the grade level of the next bench, then damage to the rock in the crest of the lower bench can be expected. Consideration also needs to be given to the depth of buffer and production holes above the lower bench as these too can damage the lower bench if subdrilled. Subdrilling presplit holes on the final lift above the grade may be necessary if there is the need for a toe trench. In highly fractured rockmasses, it may be necessary to stand holes off the design grade to keep the rock as intact as possible.

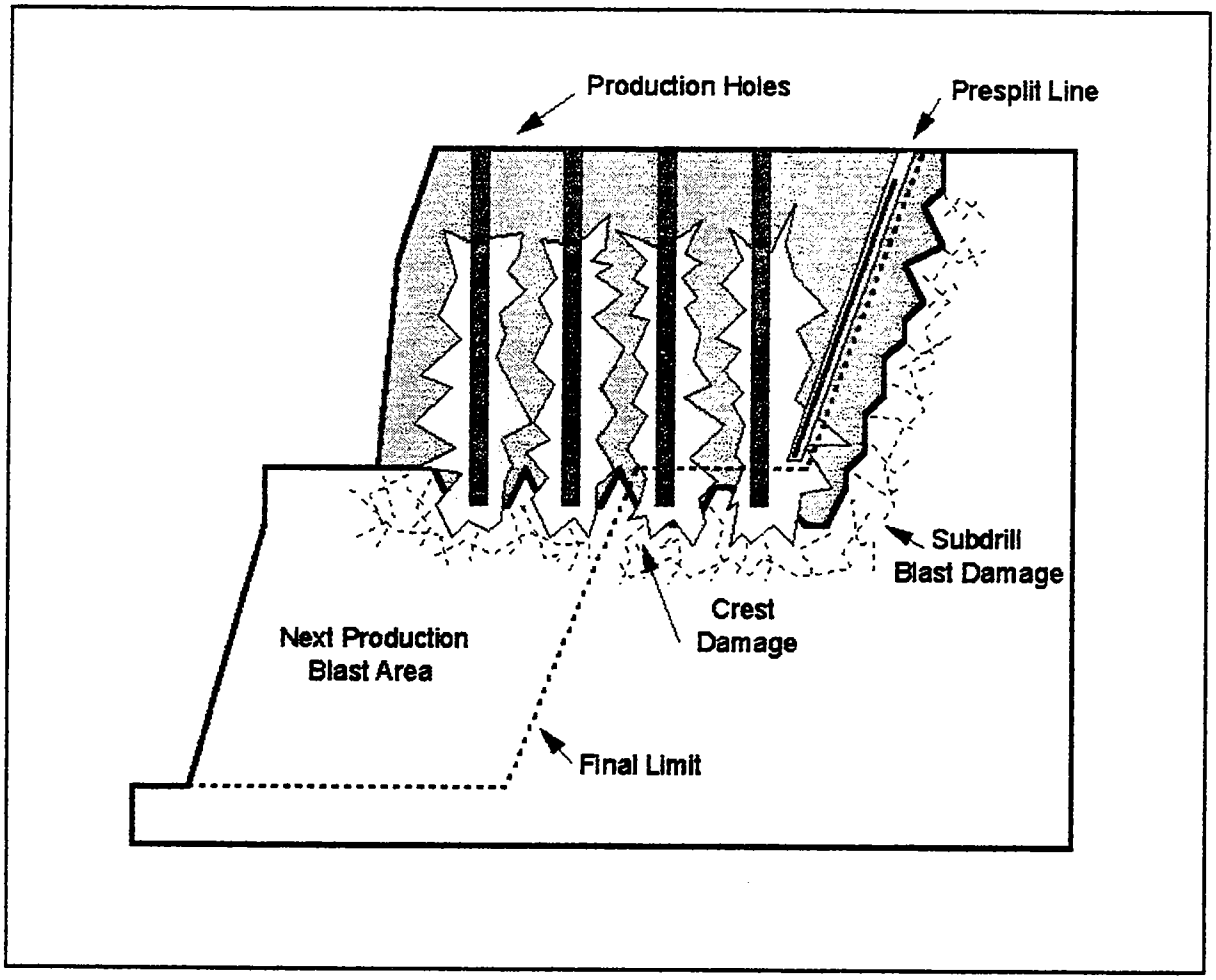

Figure 2: Crest and Final Wall Damage from Subdrill

Blast damage along the final wall is best minimized by firing unstemmed holes. Open hole firing allows the rapidly expanding explosive gases to vent directly to the atmosphere rather than to penetrate into and crack the surrounding rock. Flyrock and airblast can result from open hole blasting and should only be done in isolated areas. Under most circumstances however, flyrock and airblast control is desired, and therefore sufficient stemming is needed to confine the explosive charge.

Presplits are fired either in advance or in conjunction with the main production blast. If the presplit line is to be fired with the production, it is normally recommended that the presplit line fire at least 100 milliseconds before the nearest production hole fires.

Presplit theory suggests that the entire presplit line should be detonated instantaneously for the best result. Historically, this was achieved by tying the detonating cord downline coming from

each hole onto a surface detonating cord trunkline that was run along the row of holes. However, in addition to high airblast, this tie-in can result in large quantities of explosives being detonated at one time under confined conditions with resultant high ground vibration.

If airblast concerns exist, then in-hole millisecond delay detonators (electric or non-electric) should replace detonating cord downlines and trunklines. It must also be remembered that some presplit explosives require a detonating cord downline wrapped around the product for reliable initiation, and in this instance the detonating cord downline needs to be confined below the stemming. If vibration levels are restricted, then it may be necessary to divide the presplit line into groups of holes separated by at least 8 millisecond delay intervals.

The continuous crack opened along the excavation limit gives the subsequent production blast a plane of weakness to break to. The crack vents explosive gas from the production blast thus preventing it from expanding into and damaging joints beyond the new face. Interestingly, the presplit crack has been shown to have little effect on shockwave attenuation [1].

The presplit face can be damaged or destroyed if the last row of production holes is drilled too close to it. On the other hand, a rock web can also be left between the presplit line and the last row of production holes if they are drilled too far apart. The optimum distance between the presplit line and the production blast is usually 0.3 to 0.5 times the normal burden for the production blast.

Presplit quality is often judged, and paid for, based on the percentage of "half-barrels" visible in the final face. Despite best efforts at drilling and blasting practice, the face appearance may be controlled by geology. Massive, homogenous rockmasses are more easily controlled than those which are heavily jointed and blocky by nature. If a joint set strikes at a shallow angle to the presplit line, then the resultant face can be ragged in appearance. If the joint set strikes at an angle near normal to the presplit line, then overbreak can extend into the final wall.

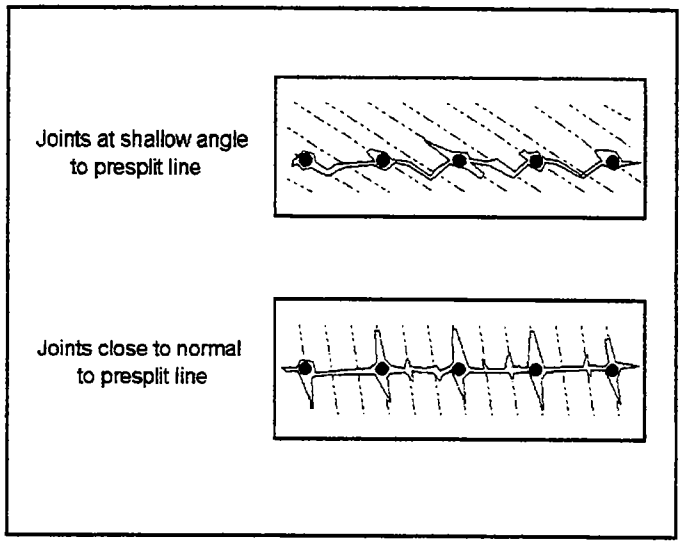

Figure 3: Affect of Jointing Orientation on Presplits

High quality presplit faces may greatly reduce small scale failures on single faces or benches. However, on large slopes with multiple benches, even with high quality presplits, there is no guarantee the overall slope angle is safe from a large scale failure. Final slope stability is largely dependent on rockmass strength characteristics, groundwater hydraulics, and the resultant potential failure geometries that are daylighted in the designed slope.

Airdeck Presplitting

Airdeck presplitting is another blasting control method for developing a defined plane along the limits of an excavation. Typically, the explosive load of an airdeck presplit hole consists of a detonator and high explosive charge, a small bottom charge, and an air gap created by placing a inflatable plug in the blasthole to hold a column of stemming at height. The stemming column minimizes airblast and seals the blasthole to create a confinement chamber for the explosive shock waves and gases to act upon.

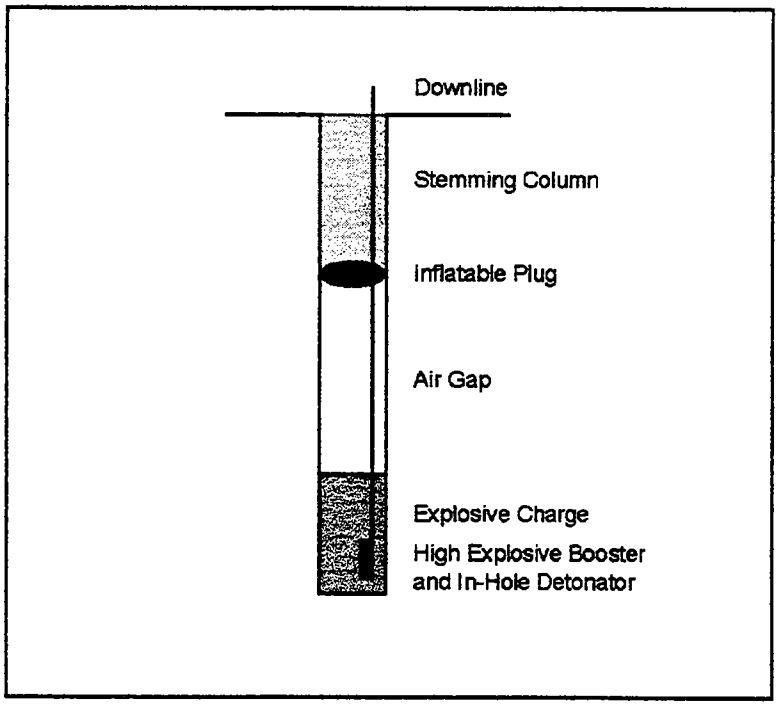

Figure 4: Airdeck Presplit Hole Load

Upon detonation, high pressure gases expand into the air column. This stresses the rock web between adjacent holes and generates the presplit fracture along the face.

Geology is the critical factor to producing quality presplit walls displaying a high percentage of half-casts when using the airdeck technique. The best airdeck presplits are achieved in rock characterized by medium to high compressive strength, horizontally bedded formation, and low

intersecting joint frequency. Rockmass characterized by low compressive strength and heavy jointing patterns usually produce poor quality presplits with the airdeck technique.

Airdeck presplits have been used to form walls on numerous highway projects. The following guidelines apply to the design of airdeck presplits with 2.5 to 4.0 inch diameter holes. Blasthole spacing (in feet) ranges from 1.0 to 1.5 times the blasthole diameter (in inches). The explosive load (in pounds) ranges from 0.08 to 0.12 times the treated area; the treated area (in square feet) being the wall height times the blasthole spacing. The stemming length (in feet) is typically 1.0 to 2.0 times the blasthole diameter (in inches). Charging ratios are similar to those previously described for decoupled presplits, with 0.10 lbs/ft^2 being a good starting point.

To achieve optimum airdeck blasting effect, an electric or non-electric in-hole detonators should be used for initiation. In-hole detonators do not rupture stemming plugs or disrupt stemming columns during initiation as detonating cord does. Firing airdeck presplits in advance of the production blast usually improves results, and as many holes as vibration constraints allow should be fired on the same delay.

Airdeck presplitting can result in significant cost saving per square foot of presplit face treated as compared to presplits made with decoupled charges. Cost savings are primarily due to the utilization of lower cost explosives and decreased labor time in loading. Airdeck presplits in competent rock have been demonstrated to allow slightly wider spacing between holes as compared to conventional decoupled presplits, thus resulting in drill cost savings.

Cushion Blasting for Natural Appearance

In competent geologies, highwalls developed by presplit blasting methods clearly show the mark of man's efforts, with rows of half cast barrels defining the excavation limits. Today, there is an increasing trend towards roadway highwall development that shows a less obvious presence of man having passed through the terrain. Many projects specify that a "natural" appearance be left behind.

High-speed photography has shown that rock is vertically uplifted in back of the last row of holes during a blast. At the end of the blast, this uplifted material settles back into place like fitted jigsaw puzzle pieces with few visible signs of disruption. In reality, completely loosened blocks of rock are present in the crest of the newly formed face.

Extensive scaling of loose material off the face and crest may be required to minimize future rock fall outs. As a result, natural appearing excavation can add significantly to the cost and time needed to complete the specified work. In some instances, ground support (rock bolts, wire meshing, and shotcrete) may be considered esthetically displeasing. Rock blocks not scaled off during the construction phase need to remain stable at their natural angle of repose and endure water erosion and freeze/thaw cycles over time. Presplit drilling and blasting costs may be reduced, but slopes excavated for natural stability may incur higher costs due to the need for additional layback and primary excavation.

From a blasting standpoint, residual natural appearance is relatively easy to achieve. Cushion blasting, also called buffer blasting, can be used to produce a stable wall with a rough looking finish. In practice, cushion blasting is a modified production blast usually consisting of two or three rows of holes fired as the last blast along the face. Well executed cushion blasts can help cut down on the disruption of material behind the excavation lines, and thus reduce the quantity of material requiring scaling.

Using the following guidelines, the cushion blast powder factor should be nearly the same as that of the production blast. The hole diameter for the cushion blast is the same as the production blast under normal circumstances. The burden and spacing are commonly reduced by 25 percent from those used in the production blast. The explosive load is decreased by 45 percent.

In addition, to reduce fracturing below grade, cushion blast holes normally do not have any subdrill. Holes along the back row are sometimes slightly staggered to promote rough breakage. In less competent geologies, airdeck loaded holes may assist in producing a ragged face.

Burden movement and ground relief during the blast are critical to minimizing the effect of gas pressure that can cause backbreak behind the design excavation limits. Progressive movement and generation of free faces is best achieved by employing hole-by-hole firing sequences (which should not to be confused with firing one hole per delay).

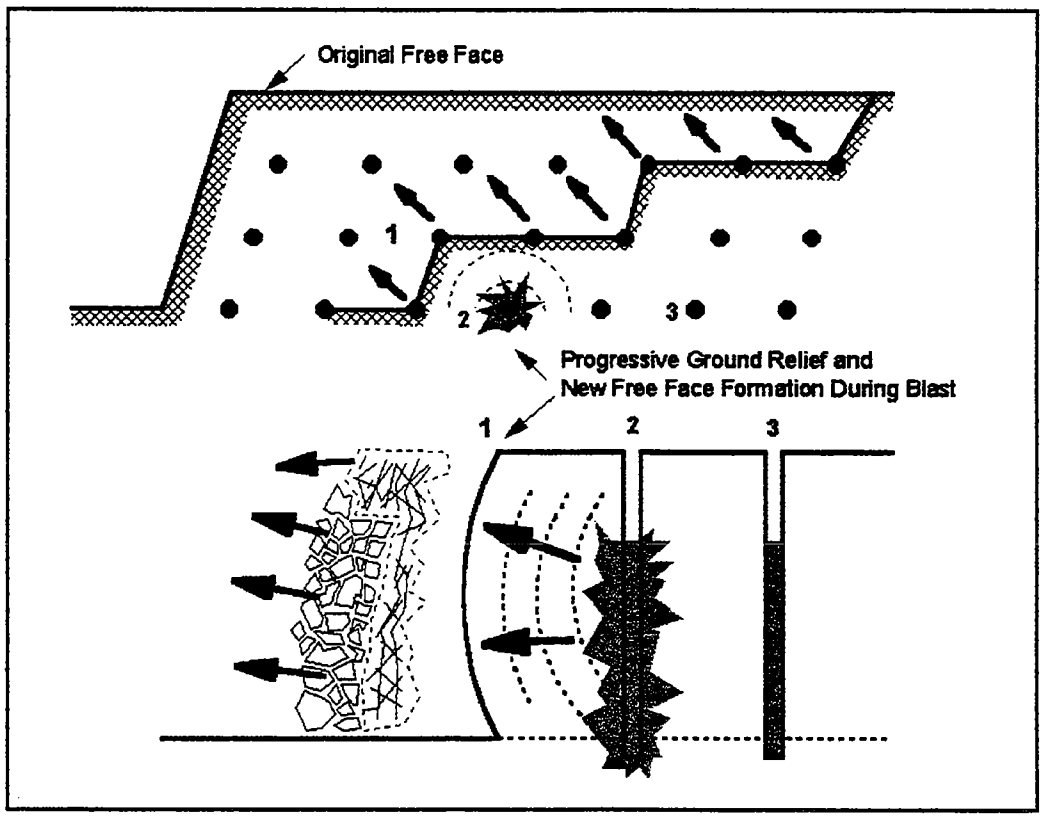

Figure 5: Burden Relief Using Hole-by-Hole Initiation

Blasting Problems Encountered with Universal Design Specifications

Blasting specifications in numerous States need review and revision as many are outdated both in technique and explosive products required. Final wall blasting specifications are copied from one project to the next. Owing to variations in rockmass characteristics and structure, a wall design that works well on one project may produce disappointing results on another.

The specified slope design often dictates at what angle the presplit holes must be drilled, and designers should give consideration to known relationships between blasting and geology. Vertical slopes are often specified by designers, as minor rock falls travel straight down the slope. Failed material gathers on catch benches or at the toe of the slope rather than rolling out into the roadway. Vertical presplits in competent, horizontally bedded rock usually have few problems breaking to the desired grade level. Conversely, vertical presplits in hard, lightly jointed ground can experience problems in breaking the toe to grade. A toe breakage problem can sometimes be addressed by loading a heavier toe charge in the bottom of the hole.

On the other hand, 1/4:1 and 1/3:1 slopes encourage angled presplits (14 and 18 degrees off vertical, respectively). Angled presplits are known to work well in competent, jointed rock and experience fewer toe breakage problems. However, on the extreme, the authors have seen specifications for 1:1 slopes with presplitting required. Drilling and loading a presplit blasthole at this angle (45 degrees off vertical) is very hard to do and would be better accomplished using stairstep drilling.

Final highwall stability is driven by geology and rock slope design. Geological investigation and rock mechanics analysis are needed to design a slope with a low risk probability of failure.

Blasting can daylight unstable wedges and planes behind the final excavation line. Ground failures of various magnitudes have happened immediately or shortly after blasting because an unidentified, but controlling, geologic feature was liberated. In many cases these failures simply occur because the friction force resisting the slide is less then the gravitational force driving slide.

Unfortunately, in many post blast slides, bad blasting practice by the contractor is the primary suspect cause cited for the failure. When a slope fails immediately or shortly after blasting, excessive blast vibration is often cited as the cause. Ground vibration levels specifications are then brought to the attention of the affected parties.

Many ground vibration specifications are written in an ambiguous or meaningless manner. Others, in an effort to absolutely safeguard the area, are being written to an overly restrictive, sometimes prohibitive, degree.

Numerous blasting specifications call for a maximum peak particle velocity (PPV) of 2.0 inches per second or less to safeguard structures of concern near the work area. However, the 2.0 inch per second peak particle velocity limitation is intended to protect residential structures at some distance outside the immediate blast area. One cannot expect ground vibrations within the

immediate blast area to be 2.0 inches per second as the onset of fracturing to intact rock by blasting begins at about 10 inches per second.

Peak particle velocities above 2.0 inches per second have to be expected and anticipated in the immediate blast area. With regard to blast induced ground vibration and slope stability, LL Oriard notes the following observations [2]:

> "At 0.05 to 0.1 m/s (2 to 4 in/s), we may expect the occasional falling of loose stones on slopes. At 0.13 to 0.38 m/s (5 to 15 in/s), we may expect the falling of partly loosened sections of rock underground and on above ground slopes, sections of rock that would remain in place for long periods of time. Above 0.6 m/s (25 ips), we would expect some damage to occur in unsound rock types, but we find that damage may not occur in sound rock at much higher vibration levels ... One can observe sections of sound rock that show no visible signs of damage even after they have been subjected to particle velocities of 2.5 m/s (100 in/s)."

The linear regression equation below is commonly used to estimate peak particle velocity assuming average ground transmission constants [3] :

$$PPV = 160(R/W^{1/2})^{-1.6}$$

where;

PPV = Peak Particle Velocity
R = Distance to Structure of Concern
W = Maximum Charge Weight per Delay (8 msec interval)

However, the accuracy of this equation breaks down when trying to predict near field effects in the immediate blast area. Furthermore, measurement of high blast vibration levels in the near field cannot be accurately recorded by standard commercial seismographs; the geophones they are normally equipped with (depending on the manufacturer) top out at vibration levels from 4 to 6 inches per second. Accurate high level vibration monitoring in the near field requires the use of a special geophone or accelerometer.

There is a strongly indicated need for increased understanding of explosives and blast engineering by many of those who are designing rock slopes and writing project specifications. Additional course work and membership in the International Society of Explosive Engineers is encouraged. Continued research into presplit techiques versus the degree of rock damage is also called for.

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CASE EXAMPLES OF ROCK STABILITY ON TRANSPORTATION PROJECTS

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Abstract - Rock Stability on transportation projects is influenced most significantly by structural geology, groundwater pressures, blasting and rock strength. Modes of failure include circular, planar, wedge, block, toppling, buckling, keyblock, ravelling and boulder fall. Numerous procedures to stabilize rock are available and described in the FHWA Rock Mitigation Manual publication SA-93-085 (Brawner, 1993).

To illustrate a number of rock stability problems five transportation case examples are described and illustrated.

Introduction

Transportation volumes are increasing at a high rate in North America. As a result, greater attention has to be paid to rock stability along transportation routes. The Federal Government has recognized the importance of safety by establishing Title 23 U.S. Highway Code and the 1991 Intermodal Surface Transportation and Efficiency Act which states:

"It shall be the National Policy to bring all of the Federal Aid systems up to standards and to increase the safety of these systems to the maximum extent". "The Secretary (of Transportation) shall not approve plans and specifications for proposed highway projects if they fail to provide for a facility that will adequately meet the existing and probable future traffic needs in a manner conducive to safety, durability and economy of maintenance".

The above clearly defines SAFETY as the primary transportation design, construction and maintenance criteria.

In recognition of this legal requirement and the desire to establish a comparable standard of practice across the United States, the FHWA contracted C.O. Brawner Engineering Ltd. to prepare a Manual on "Rockfall Hazard Mitigation Methods". The Manual was to be developed in practical terms that would be understood and usable by highway engineers and staff at the foreman and field superintendent levels. This Manual should be updated in about 5 years as a result of improving technology.

While many factors influence rock stability the most important factors are structural geology which controls the mode of failure, groundwater pressure which reduces effective strength, blasting which introduces impact and vibration forces and rock strength of weak rock and along discontinuities.

Modes of failure include circular, planar, wedge, block, toppling, buckling, keyblock, ravelling and boulder wall. Each mode of potential failure must be determined and analyzed separately since the appropriate stabilization procedure(s) can differ for each mode.

To illustrate the application of engineering principles of rock mechanics and stability, the thrust of this paper is to present five case examples from the transportation field. These include examples from outside the United States.

1. Canadian Pacific Railway, B.C., Canada

In September, 1974, an empty unit coal train was travelling near Spences Bridge from Vancouver to the south-east British Columbia coal fields. The train rounded a curve at 35 miles per hour and encountered about 20 cu. yds. of rock on the track. (Figure 1.) The two lead locomotives and several coal cars were derailed over a 100 foot high steep rock face down onto the Trans Canada Highway. (Figure 2.) The two locomotive operators were killed.

The writer was requested urgently to accompany C.P. Rail officials to the site. The initial priorities were to determine the cause of failure, evaluate the stability of the rockfall site and to implement stabilization measures required to stabilize the slope to re-open the track as soon as possible. An average of 30 trains per day used the mainline track.

A site inspection indicated the rockfall was bounded by two near vertical joints with an intersection angle of 120° resting on an irregular sand filled joint dipping about 40° out of the slope. (Figure 3.)

A review of weather conditions revealed a heavy rainfall had occurred about 12 hours earlier. About 20 minutes prior to the accident a loaded coal train passed the site.

It was concluded that the factors that contributed to the failure were:

- The adversely oriented structural geology.
- The high intensity rainfall which created water pressure in the rock joints and seepage forces in the lower joint.
- The vibration due to the passing loaded coal train.

An inspection of the rock face revealed parallel dipping joints behind the failure zone. These could also destabilize in the future. An immediate decision was made to install tensioned grouted rock bolts followed by drilling drain holes in the face. (Figure 3.)

The accident was sufficiently serious that railway officials believed a government inquiry would follow. It was decided to institute a major program to evaluate the stability of rock cuts on the rail system across the country. The program involved the following:

Development of a comparative stability classification system to locate rock slopes believed to be at greatest risk of rockfall and potential train derailment. The classification included details relating to location, alignment, climate, train frequency, slope dimensions, structural geology, history of past rockfall, evidence of seepage, work space available, stabilization recommendations and site photography. Slope stability was rated according to potential urgency of stabilization.

The rating system was purposely developed to be simple. (Figure 4.) In later years the rating system details were expanded.

A program was instituted to inspect and evaluate all rock cuts in western Canada in 1974 and in the rest of Canada in 1975. Initially the inspections were made by track motor car and walking through the cuts. In later years helicopter inspections were added to inspect the rock slopes from above, including high cliffs adjacent to the track. Tension cracks were observed from the air which were not observed from track level.

Figure 1
Rockfall on the railway tracks which caused the coal train derailment.

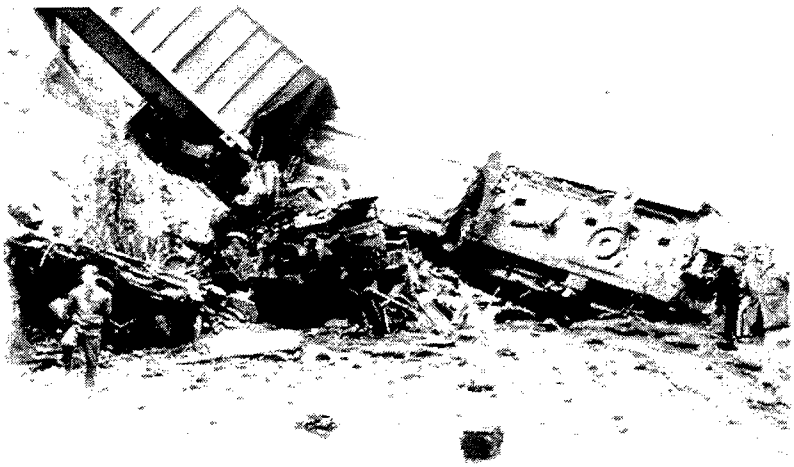

Figure 2
Locomotives derailed over a rock cliff onto the highway. Both operators were killed.

Figure 3 - Wedge failure with three joints. Rock faces were stabilized with bolts and drain holes.

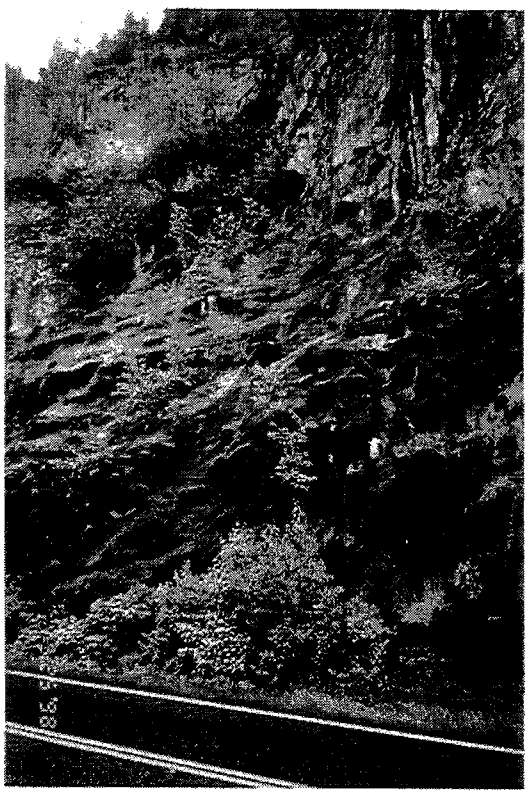

Figure 5 - Rockfall area near mile 20 north of Vancouver on the Sea to Sky Highway to Whistler.

Figure 4
PRIORITY RATINGS

A - Moderate probability of failure of sufficient volume to result in derailment if failure undetected.
B - Some probability of failure of sufficient volume to result in derailment if failure undetected.
C - Moderate probability of failure of small volumes which might reach the track.
D - Moderate probability of localized rocks or rockfalls occurring during extreme climatic conditions - Very heavy rainfall or runoff, extreme freeze-thaw cycles, *etc.*
E - Slight possibility of localized failures under extreme climatic conditions. Generally shallow cuts.

At some locations instability was considered to be sufficiently imminent that urgent stabilization was recommended - as soon as practically possible. In two instances the potential of rockfall was considered to be so serious that train traffic was halted until the unstable area was stabilized.

Based on the inspections, the railway allocated substantial funds to develop a stabilization program for 10 rock slopes where the stability rating was considered to be the most critical.

The stabilization programs included rock scaling, increasing ditch dimensions, installation of dowels, rock bolts or cable mesh, application of shotcrete, construction of the buttresses, installation of warning fences, *etc.*

Approximately 6 months following the rockfall the Canadian Transport Board held a hearing into the accident at Vancouver. The report of the Board expressed concern over the accident but commended C.P. Rail on it quick and professional response. The major recommendations were the requirement to inspect rock cuts annually and to continue the stabilization program.

This program for inspection and classification of potential rock slope instability was the first of its kind in the transportation field. The program has been expanded for use in highway engineering, particularly by the State of Oregon under the direction of Larry Pierson, and more recently by the State of Washington under the direction of Steve Lowell.

The FHWA in 1994 sponsored the preparation of a Rockfall Hazards Mitigation Manual. The manual was conceived by Ron Chassie FHWA Geotechnical Supervisor in Portland and developed by the author. Seminars which review the Manual have been presented in many States.

This example emphasizes the significance of structural geology, groundwater and vibration on rock stability.

2. Just vs the British Columbia Department of Highways

On January 18, 1982 a single rockfall measuring about 2.5 feet by 2.5 feet by 0.5 feet bounced down a rock slope about 20 miles north of Vancouver, B.C. on the Sea to Sky Highway to the Whistler - Blackcomb area.

A snow storm with winds was in progress. Vehicles had been stopped by the poor driving conditions. A Volkswagen automobile carrying Mr. Just and his daughter was stopped in the lineup. The rock fell on the roof of the car, killed the daughter and severely injured Mr. Just. His injuries were sufficiently severe that he could no longer work as a carpenter foreman.

A site inspection located the rockfall source. The face of the remaining rock showed evidence of weathering and tree roots. The fallen tree was located directly below the rockfall area. It was about 6 inches in diameter at the base and had stood about 30 feet high. Weather records indicated 8 days of freeze-thaw cycles had occurred prior to the failure.

At the time of the failure the vegetation was partially covered with a heavy load of wet snow. The wind created very high leverage on the tree roots growing in the open joint. The tree was solitary and very exposed to the wind.

The rockfall area was located on the natural slope directly above a rock cut face. (Figure 5.) The lower portion of the slope had been stabilized with shotcrete. A narrow slightly sloping bench existed between the upper rock cut face and lower shotcreted slope. The rock bounced off this bench.

A portion of the rockfall had remained near the point of origin. To assess the trajectory of the killer rock, the traffic was stopped and this rock was pushed over the slope. It impacted the roadway within 10 feet of where the original rock had hit the car.

The court case against the Highway Ministry was heard by the Supreme Court of British Columbia. The Judge ruled against the Just family based on a legal technicality. The Court of Appeal in British Columbia upheld the decision. The case was submitted to the Supreme Court of Canada were it was held that the legal technicality was unreasonable and a new trial was ordered.

The author was retained as a specialist witness on behalf of the family. Evidence was presented regarding the state of rock mechanics knowledge and practice in the transportation industry. Major emphasis was placed on the potential for isolated trees on rock faces and at the slope crest with roots growing in cracks to lever and cause local rockfall. (Figures 6 and 7.) The tree which levered the rockfall was readily observable from this highway. The Department of Highways had inspected the highway rock slopes and performed rock stabilization on slopes along the highway. For some reason the potential for instability due to this tree was overlooked.

The case was presented to the retrial Court that the rockfall was both predictable and preventable. The Department of Highways was held responsible and the family was awarded substantial compensation.

During the litigation a strong recommendation was made that all trees located on rock slopes or within 6 - 8 feet of slope crests should be removed. This does not apply to shrubbery or small trees.

From the aesthetic point of view, some agencies provide strong resistance to removal of these trees. As a result of a serious accident in Colorado where a large rock impacted a Greyhound bus and killed 8 people that U.S. Highway code Title 23 was enacted in 1991 which specifies that safety is the primary design and maintenance issue on U.S. highways. Hence, safety takes precedence over aesthetic and environmental issues.

In order to ensure that trees on rock slopes will not contribute to instability it is recommended that such trees be identified as "danger trees" and removed.

This example emphasizes rockfall potential due to wind leverage on tree roots in cracks located on rock slopes or near the crest of rock slopes.

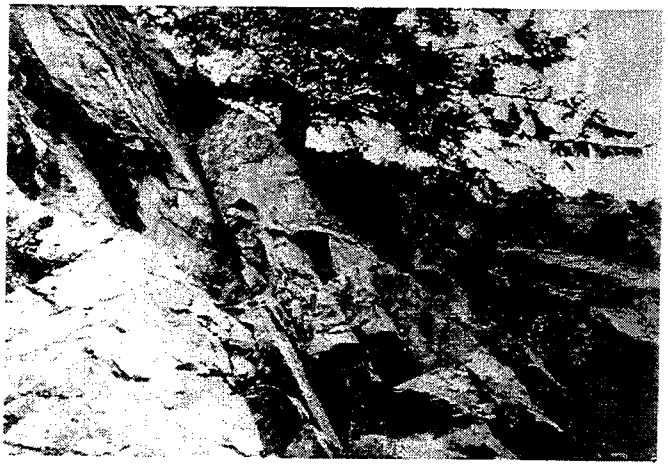

Figure 6 - Danger tree with the root system growing in the cracks. Wind leverage is high.

Figure 7 - Tree roots growing in a rock crack which have levered a rockfall.

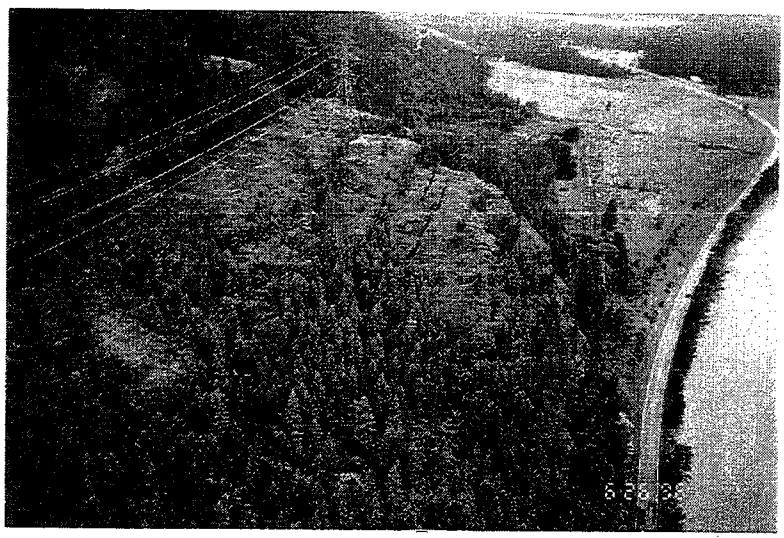

Figure 8 - Air photo of the Badrock Canyon cliff showing the highway location.

3. Badrock Canyon, Montana

A proposal has been developed by the Montana State Department of Transportation to upgrade Highway No. 2 north from Kalispell from two lanes to four lanes along the Flathead River. (Figure 8.) The alignment is very restricted between the river on the outside and a major rock cliff and state park on the inside. In addition, two major power lines exist on the top of the rock cliff.

A field program was performed to map the structural geology. The geology comprises bedded limestone - argillite. The bedding dips from 18 - 35° to the east and varies about 0 - 30° from the strike of the proposed center-line. There has been past movement along some of the beds, hence the shear strength will be at or near residual. The movement has an angular component toward the proposed highway slope. A predominant set of open near vertical joints strikes about 0 - 30° off the strike of the proposed highway center line.

The most important geologic features are a number of open tension cracks which are only observable from the air or by climbing to the top of the cliff. (Figures 9 and 10.) It is believed that these cracks represent a major combined toppling and block sliding condition which has developed as a result of lateral stress relief due to valley glacial melting and river erosion. Horizontal drilling and coring was performed near highway elevation to determine if the open tension cracks extend to substantial depths. Open cracks were not encountered during the drilling. This suggested only the upper blocks had slid latterly as well as toppled.

A major concern was whether the toppling movement was still active. If movement was still occurring rock excavation to widen the highway would destabilize the slope. To evaluate this, survey stations were developed across the river and several lines of reflecting mirrors were installed on sections on the cliff face. In addition, movement hubs were installed for periodic measurements to be taken by a micronometer tape across the upper cracks. This survey program extended over two years and has shown that no movement has occurred during that period.

Had the structural geology been favorable the intact rock strength would have been adequate to develop a vertical cut face and a wide catch ditch according to the modified Ritchie design (Brawner, 1993). Because of the adverse structural geology and evidence of past movement, the following design was developed. Flatten the overall slope angle by developing three benches. On each bench it was proposed to install dowels drilled and grouted into the bench and packed with concrete against the upper rock to prevent lateral sliding. The bench geometry was to coincide with the bedding.

Construction sequencing was considered important. The upper bench was to be developed first, commencing up dip. The second and third benches are to follow sequentially. The back of each bench is to coincide with a tension crack or near vertical joint.

To maximize stability, the impact of seismic forces due to blasting and water pressures in rock joints or along the bedding planes must be minimized.

Detailed blasting specifications have been developed. These include the use of controlled blasting. Closely spaced small drill holes using small explosive charges are to be used with each blast hole delayed. All blasts shall be to a free face, perpendicular to highway center line. Small blasts are to be used with the rock surface covered with blasting mats to control flyrock into the river or from damaging the power-lines. All blasts are to be monitored to determine peak particle velocity.

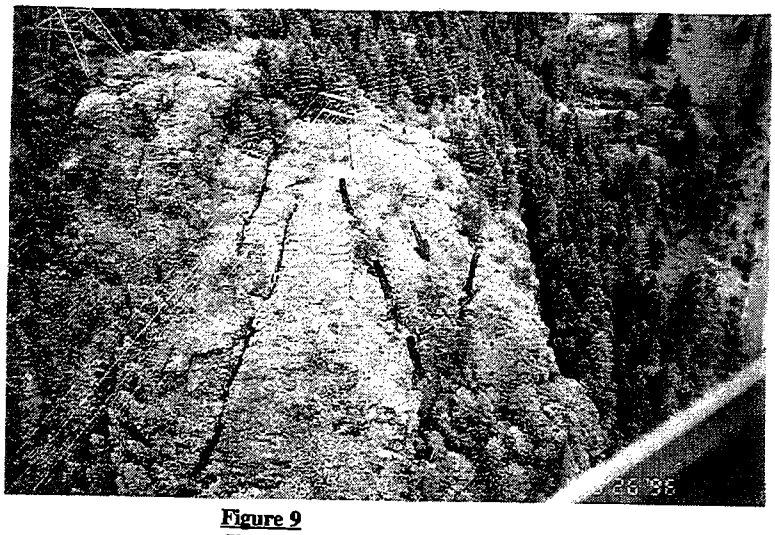

Figure 9
Closeup of the movement cracks
and power-lines.

Figure 10 - Typical crack on the top of the cliff.
The cracks are a combination of toppling
and block movement.

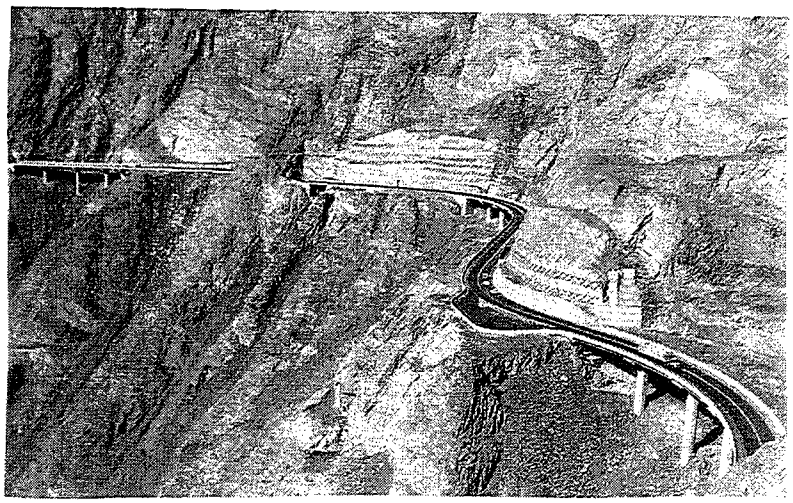

Figure 11 - One portion of the Sha'ar Descent Highway,
Saudi Arabia showing major rock construction.

The contract will require that the contractor retain a specialist blasting consultant to design and monitor blasts. A pre-bid meeting, which all contractors who bid must attend, will be established to describe the rock conditions, blasting requirements and construction sequencing.

The control of groundwater in the long term is essential. Horizontal drains are proposed from each bench and all tension cracks are to be drained at the lower end or filled to the surface to exclude water.

Because of the very high summer traffic it is recommended that rock excavation and slope stabilization be done during the winter.

Following the development of each bench the adjacent rock face is to be scaled.

At the north end of the rock area the rock cliff deviates away from the road shoulder. Below the cliff a talus slope exists. The surface slope angle is 37°, the angle of repose. This will be partially undercut by the highway widening. Talus will be removed to allow a 20 foot wide 6 foot deep catch ditch. Placement of a Jersey barrier on the outside of the should is recommended to increase the effective ditch depth.

This project has been delayed pending environmental hearings and to obtain the necessary permits.

This example emphasizes the need to determine the mode of rock failure.

4. Sha'ar Descent Highway, Saudi Arabia

The author and Larry Pierson, Chief Geologist of the Oregon D.O.T., were requested through JECOR and the Ministry of Communications of Saudi Arabia to present Rockfall Mitigation Seminars in Riyadh. It was recommended to them that site inspections of problem areas in Saudi highways be inspected first to understand Saudi rock problems.

The most serious problems were observed on the Sha'ar Descent near Abba in the westerly Asir area. This is a very spectacular section of highway (Figure 11) with rock cuts up to 450 feet high, extensive tunnels, retaining walls, bridges, shotcreted slopes, rock bolts and rock barriers.

The project was designed by an Italian Consultant and constructed by a Chinese contractor.

Two problems are described:

a) Failure of rock bolts.

Many rock faces were stabilized using 10 - 20 foot long rock bolts through previously shotcreted rock faces. The higher the slope the longer the bolts. The bolts were Dywidag non-corrosion protected steel bolts. The bolts were anchored with rapid set two component epoxy. When the epoxy had set the bolts were tensioned to about 60% yield strength. Large concrete bolt head sections were used to spread the load over the rock face. (Figure 12.) The bolts were not grouted! Commencing several years ago the anchors began to fail. Due to the high stress on the bolt when the anchor released, most shot out of the hole and across the highway, an extremely dangerous event. Some bolts stuck in the holes following anchor failure. (Figure 13.)

Not grouting the bolts was a design and construction error. In addition the high tension required was unnecessary as were the excessively large bolt heads.

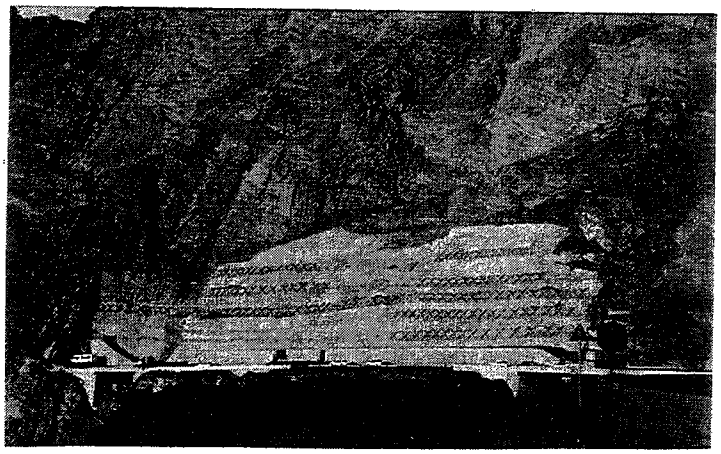

Figure 12
Shotcreted rock slope with pattern bolting. Missing concrete bolt heads indicate anchor failure.

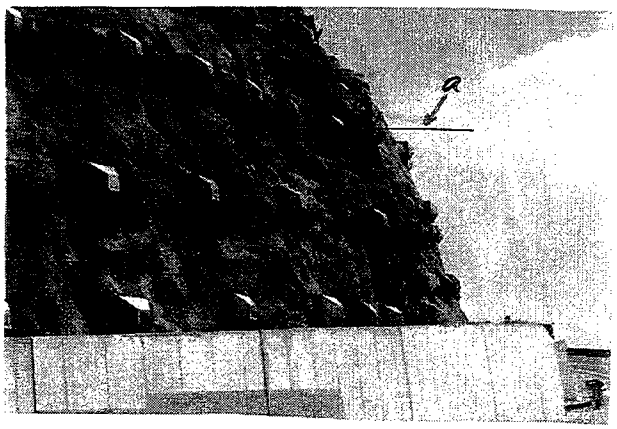

Figure 13
Bolt extruding from a hole due to anchor failure.
Most failed bolts have shot across the highway.

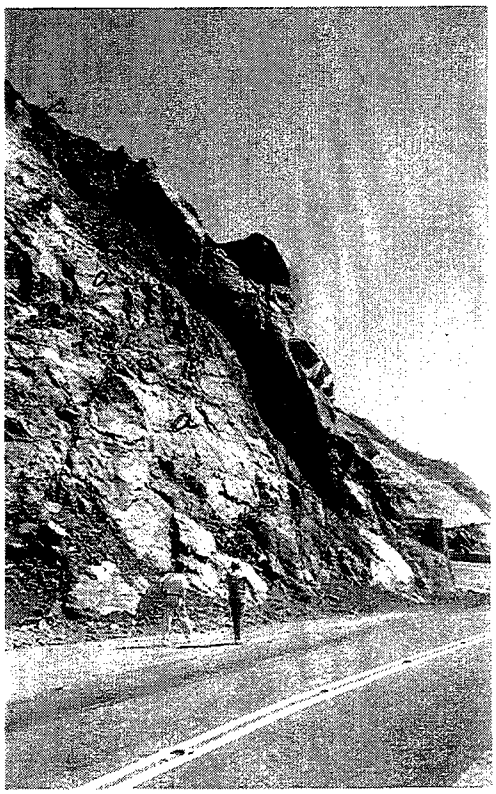

Figure 14
Rock has failed along the rock joints.
Tensioned rock bolts were all the same length.

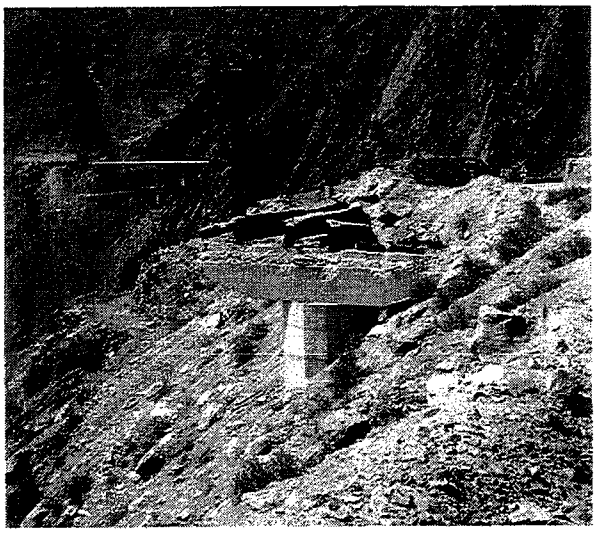

Figure 15 - The rock slide destroyed the adjacent bridge. Traffic is re-routed on a shoefly detour.

Standard practice in rock slope stabilization in North America is to grout all tensioned rock bolts to control corrosion and lock in the applied tensile stress. In addition the amount of tension is dictated by volume of rock to be stabilized. High tension is usually not required.

Standard practice also requires the angle of bolt installation to be such that minor movement places the bolt in tension - not bending.

b) Rock slide at Bridge 8A.

A rock slope failed above bridge 8A and destroyed one end of the bridge. (Figure 14 and 15.) The failure occurred primarily along joint surfaces near parallel to the slope. A concrete toe wall had been constructed and the lower rock cut face had been shotcreted and bolted with two rows of tensioned ungrouted rock bolts 16 feet long. High bolt tension was used. It has been noted previously that not grouting the bolts is poor practice. Under such circumstances highly tensioned bolts that are the same length develop a zone of tension through the anchor zone. Good practice requires such bolts to be variable length.

This example emphasizes the necessity to grout rock bolts for long term stability, to tension according to volumes to be stabilized and to use non-uniform length of bolts.

5. Crooked River Bridge Foundation, Oregon

The Oregon Department of Transportation (ODOT) plans to construct a new 4-lane bridge across the Crooked River Canyon near Bend, Oregon. ODOT commissioned the author to review the site investigation, rock coring and rock testing by ODOT and provide recommendations for foundation design for the concrete arch structure. (Figure 16.) The arch span is 134.4 meters long. The structure was designed by D.G.E.S. Consultants of Olympia, Washington.

The site geology comprises inter-canyon and plateau basalts typical of this region of Oregon. The rock mass is moderately hard to hard (R3 - R4). At some locations open joints, vesicles, near horizontal interflows and open voids were encountered.

Oregon DOT performed an extensive site investigation including eight boreholes. The core was not oriented. A laboratory investigation included unconfined compression tests and point load tests on selected core. The core was photographed. Core recovery, RQD, joint conditions and core description were logged. The core was retained for inspection by the contractor.

For a 500 year seismic occurrence at the site, the peak rock acceleration is 0.07g from a magnitude 7.75 subduction zone event and a magnitude 5.75 crustal event.

Based on the test results it was apparent that the north thrust block rock conditions were less favorable than for the south thrust block. RQD values at the north ranged from 53 - 93% and for the south thrust block area 84 - 100%. The recommended allowable design bearing values were 80Ksf for the south thrust block and 50Ksf for the north block. The north area had more voids and cracks. As a result, foundation grouting was considered necessary.

Rigid arch structures are sensitive to lateral differential settlement. Based on the recommended loads, the foundation settlement was estimated to be 13 to 20mm with differential about one-third the total. The settlement would include excavation load recompression, joint closure and elastic compression. It was a requirement that settlement readings must be taken at each foundation corner during construction to assess settlement.

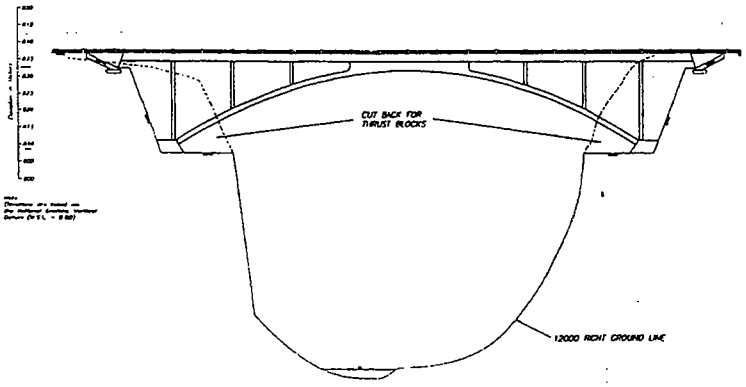

Figure 16
Cross-section of the Crooked River Gorge concrete arch bridge presently under construction.

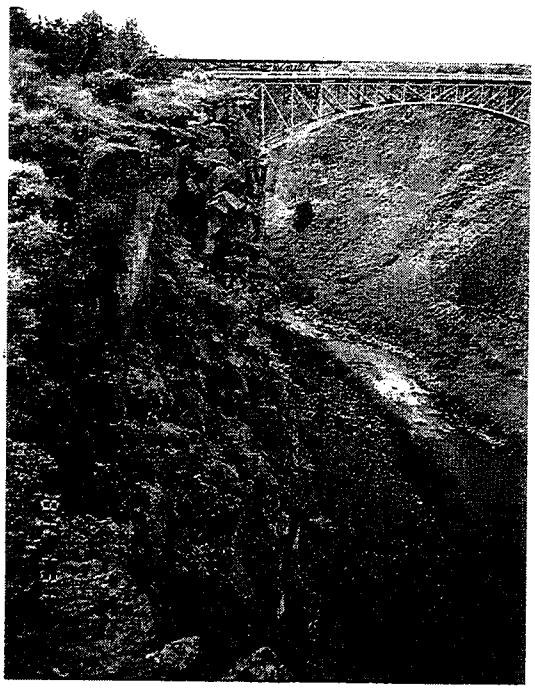

Figure 17
South abutment and thrust block area high above the Crooked River. The present bridge is in the background.

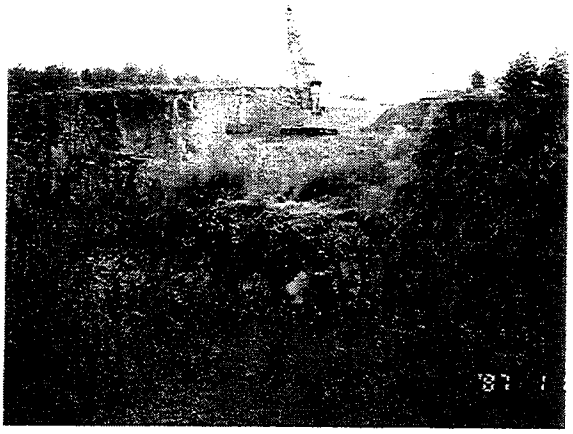

Figure 18 - Controlled blast for the thrust block excavation. The large crane lifts the rock and the excavator from the excavation.

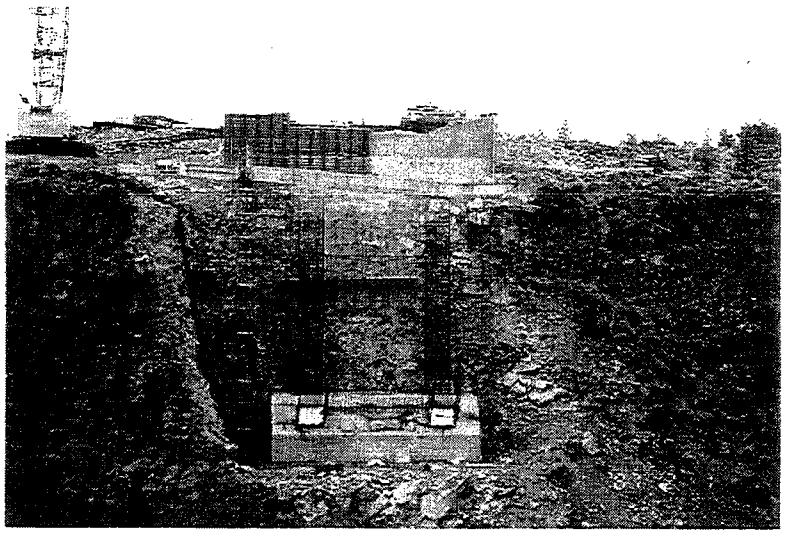

Figure 19 - North excavation and thrust block is completed. The north abutment is under construction.

The front face of the thrust blocks were not to be closer than 5 meters to the cliff face.

Due to the river below, only a minimum of excavated rock was to be allowed over the cliff face. Special blasting procedures were specified. The contractor was required to retain a specialist blasting consultant approved by ODOT. Excavation depth was about 26 meters. The rock was excavated in 5 meter lifts.

A pre-bid conference was held at the site and all proposed bidders attended.

Most rock was excavated with a backhoe which was lifted in and out of the excavation as required by a large crane. The rock faces were rock bolted and meshed for construction safety and long term stability.

The north foundation imprint was grouted to a depth of 5 meters with various cement grout mixtures. Grout pressures were retained low to not "lift" the rock. More grout than expected was required in about 10 percent of the holes. Two core holes were drilled to confirm grout had filled most cracks and voids.

The thrust block construction has now been completed. Photographs taken during construction are shown in Figures 17 to 19.

This example emphasizes the need for controlled blasting to reduce rock damage and the need for foundation grouting where voids or cracks are prevalent.

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Rock Slopes and Bridge Foundations in a Foliated Rock Mass Along Portions of State Route 69 Near Mayer, Arizona

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ABSTRACT

Geologic units exposed in cuts slopes and encountered in excavations for three new bridge sites across Big Bug Creek for the new alignment of State Route 69, just south of Mayer, Arizona consist of Precambrian metamorphic rocks of the Spud Mountain Volcanic and Iron King Volcanic sequences. The sequences are composed primarily of an interfoliated and folded assemblage of phyllites and schists. These rocks generally possess good bearing capacities for structure foundations and have adequate strengths to support relatively steep slopes in cut areas. However, less steep slopes are governed by the fracture system contained within the rock mass, since slope failures can occur along the discontinuity planes in certain slopes. The fracture system includes three major joint sets; the most persistent of which occurs along the foliation planes.

Stability of individual cut slopes is dependent upon the orientation of the discontinuities relative to the orientation of the cut slopes. Most of the slopes in the Mayer area support a 3/4:1 (Horizontal:Vertical) slope since the more persistent foliation joint set crosses the alignment of the new roadway and intersects the cut slopes at nearly 90 degrees. However, one failure occurred in the largest and southernmost cut slope during construction. The failure included an approximate 3000 cubic yard block of material. Due to a wider fracture spacing within a portion of the failure zone, some individual fragments that slid from the failure zone ranged up to 23 feet across. The failure was a planar type failure that occurred along a local cross-foliation joint set. A failure of this kind was not anticipated since the strike of the failure plane differed from the strike of the slope face by more than 20 degrees. However, the failed block separated from the intact mass along the more continuous foliation joint set. Remedial design in this slope included a benched construction with an upper 1:1 slope face to remove the material from the failure area and controlled blasting since initial blasting at the finished cut face had opened the fractures and weakened the rock mass.

Bridge Structures Nos. 1 through 3, which spanned Big Bug Creek at three locations were each supported on drilled piers socketed into the metamorphic rocks. Drilled piers socketed into bedrock were the selected foundation type for the three structures, mainly due to the presence of relatively thick overburden above the bedrock, and the variable degree of weathering of the foliated bedrock contact with respect to support of footings.

Introduction

Improvements to State Route 69 (SR-69) from Cordes Junction to Prescott, in Yavapai County, Arizona included construction of a new 4-lane divided roadway through a strongly foliated section of Precambrian metamorphic rocks just south of Mayer, Arizona. The Precambrian metamorphic rocks are exposed in about a 3-mile section extending southward from the south edge of Mayer (MP 267 to MP 270). Road cuts from 35- to 170-feet high and three new sets of twin bridges across Big Bug Creek were required to complete the construction of the roadway in this section. The largest cut was required at the south end of this area in Section 31 (T12N, R2E). The bridge designs consisted of twin, two-span, parallel structures with span lengths varying from 87 to 100 feet and overall widths of about 45 feet. The project area within this metamorphic terrane is shown on Figure 1. The three bridge sites and the location of the largest cut area also are shown on the figure. A photo of Bridge Site 2 is included on the figure.

Although the strengths of the rocks were adequate for support of the bridges and relatively steep slopes, the foliated and jointed condition contained within the rock mass required some special design considerations and modification to construction techniques to safely complete the project. The design considerations included slope stability analyses before and after the failure, ditch design to contain rockfalls and small failures and constructability of remedial slopes. The constructability of drilled piers (rock sockets) to support bridges also was evaluated.

The slope stability analyses were completed in two stages. The first stage was completed prior to construction to establish general slope

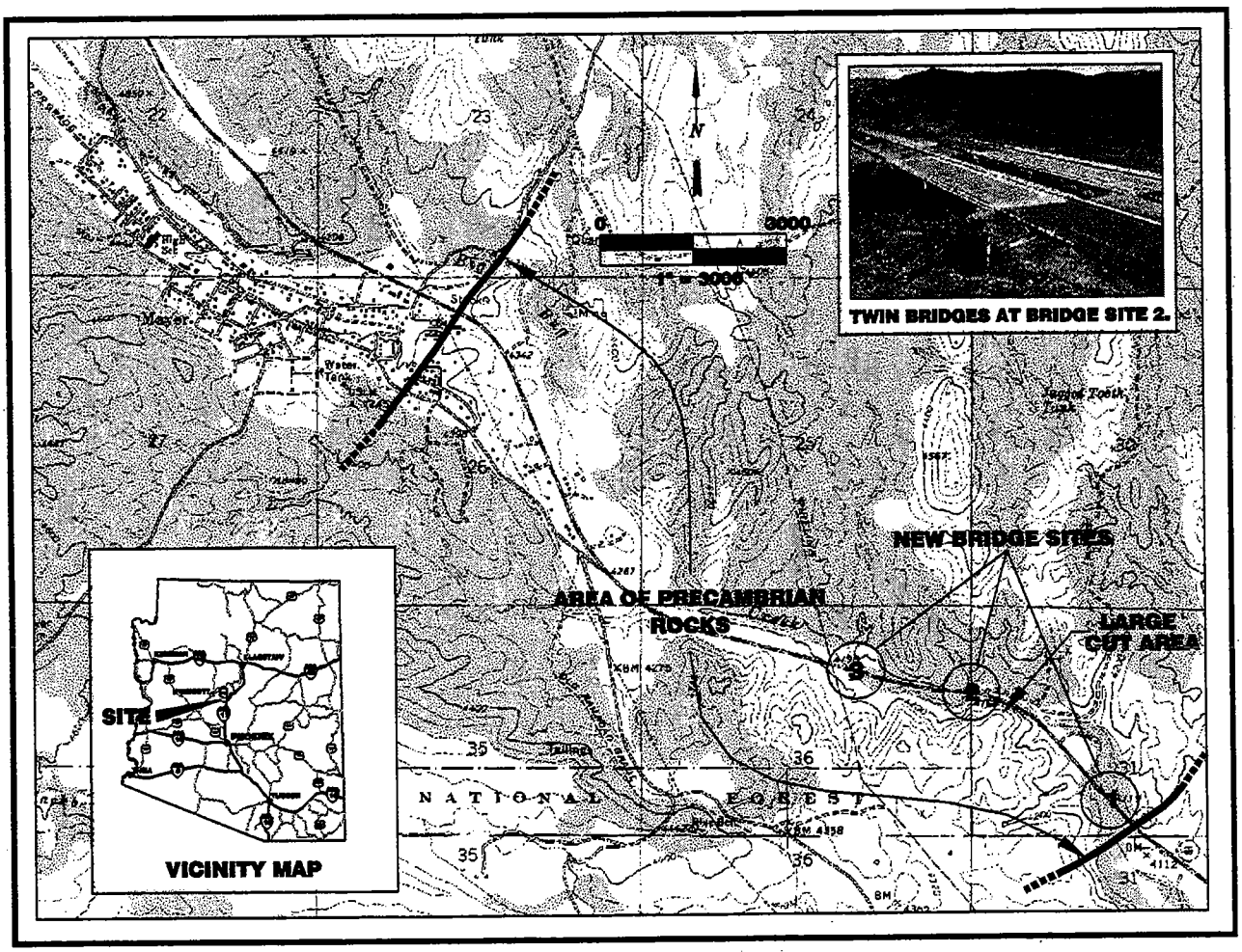

Figure 1. Location map.

configurations. This analysis was based on an evaluation of joint conditions contained within the rock mass in each major cut area, since slope failures in this rock type and condition would be the result of sliding along joints. Based on this analysis the slopes were designed at an angle of 3/4:1 (Horizontal:Vertical). The analysis indicated that some potential failures could occur in the large cut slope, but the joints involved would not contribute to a large failure due to their discontinuous nature. The small failures would be very localized and the debris would be contained within the ditch at the toe of the slopes and thus not impact safety to road users.

The second phase of the stability analysis was completed during construction of the roadway, following a failure in the upper portion of the large cut slope. The failure was a planar failure that was not anticipated since the strike of this plane differed from the slope face by more than 20 degrees. However, the block of rock in the failure zone separated from the intact mass along the foliation joint set, which appeared to be weakened during blasting activities. The slope was subsequently redesigned with a benched geometry with an upper 1:1 slope face to mitigate against further failures in this area and to remove the failed zone.

Geologic Setting

The site lies in the Transition Zone Physiographic Province along the northeastern flank of the Bradshaw Mountains in central Arizona. The Transition Zone is characterized by a series of short mountain ranges separated by small basins. The mountain ranges typically are composed of Precambrian to Tertiary bedrock and represent the uplifted structural blocks. The intervening basins are filled with Tertiary to Quaternary volcanic and sedimentary materials and represent the down dropped blocks.

The Bradshaw Mountains in the site area are orientated north to north-northwest. The geologic units exposed in the Bradshaw Mountains primarily consist of Precambrian rocks with localized Cretaceous to Tertiary rocks. The Precambrian rocks include numerous igneous intrusives and a schist belt. The schist belt trends north-south and commonly separates the intrusives to the west from the Tertiary basin-fill rocks on the east. The schist belt includes rocks of the Big Bug Group of the Yavapai Series (Anderson and Blacet, 1972) and consists of schist and phyllite that are derived from a metamorphosed sequence of volcanic and volcaniclastic rocks and lesser amounts of sedimentary and local intrusive rocks. As a result of numerous structural disturbances through geologic time, the schists and phyllites are strongly foliated, tightly folded and locally faulted. The foliation planes commonly are steeply dipping to near vertically oriented.

The rocks of the Big Bug Group are further divided into the Spud Mountain Volcanics and the Iron King Volcanics (Anderson and Blacet, 1972). The Spud Mountain Volcanics is the older sequence and consists of lower and upper units. The lower unit includes metamorphosed rhyolite, andesite and basalt flows, breccias, tuffs and tuffaceous rocks containing some chert beds and local intrusives. The upper unit includes metamorphosed sedimentary rocks. The Spud Mountain Volcanics also includes a metamorphosed quartz porphyry intrusive unit. The Iron King Volcanics is the younger sequence and consists of metamorphosed andesite and basalt flows with minor interbedded rhyolite flows, chert and limestone.

The Precambrian igneous intrusives exposed in the Bradshaw Mountains west of the schist belt consist of numerous plutons of various compositions including quartz diorite, granodiorite, quartz monzonite, and gabbro. Localized quartz veins also are present in the area.

Younger rocks exposed in the general area include Cretaceous to Tertiary intrusives in the mountain block and Tertiary to Quaternary volcanic and sedimentary rocks primarily in the adjacent basin to the east of the Bradshaw Mountain block. The Cretaceous to Tertiary intrusives include granodiorite plutons and smaller bodies of quartz latite porphyry and rhyolite porphyry. The basin fill volcanic and sedimentary rocks include the Hickey and Mill Canyon formations and consist of basalt flows and cinders, latite, conglomerate, breccia, sandstone, marl, limestone and localized travertine.

Local Geology

The geologic units exposed in the three-mile section of SR69 just south of Mayer consist of an interstratified bedrock sequence of schist and

phyllite with localized quartzite, minor chert pods and a few altered basalt dikes. Relatively thin unconsolidated deposits of alluvium and colluvium locally overlie the bedrock within the Big Bug drainage path and side hill slopes. The bedrock sequence exposed along a majority of the alignment is mapped as the Spud Mountain Volcanic Sequence by Anderson and Blacet (1972). The rocks at the northwest end are mapped as the Iron King Volcanics. Rocks exposed in the two areas are similar except that schist is more abundant with lesser phyllite in the northwest section of the Iron King Volcanics. It is noted that Anderson and Blacet (1972) use a nomenclature that refers to the dominant parent rock type in the formational name rather than the metamorphosed product as presented herein.

The metamorphic rocks dominated by the phyllite and schist rock types are strongly foliated and locally faulted. The rocks in the southern half of the three mile section lie within the Shylock Fault Zone (Anderson and Blacet, 1972). The Shylock Fault Zone trends north-south. The foliation similarly strikes nearly north-south and dips steeply to the west at angles varying from about 65 to 85 degrees. Photo 1 shows the typical dip of the foliation and interstratified nature of the rocks in a cut slope at the northwest end of the metamorphic sequence in Section 26.

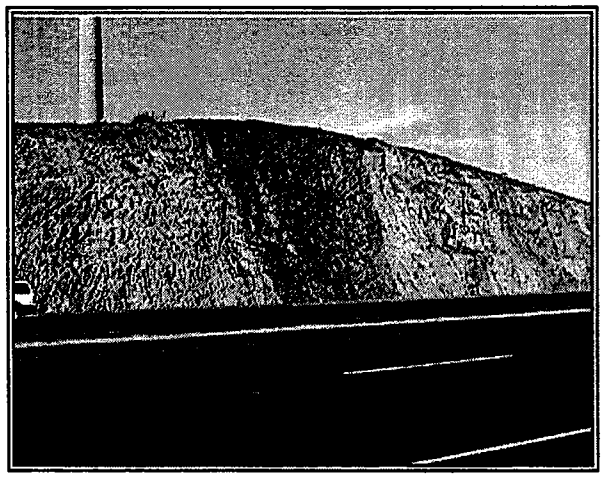

Photo 1. Steeply dipping foliation represented by near vertically oriented light and dark banding.

The phyllites and schists generally are moderately weathered to highly weathered. However, the quartzite units typically are less weathered. The phyllites and schists vary from light grayish brown to greenish gray with some light brown layers and considerable brown iron oxide staining. The quartzites are gray to dark gray with some purplish gray mottling. The phyllites and schists are soft to moderately hard with some very soft zones in the more weathered areas. The quartzites are hard to very hard.

The geologic units exposed in the larger cut in Section 31 include alternating blocks dominated by schist and phyllite with lesser amounts of the other type interstratified with the dominant type. Two localized basalt dikes also cut through the phyllite block in the central area of the large cut area. The basalt dikes are moderately altered and vary from moderately to slightly weathered in their interiors and highly weathered along their contact with the phyllite. The dikes have a porphyritic texture and are dark gray and hard. The areal distribution of geologic units along with the strike and dip of major joints in the large cut area is shown on Figure 2.

The phyllite and schist bedrock also was encountered in all the borings advanced within the bridge foundation areas located primarily within the Big Bug Creek drainage path. However, the bedrock was locally overlain by up to 43 feet of alluvium. The alluvial deposits consist of silty sand to sand and gravel with cobbles and some boulders at Bridge Sites 1 and 2. The deposits vary from moderately soft to very firm and medium dense to loose and are locally lime cemented.

The degree of weathering and hardness of the bedrock beneath bridge foundation areas varied considerably within each borehole as steeply dipping, closely spaced fracture zones of the schist/phyllite were penetrated. The weathering and hardness of the zones encountered within most borings varied considerably ranging from decomposed to slightly weathered and very soft to moderately hard. However, the rock mass predominantly was moderately to highly weathered and soft to moderately soft with the more highly weathered zones near the upper contact and harder and less weathered rock with depth. Only two borings drilled at Bridge Site No. 1, encountered highly weathered to decomposed bedrock to the full depth of investigation. Weathering of the rock was more apparent at Bridge Sites No. 1 and 2, than at Bridge Site No. 3. More widely spaced, less steeply dipping fracture zones were encountered at Bridge Site No. 3. Although core recovery from

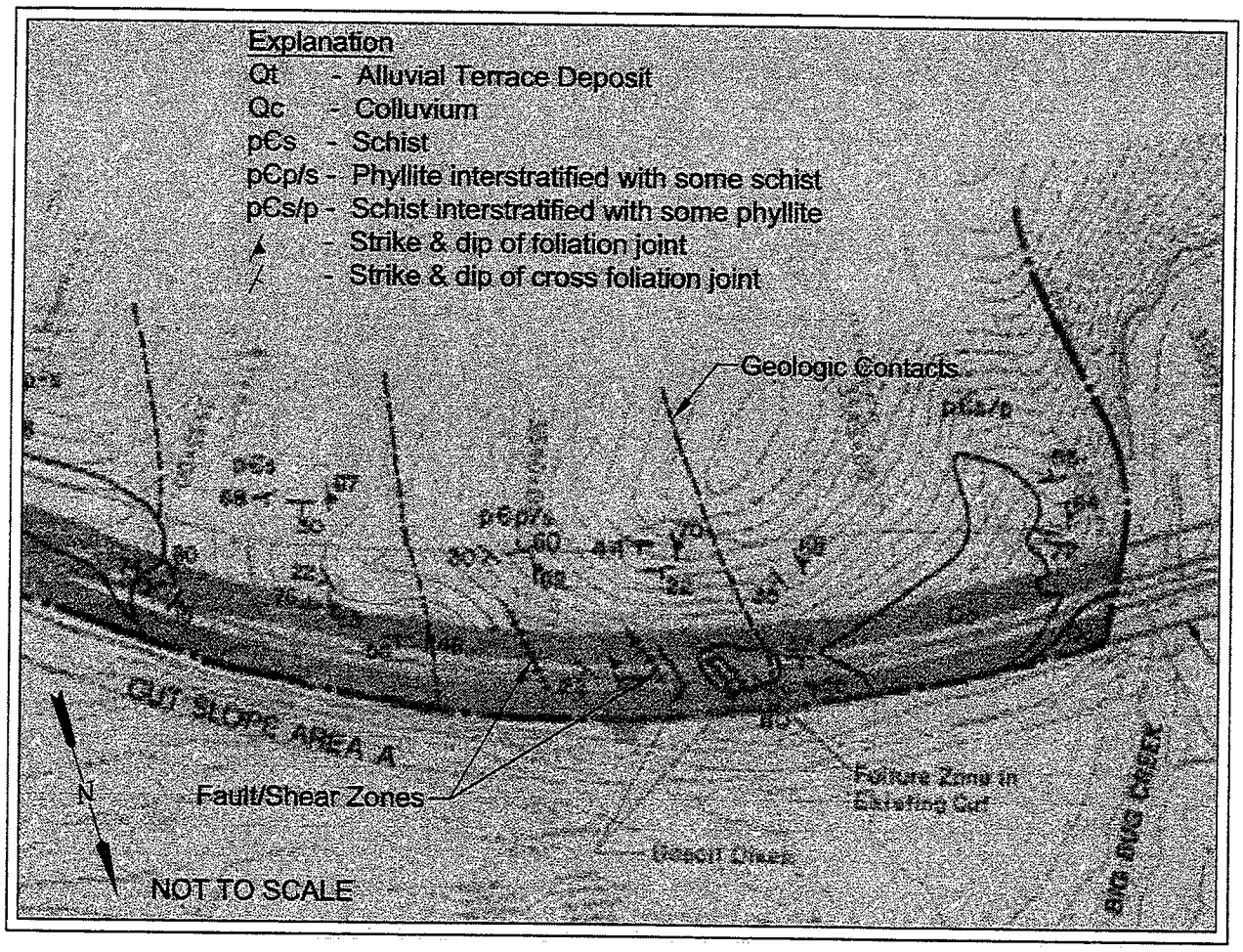

Figure 2.Geologic map of large cut area in Section 31.

borings drilled for the bridge footings varied from 0 to 100 percent, in general, recoveries exceeded 50 percent. Rock Quality Designation (RQD), which indicates the percentage of intact rock, longer than 4 inches, varied considerably in each borehole, dependent on the degree of weathering and spacing of fractures. Relative to bridge foundation design, the rock mass was, in general, considered to be highly weathered and soft.

Slope Stability of Large Cut Area

An initial slope stability analysis was performed for each proposed cut slope area along the nearly 3-mile section during the characterization. Each area was considered separate since the rock types and discontinuity conditions varied somewhat for each location. Additionally, the orientation of roadway alignment and associated cut slope face varied at each location and the most important factor to consider in the slope stability analysis is the relative orientation of joints to the cut slope face, since the most significant failures in this rock mass would occur along the primary joints. Only the analysis of the larger cut slope at the southern end of the project is presented herein, since it is the most interesting section due to a failure that occurred during construction. The cut in this area is 170 feet high and extends for a distance of approximately 1,300 feet along the alignment.

The phyllites and schists in the large cut area are mostly very closely (less than 0.2 foot) to closely (0.2 to 1.0 foot) fractured except in the failure zone where the fracture spacing widens to wide (3 to 10 feet) to very wide (greater than 10 feet). Measurements of 127 discontinuity planes were

obtained in the slope area to determine the general trends of the joints. The poles of the planes were plotted on a stereonet and then contoured with a 2 percent contour interval to locate averaged center points of the primary joint sets. Figure 3 shows the plot of the poles and Figure 4 shows the contouring with the selected center points.

The discontinuities occur both along and across the foliated fabric within the rock mass and include an occasional fault or shear zone. The joint set that occurs along the foliation (F-1) is the most dominant and consistent set of the jointed mass. The cross foliation sets (J-1 through J-3) are less continuous and are more variable in orientation as demonstrated by the larger scatter of poles on the above figures. The variation in the orientation of the cross foliation joints may be the product of folding of the rock mass with a shift likely occurring along the slope face. The orientation of the averaged center points based on Figure 4 is as follows:

SET NO.	STRIKE	DIP
F-1	N6E	64 W
J-1	N54W	38 NE
J-2	N66W	81 NE
J-3	N10W	44 NE
S-1	N14W	64 SW

Joint set S-1 represents a shear zone and is included as a separate set, although it is a single feature. A few other shear zones that occurred along the foliation joints were included in the foliation readings and not treated separately because they would behave similar to the foliation joints. The averaged planes were then plotted on a stereonet (Figure 5) along with the proposed cut slope to determine which joint sets could contribute to possible slope failures.

Since the slope occurs in a curved section of the alignment, the plot includes an orientation that represents both ends of the cut. The critical area shaded on the diagram is based on a friction angle of 29 degrees and 3/4:1 slope faces. Planes that intersect in this area represent potential wedge failures for the slope along the line of intersection. The dip of individual planes that fall within this area represent potential planar failures. As shown on the figure, two potential failures exist for this slope. One is a wedge failure created by intersecting cross foliation joint planes J-1 and J-3. The line of intersection formed by these two planes plunges 38 degrees in a N44E direction. The other is a planar failure along cross foliation joint plane J-1. The dip of this plane is 38 degrees in a N36E direction.

Factors of safety developed for the potential failures in this slope were 1.5 for the wedge failure and 3.5 for the planar failure. The factor of safety calculations were based on a friction angle of 29 degrees and cohesion of 1000 pounds per square foot (psf). These values were estimated from empirical strength parameters for a very closely fractured, argillaceous, lithified and poor quality rock mass developed by Hoek and Brown (1980). Additionally, since the cross foliation joint sets commonly are terminated on the foliation joint set and generally are not continuous for more than 10 to 12 feet, failures associated with these joints would involve only a small amount of material. Photo 2 shows the discontinuous nature of the joints in the finished face of the large cut area and typical small wedges formed by intersection of non-foliation joints.

Thus, the slope was designed at a 3/4:1 angle with a 30-foot collection ditch at the toe to contain small-localized failures and random rock falls that likely would periodically be generated from the closely fractured rock mass.

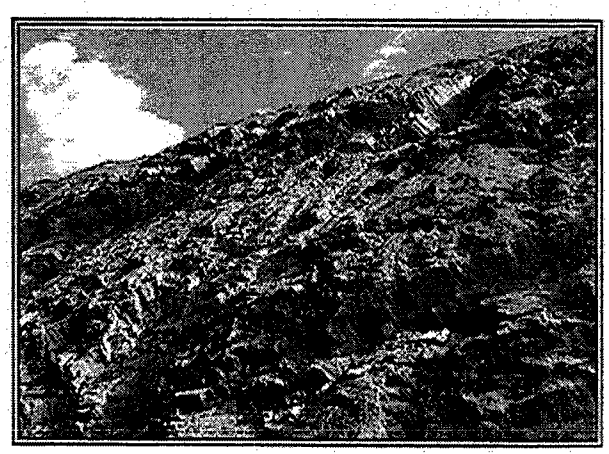

Photo 2. Discontinuous nature of joints and small wedges along intersections of joint planes.

During construction of this cut slope, a failure of an estimated 3000 cubic yards of material slid from the upper portion of the slope. Photographs of the failure at two scales are shown in Photos 3 and 4. The slide was a planar failure that slid along a cross foliation joint plane. The slide was about 90 feet wide along the slope face. Material

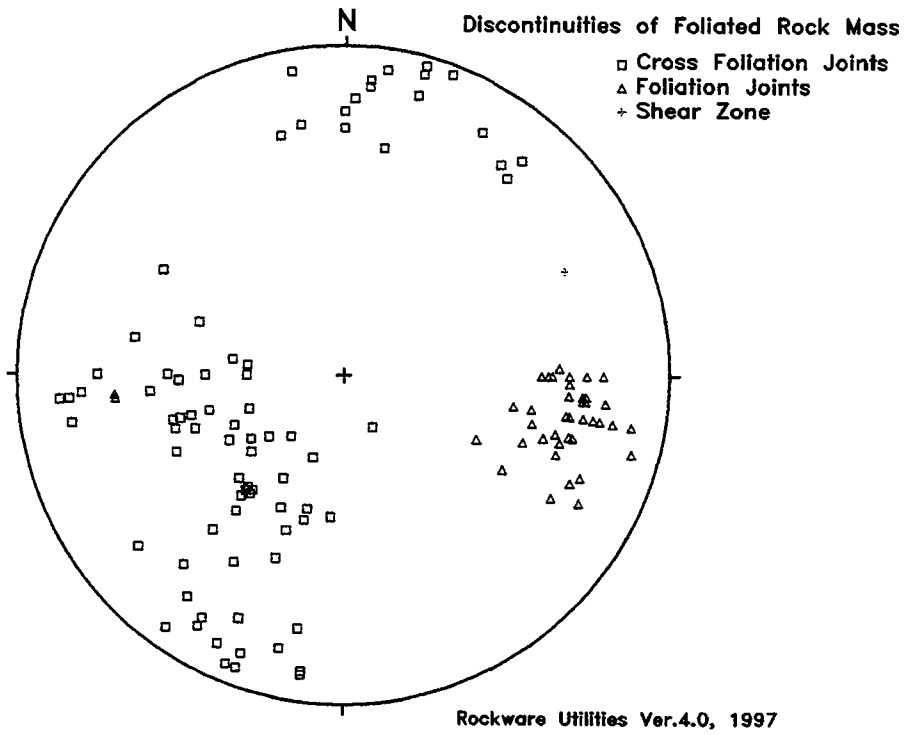

Figure 3. Plot of poles of discontinuity planes for large cut area

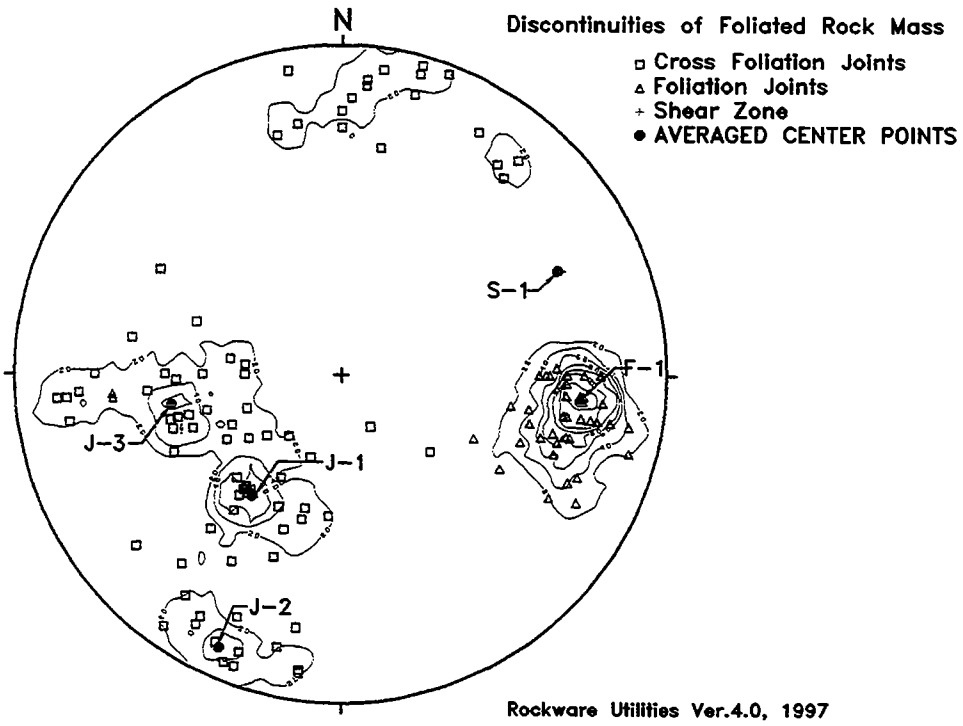

Figure 4. Contour of equal pole densities and averaged center points for large cut area

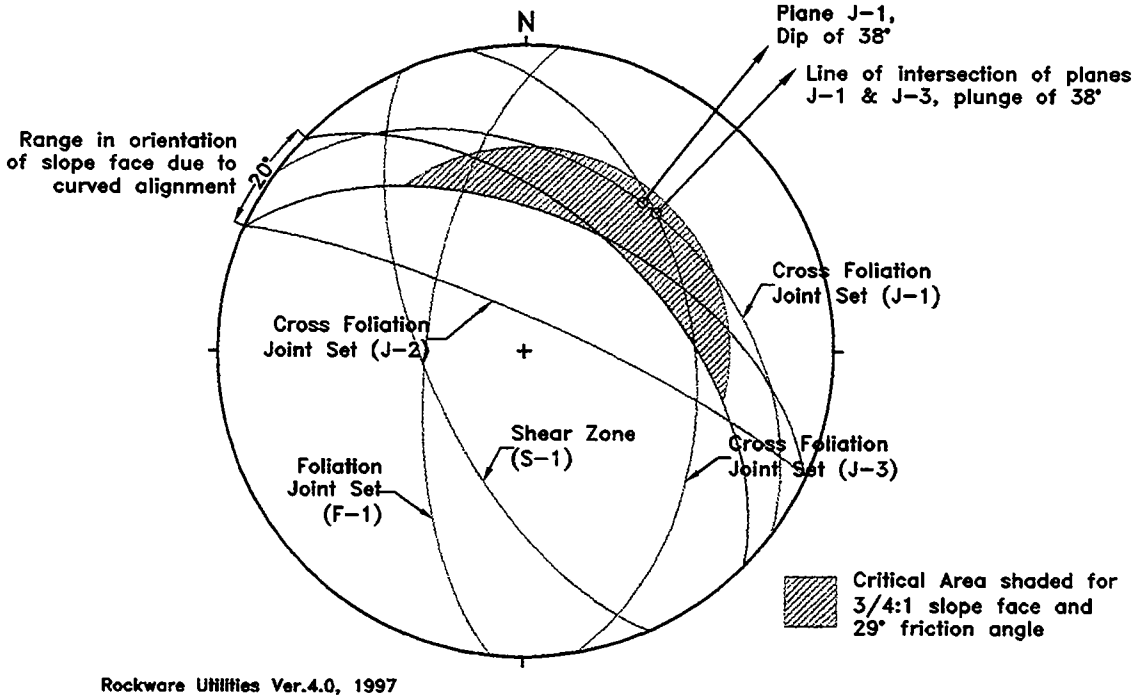

Figure 5. Stereographic solution of averaged discontinuity planes exposed in large cut area

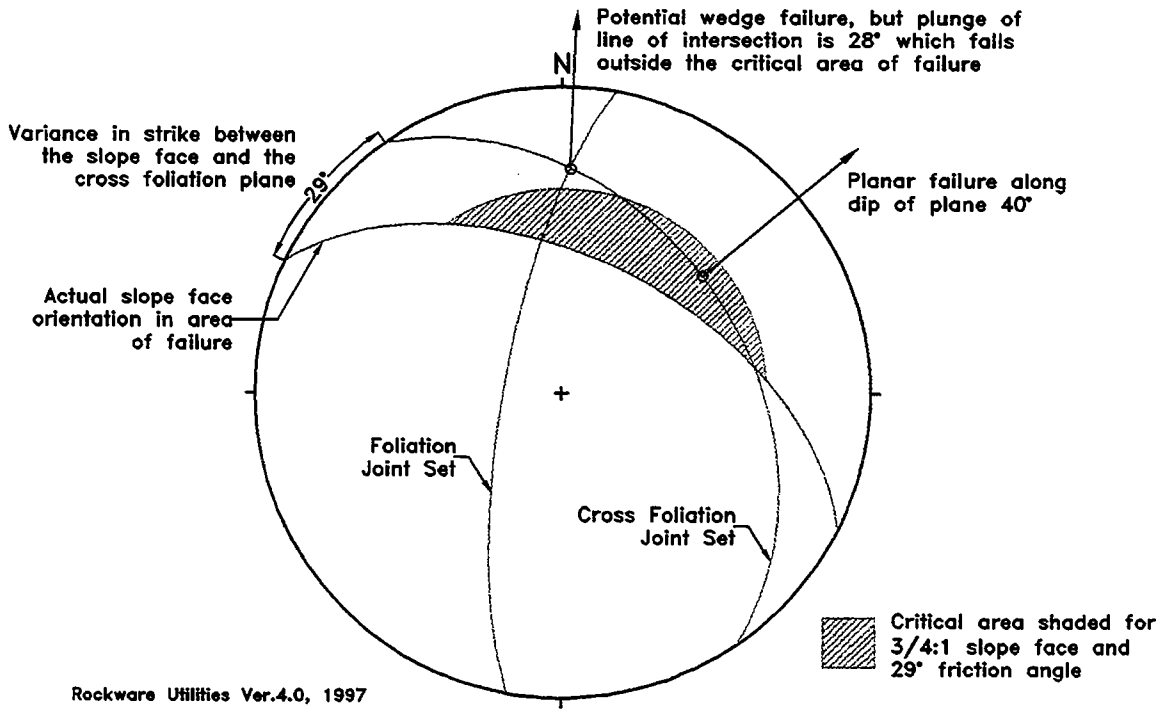

Figure 6. Stereographic solution of actual discontinuities involved in slope failure during construction of the upper slope

Photo 3. Planar failure with plane intersecting the slope face on the right and terminated on the foliation joint set at a deeper level on the left.

generated from the slid area was up to 23 feet across due to the wider spaced fracture pattern in the failure area. Figure 6 shows a stereonet plot of the planes exposed in the failure area along with the actual orientation of the slope face.

The figure shows a potential planar failure within the critical area for the slope, however, the strike of the plane varied from the slope face by 29 degrees. Classically, a planar failure will not occur if the strike of the plane varies by more than 20 degrees from the strike of the slope face (Hoek and Bray, 1981). However, in this case, the foliation joint set served to terminate the failure zone to the south. It is likely that the initial separation of the more continuous foliation set which facilitated the sliding of this large block was prompted as a result of excessive blast energies propagating into the slope along the planes. Inspection of the slope in areas adjacent to the slide revealed blast related openings of up to 6 inches wide and at least 5 feet deep along near vertical fractures projecting into the final slope face. It is possible that propagation of blast energy into the slope face may have been more pronounced in the failure area, since the fracture spacing is wider in this zone. In this case, the energy would concentrate along the few fractures rather than dispersing in a more closely fractured media.

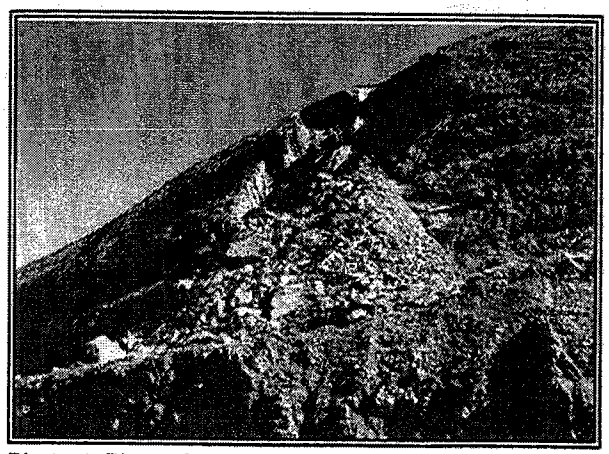

Photo 4. Planar failure in upper portion of the slope.

To remediate the slide mass and decrease the risk of additional localized failures, a benched geometry was constructed in the upper portion of the slope. A cross section (Figure 7) through the failure area shows the failed zone and remedial design geometry.

The bench was designed to fully remove the slide mass and ultimately to decrease the overall slope angle to stabilize possibly other similar conditions, if present. The bench will also serve as an additional collection point for debris from small-localized failures that may occur above the bench, thus restricting material from reaching the toe of the slope. Photo 5 shows the final slope geometry with the bench located at about mid height of the overall slope.

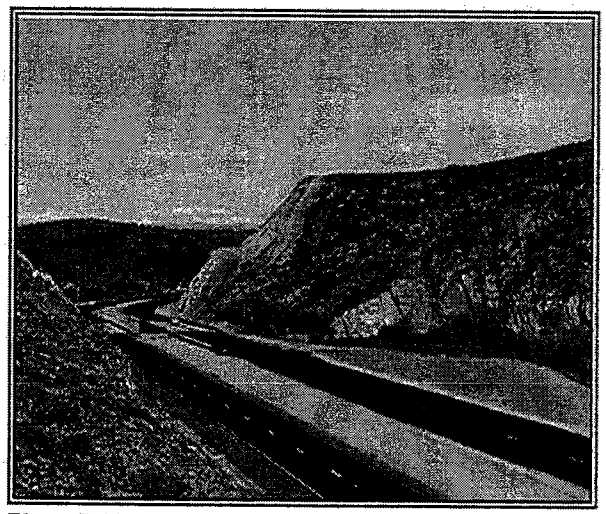

Photo 5. New benched geometry of large cut.

Concerns during remedial construction of the benched geometry included a new blasting plan so as not to weaken the rock mass behind the final

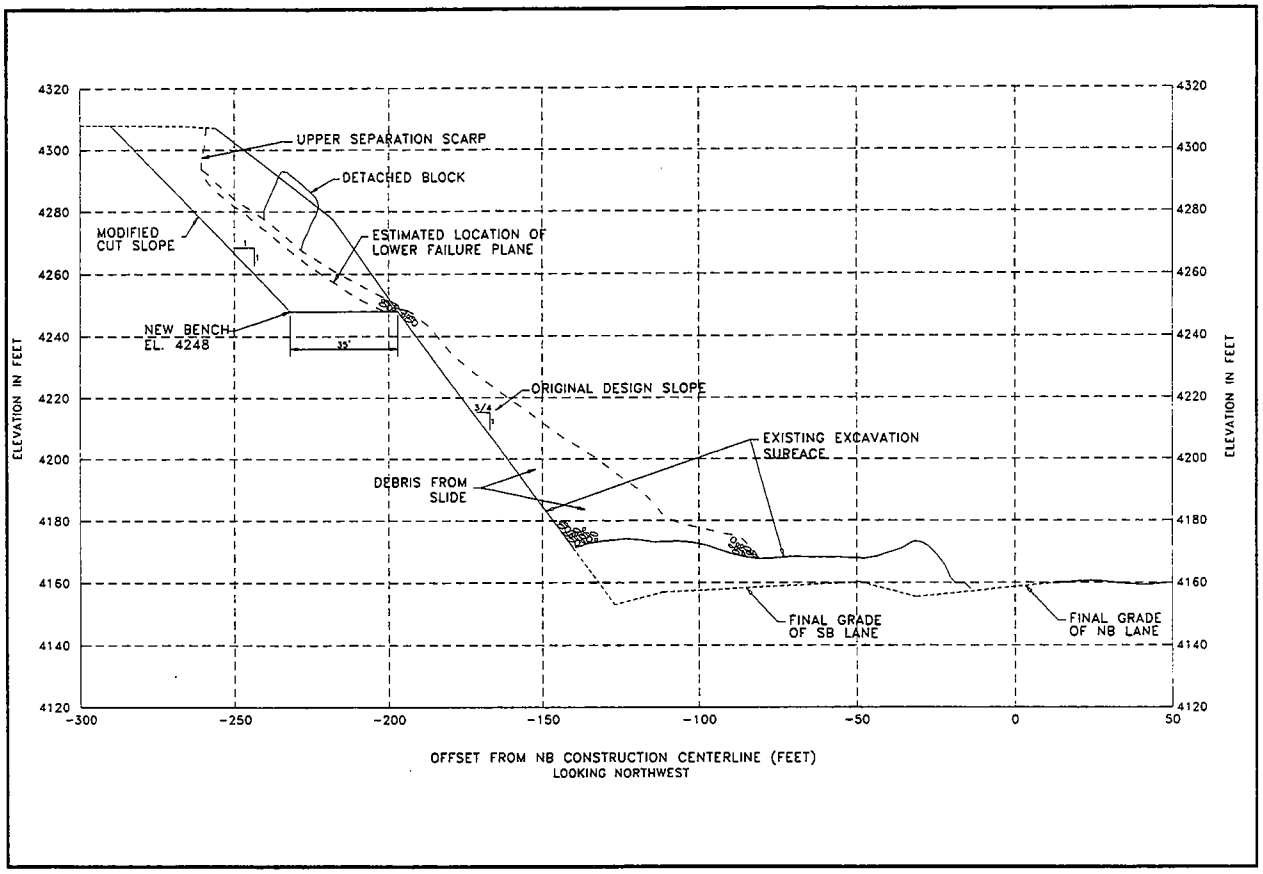

Figure 7. Cross-section of large cut area showing failure zone in the upper slope area, original designed slope face and modified cut slope for removal of slide.

slope face, and safety of both road users and construction workers. The various safety issues included:

- Preventing large blocks generated from the loose debris in the failure zone from rolling on to the traveled roadway;
- Controlling the discharge of newly excavated materials from above the bench over the partially completed slope in a manner as to prevent the debris from reaching the traveled roadway; and
- Placing excavation equipment on the narrow bench as the excavation proceeds to its final grade with a particular focus on stability over the previously failed area.

Bridge Foundations

The field investigation for Bridge Sites 1 through 3 was performed jointly by AEE and the Arizona Department of Transportation (ADOT) Materials Section. The test boring locations for each of the bridge structures were selected during a site visit by David E. Peterson, P.G. and Keith H. Dahlen, P.E., both of AEE and G. Bruce Kay, P.E., with the ADOT Materials Group. The locations were selected in terms of accessibility and for conformance to American Association of State Highway Transportation Officials (AASHTO) and Arizona Department of Transportation (ADOT) guidelines for geotechnical investigations (ADOT, 1991). In general, the guidelines require that at least one test boring be drilled within 10.0 feet of each bridge abutment and pier foundation location, for structures of less than 100 feet in width.

The depths of the borings varied from 29 to 92 feet below site grades, depending mostly where drilled. The deeper borings were drilled on or adjacent to the existing roadway and the shallowest borings were drilled near the bottom the creek, typically for the new center piers. The depths of the borings were established so that the tip elevation of each boring extended at least 10.0 feet below the bottom of the proposed foundation element (either drilled

shaft rock socket or spread-type footing). The depths of the borings were adjusted in the field, based on the contact elevation, the degree of weathering and hardness of the materials encountered. Where very soft, highly weathered to decomposed bedrock was encountered at the intended tip elevation, the depth of the boring was often increased.

The selection of a foundation type for each of the structures was based on many factors. However, scour considerations at each of the structures, dictated that the foundations be supported directly within the bedrock, with some depth of penetration to account for additional minimal scour of the rock contact surface. The highly variable elevation of the rock contact and degree of weathering made the use of drilled shaft rock sockets the preferred foundation type over spread-type footings. Rock sockets, designed as shear sockets develop relatively high capacities with minimal penetration into rock (2 to 3 times the shaft diameter, D). The capacities developed in sockets are based on the average unconfined compressive strengths of the materials penetrated. Thus, relatively consistent capacities developed from one shaft to another. Spread-type footings, on the other hand rely more on the materials directly underlying the footing for support. Differences in material hardness beneath footings could result in differential settlements, whereas settlements of properly constructed rock sockets are typically negligible (i.e. less than about 1/2 inch).

The terrain, geologic and hydrologic conditions present at each of the bridge sites, especially Bridge Sites 1 and 2, made the use of rock sockets preferable over footings as well. Considerable unstable overburden (up to 43 feet) would require massive excavations, some involving shoring to access the intended footing elevations. Shallow groundwater present most of the year would also dictate that the excavations be dewatered with pumps. The highly variable bedrock contact elevation would require that considerable bedrock be excavated in some portions of the individual footings to ensure that the entire footing was founded in rock. Photo 6 shows the final center piers supporting Bridge No. 2.

The drilling apparatus and methods of installation also make the use of drilled shaft rock sockets appealing in terms of cost. The drill rig used to advance the sockets, called "Big Stan" owned and operated by Anderson Drilling Co., is capable of developing high torque and crowd, necessary for advancement into bedrock. Using a variety of drill bits, including rock core bits, and casing with or without slurry, the large excavations, which would otherwise be required with spread footings, are not required. Construction of the majority of the shafts for this project involved the use of temporary steel casing advanced to the bedrock contact. The temporary casing is used to prevent excessive caving of the coarse grained overburden soils, and permits the installation of a much thinner walled temporary/permanent inner casing. Temporary casing, depending on the depth of overburden, and the length of available casing is often telescoped into the excavation and is not seated into the bedrock. Using this method, groundwater, if present, is not prevented from entering the shaft excavation. Within the thick walled temporary casing, the casing (either temporary or permanent) consisting of corrugated metal pipe (CMP) approximately 6 inches larger in diameter than the design shaft diameter is seated a minimum of 6 inches into the rock. The depth of penetration of the casing into the rock is based on the uniformity of the contact surface. The larger casing dimension permits the drill bit to advance to the design tip elevation within the bedrock, with less risk of hanging up on the casing when extracting drill cuttings. The intent of the permanent casing is to ensure uniformity of the shaft by preventing the influx of sand and gravel sized materials which may be present inside the temporary casing, and to seal and reduce the amount of water within the shaft

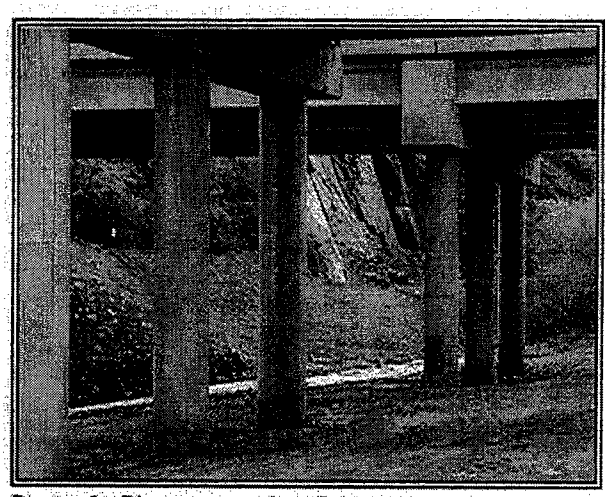

Photo 6. Piers supporting Bridge No. 2 with steeply dipping foliated rock mass in background.

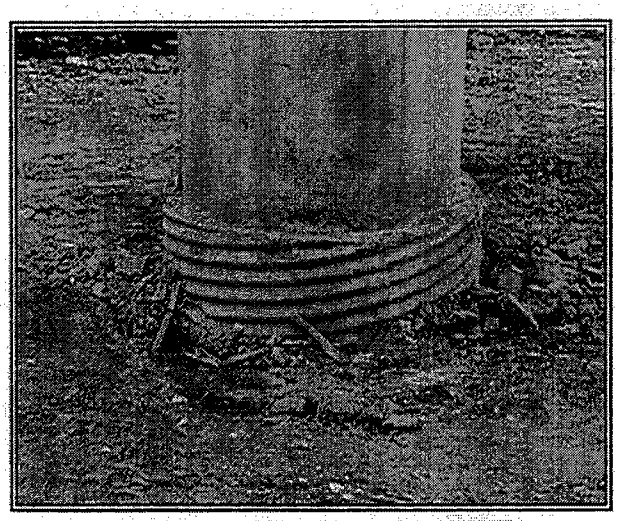

Photo 7. Permanent CMP casing left in place.

excavation. The inner CMP casing can be pulled from the excavation in concert with the concrete pouring operations (provided the temporary casing is still in-place) or be left in-place permanently as was done for this project. Photo 7 shows a CMP casing left in place on Bridge Site 2.

The rock sockets were designed for vertical capacity by methods presented by Carter & Kulhawy (1988), based on a limited number of unconfined compressive strength and point load index tests. Very few suitable intact rock cores of highly weathered schist/phyllite were available, therefore a conservative compressive strength value of 1,200 psi was selected for design. The actual unconfined compressive strength test values ranged from 837 pounds per square inch (psi) to 5,975 psi, with an average of 3,550 psi. A minimum shaft penetration of about 2D is used for design of rock sockets, depending on the hardness and uniformity of the rock. For this project, a minimum shaft penetration of 3D was recommended based on the variable hardness and weathering conditions and the assumption that the overall rock mass was generally highly weathered. Shaft capacities for rock sockets of 4, 5 and 6 feet in diameter were developed resulting in capacities of about 150 kips, 190 kips and 225 kips, per foot of embedment for the respective shaft diameters. Based on a minimum penetration of 3D the capacities of the respective shafts were estimated at about 1,800, 2,800 and 4,000 kips. The structural design consultant, Sverdrup Civil, Inc., then selected the appropriate drilled shaft diameter based on the bridge loadings, the number of shafts to be utilized and general aesthetics, scour and structural connection considerations.

Conclusion

Highway construction through a strongly foliated and Precambrian metamorphic assemblage of interstratified phyllite and schists may require some special design considerations in order to provide a safe corridor for public use. Construction of an approximate 3-mile segment of the new SR69 alignment just south of Mayer, Arizona encountered a strongly foliated and jointed rock mass. Special considerations for this section included rock mass characterization for both bridge foundations and stability of cut slopes.

Concerns for bridge foundations include depth to rock, nature of the overburden and, in this case, the variability of the weathering and hardness across and along the foliation joint sets in order to establish foundation types suited for the conditions. These conditions impact both rock strengths and construction methods. The construction method used for all bridge piers on this project consisted of drilled shafts socketed into rock, since the depth of rock was locally up to 43 feet deep and the upper contact typically was highly weathered and soft with a gradual transition into more competent rock with depth. Since the overburden materials were saturated and locally loose and subject to caving, the drilled shafts were cased to maintain borehole walls.

Concerns for stability of cut slopes in rock included characterization of the rock mass including nature of the joint system, orientation of cut face and modification of construction techniques to prevent deterioration of rock to be left in the final slope face. A detailed analysis of the joint system contained in the rock mass including significant joints or faults that could contribute to slope failures was initially performed prior to construction. An additional analysis was performed during construction following a local failure. The initial study revealed the existence of two potential failures including a wedge and planar type. However, since the joint sets involved in the potential failures were the cross foliation sets, which typically are discontinuous over short distances and highly irregular due to rock

mass folding, the failures would be very small and localized. The small failures also would be contained in the 30-foot wide collection ditch recommended at the toe of the slope.

The follow up study during construction revealed that a planar failure that normally would not be expected, occurred in the slope. This failure was not expected since the orientation relative to the slope was greater than 20 degrees. However, this failure was facilitated by existence of the more continuous crossing joint set that was opened during blasting activities and allowed separation of the slide mass from the intact rock. Variations in rock mass conditions, including a change in fracture spacing, impacted blasting effectiveness. It appears that the blast energy propagated into the rock behind the slope face in the more widely spaced zone. Remediation of the slope failure consisted of removal of the slide mass by construction of a bench at approximately mid height and building the final face without the use of blasting along the slope face in order to maintain rock strengths along the face.

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Site Characterization and Slope Stability Evaluation for the San Francisco-Oakland Bay Bridge

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ABSTRACT

The Bay Bridge connects the cities of Oakland and San Francisco by spanning San Francisco Bay. The western abutment of the east span of the bridge is founded on Yerba Buena Island. In 1989 the Loma Prieta Earthquake damaged a section of the eastern span. Studies for the seismic retrofit of the bridge started soon after the earthquake. This paper discusses the site characterization and slope stability evaluation of the western abutment, conducted as part of the retrofit study.

The bedrock in the Bay area is Jurassic to Cretaceous age rock of the Franciscan assemblage. In the Bay much of this rock is covered by poorly consolidated Quaternary age sediments. The two main structural features in this area are the San Andreas fault to the west and the Hayward fault to the east. On Yerba Buena Island the abutment is founded on a gently sloping (approximately 20°) rock surface. The rock consists of sandstone with minor mudstone interbeds. On the island the bedding dips fairly consistently to the northeast and the rock is fractured by at least four other joint sets. Paleotopography in the Quaternary sediments indicates that the present topographic configuration of Yerba Buena Island is at least 11,000 years old, and could be more than 200,000 years old. The geomorphology of the island shows no evidence (scarps, hummocky topography, closed depressions, etc.) of large-scale deep-seated slope failures.

There have been a number of large historic earthquakes on both the San Andreas and Hayward faults. The maximum earthquake magnitude for these faults is estimated to be M_w 8 for the San Andreas and M_w 7.5 for the Hayward. Both of those faults could produce earthquake accelerations of 0.5g (both horizontally and vertically) on the island. The estimated recurrence interval of those large earthquakes is 250 to 500 years for the San Andreas and 150 to 500 years for the Hayward. Therefore, the island has probably been subjected to shaking from 40 to 120 large earthquakes during the past 11,000 years (the minimum age of the island). Eyewitnesses to the 1906 San Francisco earthquake on the island did not report any landslides, scarps, ground cracks, etc.

The laboratory testing for this investigation concentrated on the rock since its shear strength controls slope stability on the island. Shear strength parameters (unit weight, friction angle, cohesion, etc.) were estimated with point load tests, unconfined compression tests, direct shear tests, and density tests performed on representative rock samples. The soil shear strength parameters for the Bay sediments were obtained from extensive testing performed for the foundation investigation for the retrofit.

Two different types of stability analyses were conducted for this investigation. First, kinematic analyses were performed to determine what types of failures (rock movements), if any, are possible in this area. Second, limit equilibrium analyses (a static resolution of forces) were conducted to analyze potential slip surfaces under pseudostatic earthquake loads.

In conclusion the geomorphology, the paleotopography, and the stability analyses indicated that the slope that the bridge is founded on will remain stable even during a major earthquake.

INTRODUCTION

The Bay Bridge connects the cities of Oakland and San Francisco by spanning San Francisco Bay. The western abutment of the east span of the bridge is founded on Yerba Buena Island. In 1989 the Loma Prieta Earthquake damaged a section of the eastern span. Studies for the seismic retrofit of the bridge started soon after the earthquake. Of particular interest were two bridge piers, E-1 and E-2, founded on rock at the eastern edge of the island.

Initially, documents related to the design, construction, and financing of the bridge were reviewed. Those documents included the preconstruction site investigation (Lawson 1932), contract plans and post construction insurance documents (California Department of Public Works, 1936). Also reviewed were some of the documents containing the data collected for the so-called "Parallel Crossing" (California Department of Public Works, 1949) which was never built. Relevant data from those documents were incorporated into this study.

Additionally, previous geologic mapping (Radbruch, 1957, and California Department of Transportation, 1993) was field checked. During the field check, data were also collected on rock discontinuities (orientations, continuity, spacing, roughness, bridges, etc.). The subsurface materials at this site were explored with 9 test borings which were drilled as part of this investigation and the investigation for the seismic retrofit of the East Bay Bridge. Besides visually logging the cores from these borings, three of the holes were logged with an oriented video. Seismic reflection work was also performed along the east span of the bridge (University of California, 1995).

GEOLOGY

Yerba Buena Island is located in San Francisco Bay midway between the cities of Oakland and San Francisco. The topography of the island's western portion is steep and rugged and the eastern portion is much more subdued, Figure 1. Pier E-1 is located on the eastern tip of the island and Pier E-2 is located just offshore.

The bedrock in the Bay area is Jurassic to Cretaceous age rock of the Franciscan assemblage. Much of this rock is covered by poorly consolidated Quaternary age sediments. The two main structural features in this area are the San Andreas fault to the west and the Hayward fault to the east (Bowen and Crippen, 1951). Between these two faults the bedrock was depressed during Pleistocene time, producing San Francisco Bay. The bedrock depression deepens to the east toward the Hayward Fault (Radbruch, 1957). In the Bay Area, the bedrock contains a series of northwest-southeast trending synclines and anticlines which were mapped by Schlocker (1974).

The geology of Yerba Buena Island was most recently mapped by Radbruch (1957). Part of this study include updating Radbruch's map, Figure 2. During the field work, the orientations of 356 discontinuities (joints, bedding planes and faults) were measured and a representative sampling of these discontinuities are shown Figure 2.

Piers E-1 and E-2 are founded on a sloping bedrock surface that dips 16° to 18° to the northeast. Out in the Bay the bedrock is covered by younger sediments. The surface and subsurface features for this site are shown in Figure 3.

Stratigraphy. For the purposes of this project three geologic units were used: bedrock, surficial (land) deposits, and bay deposits. The bedrock terminology follows the precedent established by Schlocker (1974). The terminology for the surficial deposits was established by Radbruch (1957). The stratigraphy of

Figure 1. Location map of Yerba Buena Island.

the Bay deposits shown on Figure 3 are based on the terminology established by Rogers and Figuers (1991).

The bedrock at this site is part of the Franciscan assemblage and the bedding generally dips to the northeast (Figure 2). This rock is massive- to very thickly-bedded graywacke (sandstone) with interbedded thin shale beds and it is shown on the geologic map as unit "Jkss". No mappable units were found within the bedrock due to limited exposures on the island.

The bedrock is fractured by a joint set parallel to the bedding and at least 4 other joint sets (Figure 3). Fracture lengths were measured at various bedrock exposures scattered throughout the island. The bedding fracture lengths ranged from 2 to 100 ft (0.6 to 31 m) long with the mean length approximately 40 ft (12 m). Joint lengths ranged from 1.5 to 50 ft (0.5 to 15 m) with a mean 11 ft 3.3 ft). The lengths of the intervening bridges of intact rock between the fractures were determined with detailed geologic mapping and measurements of the near vertical cut face between Piers E-1 and E-2. The bridge lengths along the bedding ranged from 0.1 to 5 ft (0.03 to 1.5 m) with an average of 1.9 ft (0.6) and along the joints they ranged from 0.2 to 4 ft (0.06 to 1.2 m), averaging 1.25 ft (0.4 m). Along the bedding plane fractures there is an estimated 5% of intact rock and along the joints there is approximately 25% of intact rock. The measured joint roughness ranged from 2° to 9.5° with a mean of 5°.

Landslide deposits (Qls) are shallow, small scale and limited to the steep topography on the western portion of the island. These deposits consist of silty sand and angular rock fragments. Colluvium and reworked colluvium (Qrc) was derived from weathered material and then reworked (by gravity, water and wind) bedrock. This deposit consists of silty sand with scattered rock fragments and varies from a few feet to over 120 ft (37 m) in thickness. Artificial fills (Qaf) consist of locally quarried materials (silty sand and rock fragments) and construction rubble.

The Young Bay Mud is the youngest (11,000 years ago to present) of the deposits in the Bay (Figure 3). This deposit consists of very soft to soft silty clay and clayey silt. This unit was deposited on erosional surfaces incised into the underlying Possey/Merrit/San Antonio formation.

What has been interpreted (Rogers and Figuers, 1991) to be the Possey/Merrit/San Antonio formation is stratigraphically just below the Young Bay Mud. This deposit was formed 90,000 to 11,000 years ago and consists of medium to dense sand. In the channel just offshore of the eastern tip of the island, recent drilling by Caltrans indicates that the Possey/Merrit/ San Antonio formation is missing. Either it was removed by erosion or was never deposited in this area.

The Yerba Buena Mud (Old Bay Mud) is just below the Possey/Merrit/San Antonio formation. This deposit is mostly stiff silty clay which was formed 125,000 to 100,000 years ago (Rogers and Figuers, 1991). This unit was deposited on an erosional surface in the underlying Alameda formation.

The Alameda formation is just below the Yerba Buena Mud (Old Bay Mud). This deposit is mostly dense silty sand which was formed 1,000,000 to 200,000 years ago (Rogers and Figuers, 1991). This formation was deposited on an erosional surface in the underlying Franciscan assemblage.

Structure. The bedding attitudes have a fairly consistent northeast dip throughout the island. This northeast dip could indicate that the axis of an anticline (possibly the same one mapped on Angel Island by Schlocker, 1974) runs just west of the island. Exposures of the bedrock in the cliffs along the southwest side of the island contain numerous bedding plane faults and high angle faults. These faults do not appear to extend into the surficial deposits. Therefore, these faults are not considered to be active. Drag folding is associated with some of the high angle faults.

Paleotopography. A channel up to 200 ft (61 m) deep was eroded in the Alameda formation just east of Yerba Buena Island (Rogers and Figuers, 1991) at the end of Alameda time (approximately 200,000 years ago). By Wisconsin time (approximately 11,000 years ago) the main trunk of the south Bay drainage flowed to the north between the island and San Francisco. Also at that time Temescal Creek made a sharp turn to the south passing east and south of Yerba Buena before connecting with the main north flowing drainage (Rogers and Figuers, 1991). Therefore, from above data it can be concluded that the present topographic configuration of Yerba Buena Island is at least 11,000 years old and it could be more than 200,000 years old.

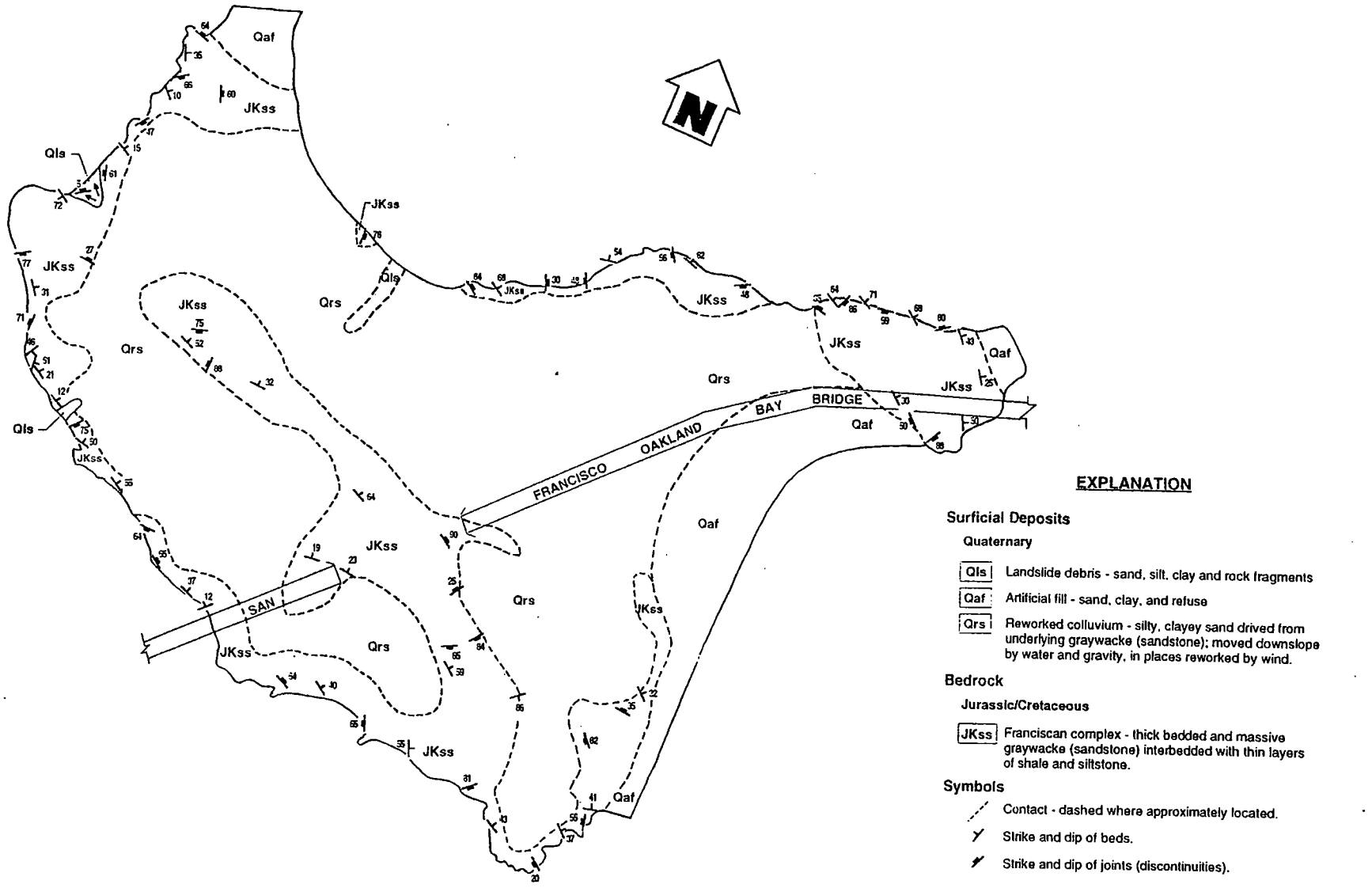

Figure 2. Geologic Map of Yerba Buena Island (After Radbruch, 1957).

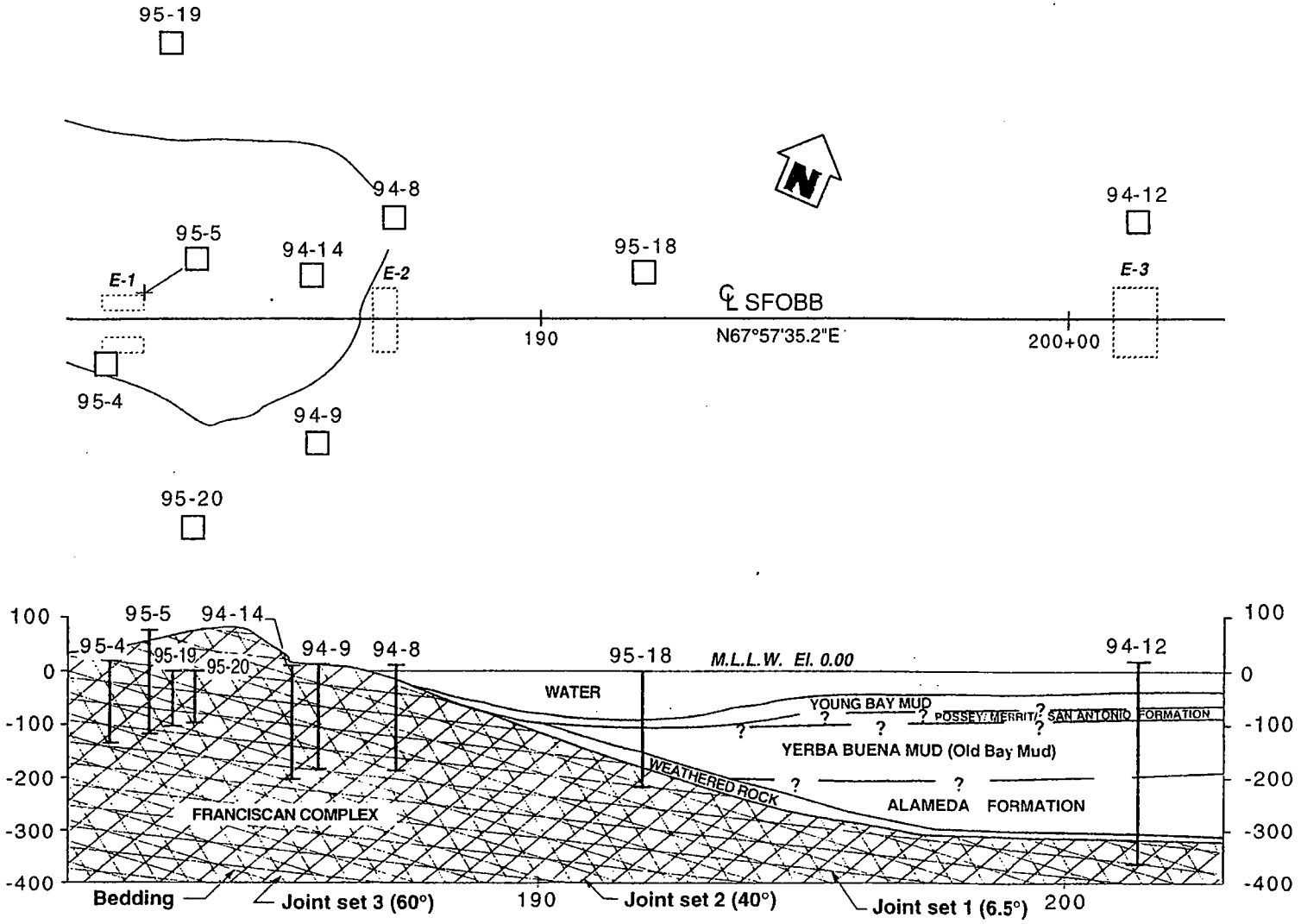

Figure 3. Geologic cross-section of study area.

Geomorphology. There are several shallow small-scale landslides confined to the steep topography on the western portion of the island. However, neither the field work nor a review of the air photo stereo pairs indicate that there is any evidence (scarps, hummocky topography, closed depressions, etc.) for large-scale deep-seated slope failures on the island.

SEISMICITY

Yerba Buena Island is located 11.5 mi (18.5 km) northeast of the San Andreas fault and 7.2 mi (11.6 km) southwest of the Hayward fault. There have been several large historic earthquakes in this area, including the 1838, 1865, 1906 and 1989 events on the San Andreas fault (Tuttle and Sykes, 1992) and the 1836 and 1868 events on the Hayward fault (Oakeshott, 1978). The maximum earthquake magnitude is estimated to be Moment Magnitude (M_w) 8 for the San Andreas fault and M_w 7.5 for the Hayward fault (Mualchin, 1995). Previous work (California Department of Transportation, 1995) concluded that either of those faults could produce earthquake accelerations of 0.5g (both horizontally and vertically) on the island.

The estimated recurrence interval is 250 to 500 years for a M_w 8 earthquake on the San Andreas fault (Bakun, 1992; Youngs, et al, 1992) and it is 150 to 500 years for a M_w 7.5 earthquake on the Hayward fault (Lienkaemper and Borchardt, 1992; Williams, 1992; Borchardt, Lienkaemper, and Budding, 1992). Therefore, Yerba Buena Island has probably been subjected to shaking from 40 to 120 large earthquakes during the last 11,000 years (the minimum age of the island).

Eyewitnesses to the 1906 San Francisco earthquake (M_w 8) on Yerba Buena Island did not report any landslides, scarps, ground cracks, etc. (Lawson, 1908). Since there is no geomorphic evidence for landslides it is reasonable to conclude that the island is not susceptible to earthquake induced landsliding.

MATERIAL PARAMETERS

The testing for this investigation concentrated on determining the shear strength parameters of the rock on Yerba Buena Island, since rock strength is a controlling factor in the slope stability in the area of Piers E-1 and E-2. In order to quantify rock shear strength parameters, representative core samples were tested.

Approximately 90 point load tests were performed on the core samples from this site. The point load test method used was based on that proposed by the International Society for Rock Mechanics (1985) and used to estimate unconfined compressive strength (q_u). The mean Point Load Index of these tests was approximately 58 ksf (2.8 MPA) and the mean q_u for the rocks at this site was estimated to be 1368 ksf (65.5 MPa). A cohesion of 375 ksf (18 MPa) was calculated for these rocks using the unconfined compressive strength and the friction angle.

Core samples were measured and weighed to determine the unit weight for the rock on the island. The average unit weight for the 43 samples measured was roughly 166 pcf (26.1 kN/m^3).

A total of 36 unconfined compression tests were conducted on the core samples from this site. The mean unconfined compressive strength from these tests was approximately 950 ksf (45.6 MPa). The samples selected for these tests were skewed toward near-surface, weaker rocks to fit the purposes of the retrofit studies.

Direct shear tests were performed on 5 sandstone and 4 siltstone/shale core samples from this site. As suggested by the Federal Highway Administration (1989), these tests were done on sawn surfaces. The mean residual friction angle from these tests was determined to be 33° for the sandstone and 26° for the siltstone/shale.

The soil shear strength parameters (unit weight, friction angle, cohesion, etc.) for the bay deposits were determined from testing conducted during previous investigations. Insitu and field testing included standard penetration tests, pocket penetrometer tests, torvane tests, and pressuremeter tests. Laboratory testing determined unit weights, moisture contents, void ratios, grain sizes, Atterberg limits, consolidation, direct shear strengths and unconfined compression strengths. For this investigation, the test results were reviewed and representative shear strength parameters were chosen for the soils at this site. Table 1 lists the subsurface deposits and their shear strength parameters.

Table 1. *Soil properties used in stability analyses.*

Formation	Soil Type	Unit Weight pcf (kN/m³)	Friction Angle deg	Cohesion ksf (kPa)
Young Bay Mud	Very Soft Silty Clay	100 (15.7)	NA	0.25 (12)
Possey/Merrit/San Antonio Formation	Medium to Dense Sand	115 (18.1)	32	NA
Yerba Buena Mud (Old Bay Mud)	Stiff Silty Clay	105 (16.5)	NA	1.50 (72)
Alameda Formation	Dense Silty Sand	125 (19.6)	35	1.00 (48)
Weathered Rock	Very Stiff Silty Clay	120 (18.8)	NA	2.20 (106)

STABILITY ANALYSES

Two different types of stability analyses were conducted for this investigation. First, a kinematic analysis was performed to determine what types of failures (rock movements), if any, are possible in the area of Piers E-1 and E-2. Second, a limit equilibrium analysis (a static resolution of forces) was conducted to analyze potential slip surfaces under pseudostatic earthquake loads. The following is a more detailed discussion of these stability analyses.

Kinematic Analysis. Hard rock slopes usually fail along discontinuities (joints, bedding, schistocity, faults, etc.) that form blocks which can slide or rotate out of the slope. There are three common types of slope failures for fractured rock: (1) plane failures, (2) wedge failures and (3) toppling failures (Hoek and Bray, 1974). Plane failures occur when: the dip directions of the fractures and the slope face are within 20° of each other; the fractures intersect ("daylight") in the slope face; and the fractures dip steeper than the friction angle. Wedge failures occur when a line of intersection of two fractures "daylights" in the slope face and it dips steeper than the friction angle. Toppling failures occur in closely fractured rock where: the fractures dip into the slope; the dip directions of the fractures and the slope are within 20° of each other; and the fractures dip steeper than the friction angle.

Kinematic analysis is a method of determining whether those types of failures are possible (Goodman, 1989). This method uses a graphical device, stereographic projection, in which the lines of latitude and longitude from the lower half of a sphere have been projected onto a piece of paper (a stereonet). With the stereonet, planes (joints, bedding, faults, etc.) of any orientation can be represented as great circles (curved lines) and lines (discontinuity intersections, discontinuity poles, etc.) as points.

Figure 4 is a density contour, developed from a stereonet plot, of the joint and bedding attitude poles that were measured on the island. Also shown on the density contour are the great circles representing the four joint sets that appear to control the slope stability at this site. In addition, the great circles representing the bridge centerline and the bedrock surface are shown. The stereonet plot indicates that the common types of rock slope failure are extremely unlikely in the area of Piers E-1 and E-2.

However, Goodman (1989) describes an uncommon type of rock slope failure, called a sliding block failure, that kinematic analysis cannot predict. Fortunately, Goodman states that limit equilibrium methods can be used to analyze the stability of slopes against those type of failures. Figure 5 shows a type of block failure which might be possible in the area of piers E-1 and E-2.

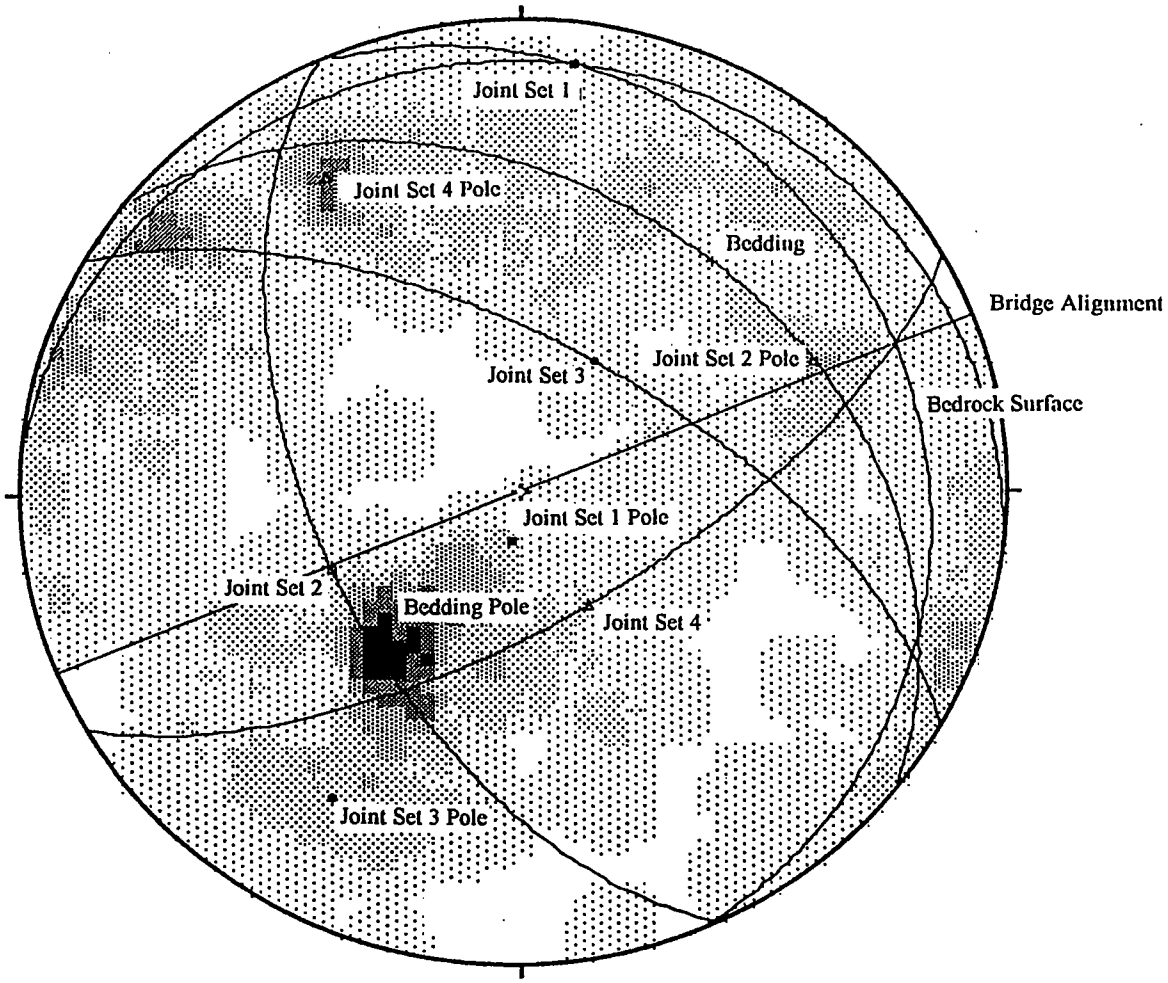

Figure 4. A density contour of the discontinuity poles measured on Yerba Buena Island. Also shown are the great circles representing the bedding, the joint sets, the bridge alignment and the bedrock surface.

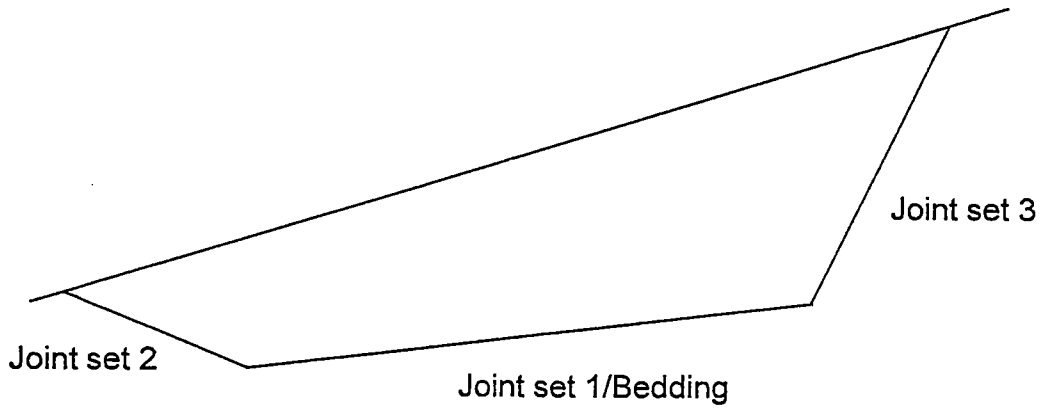

Figure 5. A diagrammatic representation of a potential slip surface formed by the joint sets in the area of Piers E-1 and E-2. Joint Set 4 is parallel to the plane of the page and it is not shown.

Limit Equilibrium Stability Analysis. The stability of hard rock slopes, which usually fail along discontinuities, can be analyzed with limit equilibrium methods that assume one or more planar slip surfaces (Chowdhury, 1978). These methods assume that the material above a potential slip plane can be considered a "free body" for a static resolution of forces. The ratio of the resisting forces to the driving forces (the factor of safety) is then calculated to give quantitative information on the stability of the slope. This process is then repeated for a number of assumed slip surfaces to determine the lowest or critical factor of safety.

The method of slices, a specialized limit equilibrium analysis method, was developed to handle irregular slopes, non-homogeneous subsurface materials and varying shear strength parameters (cohesion, friction angle and unit weight). Janbu (1957) modified the method of slices to analyze arbitrary shaped slip surfaces. The XSTABL (Sharma, 1990, 1992) computer program is a menu driven version of the STABL4 program which uses the simplified Janbu method to calculate factors of safety.

The sliding block analysis feature of XSTABL was used for this investigation. The profile developed for this analysis used the same subsurface units as shown on Figure 6 except that it is a mirror image due to constraints within the program. The shear strength parameters of the Bay deposits were summarized previously. The rock, due to its fractured and bedded nature, was modeled as a material with anisotropic shear strength parameters. The intact rock was assumed to have a unit weight of 166 pcf (26.1 kN/m^3). Its shear strength parameters are summarized in Table 2.

The cohesion for the intact rock was calculated based on the point load data and the direct shear data, discussed previously. The cohesion was added to the joints and bedding to model their discontinuous nature. Based on field measurements at Yerba Buena Island, potential slip surfaces along the joints must somehow rupture 25% intact rock and along the bedding must brake 5% intact rock. The friction angle for intact rock was estimated based on the sandstone residual friction angle and published values (Federal Highway Administration, 1989; Goodman, 1989;). The friction angle for the joints was estimated to be the sandstone residual friction angle plus the joint roughness coefficient, discussed previously in the Bedrock portion of the Geology Section. A range of bedding plane discontinuity friction angles was estimated based on the shale residual friction angle and published values.

The loads applied by the structure to the slope were modeled as distributed loads. Pier E-1 was assumed to have a load of 13.2 ksf (634 kPa) and Pier E-2 a load of 17.6 ksf (845 kPa) (Office of Structure Design, 1996) at the support

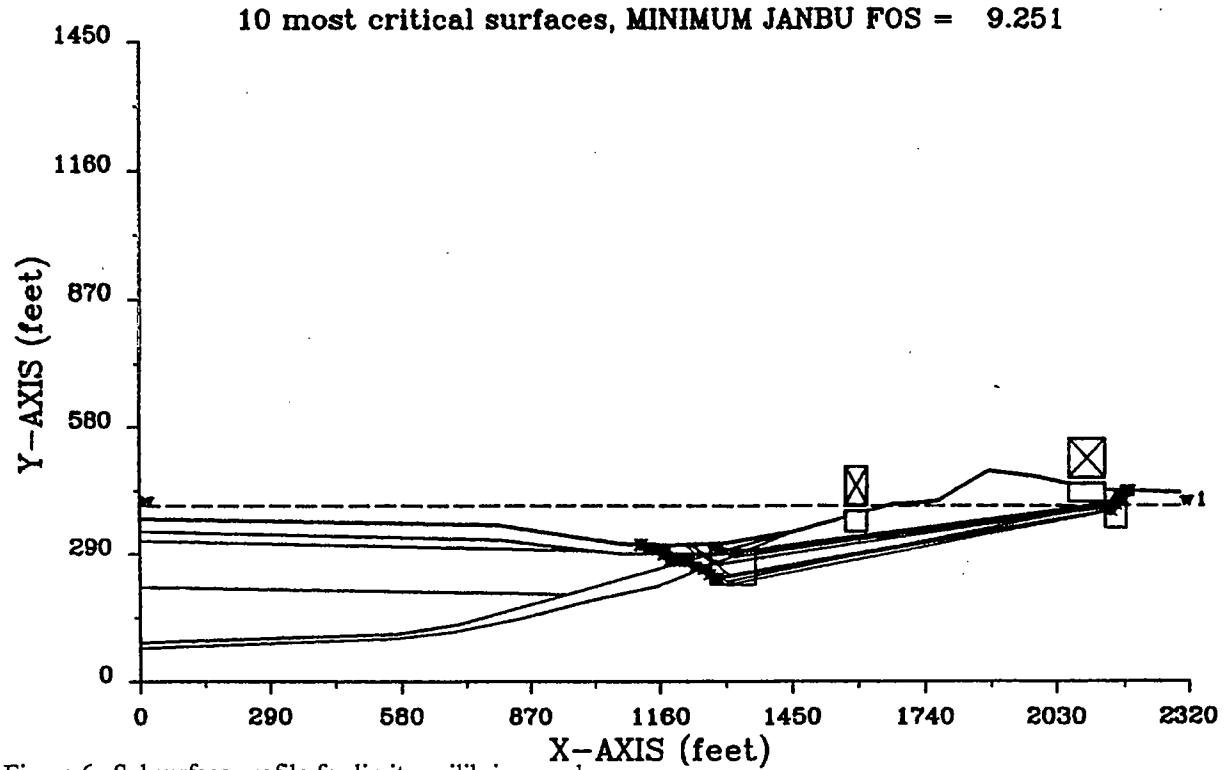

Figure 6. Subsurface profile for limit equilibrium analyses.

Table 2. *Shear strength parameters for limiting equilibrium.*

Structural Feature	Cohesion ksf (kPa)	Friction Angle (deg)
Joints	90 (4,320)	30 - 35
Bedding Plane Discontinuity	18.75 (900)	15 -25
Intact Rock	375 (10,000)	35

locations. The effect of an earthquake on the slope stability at this site was modeled as pseudo-static load (seismic coefficients $k_h=k_v=0.5$). A worse case situation was assumed for the earthquake loads. The maximum horizontal acceleration would be directed eastward along the bridge axis and the maximum vertical acceleration directed upward. Additional horizontal loads of 13.2 ksf (634 kPa) and 17.6 ksf (845 kPa) were applied at the bridge support locations of Piers E-1 and E-2, respectively. These were directed eastward along bridge alignment to model the additional forces the structure might impart to the slope during an earthquake.

A number of stability analyses were run for various scenarios:

1. Deep seated (global) slips involving Piers E-1 and E-2 on either the bedding or Joint Set 1

2. Those same global slips subjected to a pseudostatic earthquake load; those same global slips subjected to a horizontal structure-induced load and a pseudostatic earthquake load

3. Localized slips involving Pier E-2
   a. Localized slips involving Pier E-2 subjected to a pseudostatic earthquake load
   b. Localized slips involving Pier E-2 subjected to a horizontal structure-induced load and a pseudostatic earthquake load.

4. Pier E-1 was not analyzed for local slips because it is founded on a nearly flat surface.

Table 3 is a list of the results from those analyses. The results of these calculations indicate that this slope is stable even under some extreme conditions. The calculated Factor of Safety is incrementally decreased by: lowering the discontinuity friction angle; applying loads at the bridge supports; applying the pseudostatic earthquake loads; and applying an earthquake induced horizontal structure load. However, even in the unlikely event that all these conditions are applied simultaneously the calculated limit equilibrium Factors of Safety indicate that this slope will be stable.

Other numerical methods, termed "Discrete Element Methods" (Cundall and Hart, 1989), can be used to model either the static or dynamic behavior of this slope. Discontinuous Deformation Analysis (DDA) is included in this group. There are at least seven computer programs available to do these types of analyses. Discrete element methods are not yet common practice among geotechnical professionals and are beyond the capabilities of this office. However, the additional money and effort to do discrete element method analyses might be justified because of the magnitude of the Bay Bridge seismic retrofit project.

Table 3. *Results of limit equilibrium analyses.*

Scenario	Factor of Safety
Global Slip on Bedding	15.4
Global Slip on Joint Set 1	94.7
Global Slip (Bedding) subjected to earthquake load	4.1
Global Slip (Joint Set 1) subjected to earthquake load	15.8
Global Slip (Bedding) with earthquake and structure horizontal load	3.5
Global Slip (Joint Set 1) with earthquake and structure horizontal load	11.9
Localized Slip at Pier E-2	91.3
Localized Slip at Pier E-2 subjected to earthquake load	85.5
Localized Slip at E-2 with earthquake and structure horizontal loads	29.2

CONCLUSIONS

This study was conducted using a rational methodology for examining the stability of two piers of the San Francisco - Oakland Bay Bridge founded in rock. The approach was to:

1. Examine the geomorphology of Yerba Buena Island

2. Review all historical accounts and evidence of previous earthquakes, in particular the San Francisco Earthquake of 1906

3. Determine material properties
   a. Perform laboratory testing to determine rock properties
   b. Estimate soil shear strength parameters from previous investigations

4. Conduct stability analyses:
   a. Determine possible failure modes by kinematic analysis
   b. Analyze potential slip surfaces by limit equilibrium using pseudostatic earthquake loading

This approach revealed that there is no evidence of past large-scale instability at this site. Limit equilibrium stability calculations indicated that the slope on which Piers E-1 and E-2 are founded will be stable even during the maximum credible earthquake proposed for this site. It was recommended that no slope stabilization or remediation was required for the existing or retrofitted bridge.

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A Large Wedge Failure Along Interstate 40 at the North Carolina - Tennessee State Line

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ABSTRACT

On July 1, 1997, a rock slide occurred on Interstate 40 in North Carolina near the Tennessee State Line after heavy rains during the month of June. Although most of the slide was contained on a catchment area, failures continued throughout the day resulting in the decision to close the interstate until the slope could be stabilized. After 10 weeks, one lane in each direction reopened while cleanup and repairs continued until all four lanes were opened in mid-December 1997.

A slope repair project began at this site in the 1980's. The work included a relocation of the highway to provide a catchment area, excavation of unstable material, scaling and installation of rock bolts, wire mesh and horizontal drains. Until this event, these measures had proved to be successful.

The slide occurred along bedding surfaces dipping 30 to 35 degrees southwest and an intersecting joint set dipping northeast. The line of intersection formed by these two discontinuities dips into the roadway at about 40 degrees. The rock unit at the slide is the Pigeon Siltstone, a metasediment of Late Precambrian age, covered by thin deposits of colluvial material.

To mitigate this slide, over 200,000 cubic yards of unstable material was excavated down to the two major failure surfaces. The final excavation was about 350 feet across and extended up the mountain a vertical distance of 500 feet. Rock bolts and horizontal drains were used to stabilize large rock outcrops around the head of the slide and down the line of the wedge intersection. Hanging nets suspended across the slope slow falling rocks and direct them into an existing catchment area. The catchment area was improved by construction of a MSE wall at the edge of the roadway.

Introduction

On July 1, 1997, a large rock slide occurred in a cut slope on Interstate 40 less than a half mile from the North Carolina - Tennessee state line. The slide happened at about 3:00 AM in a hard, steady rainfall, and involved several hundred cubic yards of large rock, soil material, mud and water. The material filled and overran a wide area at the toe of the cut slope, spilled over into the

west bound lanes and partially into the east bound lanes. Large slide debris hit several vehicles and three injured people required transport to the hospital.

Figure 1. Slide debris in the median of Interstate 40, July 1, 1997. Shortly after this boulder was shot and removed, more slide material filled the roadway.

North Carolina Department of Transportation maintenance forces were able to clear the travel lanes once the damaged vehicles were removed. By mid-morning preparations were made to resume traffic, which by then had backed up for miles in both directions. After a few vehicles passed through the slide area, isolated falling rocks quickly developed into another major slide, filling the travel lanes and again stopping traffic. The slope continued to fail throughout the day as maintenance forces attempted to clear the slide. It was clear that it would be impossible to allow any traffic to cross the slide area and it was becoming dangerous to even work under it. The decision was made to close the roadway, detour traffic and evaluate the situation. Interstate 40 through the Pigeon River Gorge closed for an indefinite period.

Geology

Rocks throughout most of the Pigeon River Gorge are late pre-cambrian metasediments belonging to the Snowbird Group of the Ocoee Supergroup. Beginning with the oldest, formations are the Wading Branch Formation, Longarm Quartzite, Roaring Fork Sandstone and the Pigeon Siltstone. Except for the Longarm Quartzite, these units consist of metamorphosed, interbedded siltstones and sandstones.

At the slide site, the rock unit is the Pigeon Siltstone, a gray, thin to thick bedded metasiltstone containing some layers of fine grained metasandstone. Bedding layers at the slide strike N20W and dip SW 30 to 35 degrees. Discontinuities generally striking NE and dipping 60 to 65 degrees

to the SE intersect the bedding to form an unstable wedge of rock material dipping obliquely into the roadway. Wedge failures have been a common failure mode along this section of Interstate 40 in the past because of the rock structure.

The regional structure is a broad, northeast-southwest trending synclinal fold with its axial trace a short distance west of the state line. Because of this, failures in Tennessee are often mirror images of those in North Carolina.

History

Interstate 40 follows the Pigeon River though it's deep gorge for about 20 miles in western North Carolina into Tennessee. The route crosses some of the most rugged terrain in the Smoky Mountains, where relief between the river and the surrounding mountain tops often exceeds 2000 feet.

Roadway construction through the gorge resulted in high, steep cut and fill slopes. Since the route follows the river, it was necessary to cut across many steep ridges and fill across the adjacent narrow stream valleys. Cuts up to 300 feet in height and as much as 1000 feet in length are not uncommon. Long narrow fills toe up in the river and range from 80 to 250 feet in height. Construction started in the early 1960's and the project opened to traffic in mid-October, 1968.

Several large slides occurred even during construction, and once the road was opened, rock fall somewhere within the gorge was almost a daily event. Several small slides were reported each year, and there was a major failure every few years. A slide near Milepost 16 in May of 1978 blocked the eastbound lanes for a distance of 600 feet. This failure took nine months to repair and involved removal of 300,000 cubic yards of material. It was this slide that led to an investigation of cut and fill slopes along Interstate 40 through the Pigeon River Gorge in North Carolina.

This investigation led to the following conclusions concerning stability problems through the gorge.

1. Wedge failures seemed to be a common failure mode, but the numerous and varied discontinuities in the rock mass were also responsible for rock fall, toppling and planar sliding.

2. Undercut colluvial deposits resulted in failures ranging from isolated rock fall to some of the largest slides in the area,

3. Uncontrolled blasting left rock in the cut slopes shattered, loose and extremely unstable.

4. Slides occurring both before and after construction undermined many of the natural slopes above the cuts.

5. Rainfall in the area is 50 to 55 inches a year, while the average number of freeze-thaw cycles is about 70 a season.

6. Narrow benches in the slopes soon failed or filled up with rubble. All falling material collected in the travel lanes, as there was no bench or catchment area at grade.

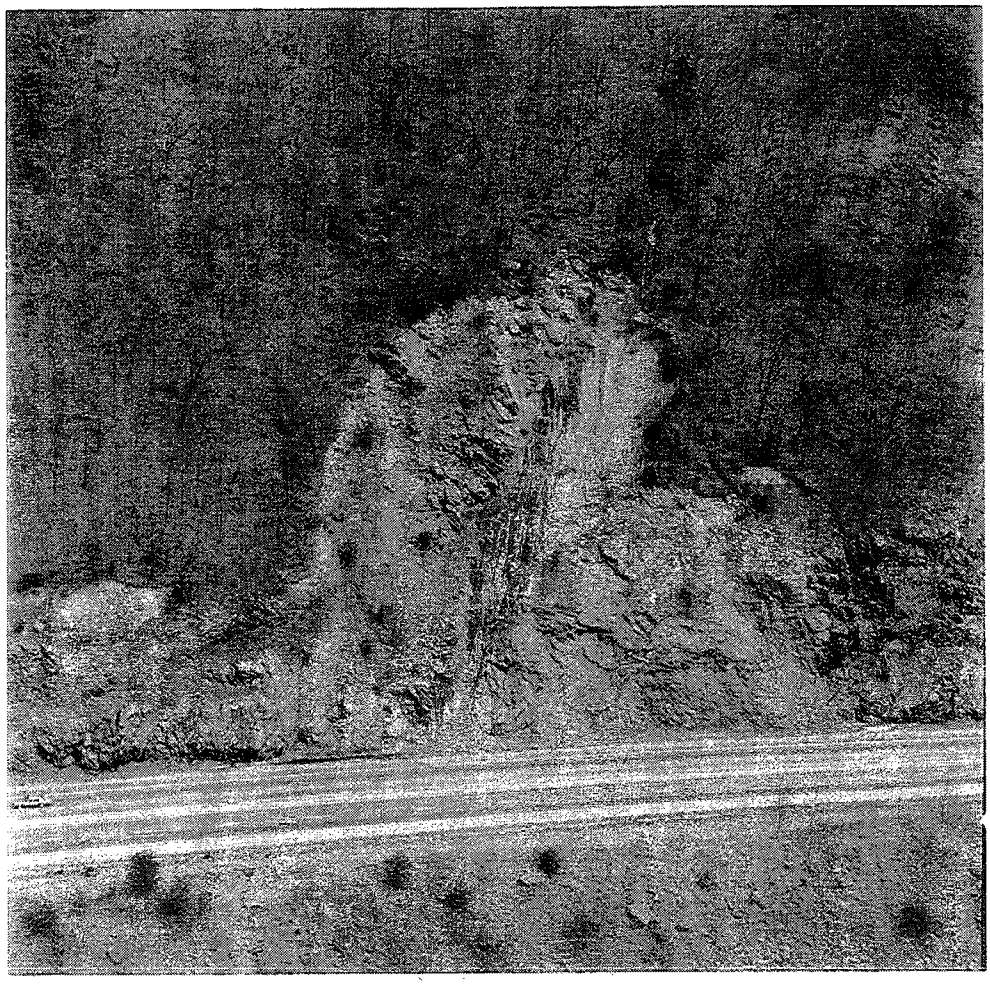

Figure 2. The slide area about 1980. Sliding probably started during 1960's construction and continued sporadically until the large failure in 1997. After the roadway was relocated during a slope repair project, the west bound travel lanes shown in this photograph became a catchment area at grade.

The study also concluded that the worst conditions were between the state line and the western portals of the tunnels at Milepost 4. A slope repair project started in the early 1980's dealt with the serious problems within this four mile section and accomplished the following.

1. Relocating the roadway laterally 24 feet onto the existing embankments provided a catchment area 30 to 35 feet wide at the toe of the cut slopes. Retaining walls were constructed at locations where there was insufficient embankment to accommodate the widening.

2. Unstable material in the cuts was excavated, and the slopes were scaled mechanically and by hand.

3. About 40,000 feet of rock bolts were installed in the slope throughout this four mile section. After the slopes were excavated and scaled, rock bolt locations and depths were determined.

4. Wire mesh and horizontal drains were used as needed.

This work was nearly complete in early 1985 when a major slide occurred at the eastern portals of the tunnels at Milepost 4, just outside the limits of the slope repair project. Interstate 40 was closed for about two weeks while the slide was cleared from one lane in the east and west directions. The cleanup and repairs necessary to open all four lanes took until December 1985. Although failures continued to occur along Interstate 40 from 1986 to July 1997, they were less frequent and of lesser magnitude. Slides occurring within the repaired four mile section were nearly all contained in the catchment area, and what little material fell into a travel lane was quickly removed by maintenance forces. Wrecks and rehabilitation projects were more of a disruption to traffic than slope failures.

Cleanup Work Begins, July 1997

The search for a contractor to clean up and repair the slide was begun immediately after it was realized that the slopes were too unstable to allow traffic to continue. As this proceeded, the slide began to migrate upslope into shallow colluvial deposits in the upper northeast corner. Tension cracks and small scarps appeared as much as 150 feet above the highest scarp of the original slide. Access to the top of the slide was extremely difficult, and after much ground reconnaissance, use of a helicopter confirmed the fact that the only way into the slopes was from the highway below.

The Department of Transportation hired Phillips and Jordan, Inc., of Knoxville, Tennessee, to clean up the slide. They immediately began construction of an access road to the top of the slide, over 500 feet above the roadway. This work took about a week. Starting the excavation in the shallow colluvial deposits above the slide presented a problem, as the material was only marginally stable in the natural 2:1 slopes. Excavation above the slide began, although the colluvium in the upper northeast corner could to continue to be a problem.

As the excavation proceeded, it uncovered the southwest dipping bedding surface, along with the intersecting southeast dipping joint set, which formed the wedge configuration. Although the bedding planes were prominent features, the intersecting joint set was less well defined because

of variations in orientation and the fractured and weathered nature of the rock. Even so, a well-defined line of intersection was becoming apparent. The Department of Transportation's plan at this time was to continue the excavation, following the bedding surface down the east side of the slide. On the west side, the goal was to find a joint surface stable enough to leave as the west side of the wedge. The strategy was to let the rock structure determine the configuration of the excavation, rather than to use a specific cut slope design. This required daily decisions as to what material to remove and what to leave.

As the work continued, a water bearing, clay seam, up to several inches in thickness, was discovered along the major bedding surface. This seam was under the highly fractured and weathered rock mass within the wedge. All this mass was undercut during construction of the original cut slope and would remain unsupported if left in place as the slide was cleaned up. It was decided to remove whatever material was necessary to leave a stable slope. The quantity of this material was first estimated at 20,000 cubic yards, but it was probably much more than that in the end.

Golder Associates, Inc. of British Columbia was asked to visit the slide site, assess the situation and offer recommendations concerning the current and future work. A summary of their preliminary recommendations includes the following.

1. No heavy blasting during excavation to avoid damage to the final cut faces.

2. Survey the face as the excavation proceeds in order to determine what face angle is needed to produce the required catchment width at road level.

3. Monitor prisms set on the remaining rock faces for movement as the excavation continues. This became critical as some sort of stress relief action occurred along a near horizontal shear zone high in the west side cut face.

4. Assess the stability of the remaining wedges and determine what effect rock bolts and horizontal drains would have on the factor of safety. Explore the feasibility of installing bolts and drains in these extreme conditions.

5. Develop some preliminary recommendations regarding drainage, scaling and bolting.

6. Once the width of a catchment area at roadway level is determined, plans for a rock fall protection structure should begin. Options included a MSE wall and/or a Brugg fence and Jersey barrier.

Much of this work could be done as the excavation continued, especially the scaling and installation of rock bolts and drains. However, the public, political, commercial and media pressure to reopen Interstate 40 was such that the cleanup operations could not be slowed or stopped long enough to allow it. Excavation continued, still following the bedding planes and the wet clay deposited on them. Although some explosives were necessary to loosen the rock, most

Figure 3. Slide clean up in mid-October, 1997. Bedding surfaces dipping 30 to 35 degrees are exposed on the right. Intersecting discontinuities dipping at 60 degrees form the west side of the wedge. Rock bolts and horizontal drains are being installed in the rock faces above the bedding planes.

of it was extremely fractured and could be dug out with a large track hoe. By mid-August, the contractor was operating 24 hours a day, pushing material off the slide during the day and hauling to the waste area at night.

Much of the remaining rock mass needed to be scaled and rock bolted before the roadway was opened to traffic. However, the speed of the work left all these areas high above the excavation, with difficult or no access into them. In late August, Pacific Blasting and Demolition, Ltd. from British Columbia moved their crews and equipment to the job to begin scaling operations and the installation of rock bolts and horizontal drains. This company had the experience and specialized equipment necessary to work in steep and inaccessible areas.

By September 11, 1997, much to the relief of the traveling public, one lane of Interstate 40 in each direction reopened after having been closed for over ten weeks. The scaling and bolting operations continued, but excavation of the slide debris had cut off all access and it was necessary to use a helicopter to lift supplies and equipment up to the drilling operations. The excavation and removal of nearly 200,000 cubic yards of slide debris and unstable material was essentially complete by early November. The bedding plane uncovered at the top of the slide proved to be the same surface exposed in the cut slope before the slide occurred, and formed the continuous east side of the large wedge. The joint set forming the west side of the wedge was not as regular, but once the material was removed, the classic wedge shape was exposed.

Continuing cleanup operations included restoration of the drainage structures and repair to the retaining walls supporting the east bound lanes. These walls were severely damaged by material pushed from high off the slide. The original catchment area at grade, 35 to 70 feet wide, was reestablished, and on December 19, 1997, all four lanes were opened to traffic in time for Christmas.

Before opening four lanes of traffic, a priority was to install three hanging cable nets across the slopes. Their purpose was to absorb the energy of rolling rocks and to direct them into the catchment area at grade. Brugg Cable Products, Inc. was contacted to help with the design and fabrication of the nets, which were to be installed by Pacific Blasting.

The system consists of three separate nets hung on 1.25 inch diameter cables suspended across the main wedges about a third of the way up the slope. The cables are anchored on one side of the wedge, run through a pulley on the other side, and down to road level where they could be tensioned and tied off to a second anchor. The nets consist of chain link secured to 10 by 20 foot panels made up of 5/8 inch cable woven into an eight inch square pattern, each weighing about 300 pounds. The panels are clamped to the suspension cable, and then laced together to form a solid net. All this work was done in the slopes by hand.

Additional rock fall protection is provided by an MSE wall constructed along the outside edge of the road level catchment area. This wall is 15 feet wide and 20 feet high with vertical front and

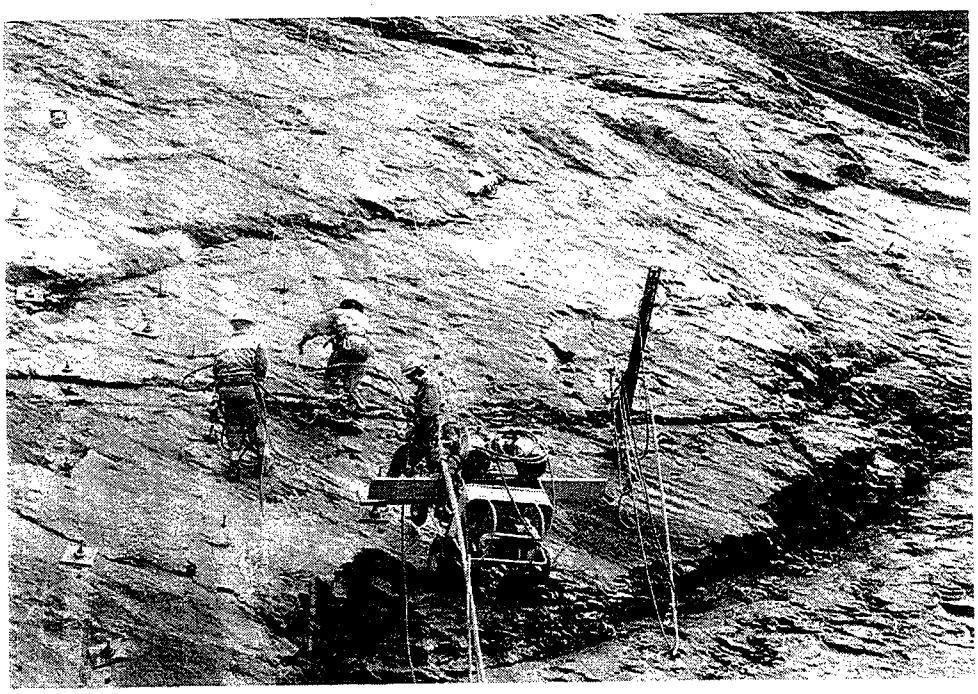

Figure 4. Installing rock bolts in the rock face high in the slide. Due to some sort of stress relief or movement along the horizontal discontinuity, the face was monitored for movement and bolted before the road was reopened to traffic.

Figure 5. Hanging cable nets for rockfall protection. Panels of woven cable and chain link are secured to the suspension cable and laced together.

back faces. Pre-cast concrete blocks form the front face, while rock filled wire baskets designed to resist the impact of falling rocks were used on the back. On top of the wall is a 15 foot high Brugg fence with guy wires attached to the tops of the fence posts and anchored back to the rock face.

During the winter and early spring of 1998, rainfall was well above normal, nearly duplicating the conditions that existed at the time of the original slide. Colluvial deposits above the northeast corner of the slide began to fail, with some of the slide passing under the nets to collect in the catchment area. Although none interfered with traffic, the slide damaged a piece of the contractor's equipment parked below. The hanging nets were in place at the time, but no work had been started on the MSE catchment wall. The only damage to the hanging nets occurred when the clamps holding the panels to the suspension cable slipped, causing them to slide to the middle of the cable. The panels were quickly pulled back into place and refastened by Pacific Blasting.

During the heavy rains of April 1998, a larger failure occurred in colluvial and weathered residual soils as the slide moved upslope. Department of Transportation maintenance forces estimated they removed at least 300 cubic yards of rock, soil and mud that had collected in the catchment area and some was left in the slope. All this material had passed under one of the hanging nets with no damage and none reached the traffic lanes.

Summary

At the beginning of this paper some reasons for failures in the Pigeon River Gorge were summarized. All contributed to some degree to the events at this particular slide.

1. The slide occurred along prominent discontinuities in the rock. In this case, the discontinuities combined to form a wedge configuration.

2. Undercut colluvial deposits contributed to sliding occurring during and since construction of this cut, and will continue to be a problem in the future.

3. Evidence of heavy blasting during construction is seen in the remaining cut faces. Explosives were used carefully during cleanup operations to avoid further damage.

4. The natural slope above the highest northeast corner of the slide continues to fail because it was undercut at the start of cleanup operations below. This was unfortunate, but necessary because of the steep terrain.

5. Rainfall in the weeks before this slide occurred was exceptionally heavy. When water sheets off the cut slopes and the small drainage courses run full, stability problems are usually not far behind.

6. The fact that most of the slide material stayed within the catchment area at grade rather than interfering with traffic demonstrated its value. This area also provided the working room out of traffic necessary to clean up the slide and implement corrective measures.

Although some smaller failures may continue to occur at this site, removal of the greater mass of unstable material and the use of other slope stability methods should provide protection to traffic.

Acknowledgments

Thanks to personnel with the following organizations who were involved with this slide repair project.

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Pacific Blasting & Demolition, Ltd, Burnaby, British Columbia
Phillips and Jordan, Inc., Knoxville, Tennessee

Performance Review of Rockfall Mitigation Measures

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ABSTRACT

The subject of rockfall has made many advances over the last decade. Computer models have been developed and new designs have emerged, all of which have allowed practitioners to increase their ability to solve and mitigate rockfall problem areas. As a result dozens of these new rockfall mitigation designs have been implemented. Many have been in place for years and now afford the opportunity to evaluate performance and effectiveness. California has installed many of these new designs which have been in place through several storm seasons. Controlling measures, such as drapery and flexible barriers have been most prevalent. While many of the traditional measures continue to be used the purpose of this paper is to review the performance of the newer flexible barrier designs and drapery designs. Five rockfall design installations are discussed, Gaviota, Highway 9, Waddell Bluffs, the Easter Slide, and Lamb Canyon. Low to high-energy flexible barriers, cable mesh, and double twisted wire mesh are discussed. These systems were installed to mitigate rockfalls, debris avalanches, and debris flows along California State routes. Systems and materials, in some cases, were provided by Macafferri, Tera Aqua, Brugg, and L'Enterprise Industrielle. Several systems were designed in house. Overall, performance has been higher than expected. Protection has been provided and maintenance reduced. At some locations significant repairs were required implemented through maintenance contracts, however, public safety was always maintained. Based on performance of these systems, use of such designs will continue.

INTRODUCTION

Caltrans has installed numerous rockfall mitigation designs throughout the state. These systems include flexible rock nets, double twisted wire mesh drapery, and cable mesh drapery. Performance of these systems has been as expected and have enhanced highway safety while reducing maintenance costs. This past winter "El Nino" caused above average landsliding throughout the state including the areas where many rockfall mitigation measures were installed. What was discovered was that these systems performed well above expected levels. In many locations impacts were on the order of two to three times design loads. The systems held up and prevent hundreds of cubic meters of rock, mud, and debris from reaching the roadway. Five of theses installations are discussed, Gaviota, Highway 9, Waddell Bluffs, the Easter Slide, and Lamb Canyon. Low to high-energy flexible barriers, cable mesh, and double twisted wire mesh are discussed. These systems were installed to mitigate rockfalls, debris avalanches, and debris flows along California State.

PROJECTS

Gaviota Pass, California State Route 101, Santa Barbara County, PM 47

The site is located approximately 48 kilometers north of Santa Barbara on California State Route 101. At this location the roadway passes through Gaviota Pass. Gaviota Pass is characterized by steep canyon walls plunging into Gaviota Creek. In 1987 Caltrans under took a study to mitigate rockfall in the area in response to increasing rockfall related accidents.

The cuts are comprised of sandstone, colluvium, and landslides. The cut slopes were designed with mid slope benches and cut slope ratios as steep as vertical to 1: 0.5 (H: V). Rockfalls are developing in the landslide and colluvium as the result of differential erosion. Within the sandstone cuts, rockfalls are developing as planar and wedge rock block failures. The mid-slope benches are also failing and filling with debris creating launching ramps. Although some catchment is available rockfalls were reaching the roadway. Rockfall activity was most prevalent during heavy rainfall periods but also as high winds are funneled through the canyon.

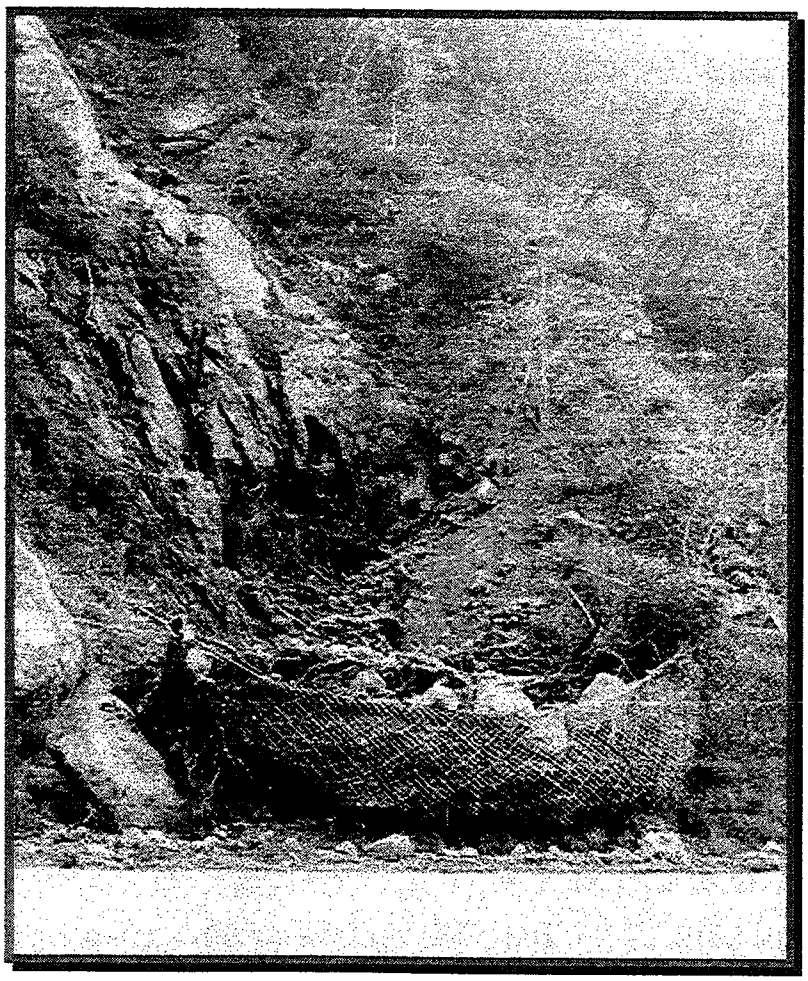


Photo 1: Rock Net Filled to Capacity

All the cuts in the canyon were analyzed and rated. Rating was done using the Oregon Rockfall Hazard Rating System to prioritize the area. The analysis of the cuts consisted of detailed mapping, historical events, and a CRSP analysis (Colorado Rockfall Simulation Program). While many alternatives were considered in the final analysis a combination of flexible barriers and double twisted wire mesh drapery were used. To a lesser degree rock bolts and shotcrete were used. As the result of this study more than a 300 meters of Brugg and L'Entreprise Industrielle rock nets and 2700 square meters of Tera Aqua wire mesh drapery were installed in 1992.

Since installation dozens of rockfalls have impacted these systems. No rocks have reached the roadway and little to no maintenance has been required. Most of the impacts were within design load. During this winters storms rock slides and rock avalanches were a common mode of failure and at several locations impacted the rock nets. Average volumes ranged from 5 to 10 cubic meters of material consisting of rock, soil, and trees. All of the material was stopped and many cases filled the barriers. These systems, design for 200 kilo-joules of impact energy, stopped impact energies in excess of 400 kilo-joules with masses of 6 tonnes of material. The drapery systems were equally successful in stopping and controlling 3 to 5 cubic meter avalanche events. However, where individual rock sizes exceeded 600 to 700 mm some of the mesh was torn and the seams were separated.

The nets, many of which filled to capacity, in some cases needed to be cleaned, friction brakes reset, and the chain link backing replaced. Where chain link and double twisted wire mesh were repaired Spanex and Tiger Tite wire mesh fasteners were used instead of hog rings.

Highway 9, California State Route 9, Santa Cruz Co., PM 2.5.

Along Highway 9 outside the city of Santa Cruz the roadway traverses steep mountainous terrain within thick redwood forests. The roadway is a two lane road with little to no catchment. Following heavy winter storms in 1995 rockfall activity increased due to slope erosion. Under emergency storm damage contracts Caltrans undertook a study of the area to mitigate rockfall.

The slopes above the roadway consist of weathered fractured and sheared sandstones. Slope ratios are vertical to 1:1.5 with slope heights in excess of 60 meters. Slope erosion in the form of surficial landsliding is occurring causing rock blocks to erode out of the slope. Rockfalls are occurring during periods of heavy rainfall.

As part of storm damage repair an emergency project was initiated to instal rock nets at the site. Rock nets were chosen due to the limited room andenvironmentla constraints. An analysis using the Colorado Rockfall Simulation Program was performed in conjunction with rock rolling tests. The results of this analysis indicated that a 200 kilojoule barrier located approximately 9 meters upslope would be the most effective. The barrier was installed at this location with double twisted wire mesh drapery attached to the base of the net extending downslope to grade. Thrity meters of Brugg rock net barrier was placed in the rockfall zone between the trees.

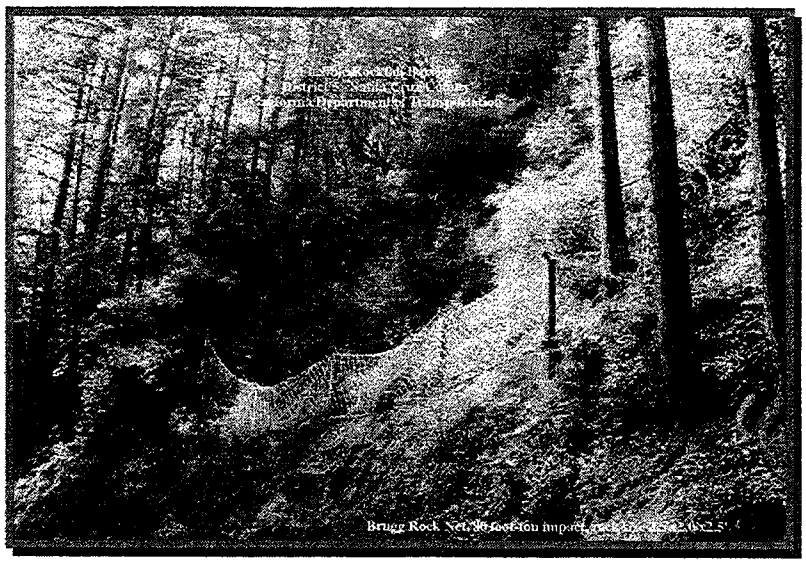


Photo 2: Highway 9 Rock Net Installation

Up to and through this years "El Nino" event the nets have been stopping rockfalls effectively and without requiring maintenance. During this winters unusually high precipitation rock and debris moved downslope into the nets with no material passing through. No maintenance was required and roadway safety was maintained at a critical. All the events were within design load impacts.

Waddell Bluffs, California State Route 1, Santa Cruz County, Post Mile 36.3/37.4.

Waddel Bluffs is located along California State Route 1 between Santa Cruz and San Francisco. The site is located on the western flank of the Santa Cruz Mountains where the mountains meet the Pacific Ocean. Originally the slope was constructed on a 1: 3/4 (V:H) slope with a 4.5 meter wide, and 2 meter deep catchment area at road grade. Today, after decades of maintenance cleaning, the ditch is, on average, 10 meters wide and 3 meters deep. Following a history of rockfalls reaching the roadway Caltrans conducted an investigation to mitigate rockfall. This study was conducted over a one year period starting in September 1989 and ending in October 1990. The investigation included a rockfall survey, rock rolling tests, computer modeling, and geologic mapping.

The existing cut slopes occur along approximately 1800 meters of roadway with slope heights between 60 and 120 meters and slope ratios between near vertical and 1: 3/4. The rock type composing the bluffs are interbedded mudstones and siltstones. This formation, known as the Santa Cruz Mudstone, is highly erodible, sloughing from the bluffs in 25 to 50 mm pieces forming tallus piles on ledges and in the catchment ditch at road level. The more competent siltstone is undercut by the eroding mudstone which spalls from the bluffs in 0.3 to 1.0 meter blocks or occasionally as small rockslides (3 to 11 cubic meters). These conditions are filling the catchment ditch to the extent that every year as much as 40,000 cubic meters of material fills the ditch each year. Rainfall, wind, surface water runoff, and chemical decomposition of the rock are the most active weathering agents causing rockfall at this site. The most noticeable cause is rain and during periods of heavy rainfall rockfalls can be expected. Less predictable, but as important, wind and chemical decomposition are causing rockfalls during calm and dry periods.

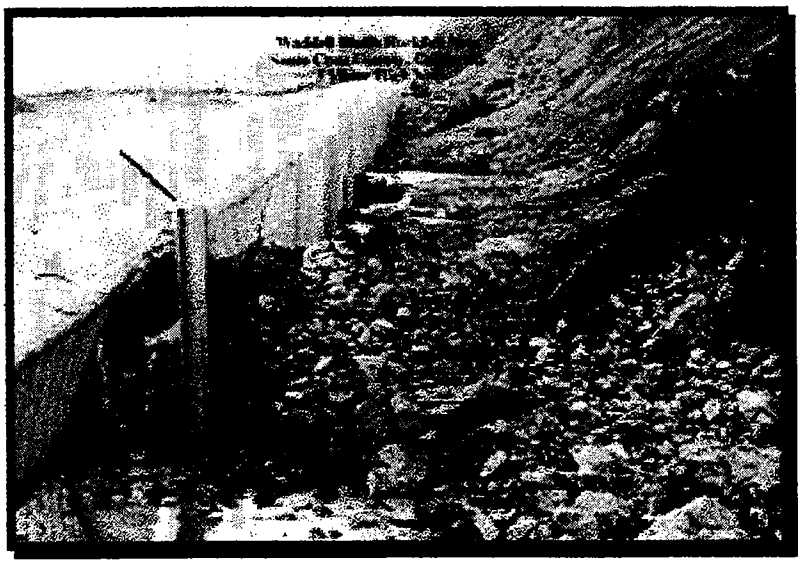


Photo 3: Waddell Bluffs Rock Net Containing A Rock Slide

In the final analysis rock net barriers were the recommended solution. Fifteen hundred meters of 200 kilojoule Geobrugg nets were installed in the Fall of 1997. One thing unique about this design was there are no upslope supporting cables. This was required due to the geometry and rock quality. As part of the installation the catchment ditch was cleaned and a jersey barrier placed between the net barrier and the roadway. During construction rocks were hitting the net protecting the highway and the workers.

Immediately upon completion design load impacts hit the barrier without damage to the barrier. Within two months after installation "El Nino" storms hit the coast and this area was hit particularly hard. Rockslides, mud flows and rockfalls were occurring regularly. The nets performed well above exceptions. In several areas 300 to 500 cubic meter rockslides translated downslope, from heights in excess of 60 meters, into the ditch and nets. Impacts energies were above 600 kilojoules. All the material was stopped including the mud and trees. Above design rockfall impacts were stopped as were mud flows. However, at one location a 4000 cubic meter rockslide topped over the net and reached the roadway. The net was destroyed. While most of the adjacent roadways were closed due to sliding this 1.8 kilometer section was largely clear of debris.

During the cleanup a 30 meter section had to be replaced where the large slide occurred. In the other areas the nets were cleaned with a excavator from the front and in the heavily hit sections the friction brakes had to be re-set or replaced as did the chain link backing the cable net.

Easter Slide, California State Route 41, San Luis Obispo County, PM 7.0

Along State Route 41, between Atascerdero and Morro Bay, California approximately 60 meters of barrier was installed in an area which had been previously burned during a wild fire in 1994. Following the burn Caltrans investigated the site and determined that within this section of roadway several locations had an extremely high potential for debris flows, mud flows, and rockfalls.

Photo 4: Highway 41 Rock Net Containing a Debris.

The slopes above the roadway consist of colluvium with occasional bedrock outcrops of meta-volcanics. Slope heights exceed 500 meters and slopes ratios average 1: 1. These slopes are covered with hundreds of loose rocks and numerous small drainages and swales. As part of an emergency program to prepare for the onset of winter rains Caltrans Maintenance forces installed a Brugg and L'Entreprise Industrielle flexible barrier along a 60 meter section of roadway through this area. With little catchment available and steep slopes above the roadway the barrier was installed at grade with a jersey barrier between the net and the roadway. As in the Waddell installation no upslope support cables were used in the design.

The following winter the storms were severe in this area, eroding many of the slopes in the burn area. At the location of the rock net a series of debris flows impacted the barrier. An estimated 60 cubic meters of debris flow material accumulated behind the barrier as the result of three to six individual flows. Estimated impact energies ranged between 80 and 130 kilo-joules of impact energy. Less than 3 percent of the material passed through the

barrier while in adjacent unprotected areas debris flows covered the roadway requiring closure.

During this winters "El Nino" storms smaller mud flows and rockfalls impacted the nets with no material passing through. Again, another section of roadway was clear of debris during a critical time allowing maintenance forces to concentrate on other areas.

Lamb Canyon, California State Route 79, Riverside County, PM

Lamb Canyon is located between Los Angeles and Palm Springs along California State Route 79. This section of highway was part of a widening project through several miles of mountainous terrain. This project was contracted out at all levels with Caltrans as an overseer. Dozens of cut slope excavations were proposed as part of the widening. During construction several slope sections began to show signs of instabilities. In response, Caltrans participated with the consultants to mitigate the problems areas. The investigation, performed in 1995, consisted of a detailed mapping project in conjunction with a computer analysis using the Colorado Rockfall Simulation Program (CRSP) and actual rock rolling tests.

Cut C-34 was originally 21 meters high with 1:1/2 (V:H) slope ratios. The plans called for 27 meter high cuts and 1: 3/4 slope ratios with 6 to 10 meters of catchment area provided at grade. The final cut slopes ended up with approximately 33 meter high slopes with 1: 1/2 slope ratios. The rock type in this cut is granitic with various distributions and grades of decomposition/weathering ranging between Grade I and III/IV causing differential weathering. The excavation was blasted and, probably due to the heterogeneity of the rock, the cut slope was destabilized causing rockslides and over break. The finished cut slope is ragged, over steepened in some locations, and loose rock covers much of the slope.

A barrier at grade, to prevent rocks from rolling onto the traveled way, was recommended and installed. The recommended barrier was a 200 kilojoule flexible wire rope rock net. Approximately 146 meters of 2 meter high barrier was installed approximately 4 meters from the edge of shoulder. A Geobrugg barrier was installed.

Cut C-13 was originally 22 meters high with a slope ratio of 1: 1/2. The plans called for 48 meter high cuts with a 1: 3/4 slope ratio and 7.6 meters of catchment width. The final cut slopes are approximately 48 meters high with 1: 0.75 slope ratios and 6 meters of catchment. A portion of the cut slopes consists of Paleozoic metamorphic shales with occasional interbeds of limestone. Cut slopes within the metamorphic shales are raveling, resulting in small rock pieces sliding downslope. Larger blocks are eroding out of the limestone units and working downslope. No large scale instabilities were observed. There also occurs a granitic intrusion approximately 100 meters in width. The cut section within the granitics was blasted for the excavation. The blasting operation resulted in over-break of the slope face. The finished granitic cut slope is ragged, over-steepened in some locations, and loose rock covers much of the slope. Large rock blocks are being destabilized and moving downslope.

At these location based upon field observations and computer modeling the catchment width alone is not adequate rockfall mitigation. The recommendation was to stabilize the area by covering the outcrop with cable net drapery. Approximately 6300 square meters of cut slope face was covered with Geobrugg cable mesh drapery.

Cut C-8 was originally 16 meters high with 1: 3/4 slope ratios. The plans called for 44 meter high slopes with 1: 3/4 slope ratios and 6 meters of catchment provided at grade. The existing cut slopes are approximately 40 meters high with 1: 0.67 or steeper slope ratios and 6 meters of catchment. The cut slopes consist of Paleozoic metamorphic shale with a granitic dike intruded into the metamorphic rocks. To the north of the dike the metamorphics are relatively homogeneous. To the south of the dike the cut consists of a sheared zone of metamorphics and granitics. Adjacent to the shear zone is a melange of metamorphic rocks, gouge, and granitics. The existing slope is in a ragged condition with an abundance of loose rock lying on the slope. Draping the slope with double twisted wire mesh was recommended to mitigate rockfall from reaching the traveled way. Approximately 2100 square meters of cut slope face was covered with Macafferri double twisted wire mesh drapery.

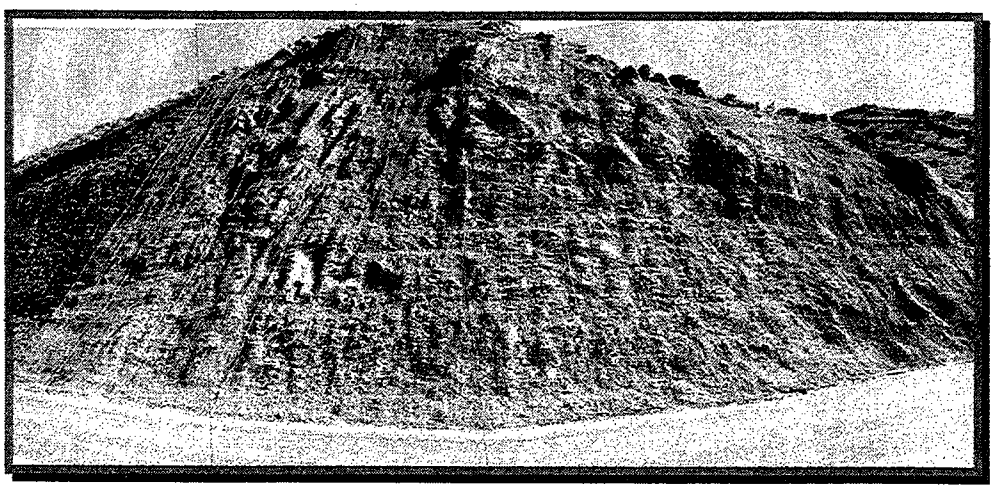


Photo 5: Lamb Canyon Cable Drapery Installation

During the winters of 1995 and 1998 heavy rains hit this canyon causing considerable erosion of the cut slopes in every case the mitigation installed prevented rocks from reaching the traveled way. These events are estimeted to be within design loads. The cable drapery was the most effective to the extent that very little material worked downslope to the catchment. It appears that the meshes weight dramatically arrests movement of the rock and that this was its greatest attribute to controlling the rockfall. Maintenance callouts and operations in these locations were eliminated. However, in areas were catchment area was the principle mitigation measures rocks were reaching the traveled way.

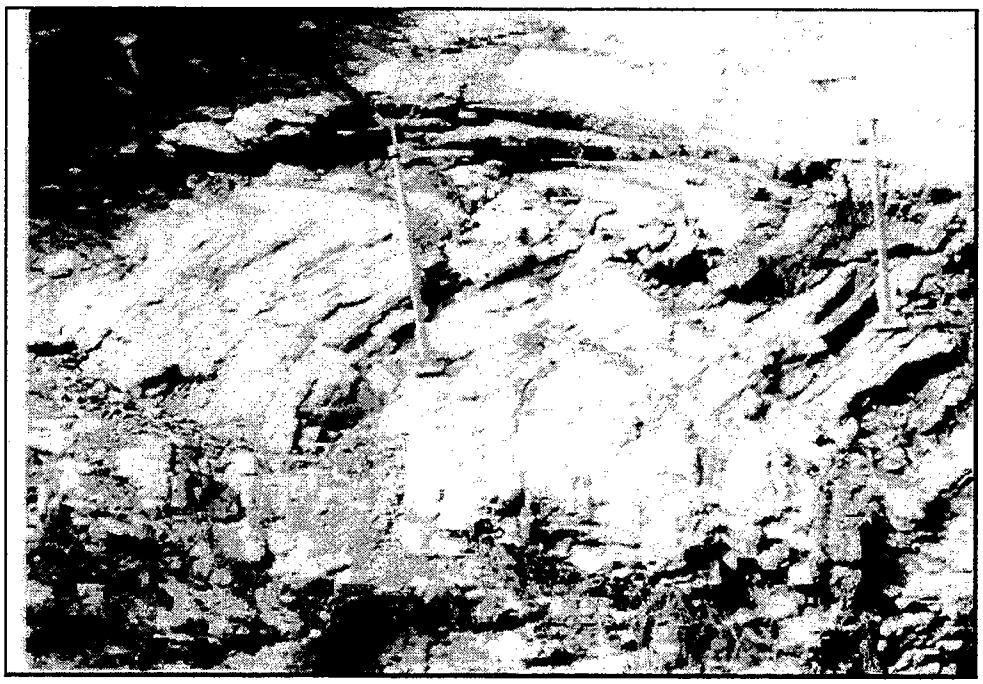
In Conclusion

The performance of many of these systems has been tested beyond design load capabilities and in extreme maintenance conditions. These relatively new designs and recently improved designs are proving to be a very valuable method for protecting highways from slope instabilities such as rockfalls, debris flows, and debris avalanches. The maximum potential for these types of systems is as yet untapped. Research continues around the world on increasing the capabilities of these designs. Case histories, such as these, help in those pursuits and certainly illustrate the value of using these systems to protect highways.

ROCKFALL CONTROL FENCE

UNAKA SPRINGS ROAD

UNICOI COUNTY



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ROCKFALL CONTROL FENCE

UNAKA SPRINGS ROAD

UNICOI COUNTY

ABSTRACT

Rockfall problems have plagued a short section of a county road in Unicoi County, Tennessee. Unaka Springs Road is located in a canyon cut by the Nolichucky River in a mountainous section of East Tennessee.

Maintenance history indicated that rocks would roll and bound onto and over the narrow roadway causing numerous road closure events and some property damage occurrences. A rockslope scaling contract was let in August of 1996 to help alleviate the rockfall hazard.

During the scaling project it was determined that a safety hazard would still exist after the project was completed. A rockfall catchment fence was designed and added to the scaling project during the construction phase.

The rockfall control fence was 195m (640 feet) in length and cost $603.55 per linear meter ($183.90 per linear foot). Total project length was 31 working days.

INTRODUCTION

Unaka Springs Road is located along the Nolichucky River as it cuts a canyon through the mountainous Blue Ridge Province in Unicoi County, Tennessee (Figures 1 and 2). The county road is located near Erwin, Tennessee and is also crossed by the Appalachian Trail at Chestoa.

Numerous rockfall events along a short section of this road resulted in a number of road closures and some property damage incidents. It was decided by Tennessee Department of Transportation to remove as much of the lose rock along the affected roadway as possible under a controlled situation.

The general geology of the site involves tilted and metamorphosed strata of the Cambrian Age Chilhowee Group. The rock consists mainly of metasandstone, metasiltstone, and some phyllite all of which have been greatly deformed by tectonic movements. The strata strike generally NE/SW with varying degrees of dip, typically to the southeast. The section of Unaka Springs Road that is under discussion cuts across the general strike of the rock strata, exposing numerous layers of the rock in the roadway cut slope.

The rock strata is very broken with numerous fractures throughout (Figure 3). Faulting and folding of the strata emplaced the broken character of the rock which becomes greatly manifest during weathering. A result of the fracturing in the rock is the position of fracture discontinuities in an arrangement that results in the formation of wedge-shaped blocks of rock material. Depending on the orientation of these wedge

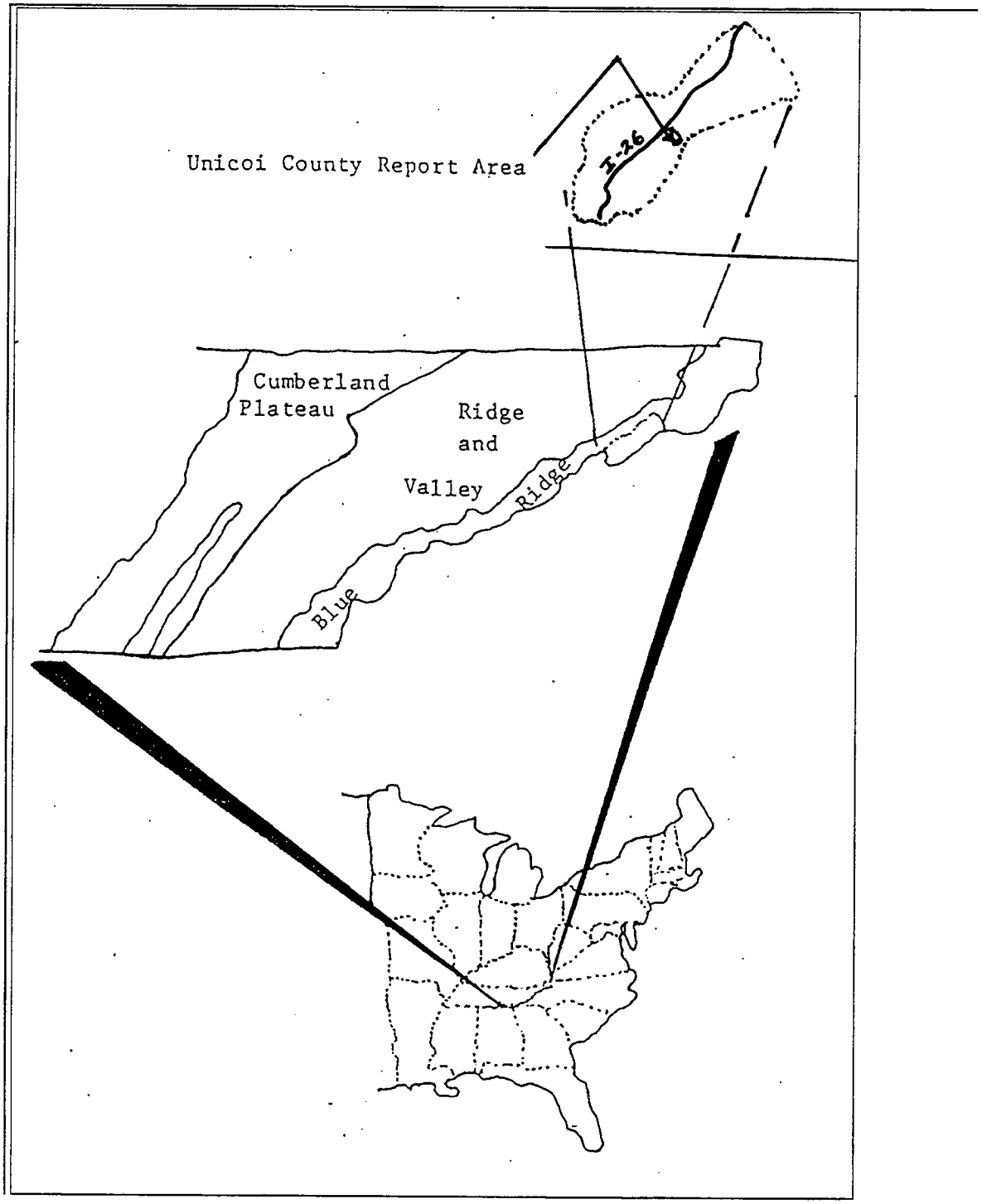

Figure 1. Location of project area.

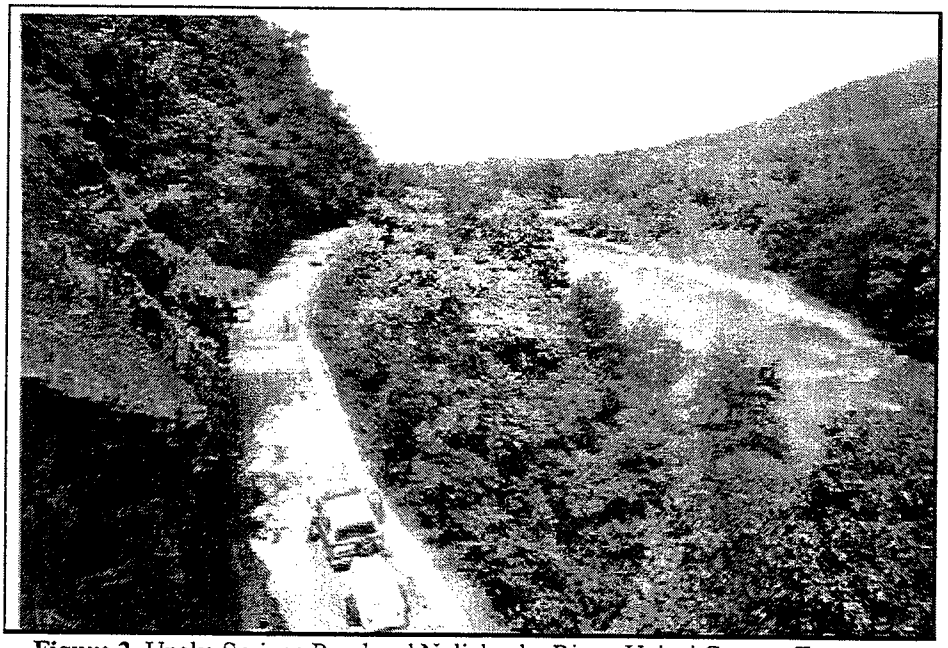

Figure 2. Unaka Springs Road and Nolichucky River, Unicoi County, Tennessee

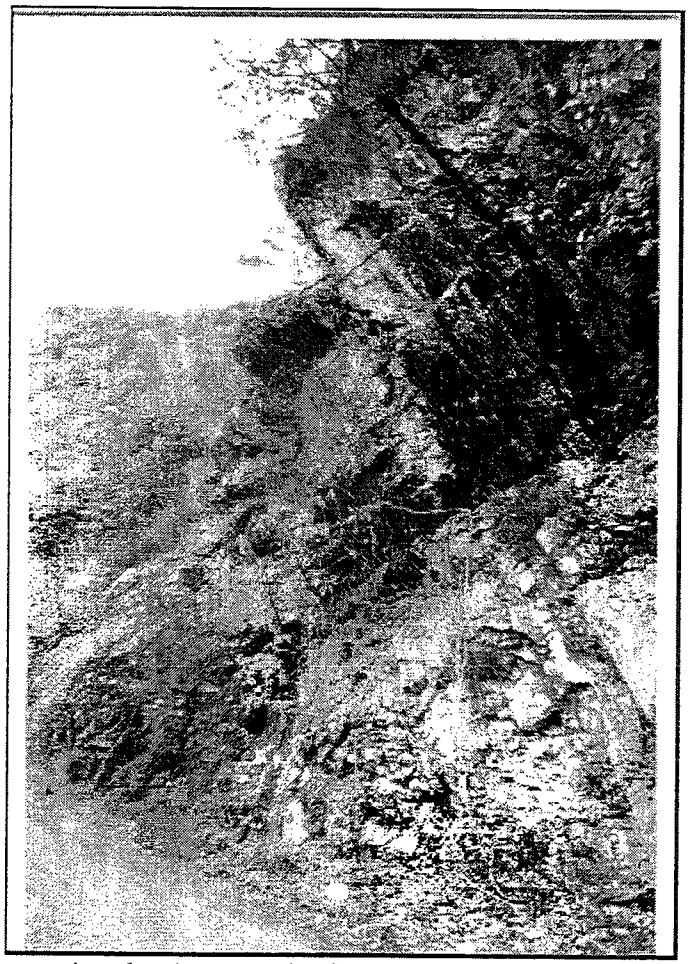

Figure 3. Rock slope on project showing example of overhang to be removed. Overhang of rock is actually a small topple failure that contributed to the rockfall problem.

blocks, slope failure may or may not occur. At the project site no wedge failures had occurred, although there were wedges of rock that were oriented such that their axis of movement would have been into the cut slope.

PROJECT CONCEPT

The project involved the removal of loose and broken rock from the cut slope surface in a controlled manner. This process is referred to as "rock scaling". The rock material is removed by hand scaling, mechanical removal, or by blasting. Most of the rock debris was removed by mechanical methods using a crane suspended implement which was dragged over the rock slope surface (Figure 4). This method of removal can liberate from the surface pieces of rock that range up to several tons.

In addition, small pieces of rock were removed by hand using pry bars. Small scale blasting was used to remove several large overhangs. These overhangs of rock were the result of a type of rock slope movement known as toppling. Toppling is the forward rotation out of the slope of a mass of soil or rock about a point or axis below the center of gravity of the displaced mass (see Figure 3). These large blocks of rock gradually rotated away from the slope and broke loose from the main rock mass and tumbled down to the roadway.

As the project scaling was underway, it was determined by field examination and experimentation that the roadway traffic would still be in danger of falling rocks after the scaling project was completed. Therefore, it was decided that some method of protecting the motoring public from rockfall was needed.

A rockfall catchment fence was decided as the best treatment method. Due to the narrow width of the road, it was decided that a design that would place the fence upon the slope would be more acceptable.

ROCKFALL CONTROL FENCE

The rockfall control fence used on this project consists of a series of posts mounted on the rock slope and connected together by a cable network. A wire mesh is then attached to the cable network and draped down over the remaining rock slope, down to the roadway. The concept is to catch rolling or bounding rocks with the fence/net combination and let the rock material then fall down to the roadway ditch behind the wire mesh.

The rockfall control fence was placed along the edge of the cut slope where the rock face slope changes from about a 30 degree plane to a vertical. This ranged from 3.65m to 6.1m (12 to 20 feet) above the roadway. The posts are 1.524 m (five feet) in height and are anchored at the bottom by a rock bolt (Figure 5).

Approximately 1.5 to 2.4m (five to eight feet) up slope another anchor is located where a cable is attached for secondary anchor support. All support posts and anchors are connected with a series of wire ropes that lace through the top of the posts and is then anchored at each end of the cut interval. In addition, a cable is also laced from an anchor to the top of the next post then back down to an anchor to provide support to the whole fence. This is needed in order to prevent any one rockfall event from taking the entire fence down.

A 1.9cm (¾ inch) diameter cable was used having a nominal strength of 26 010.88 kg (25.6 tons) and meeting federal specifications for wire rope and strands RR-w-410D. The wire rope was zinc-coated 1.9cm (¾ inch) diameter class 6 x 9 (IWRC).

The fence posts used on the project were6.0cm (2 3/8 inch) diameter , 1.524m (five feet) in length with a wall thickness of 1.1cm (7/16 of an inch), and be galvanized. The pipe met the specifications for ASTM A-53, Type E, Grade B pipe (schedule 120 pipe).

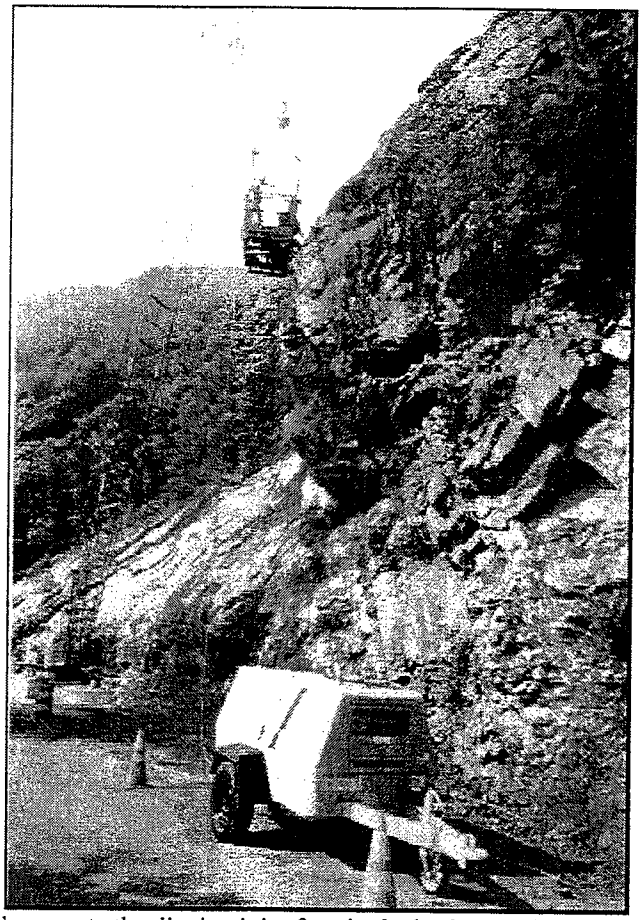

Figure 4. Scaling rock slope; note the dipping joint face in the background where the proposed fence is to be constructed.

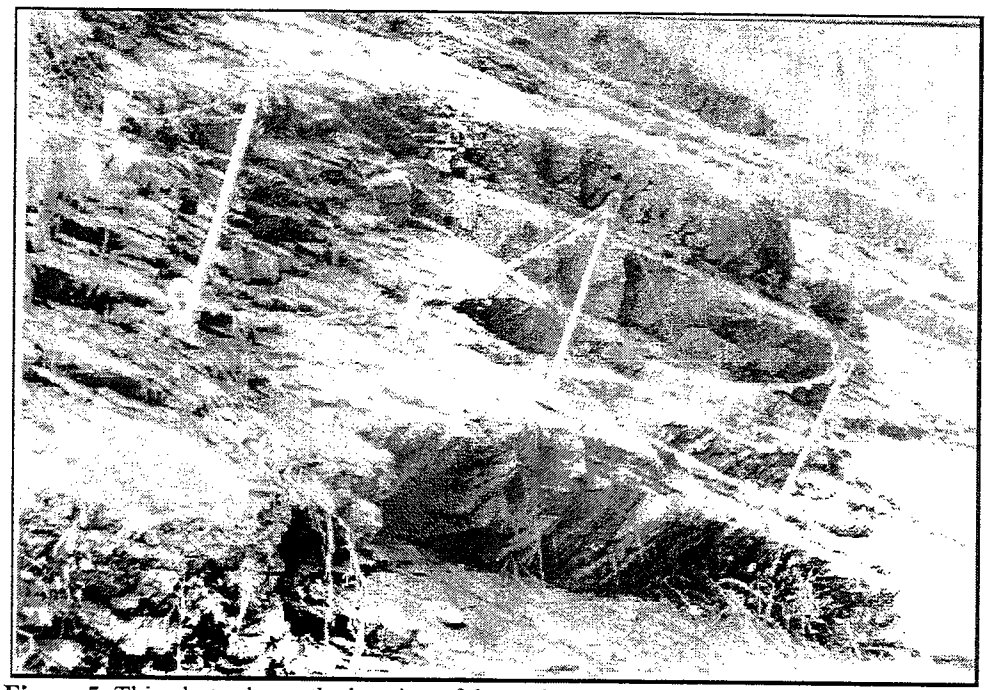

Figure 5. This photo shows the location of the rock anchors and the fence posts with cables.

The wire mesh consisted of galvanized, hexagonal, double twisted steel wire, approximately 0.30cm (0.1181 inch) in diameter with a minimum tensile value of 413 683.2kPa (60,000 psi). The maximum size of the mesh opening was 11.43cm (4.5 inches) in the largest dimension.

CONSTRUCTION SEQUENCE

Initially, the contract required that the affected cut slope be scaled of all loose rock debris and overhangs as indicated on the plans. This required the use of mechanical scaling devices using a crane hoisted dozer tracks and hand scaling with pry bars. Several precarious rock overhangs were removed by small scale drilling and blasting - referred to as trimming. Once this was achieved, several rockfall tests were conducted to check the safety of the remaining cut slope.

The tests revealed that rocks (when dislodged by weathering processes) would indeed roll down the cut slope and into the traffic lanes. This was decided to be very unsafe and unacceptable. Therefore, it was decided to employ the use of the rockfall control fence in order to prevent the rocks from reaching the traffic.

The construction sequence required that the mesh be installed in vertical strips, each lapped over the other by a minimum of 30.48cm (12 inches). The wire mesh was attached to a horizontal cable that was located at the top of the posts and was laced through eye nuts at the top of each post (Figures 6 and 7). The horizontal cable was then attached by vertical cables to anchors located up slope and spaced at maximum 6.1m (20 foot) intervals. The wire mesh was draped over the horizontal cable so that a 30.48cm (one foot) overlap was achieved. The wire mesh was attached to the horizontal cable by clips called " Tiger Tites" and continually woven tie wire. The vertical overlaps were also connected with the tie wire and 'Tiger Tite" clips.

The wire mesh was extended down the slope to approximately 1.524m (five feet) above the roadway ditch line. This enables the maintenance crews to clean the ditches without destroying the wire mesh (Figure 8).

SUMMARY

The project was completed in 31 working days during the months of September and October, 1996. A total of 5 922.274 cubic meters (7,746 cubic yards) of rock were removed by the scaling and trimming process. The completed rockfall control fence was 195m (640 feet) in length and cost $603.55 per linear meter ($183.90 per linear foot). A total of 55 anchor sets were installed. Each anchor set consisted of two anchors and a base plate for the post. The final contract costs including trimming and scaling, rockfall control fence, and clean-up and paving was $514,152.98.

The only item that will need to be changed for future projects was the eye-nut detail. The eye-nut ring that the cable threads through is only 1.27cm (½ inch) in diameter. This is the weakest point in the fence and most likely to be a maintenance problem. For future projects it was decided that this eye-nut ring should be a minimum of 1.9cm(¾ inch) in diameter and preferably 2.54cm (one inch) in diameter.

Several large boulder hits have occurred since completion of the project. One "hit" involved a boulder 1.5 meters (5 feet) in diameter. This boulder, after hitting the fence, dropped to the ditch and broke into several pieces. The other "hit" involved a boulder about 1 meter by 0.5 meter (3 feet by 1.5 feet) which became trapped between the fence and the rock slope. No appreciable damage to the catchment fence has been observed.

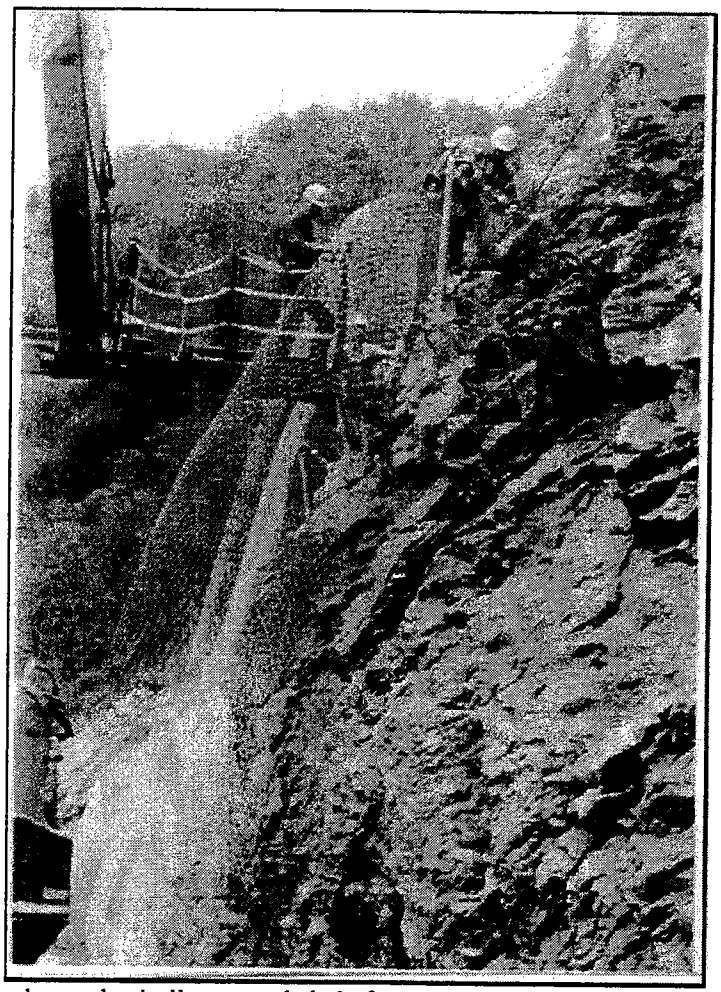

Figure 6. Workers used a mechanically suspended platform to attach the wire meshing to the cable network.

Figure 7. The rockfall control fence showing the draping nature of the wire meshing.

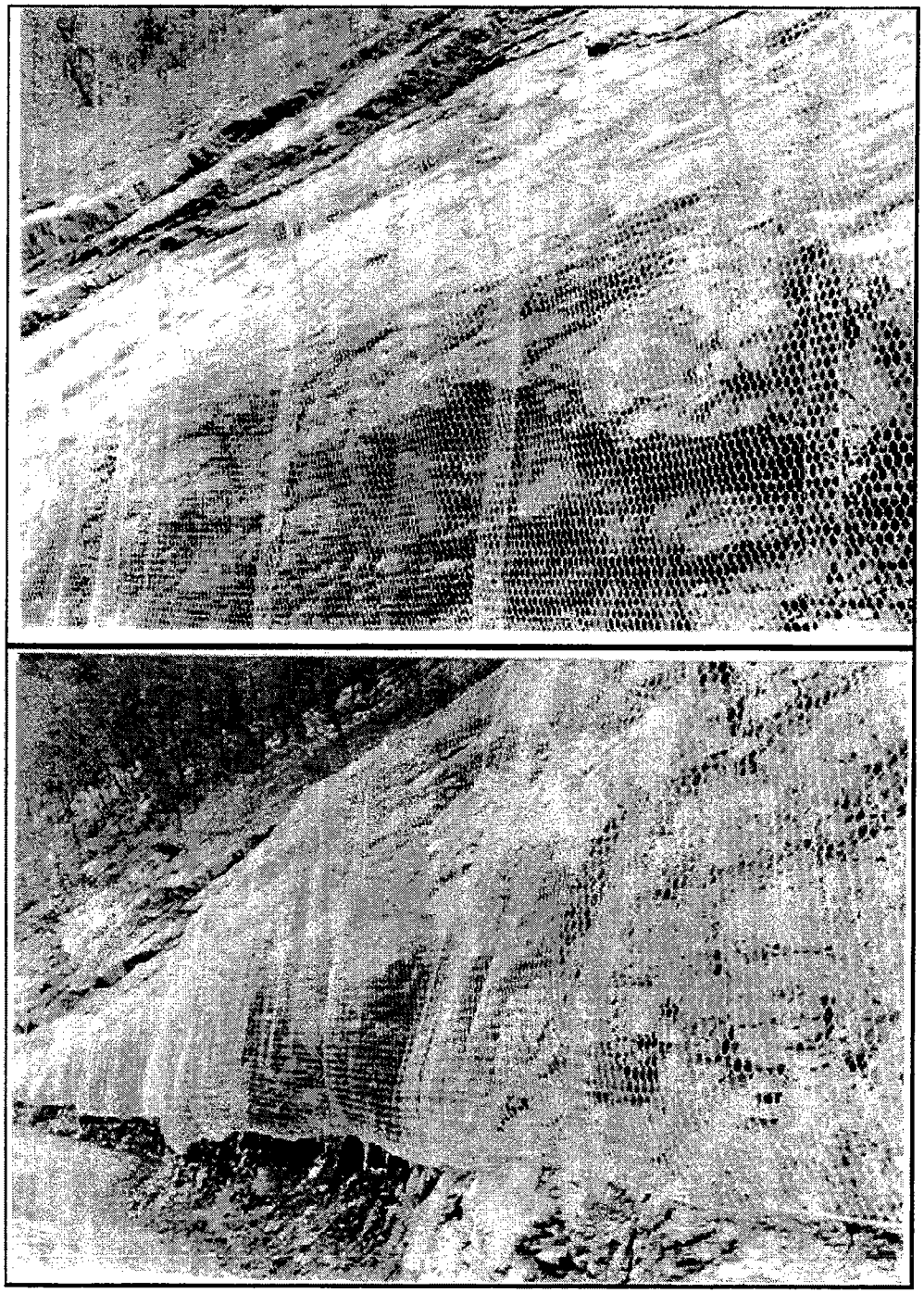

Figure 8. The rockfall control fence was allowed to reach within 1.524m (five feet) of the ditch line, which enables access by maintenance personnel in cleaning the ditch.

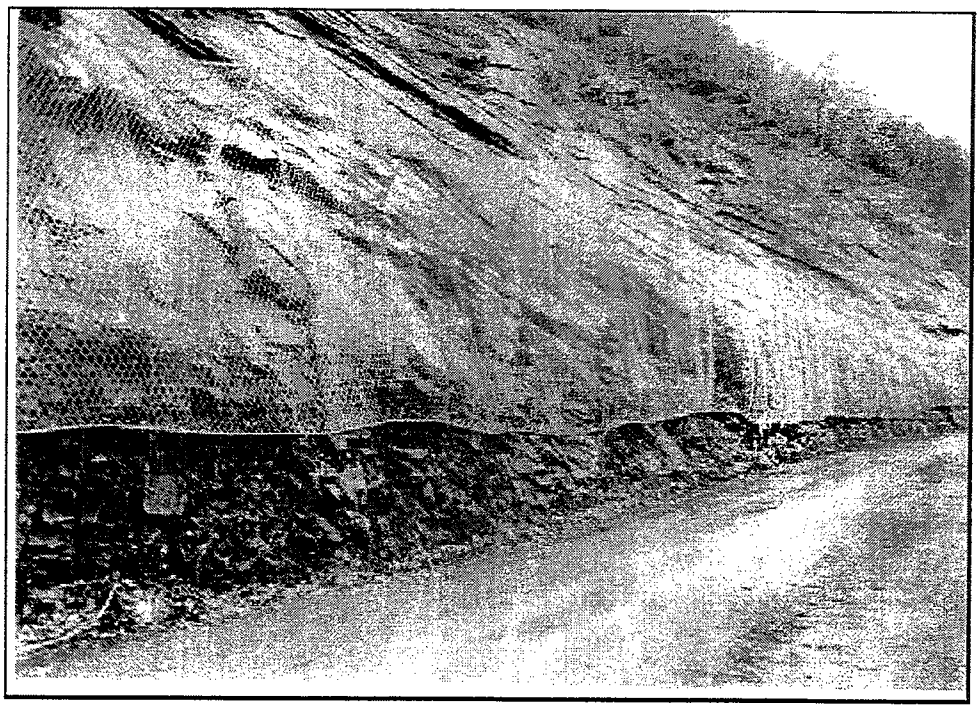

Figure 9. The completed fence is shown in this photograph; note the cable along the bottom of the wire mesh which is designed to keep the mesh from flaring out on impact from falling rocks.

The Performance of Rockfall Mitigation Measures in New York State

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ABSTRACT

Prior to 1968, when New York State began to use presplit blasting methods, rock cuts along roadways were constructed by production blasting. As a result, many of the older rock slopes are over-blasted and highly fractured.

Over the years, weathering and erosion have affected these constructed slopes, leading to deterioration which has resulted in an increasing number of rockfall problems. These problems are addressed by the use of rockfall mitigation measures which have evolved over the years. New York State uses three (3) approaches to mitigate rockfall problems. These consist of: 1) constructing stable rock slopes by recutting the slope utilizing presplit blasting, 2) reducing the number of rockfalls by removing or securing unstable material, and 3) increasing rockfall protection by modifying the rockfall catchment areas. Additionally, the New York State Department of Transportation uses a rock slope rating system to evaluate rock slope conditions and determine what mitigation measures are appropriate to remediate hazardous rock slope conditions

Through the use of proper design and construction practices and by incorporating improved mitigation measures, this approach has yielded a cost-effective solution to problems of rockfalls along roadways.

INTRODUCTION

The performance of rockfall mitigation measures has improved with the evolution of these measures. For this presentation, one mitigation method from each of the three (3) approaches was chosen and its evolution and improvement in performance is briefly described.

BLASTING

In the late 1960's New York State was in a major roadway construction period. A large part of this work required road cuts through rock and the stabilization of roadway slopes. Blasting was required to remove the rock in a reasonable time. The standard blasting practice at the time was production blasting. Production blasting was performed by drilling holes in the rock, adding explosives and detonating the blast. This method not only fractured the rock required for the construction of the roadway proper but also fractured rock that was not intended for removal and was left in place as backslopes. This excess fracturing of the rock, or back-break, required removal of addition material which added to the cost of the projects and left fractured material next to roadways.

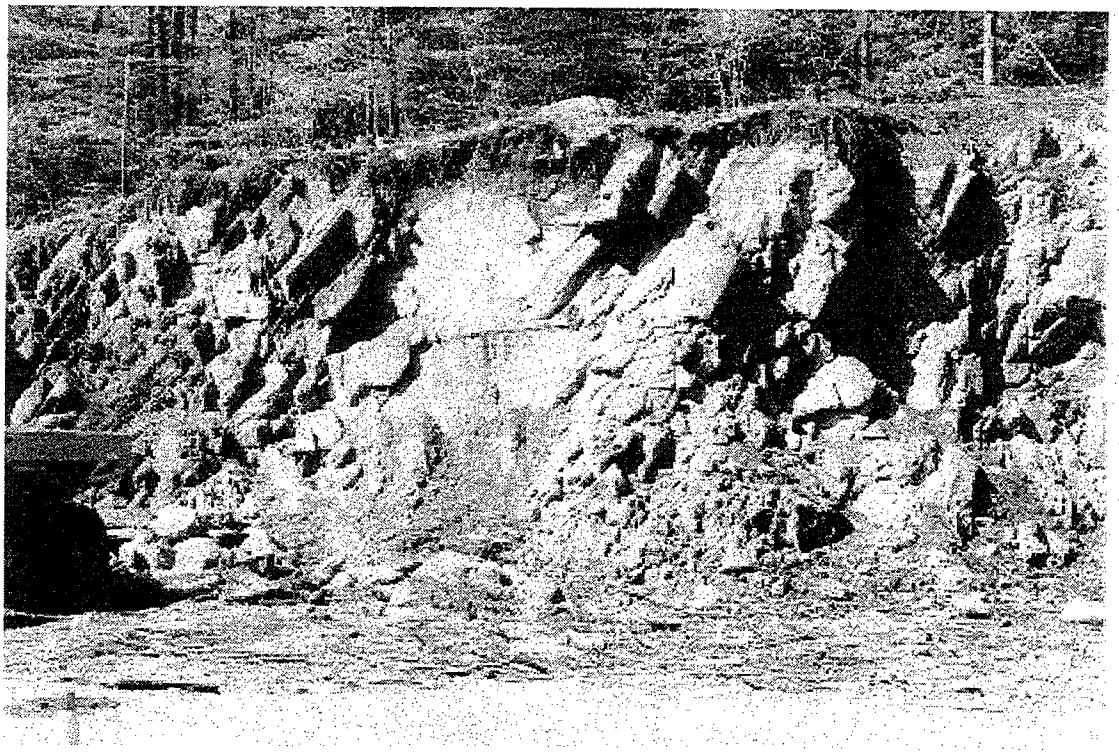

Figure 1. Production Blasted Rock Slope

In order to reduce the cost and amount of rock removal and to improve the stability of the rock face, a new blasting method, known as controlled blasting, was tried. The controlled blasting method initially tried by NYSDOT involved the use of standard production explosives and short-interval (milliseconds) delay blasting caps. The first part of the controlled blasting method was to drill a line of control holes five (5) feet on center along the desired plane of rock removal. All the rock in front of the control line, on the excavation side, was to be removed. The rock at and behind the control line was to remain as the finished face. The second part was to reduce the amount of explosives to only shear the web of rock between adjacent holes and not shatter the rock behind the control line of drill holes. This was attempted by decking or spacing out the explosives to reduce the amount of explosives per unit length of drill hole. Standard 1¼"dynamite was attached to detonation cord with a space of one foot between every stick of explosive for the length of the hole. This resulted in less explosives per length but still left a high amount of explosives in concentrated areas. This concentration, along with the large production blast in front of the control line, still shattered the rock in the backslope. Over time, these fractured areas would weather out and cause deterioration of the slope. This deterioration was more evident if the explosives happened to be placed near natural joints or fractures in the rock. Gases generated by the blast escaped into these fractures and joints, causing larger amounts of rock to fall out, destabilizing the slope even faster. While this controlled production blasting was achieving better results, over-blast was still occurring. Therefore, a better method was sought to solve the problem.

In the early 1970's the NYSDOT decided to try a new and improved type of controlled blasting called presplit blasting. Presplit blasting uses a row of closely spaced drill holes, spaced three feet on center for NYSDOT, called the presplit line. These holes are loaded with less explosives than normal production holes and detonated before or (pre) the production section of the

blast. This preblasting of the presplit line holes cracks the rock between the holes and forms a gap or split between the final face of the slope and the larger more powerful production blast. This way, when the production blast is detonated, the gases generated by it escape through this split, thereby reducing the damage to the rock behind the presplit line.

NYSDOT's current presplitting specifications require drill holes for presplitting be spaced no further than three feet on center, no larger than four inches in diameter, and drilled to a maximum depth of 60 feet per lift. Our pre-split explosives are no greater than 0.35 pounds per foot(we feel that 0.25 pounds /foot is good, 0.19 was even better), attached to detonation cord (25 grain/foot preferred), with a maximum of a 3.0 pound base charge per hole. The base charge must be placed below finished grade. The detonation cord from each hole is tied to a trunk line running on the surface to all the presplit holes. The presplit holes are detonated 25 ms ahead of any production holes in that blast. After blasting, the presplit rock face is scaled of loose material and evaluated for stability. It is then determine what, if any, additional stabilization to the slope is necessary. Each lift must be stabilized before any drilling is begun on the lower lift. Presplitting is the standard method for cutting or recutting rock slopes that require a one vertical on one horizontal (1v/1h) or steeper slope on NYSDOT projects.

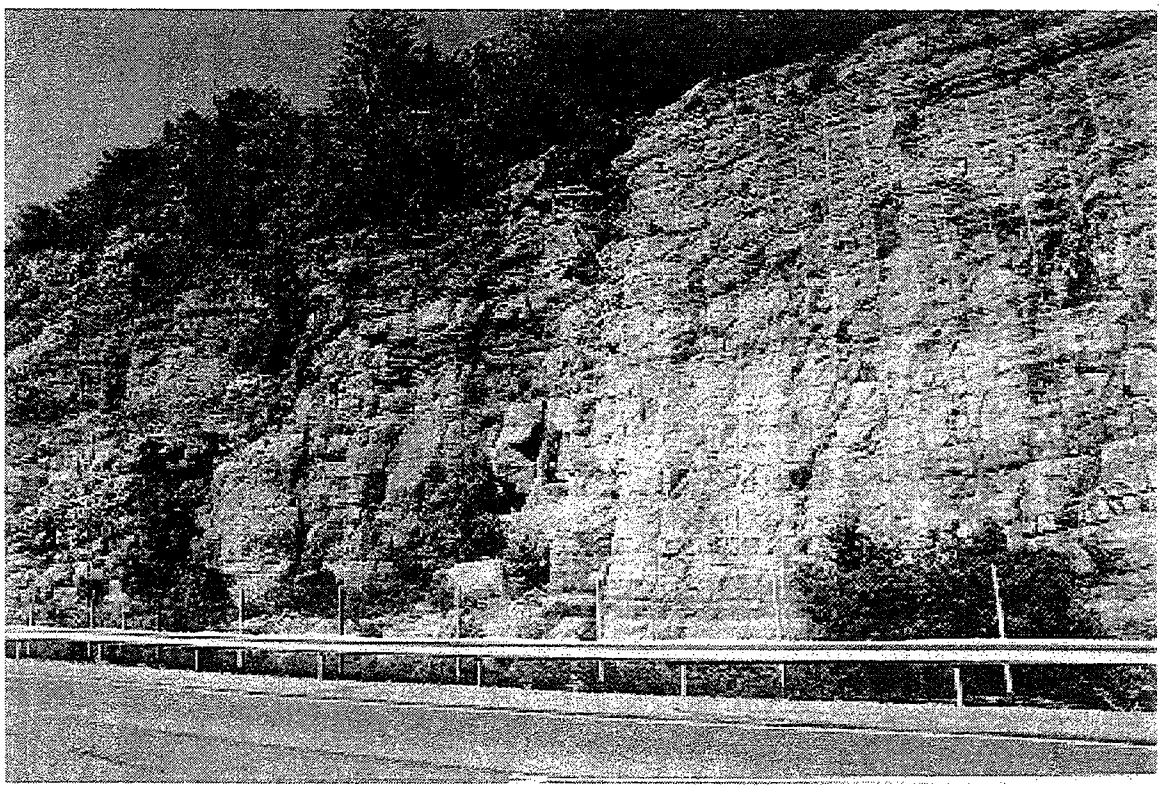

Figure 2. Rock Slope Exhibits Production Blasting on the Left and Presplit Blasting on the Right of the Slope.

We have found presplitting to be a highly effective method for controlling excessive over-break when blasting new rock cuts and recutting existing slopes. However, its effectiveness diminishes when recutting thin sections or sliver slices from existing rock slopes. Problems may develop when it is necessary to remove 5 to 10 feet of rock by presplitting on a previously over-blasted slope. This over blasted slope may have sustained damage beyond the 5 to10 foot scheduled

for removal. In such cases it may be necessary to remove more rock or use one of the other methods of stabilization after blasting is completed. A second problem with blasting thin slices is that of not enough rock (burden) in front of the explosives to force breakage of the web between the presplit holes and to not cause excessive flyrock.

Recently NYSDOT was required to construct road cuts in a State park and was requested to blast without leaving a presplit face. The park agency felt that a presplit face resulting from blasting was quarry- like in appearance and was not a "natural looking" slope. The park agency preferred the look of slopes that were production blasted years ago. In order to achieve a slope that did not look like a straight line quarry face and, at the same time, is more stable than with production blasting, was a challenge. It was decided to use a modified presplit line. Two presplit lines, one staggered in front of the other, would be used. This modified preseplit method is referred to as "staggered presplit'"

Staggered presplitting gave the rough appearance wanted by the park and more stability than with production blasting alone. These slopes are being monitored for long term effects of weathering and destabilization. There have been no major problems in the four years since they were constructed.

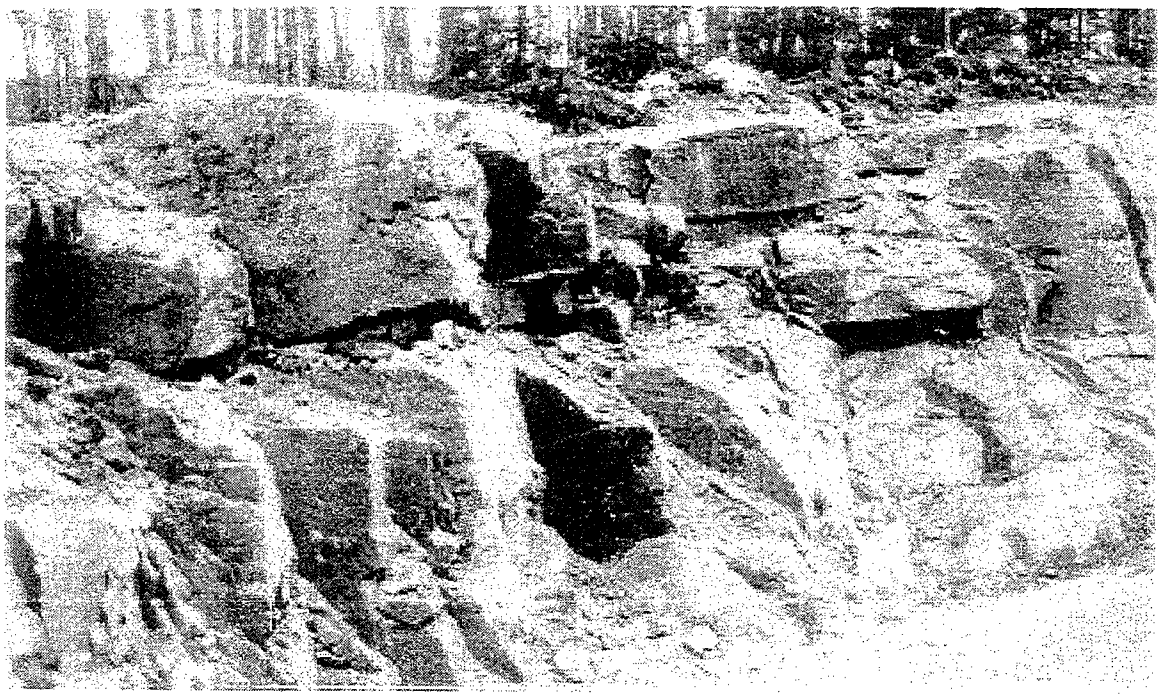

Figure 3. Staggered Presplit Rock Slope

ROCK BOLTING

Rock bolting as a method has evolved greatly over the years. New York started out using wedge anchor bolts, then used serrated expansion shell anchor bolts, and now uses prestressed resin encapsulated bolts. Wedge bolts were installed by drilling a hole in the rock to a depth three (3) inches less than the length of the bolt. A steel wedge was then inserted into the slotted end of the bolt. The bolt was placed through the opening in a steel plate and driven into the hole. A driving dolly was used to protect the threads. The wedge expanded the slotted end of the bolt providing anchorage. The nut was then tightened with the plate up against the rock. Most of the bolts used were 6 to 10 feet in length and were installed as shear pins. There are approximately 7,500 feet of installed wedge bolts in our inventory.

Serrated expansion shell anchor bolts were used next. Holes were drilled to the required depth and bolts installed perpendicular to the bearing surface. The assembly included a square-headed bolt, square plate, and expansion shell and steel plug. The bolt is tightened with a wrench and as the bolt turns, the plug is drawn down on the threads, thereby expanding the serrated section of the shell. These bolts were not grouted on NYSDOT projects. There are approximately 5,000 feet of installed serrated or expansion shell bolts in our inventory.

While investigating a rockfall, rock bolts were noticed to be among the debris material on the roadway. This fall occurred after a heavy rain storm. Large slabs of rock had slid down a failure plane and landed on the road. Rock bolts had been installed to pin a top layer of rock to a lower layer of rock with a layer of mylonite between them. All these rock bolts were found to have been installed 20 years previously. Therefore, a survey of nearby rock slope sites was performed to review the rock bolts in the area of the slide. Upon review it was found that other bolts in the area had failed. However, the majority of other bolts installed were still holding the rock. A contract was let to stabilize the slope by installing one and one-quarter inch nominal diameter rock bolts. The old expansion shell anchorage type rock bolts were saved and studied for corrosion. A review of their condition was performed and entered into our data base for analysis.

Epoxy resin bolts were used by NYSDOT only for a short time. Their use by NYSDOT was abandoned because an inordinate number of them failed the required pull test. A large, unpredictable range in curing time, probably due to temperature variations affecting the epoxy, was the reason the use of the epoxy resin bolts was discontinued.

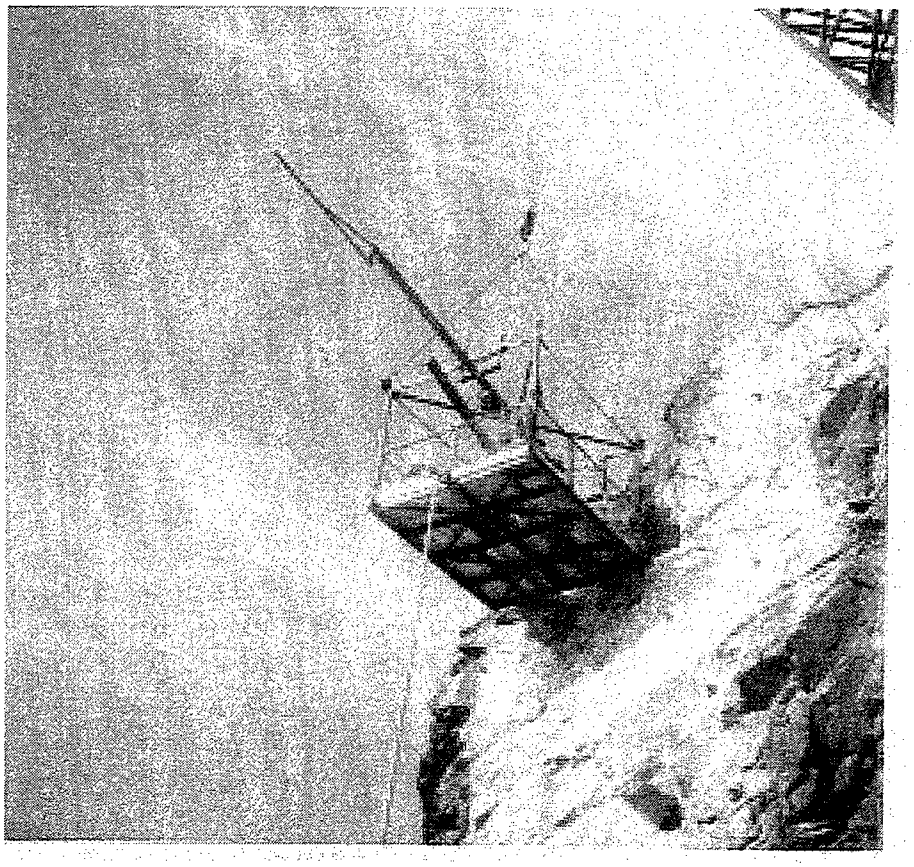

Figure 4. Installation of Thirty Five (35) Foot Rock Bolt

Today almost all the rock bolts used by NYSDOT are grade 150 1"or 1¼" nominal diameter prestressing steel. The bolts must have rolled thread-like deformations over the entire length. The high strength steel allows the use of fewer bolts and the continuous threads allows for field cutting of lengths to accommodate the length of bolt needed. Resin of the two component type, and of two different setting times, specifically manufactured for rock bolting, gives good bonding strength and is easy to install with little specialized equipment required. The anchor length can be varied to adjust for different rock types by adding different numbers of the faster setting cartridges. The slower setting resin is added to fully encapsulate the bolt after tensioning. There are approximately 25,000 feet of resin rock bolts on NYSDOT projects. The performance of these rock bolts has been good and we intend to continue using them.

NYSDOT has approximately 300 feet of cement grouted rock bolts in place today. These bolts have been used in areas of highly fractured rock where resins would not have guaranteed full contact in the hole.

NYSDOT maintains an inventory of rock bolts by their type and locations.

ROCK CATCHMENT

The third mitigation approach NYSDOT uses is to increase the catchment area for rock falls. One method is to construct a new or recut an existing rock slope with a wider and deeper ditch similar to a "RITCHIE DITCH" to catch any falling rock. Sometimes it is either too expensive or impossible to construct a Ritchie type ditch.

In the early 1980's the NYSDOT was asked by the rail line Metro North to assist them on a project. Rock and other debris was reaching a set of main line tracks. The rail line was bounded on its west side by the Hudson River and on the east by a slope, part of Manitou Mountain. Part way up this slope are State Routes 6 and 202. After investigating the area, which had no reported history of rock falls, the origin of most of the material was determined to be from the scenic rest area on the highway above. People were either throwing or rolling the rocks and other debris down the slope. Literally anything that could be thrown off the slope was, including a safe from a robbery! Stabilization of rocks at the points of origin was impractical due to their size, most of the rocks being less than 2 feet in diameter. Stabilization was impossible for the debris. Construction of a catchment ditch was not possible due to a high rock slope next to the train tracks that would require a large amount of excavation. Access to the site was limited due to the shear cliff faces, dense vegetation, and talus that covered the slope. It was determined that a low impact rock catchment fence would be a viable solution. The location, along with the availability of construction materials, were among factors considered during design of the fence. Fence placement location was determined via the use of a computer program written by NYSDOT personnel along with photos taken from a good vantage point across the river. The computer simulated rocks rolling and sliding down the slope. The photos were used to locate the best terrain for the placement of the fence. The fence, for the most part, was designed using the standard specifications for highway cable guide rail components. Some of these components, along with fence fabric, and re-bar posts, make up the fence. The fence was intended to deform and stretch upon impact, attenuating the kinetic energy of the rolling rocks in the process, while restraining them from further movement. For construction, most of the components were carried in by hand from the top of the slope by laborers. The terrain was so rugged that the workers had to string cables to tie off to for safety while working. The fence was nearly complete when a rockfall occurred. The cable splicing hardware and heavy cable clamps that were to attach the tie-back cables to the re-bar tie-back anchors were not in place yet. Approximately fifty (50) tons of rock fell from the rock face 180 feet upslope from the fence. This mass slid and tumbled down the thirty eight degree(38) talus slope, knocking other rocks loose in the process. Most of the rock remained behind the fence, the largest single piece having a volume of over six (6) cubic yards. The fence suffered severe deformation in the center of the impact area. Other rocks weighing approximately 1,000 pounds which struck the fence outside the center of the slide area did not do any serious damage to the fence. However, rocks still reached the tracks, the fence having been overtopped by the sheer volume of the rock which fell. Even though the fence was not designed for a major slide and was not quite complete in the vicinity of the slide, the overall performance of the catchment fence was quite impressive. The rock fall provided an unexpected large scale test of the rock catchment fence.

Since that time, we have installed more low impact fencing, and have had additional rock falls at other sites. The fences having performed well. NYSDOT also uses medium impact wire rope fences on some of its projects with good results. These medium impact fences are also used as temporary rock catchment fences on construction projects during scaling and blasting operations.

These fences have performed very well. Installation of fences, especially in areas of right-a-way problems where recutting is difficult or impossible, is a preferred alternative.

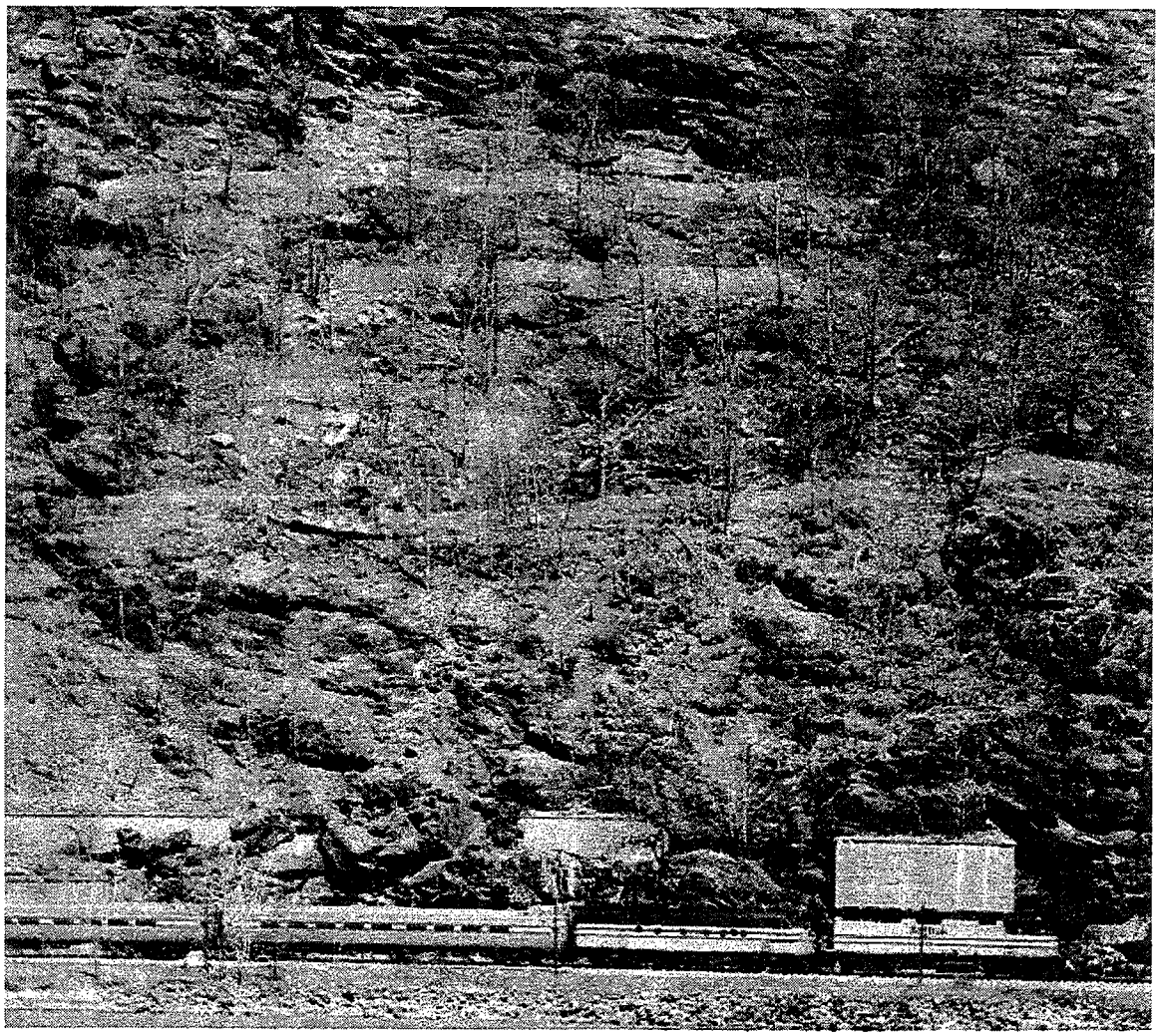

Figure 5. Rock Catchment Fences With Train Passing Below

CONCLUSION

These three mitigation measures are only part of a much larger system. NYSDOT's rock fall mitigation system includes a rock slope rating system to evaluate rock slope conditions and determine which mitigation measures are appropriate to remediate potentially hazardous rock slope conditions. In addition, an inventory of all reported rock falls is maintained. This inventory includes reports that contain the location and digital photos of the fall. The amount of material and area covered, along with any information on resulting accidents, are included. This information allows effective utilization of available resources for the construction and maintenance of rock slopes along roads by documenting rockfall events by frequency and location.

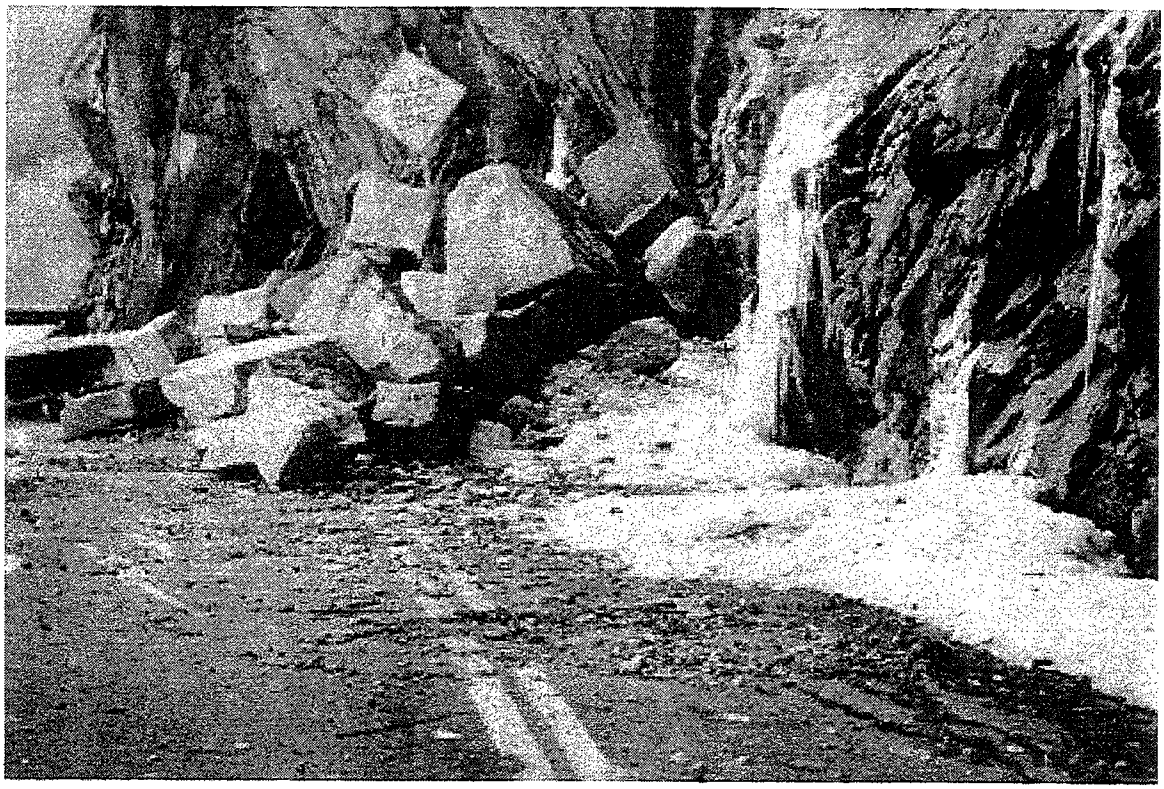

Figure 6. Rockfall on Route 218

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TUNNEL TO THE 21st CENTURY

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ABSTRACT

The current alignment of two-lane US 71 across the rugged Ozark plateau, between Ft. Smith and Fayetteville, Arkansas, was designed in the 1920s. With population, traffic, and trucking growth since the 1960s, this winding route has become known for travel delay and accidents. The Arkansas State Highway and Transportation Department proposed a new route in the 1970s, but it was not until the late 1980s that environmental interests could be assuaged and construction began on a replacement highway for the old US 71 route.

This new four-lane divided highway is being built to Interstate standards. The roadway will consist of two-3.6 m lanes, with a 3 m right shoulder and a 1.2 m left shoulder. The combination of the high design standards and the rugged terrain necessitated mining an approximately 0.42 km long, twin-bore tunnel near Winslow. There is an additional 67 m of cut-and-cover tunnel on the ends. The maximum cover from the tunnel top to the ground surface is 54 m. Each of the two flattened-arch bores was excavated to a width of 14.4 m at the base.

The tunnel was constructed through shale laminated with sandstone seams. The contractor used the drill-and-blast construction method. Work began with removal of the top 5.5 m; followed by the slash vertical mining of the center section of the remaining 3.1 m bench. Then, the lower corners of the bench were excavated. Work progressed from both ends, with side work staggered. The opening was lined with a waterproofing membrane, then lined with 12,000 cu. m. of concrete.

A 14.6 m long roof cave-in occurred in one of wettest parts of tunnel, where there were 14 unsupported sets (steel ribs) spaced on 1.2 m centers. Fortunately, no one was near the area when the cave-in occurred. The void was backfilled with concrete.

Mining began in August 1996, and was completed in the summer of 1997. The estimated completion date is September 1998. The construction contract was for $37,000,000, which included about 20 m of external work on the south side and about 550 m on the north end to get down to grade (e.g., outside-of-tunnel retaining walls, excavation). The contractor is J.F. Shea of Walnut Grove, CA., and the project manager is Parsons Brinckerhoff of Dallas, Tex.

INTRODUCTION

Since the days of the Butterfield Stage Line, travel from St. Louis to the southwest has been difficult. This difficulty is due to the barrier created by the rugged Ozark Plateau in an otherwise flat-to-rolling landscape across the mid-section of the United States. The pre-Civil War stage line from St. Louis to Ft. Smith and on to California deviated west, almost to the border with what was then Indian Territory, to avoid the terrain directly south of Fayetteville, Arkansas. When the St. Louis-San Francisco Railroad constructed its first extension from near Springfield, Missouri into north central Texas, the engineers dealt with the problem by constructing a 519 m (1702 ft) long tunnel under the summit of the Boston Mountain range at Winslow, Arkansas in 1882 *(Bryan, 1987)*. Daily freight trains still trudge through this tunnel.

The current two-lane US 71 route between Ft. Smith and Fayetteville, Arkansas was designed in the 1920s. This route twisted and turned to cross the mountains, but did so without the need of a tunnel. As population, traffic, and trucking grew after the early 1960s, this winding route was becoming inadequate.

In the late 1960s, local officials identified the need for a replacement route. The Arkansas State Highway and Transportation Department (AHTD) proposed alternatives in the early 1970s, but there was opposition *(Martin, 1994)*. Part of the opposition was based on concerns about disturbing the environment and natural beauty of the area. A proposed route that lay to the west of the present US 71, in less rugged terrain, also passed through a national forest and a state park. Opposition effectively stymied progress. The first construction contract for a replacement highway for the old US 71 route between Fayetteville and Fort Smith was finally let in 1987.

By the mid-1990s, average daily volumes on the old US 71 ranged from 9,000 to 12,000. The inadequacies of the road manifested themselves not only by delay, but also by crashes and deaths. In 1992 there were 7 fatalities on this 46 mile segment, and in 1993 there were 9. Of these 16 people killed, 9 resided outside of the two counties which the route occupies *(Robinson, 1994)*. This fatality rate is more than double the national rate of 1.75 fatalities per 100 million-vehicle miles for 1992 and 1993 *(Gross and Feldman, 1995)*.

The need for a new highway was indisputable. The route chosen and on which construction began in the late 1980s was east of the route that had been opposed in the 1970s, and therefore was in more rugged terrain. The replacement four-lane divided highway is being built to Interstate standards. The maximum grade is limited to 5 % and the minimum radius is 698 m (2291ft). The new 69 km (43 mile) long roadway will consist of two-3.6 m lanes, with a 3 m right shoulder and a 1.2 m left shoulder. The combination of the high design standards and the rugged terrain necessitated choosing between either a massive cut or a tunnel. After considering initial costs and long term maintenance costs of both options, engineers decided to mine a 0.42 km long, twin-bore tunnel near Winslow, AR.

While highway tunnels are not uncommon in the Appalachian or Rocky Mountain ranges, mountain tunnels are seldom found in the mid-section of the United States. This was the first highway tunnel ever undertaken by the State of Arkansas.

SELECTING THE DESIGN APPROACH

During the preliminary design stages of the route that was to replace the old US 71 alignment, serious consideration was given to restricting earth moving activities to cut and fill, and avoiding the need for a tunnel. The cut-and-fill approach would have entailed construction of a 366 m bridge for County Road 41 across the proposed alignment and the environmental problems associated with

disposing of over 2.7 million m^3 of spoil resulting from a 60 m cut through Bunyard Mountain. In the final analysis, the tunnel approach was deemed less disruptive.

Since AHTD had no previous experience with the design or construction of tunnels, officials chose to contract with engineering firms to design the tunnel and manage tunnel construction. The primary design consultant for the portion of the relocation project in Washington County was Garver and Garver of Little Rock, AR. Garver and Garver subcontracted the tunnel design to Sverdrup Civil, Inc. of St. Louis, MO. This was also the first construction project ever undertaken by AHTD where an external construction manager was used. The project manager selected for the tunnel construction was Parsons Brinckerhoff of Dallas, TX.

The final design submitted by Sverdrup Civil called for two parallel concrete lined, mined rock tunnels, each approximately 420 m long. The twin bores would be connected by five small cross-passages located on approximately 80 m centers. The tunnels would have a maximum rock cover approaching 54 m. Each tunnel was to support the full standard traffic lanes and shoulders within an 11.6 m traveled way. Each tunnel required an excavation approximately 13.7 m wide and 8.9 m high. The mining method was to be a variant of the New Austrian Tunneling Method (NATM), where rock bolts and shotcrete provide the initial ground support. The drive was to be accomplished by staggering the advance in five sections across the face of the heading. The ground support design called for 3.6 m by 35 mm bolts on 1 m centers along with a 200 mm reinforced shotcrete liner. In addition to the mined portion, each tunnel included a 67 m cut and cover section at the north portal and a 6 m cut and cover section at the south entrance.

CONSTRUCTION STRATEGY

The $37,000,000 construction contract for the tunnel was awarded to J.F. Shea of Walnut Grove, CA. This contract included about 20 m of external work on the south side and about 550 m on the north end of the tunnel to get down to grade (e.g., outside-of-tunnel retaining walls, excavation).

The contractor immediately proposed a no-cost change to the specified mining technique for the tunnel. Instead of using the staggered advance proposed by the designer, the contractor elected to utilize a full face mining technique and install a series of steel arches with concrete lagging to supply the necessary ground support. A reinforced concrete-finish liner would follow this initial support system.

Mining

Mining began in both bores at the south portal in August, 1996. In September, mining from the north portal commenced. Figure 1 depicts the initial mining at the southern portal. During the bulk of the mining operation four headings were worked simultaneously. The revised design called for an overall excavated height of 8.8 m. Each of the two flattened-arch bores was excavated to a width of 14.4 m at the base. The southern headings were advanced at a 2.5 percent grade while the bores from north to south were made on a 4 percent grade. The design called for the two drives to meet at approximately the 60 percent point in the tunnel, as measured from the north end. The invert elevation of the north portal is 5 m lower than that of the south portal.

The tunnel was constructed through a Mississippian-age formation of interbedded sandstone and shale. The beds dipped generally from south to north and the sandstone seams ranged from 2.5 m to only 1 or 2 mm in thickness. Virtually all of the shale was classified as soft or very soft rock, and approximately 85% of the mined material was shale.

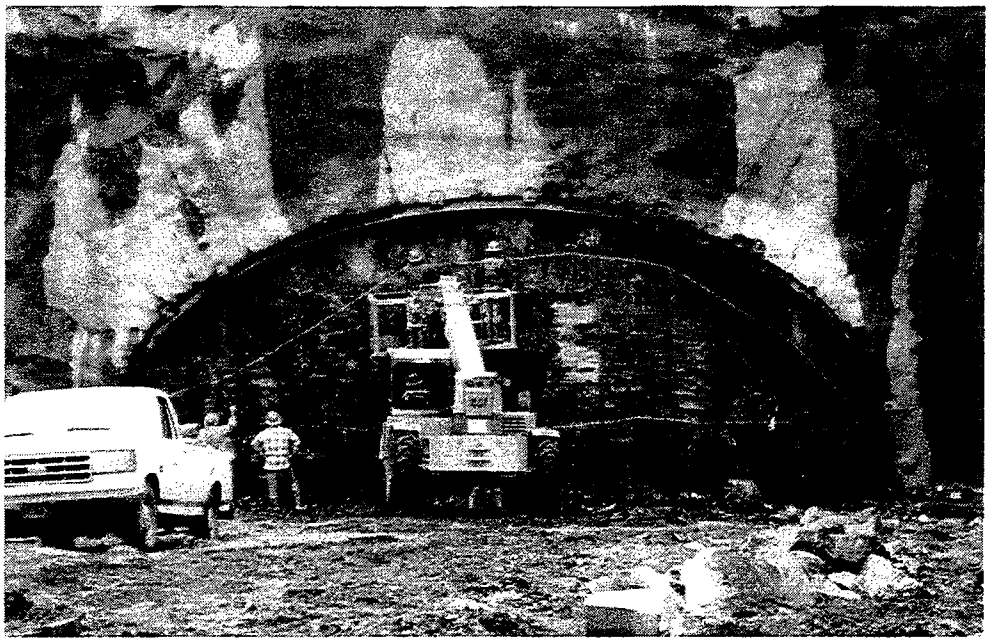

Figure 1. Initial mining operation at the face of the north portal.

The contractor used the drill-and-blast construction method. Each heading was advanced by removal of the top 5.5 m of the tunnel cross-section. Advancement from the northern headings generally proceeded more rapidly than from the southern headings. The slope of the drive from the north was nearly parallel to the bedding planes of the shale and sandstone, which resulted in very little overbreak during blasting. By contrast, the slope of the drive from the south crossed the bedding planes, resulting in some lost ground after every blast. An additional factor aiding the drive from the north was a 2 m sandstone seam in the roof area that miners were able to follow for a substantial distance. This seam allowed for easier mining and longer stand-up times before the installation of the concrete lagging. Figure 2 illustrates the method of providing ground support.

All headings were advanced in up to 4 m increments by drilling between 90 and 120 holes 3.7 m into the full face and charging them with approximately 1 Kg/m^3 of explosive. The blast left between 5 and 6 meters of tunnel unsupported during the mucking operation. After the mucking was completed, four W8x40 steel arches were set on 1.2 m centers and the concrete lagging was placed between each. Support was brought to within one meter of the face before the next cycle of drilling and blasting was undertaken. Working in this fashion, crews at each heading worked in a feverish competition to see who would hole-through first. Both bores holed through at virtually the same time in March 1997. The slash vertical mining of the remaining 3.1 m bench followed mining of the initial bores. Mining of the bench section was accomplished by rubblizing the middle third of the bench by vertical drilling and blasting. The resulting rubble was left in place. Once the middle section was rubblized, the corners of the arch sections were mined by horizontal drilling and blasting. The entire bench face was then mucked. As depicted in Figure 3, the W8x40 steel ribs acting as the ground support were extended with stub columns as the mining of the bench progressed. The total volume of excavated material was 95,000 m^3.

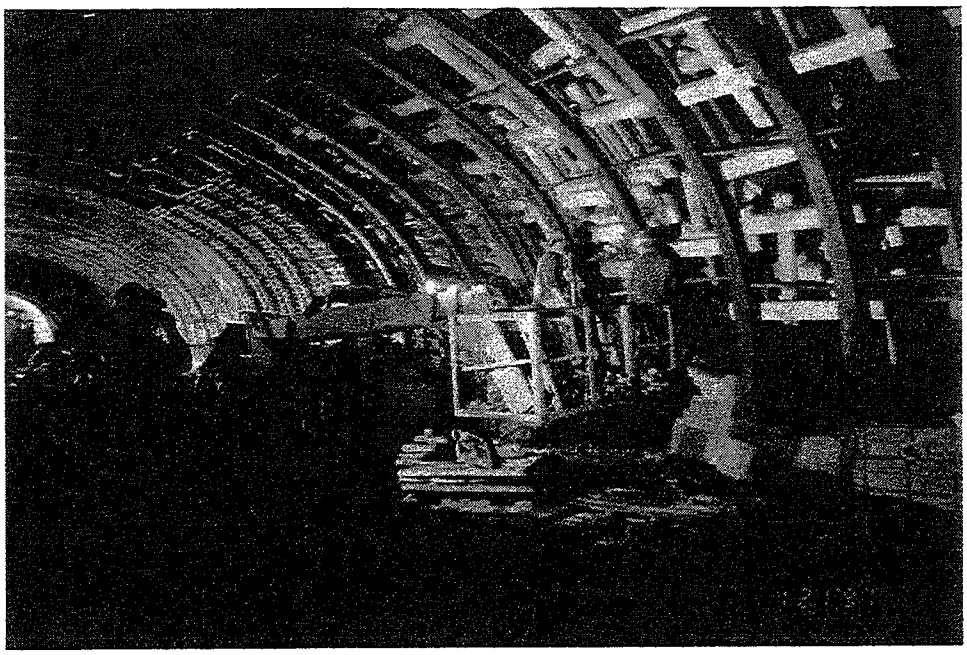

Figure 2. Wide flange sections, formed into ribs, provide the initial ground support. Timber forms and concrete lagging are shown in the background.

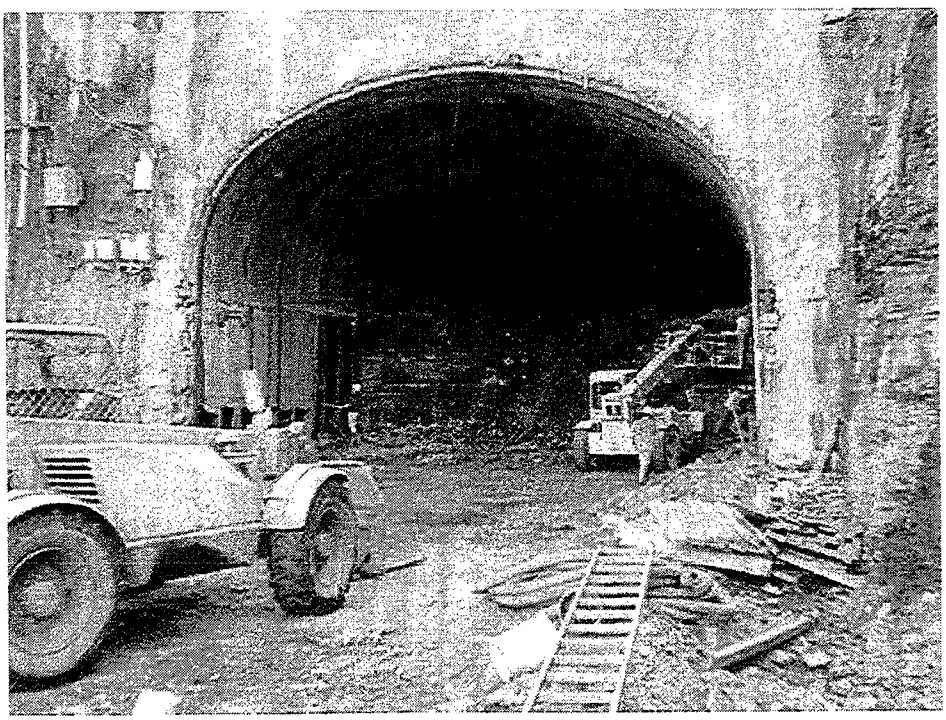

Figure 3. Workers drill the corner sections of the arch bench In the foreground while, in the background, others install extensions to the steel ribs of the ground support system.

Tunnel Liner

After the mining operation was completed in the summer of 1997, the walls of the initial ground support system were lined with a 1.5 mm thick PVC geomembrane. This waterproofing membrane, depicted in Figure 4, was designed to divert overhead ground water into a longitudinal drainage pipe located at the base of each arch leg. Over 20,000 m^2 of liner material was attached to the ground support system with adhesive-faced Hilty nails that were driven into the concrete lagging. The membrane was seamed transversely at 1.5 m intervals using a double-wedge heat fusion process. Construction of the inner liner commenced in the cut and cover section of the north portal in August of 1997. Initially, a 2.5 m barrier and stem wall combination was placed on each side of the arch cross-section using an 18 m curb form. These stem walls served as a track and guide for the traveling arch form, illustrated in Figure 5, which was used to produce the smooth inner liner in 18 m sections.

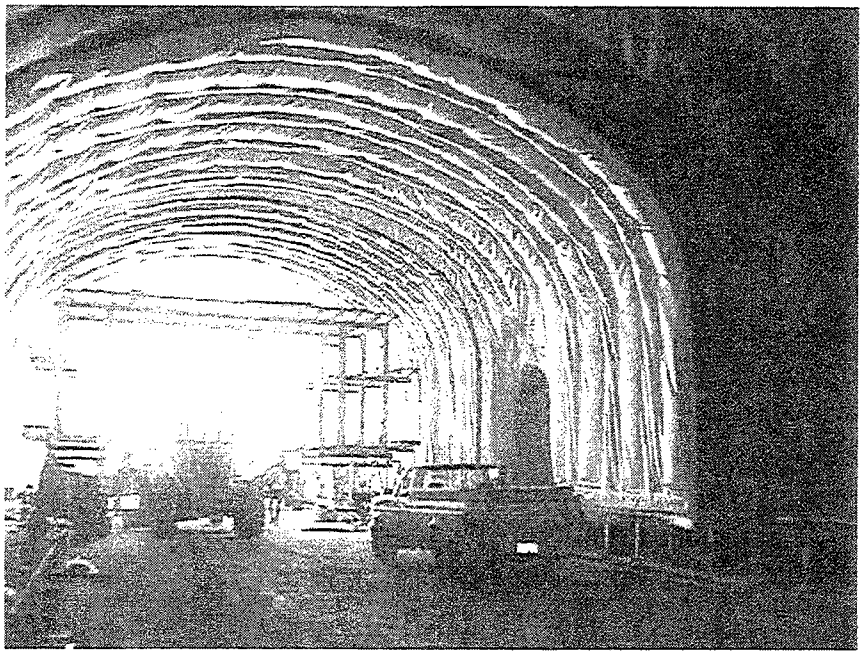

Figure 4. Attachment of the PVC waterproofing membrane to the initial ground support system.

While the cut and cover sections were being cast the cross passages within the tunnel were mined. The liner system was completed in the third week of January,1998. In all, approximately 12,000 m^3 of concrete were placed in the 470 mm reinforced concrete inner liner and cross passages.

Other Associated Work

In addition to the 420 m twin-bore, there was an additional 67 m of cut-and-cover tunnel on the ends and 250 m of reinforced concrete earth retaining structures at the portals. Five cross passages connect the adjacent twin bores. Ten 75 horsepower fans, five in each bore, vent the tunnel. These fans are electronically controlled and force air out of the tunnel in the direction of traffic when CO levels reach a predetermined level. Water for fire fighting is stored in two 500 m^3 steel storage

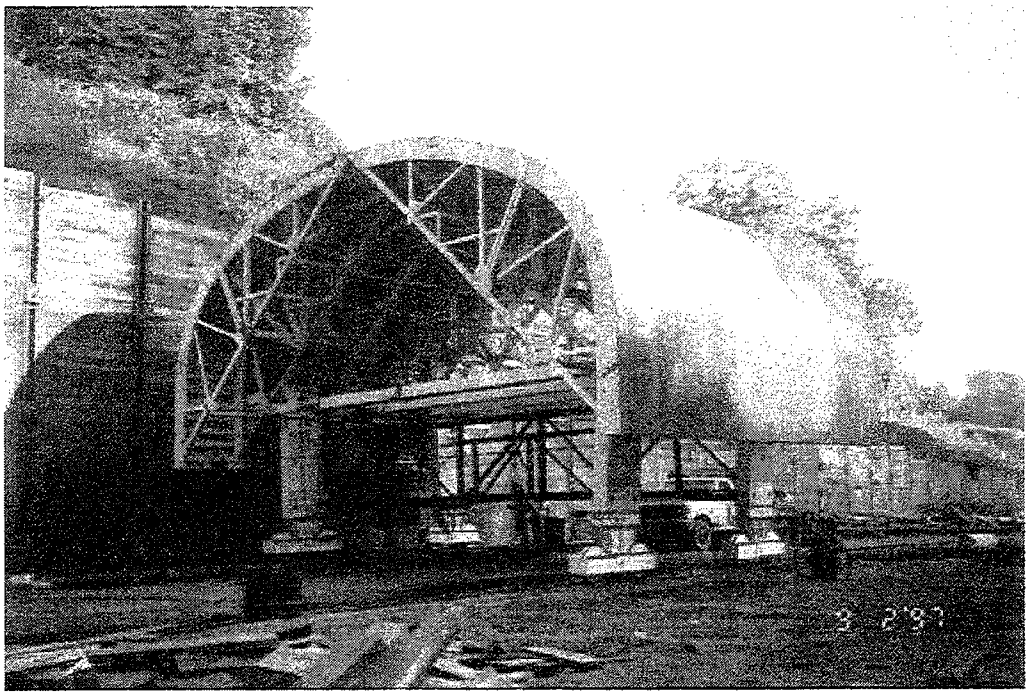

Figure 5. Traveling arch form use for placement of the final reinforced concrete tunnel liner.

tanks located 55 m above the mid-section of the tunnel. This water is supplied by gravity to two 203 mm standpipes that run the length of each bore. Ten video cameras will monitor the inside of the tunnel and motorists will have 29 call boxes available in case of an emergency. All monitoring and control of the electrical and mechanical devices will be performed at the highway department's Ft. Smith district office via a fiber-optic cable. A typical tunnel bore cross-section is illustrated in Figure 6.

PROBLEMS ENCOUNTERED

The most newsworthy problem encountered during the construction of the tunnel was a 14.6 m long roof cave-in. The cave-in occurred in the eastern (northbound) bore approximately one third of the way through from the southern portal. Even though both the contractor and construction manager described this tunnel as one of the driest they had ever seen, the failure occurred in one of the wettest parts of tunnel. At the time of failure, there were 14 unsupported sets (steel ribs) immediately below the cave-in that had not had received the concrete lagging. Fortunately, the cave-in occurred over a weekend and no one was near the area. The cause of the failure was believed to be the result of a major seam of shale becoming weakened through exposure to water. Investigation of the failure revealed that the bulk of the failed material was very weak shale and the roof of the failed cavity was a solid seam of sandstone of undetermined thickness. Basically, the weak material broke out to a zone of stronger material. To correct the cave-in problem, the void and rubble pile were pressure grouted with over 1000 m^3 of lean concrete. Return pipes were placed in the upper reaches of the void so the contractor could verify that the void was completely filled during the grouting process. The heading was then re-mined in the normal fashion. It is estimated that this

failure resulted in approximately two weeks of lost time to one of the four hedings, or three days to the overall project.

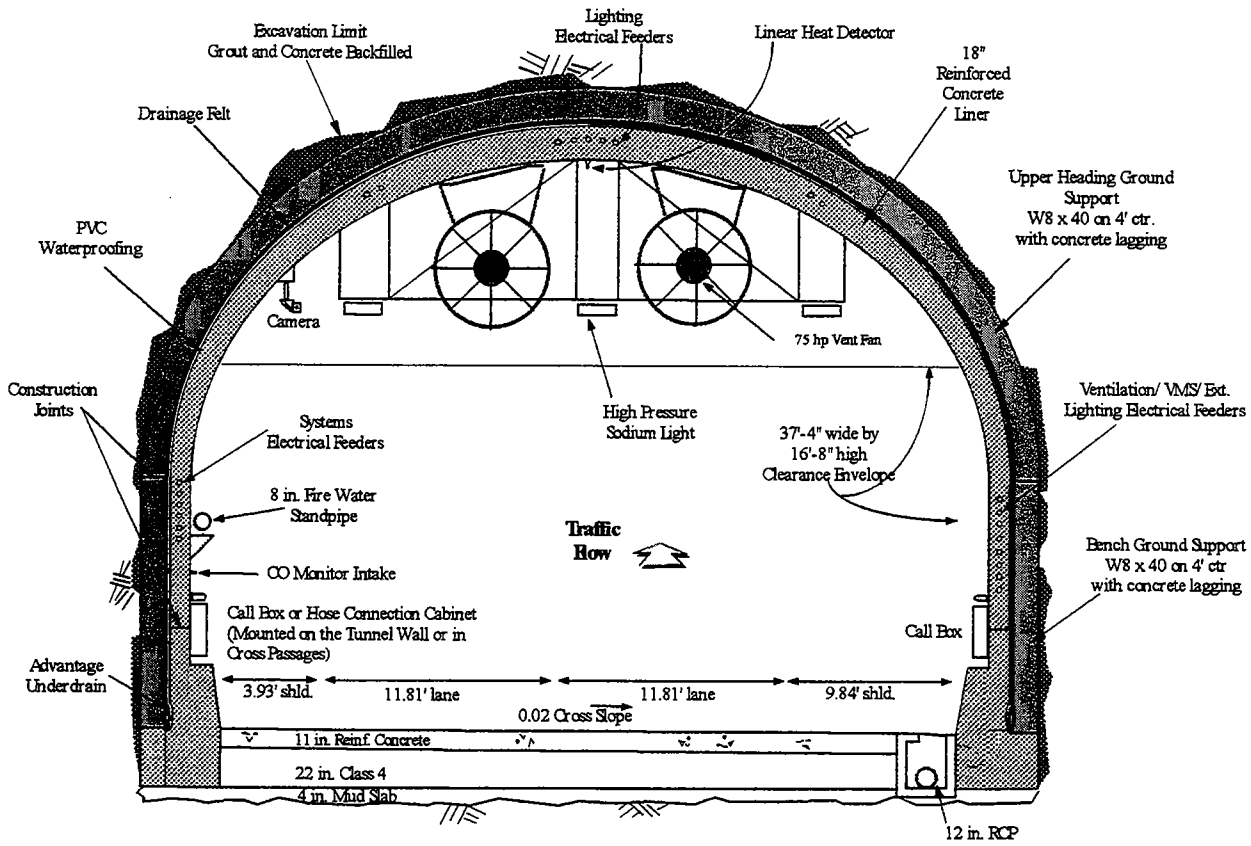

Figure 6. Cross-sectional view of one tunnel bore, illustrating the placement of mechanical and electrical devices.

CLOSING

Now nearing completion, the tunnel was recently named "Bobby Hooper Tunnel" in honor of the state highway commissioner who represents that part of the state. The estimated tunnel completion date is September 1998. Both tunnel portal areas will offer the traveler scenic mountain vistas.

Travelers are eagerly anticipating the completion of the replacement to old US 71, which is scheduled for late 1998. Area newspapers and television stations regularly report the progress of construction. The replacement will not come soon enough for the survivors of crashes and the families of those killed on the present outdated highway. The centerpiece of the new highway, built to standards expected by today's motoring public, is the twin-bore tunnel. The new highway and tunnel should save not only traveler's time and money, but also their lives.

ACKNOWLEDGMENT

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TUNNEL ENLARGEMENT AND REHABILITATION

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Abstract

Clearance improvement or rehabilitation of old tunnels can be a significant challenge. The stability of the lining, the condition of the ground behind the lining, and the backfill behind the lining are often difficult to investigate. Removing the old lining and supporting the surrounding ground poses unique problems. The investigations and construction of improvements must often be conducted during short construction windows so that the tunnel can be open to traffic for the remainder of the day. Consequently, flexible, innovative approaches are required for exploration, design, and construction of these projects.

Introduction

As road and rail traffic increases throughout the country, many old tunnels need to be enlarged or repaired. This work is often more challenging than driving new tunnels. Many existing tunnels, especially railroad tunnels, are more than 100 years old and have no as-built design drawings or construction records. The attention to maintenance has usually varied over the life of the tunnels, and in some cases, has been discontinued entirely. There are many possible effects of aging, most of which can be accelerated by discontinuous maintenance. In some cases, the methods of maintenance actually damaged older structures.

The investigations and construction for tunnel improvement projects are often greatly complicated by the necessity to conduct the work under "live" conditions, where work is allowed only during limited intervals each day, and the tunnel must be returned to service in a stable condition at the end of each work period. In many cases, contracts stipulate the assessment of penalties to the contractor for returning the tunnel to service later than the specified time. Where commuter traffic is involved, these penalties are sometimes tens of thousands of dollars per minute.

This paper presents experiences on several recent projects involving tunnel rehabilitation and clearance improvements. Most of these projects have involved upgrading old rail tunnels for railroad companies who wish to increase the clearance on their lines so that they can transport double stacked freight containers. However, several of the projects have comprised rehabilitation and enlargement of highway and rapid transit tunnels to improve safety or to increase traffic envelope.

Causes of Aging and Deterioration in Tunnels

Many old tunnels are in good condition. The strength of some types of brick, mortar, masonry, and concrete actually increase with age, and most types of rock do not weather significantly in the life span of a tunnel. However, the lining of many old tunnels and the ground conditions in some tunnels have deteriorated markedly.

When investigating the conditions in old tunnels, the four broad categories of alteration of materials need to be evaluated. These are physical processes, chemical processes, biotic processes and human activities. The cause of deterioration must be fully understood before repairs or improvements can be implemented in order to avoid the same problems recurring.

A detailed description of each of the four categories of material alteration listed above is beyond the scope of this paper. However, from the experience gained on the projects described later in this paper, the primary causes of aging and deterioration in tunnels are:

- Water: Undoubtedly, water causes the greatest number of problems in old tunnels, mainly in the form of ice damage; ice build-up; freeze-thaw damage; water borne chemical deterioration of bricks, mortar and wood; poor drainage; and waterlogging of track/roadway subgrades.

- Human Activities: Changes to loading conditions due to fill placement or excavation can cause distress in the lining or portal structures. Drainage modification can result in increased seepage into a tunnel, ponding of water, or ice build-up.

- Inadequate Repairs: Repairs to tunnels have often been made at minimal cost and by personnel who do not understand tunnel support. In many instances, the repairs made by applying a thin skin of gunite, pointing mortar or replacing damaged bricks have themselves become hazards.

Support and Lining of Old Tunnels

Many older tunnels were constructed using combinations of the following two ground support methods:

1) Where the rock was sound, no rock support was installed. Portal structures were constructed of stone masonry or concrete for architectural reasons, or to prevent rockfalls.
2) Where ground conditions within tunnels required support, a brick, stone masonry or lagged timber lining was installed. In most cases, the linings were constructed as free standing structures, leaving voids behind the lining. These voids were occasionally backpacked with shot rock, soil materials, or wood, but were often left open. Where voids or compressible materials were left behind lining structures, the excavation often relaxed and failed progressively upward, or stoped. In some cases, the weight of loose rock on the liners caused damage or failure of the lining. In other cases, linings were strong enough to support the added weight, but modifications to the lining may result

in damage or failure. The loose material between the lining and the in-situ rock or soil often collapses around drill bits making it particularly difficult to investigate the tunnel condition.

Typical Issues

Each old tunnel has unique problems and issues that need to be addressed when repairs or enlargements are being considered. A few of the more common concerns are discussed below.

Some tunnels have had a degree of repair installed over the years. These repairs were generally in the form of passive restraint systems such as installation of timbers, concrete linings, or a thin skin of gunite. These repairs, often effected as emergency measures, now can contribute to problems as they themselves begin to fail.

A particular concern in many tunnels is seepage and ice buildup. Seepage can have deleterious effects on any of the lining systems, and can increase the need for maintenance of the roadway or the track. Ice in tunnels can have serious effects on bare rock and, especially, on lining structures. Ice jacking has been observed to cause failures of concentric layers of masonry lining, movement of massive stone blocks in masonry lining, and spalling of poorly drained concrete and shotcrete linings. Where temperatures remain below freezing for significant lengths of time, ice can build up in tunnels, reducing clearance. When ice buildup becomes critical, manual labor is sometimes employed to remove ice from track structures. Such ice removal efforts are time consuming, can be extremely hazardous to labor, and often damages the lining or drainage system.

An additional concern can be the presence of portal or other structures that have been declared historically significant. Special care may be required to preserve these structures, and certain types of clearance improvement and ground support may be required by historic preservation organizations. In some cases, the historic structures may need to be dismantled and reconstructed using the same materials.

For rehabilitation or clearance improvement of tunnels which will be used for motor vehicles or mass transit, there is little tolerance for rockfall, uncontrolled drainage, or ice buildup. In most cases, such projects must be finished with full lining systems and enclosed drainage. Where tunnel rehabilitation or clearance improvements are being conducted strictly for freight railroad use, a greater degree of tolerance for minor rockfalls usually aids in effecting a more economical solution for final ground support. In some tunnels, rockfalls of less than three cubic feet were considered to be of only minor concern.

In some instances, it is possible to lower the roadway or track bed to provide additional tunnel clearance. However, this usually requires the tunnel to be taken out of service for many weeks. In many instances, the approach grades of bridges near the portals cannot be modified to accommodate lowering the invert of the tunnel. Consequently, this paper focuses on stabilizing and raising the crowns of old tunnels, rather than lowering the inverts of tunnels.

The Projects

The following section describes a few of the tunnel rehabilitation and clearance improvement projects that have been completed over the last ten years. The methods used include rock and lining stabilization and reinforcement; lining removal; notching in lining and rock crowns; open cut excavation; portal removal and replacement; drainage improvements; and prevention of ice build-up.

Tunnel: *"Little Tunnel"*
Scope of Work: *Stabilization*
Location: *Cumberland Gap, Tennessee*
Owner: *Federal Highway Administration and National Parks Service*

As part of the Cumberland Gap Highway Tunnel Construction Project, the National Park Service needed to stabilize a 100-year old, disused railroad tunnel. This tunnel, called "Little Tunnel," is currently being used as a utility corridor during the project, and will shortly be converted for use along a bicycle path.

Two ground conditions were encountered within the tunnel. The western half of "Little Tunnel" was constructed through poor quality shale, but had been lined with a brick masonry arch which was in excellent condition. By contrast, the eastern half of the tunnel had been constructed in good quality limestone, but had been lined with timber arches and wooden lagging. The wooden lining and timbers were in varying states of decay, with wood structures sound in some areas, and completely rotten in other areas. Rock above the lining had stoped up to form a stable arch, but the relaxed rock was applying a significant loading to the timber arches in some locations.

To facilitate the immediate use of the tunnel for utilities, stabilization of the tunnel involved removing the worst timber arches, stabilizing the newly exposed rock in the tunnel crown with rockbolts and shotcrete, and strengthening other, marginally stable timber arches. The permanent tunnel stabilization, for use as a bicycle path, will consist of a large steel culvert pipe installed in the tunnel, with the surrounding void backfilled with pea gravel.

Tunnel: *Montgomery Tunnel*
Scope of Work: *Vertical Clearance Improvement*
Location: *Montgomery, Virginia*
Owner: *Norfolk Southern Railroad*

As part of a regional track clearance improvement program to carry double stacked containers, Norfolk Southern Railroad needed to raise the crown of Montgomery Tunnel by 2.5 feet. The 100 year old, 650 foot long tunnel was fully lined with concrete. Challenges included investigating the ground conditions behind the concrete lining, developing designs and construction methods for ensuring the stability of the tunnel crown when the concrete arch was removed to increase clearance, and conducting the work in four to eight hour daily windows of track time. After each work window, the tunnel had to be left open and sufficiently stable for passage of trains.

The design selected consisted of two steps:

1. Side walls were stabilized in place using rockbolts, and the voids behind the walls were filled with grout.
2. The concrete arch crown lining was removed in short sections using a roadheader (Photograph 1), and the crown rock was supported using either rockbolts and shotcrete or lattice girder and shotcrete arches.

Photograph 1: Roadheader used for removing the old concrete lining.

Tunnel: *Berry Street Tunnel*
Scope of Work: ***Rehabilitation and Horizontal Clearance Improvement***
Location: *Pittsburgh, Pennsylvania*
Owner: *Port Authority of Allegheny County*

As part of a mass transit corridor project, the Port Authority of Allegheny County rehabilitated the abandoned Berry Street Tunnel. This 140 year old brick masonry lined tunnel was constructed through gently dipping Pennsylvanian cyclothemic beds, with weak underclays and shales present through much of the tunnel length. The masonry lining was in poor condition, with as many as three courses of masonry having failed in some locations. Fall of bricks was a safety concern during exploration and construction. The tunnel also experienced significant inflows through the lining, and poor drainage led to pooling of over a foot of water across the entire tunnel invert at the downslope end of the tunnel. A residential neighborhood had been constructed above the tunnel after the tunnel was completed.

Because the tunnel was to be used for bus traffic, vertical clearance which had been improved prior to 1900, was adequate. However, horizontal clearance improvements were required, as well as crown stabilization. The geologic conditions in the tunnel were evaluated by probe hole drilling through the lining, borehole camera surveys in the probe holes, rockbolt anchorage testing, and trial grouting behind the lining. Camera surveys and drilling conducted for the surveys indicated that significant voids had been left behind the tunnel lining. In some cases, these voids had been backfilled with rock, wood, or other materials. In other locations, rock had stoped up, and detached, deteriorated rock was present above the lining.

The tunnel was stabilized and enlarged by removing the entire perimeter of the existing brick lining and replacing the lining with a lattice girder and shotcrete arch. The lining had to be removed in short sections so that the unsupported ground could be stabilized as soon as it was exposed.

Tunnel: *Jeddo Tunnel*
Scope of Work: *Rehabilitation and Ice Mitigation*
Location: *Hazelton, Pennsylvania*
Owner: *Reading & Northern Railroad*

For several years, Reading & Northern Railroad (R&NRR) had experienced difficulties with seepage, freezing, ice buildup, track structure deterioration, and deterioration of the concrete lined south portal at the Jeddo Tunnel, near Hazelton, Pennsylvania. The tunnel is located on a rail line that serves a single coal breaker, and consequently is only used for a single round trip per week. During some recent winters, ice build up in the south portal area was so severe that the tunnel became impassable. Conditions prior to rehabilitation are shown in Photograph 2.

Due to the limited use of the tunnel, the company had a very restricted budget for tunnel repairs. Several portal rehabilitation options were developed and presented to the railroad. Two options were let to bid for construction: support and insulation of the most deteriorated portion of the portal lining, and open cutting (daylighting) of the portal area.

Photograph 2: Conditions at Jeddo Tunnel during a mild winter prior to rehabilitation.

The low bid for the project was obtained from a contractor associated with the adjacent strip mine, who also owned the coal breaker serviced by the rail line. Consequently, close cooperation in scheduling a track shutdown was possible. The contractor elected to daylight the portal. Rippable rock was removed first, followed by presplit blasting of a new portal face and production blasting of remaining rock and the concrete lining. Tunnel traffic was eliminated and the track structure removed from the tunnel and approach cut prior to construction, allowing blasting of the overburden directly into the tunnel invert. Though a significant volume of soil and rock was excavated, use of large equipment from the adjacent mine made daylighting of the portal area a cost-effective alternative.

Tunnel: *Rockport Tunnel*
Scope of Work: *Vertical and Horizontal Clearance Improvement, Ice Mitigation*
Location: *Pocono Mountains, Pennsylvania*
Owner: *Consolidated Rail Corporation (Conrail)*

As part of the clearance improvement of Conrail's Philadelphia to Buffalo line, Rockport Tunnel required both vertical and horizontal clearance improvements. Vertical clearance was required for passage of double stacked container cars, while horizontal clearance was required to allow transport of large heat exchange vessels though the tunnel.

The 110 year old tunnel was largely unlined except for portal structures and two lined sections in the northern half of the tunnel. All lining structures were comprised of stone masonry blocks. The tunnel experience severe seepage and ice buildup.

The exploration and clearance improvement work were to be conducted under "live" track conditions, with 8-hour work windows. An exploration program was conducted consisting of geologic mapping of the tunnel and surrounding outcrops, survey measurements of existing clearances, evaluation of existing linings and portal structures, and use of a jackleg drill and borehole camera, operated from a rail-mounted truck, to evaluate conditions behind the lined portions of the tunnel.

The survey and mapping indicated that the two inner sections of lining provided ample clearance, but that minor repairs were required to one section of the lining due to undercutting of the masonry foundation during ditch clearing operations. Clearance improvements were found to be required in several unlined areas of the tunnel, and at both portal structures.

The sandstone bedrock in the tunnel was generally of very good quality. The shape of the tunnel was generally structurally controlled by bedding planes, jointing, and minor inactive faults. Rock was too hard to permit excavation by roadheader. Therefore, drill and blast excavation was used for the clearance improvement. Rock support was provided by spot installation of resin grouted rockbolts. The contractor elected to blast directly into air dump rail cars which were widened by installation of steel "wings." This method of excavation and muck removal was generally effective, but eventually resulted in significant damage to the rail cars.

The north portal of the tunnel required minor clearance improvement. Original designs called for grouting behind the lining, and support of the arch with shotcrete and rock anchors. However, while drilling holes for the grouting program, it was discovered that the masonry blocks had deteriorated to the point that stability of the arch would be marginal should any notching be employed. Consequently, the portal structure was removed using a large excavator to "pluck" the blocks from the structure, and the portal opening subsequently supported with shotcrete and resin grouted rockbolts.

The south portal of the tunnel required significant clearance improvement, both in the crown and in the east rib. This, combined with serious deterioration of the masonry due to seepage and freezing, resulted in the need to remove the portal structure. For removal of the portal structure,

Conrail allowed a single, 48-hour window of track closure. Due to steep rock slopes above the portal which were prone to rockfall, a new portal structure was constructed.

To mitigate ice problems in the tunnel, Ethafoam plank insulation was installed in seepage areas, and seepage was directed into an insulated invert drainage system. Insulation methods are discussed further in the discussion of White Haven Tunnel below.

Tunnel: *White Haven Tunnel*
Scope of Work: *Vertical Clearance Improvement and Ice Mitigation*
Location: *White Haven, Pennsylvania*
Owner: *Consolidated Rail Corporation (Conrail)*

Clearance improvement of White Haven Tunnel was conducted during the same program which included Rockport Tunnel. Similar exploration methods and work windows were employed.

The 130 year old tunnel was constructed in sandstones and siltstones of varying quality, and was unlined except for the north portal area. Deterioration of the tunnel lining was evident near the portal, where previous repairs using gunite had been unsuccessful and were failing. The outside face and wing walls of the north portal structure consisted of hand-cut stone masonry, and were considered historically significant. Consequently, visual impacts to this portal structure were to be minimized. The south portal was entirely unsupported and subject to slabbing of rock above the portal face. Ice buildup problems were significant, particularly in the vicinity of two vertical shafts that intersected the tunnel. Minor rockfalls in areas of weaker rock in the southern portion of the tunnel could only have been prevented by lining a significant portion of the tunnel. Because the rail line is used strictly for freight, Conrail decided that small rockfalls could be tolerated.

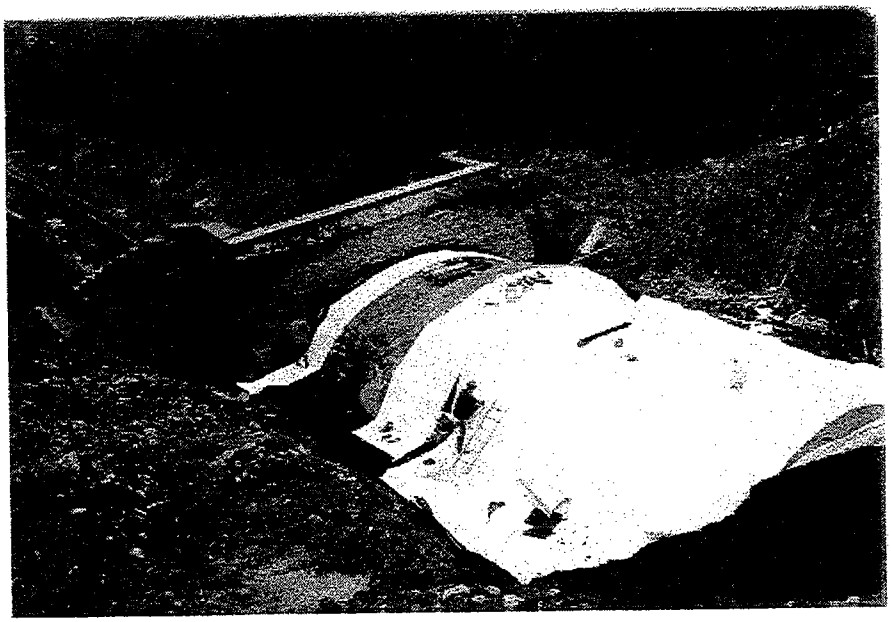
Vertical clearance improvements of up to three feet were required throughout much of the tunnel, including the south portal. Minor clearance improvements were also required in the north portal.

Initial clearance improvement in unlined areas of the tunnel was conducted using a roadheader. However, the roadheader was inadequately supported on a rail car, and consequently cutting rate was very slow. In addition, the roadheader used did not have sufficient range of side-to-side motion, resulting in the need to blast narrow slashes along both sides of the roadheader cut. For these reasons, the remaining excavation was completed using drill and blast methods similar to those used at Rockport Tunnel. Support was provided by pattern resin grouted rockbolts installed in the haunches of the newly excavated notches, with strategically placed rock bolting in the crown and shotcrete installation where necessary.

The rock faces above and adjacent to the south portal were thoroughly scaled. The portal face and tunnel crown were supported with resin grouted rockbolts and shotcrete, using strip drains to control seepage.

Much of the deterioration of the north portal structure appeared to be due to seepage and ice jacking. It was necessary to support the structure prior to notching for clearance improvement. Support and rehabilitation of the north portal were provided using the following procedure:

1. Shotcreting the inside of the tunnel arch;
2. Installation of resin grouted rockbolts in the tunnel crown where bedrock was present;
3. Construction of steel-reinforced shotcrete walls in the bulged area of the lining side walls;
4. Coring through the new walls and the masonry lining and installing cement grouted ground anchors;
5. Carefully excavating soil slopes above the portal to expose the last 30 feet of masonry lining and cleaning the exposed surface with a blowpipe;
6. Installation of bolts through the worst portion of the masonry, which was discovered to have been the original portal location;
7. Installation of French drains from the outside haunches of the tunnel, around the portal;
8. Shotcreting the outside of the tunnel arch;
9. Installing a PVC liner over the arch to prevent inflows (Photograph 3); and,
10. Replacing, compacting, grading and seeding the cover soils over the portal structure.

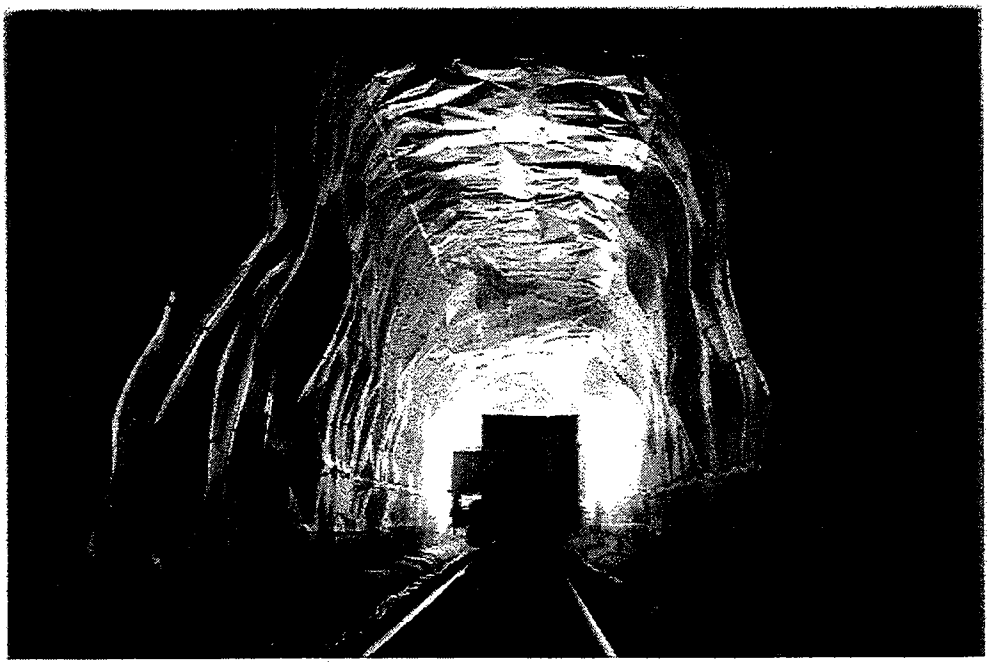


Photograph 3: Installation of PVC liner over shotcreted north portal arch.

After the existing arch had been strengthen, clearance improvement of the lining could commence. The vertical clearance envelop required a small notch to be cut in the masonry on each side of the crown. Several methods of accomplishing this cut were examined, including saw cutting and grinding. Because vibrations had to be kept to a minimum, a system of saw cutting was selected. Notching was conducted in four foot lengths, using alternate spacing (cut four feet, skip four feet, cut the following interval, and so on). A saw guide track was installed along the desired notching area, parallel to the tunnel alignment. A circular concrete saw mounted on the guide track was used to make a shallow cut, about one inch deeper than required for clearance, along the guide track. The track was then offset two inches, and the cut duplicated. This procedure was repeated until saw cuts were completed over the full width of the desired notch. A hydraulic splitter was then used to break out the stone between the notches. The notch was cleaned and a thin layer (1

inch) of fiber reinforced shotcrete was installed to knit the masonry together within the notch. After alternating sections were notched and shotcreted, the guide track was re-set, and the intervening sections notched. In this manner, clearance improvement was conducted without significantly weakening the masonry arch and with minimal aesthetic impact on the historically significant portal face.

Ice buildup was remedied by installing Ethafoam plank insulation installed comformably to the tunnel walls with galvanized pins. The insulation was provided in two inch thick by 8 foot by 32 foot panels. Joints between panels were sealed by using an industrial hot air gun to weld on strips of 0.25 inch thick by eight inch wide insulation material. Where installed in the crown, it was necessary to provide spark protection for the insulation. The spark deflection material consisted of soft aluminum bonded to insulation material. The ends of insulated sections of the tunnel were secured by tightly bolting mine strapping around the perimeter. Finished tunnel insulation is shown on Photograph 4.



Photograph 4: Finished section of tunnel insulation

Conclusion

Each tunnel enlargement or rehabilitation project presents unique challenges that require ingenuity, flexibility, and innovative solutions. A clear understanding of the owner's goals regarding scheduling, work conditions, budgetary concerns, intended end use, and level of acceptable risk is vital. "Boilerplate" types of approach cannot be applied to rehabilitation work on these historic tunnels. Successful rehabilitation work can only be accomplished using flexible designs and close cooperation between owner, designer, and contractor.

High Altitude Slope Reconstruction and Restoration on Colorado State Highway 82 at Independence Pass - A Continuing Case Study

by
Jonathan L. White
Colorado Geological Survey

ABSTRACT

The 'Top Cut' is a mile long corridor of cut slopes and steep eroding ground along Highway 82 in Pitkin County, Colorado, from elevations of 11,600 to 12,000 feet, near Independence Pass. The site lies within the core of the Sawatch Range in Precambrian rock terranes. Glacial morphology, geologic conditions, poor early road cut design, and climate set the stage for existing stability problems. Continuing erosion and detrimental maintenance procedures have created degraded environmental conditions and rock slide hazards. The Independence Pass Foundation, a nonprofit environmental organization based in Aspen, has formed the Independence Pass Restoration Team Partnership. This partnership includes private individuals, various state and federal agencies, Pitkin County, and Aspen who are working to mitigate these problems and restore the eroding slopes. In 1995 an engineering geological study of the Top Cut corridor was conducted that included the utilization of accurate Digital Elevation Modeling and GIS mapping. Later a hydrologic study was also commissioned for the Top Cut area. These data have been used to select and develop project areas to mitigate rockfall hazards and reclaim heavily eroded slopes. The primary factors in design concepts were costs and natural appearances, the Foundation relies entirely on grants and private donations. In 1996 and 1997 the Independence Pass Foundation and the Colorado Department of Transportation implemented and shared costs in a rockfall mitigation and prototype slope reconstruction, restoration, and revegetation project at the Top Cut. Costs were kept down by the use of rock buttresses, reinforced by geotextiles, for slope reconstructions and restorations. Rock-faced geotextile-reinforced reconstructions offer design flexibility and natural appearance more suitable for the area. The success of this project has led to larger, more ambitious, projects planned for the future.

INTRODUCTION

State Highway 82 crosses Independence Pass at an elevation of 12,093 feet at the Continental Divide. Originally an early stagecoach route from Leadville, the Independence Pass Route was improved for vehicles in 1927 and further modernized in the 1950's. See Figure 1 map. It was the excavation for the modern two lane corridor that created conditions for later erosion of cut slopes and native slopes that occur today.

In the early design and construction of Highway 82, as in many highways of Colorado, it was not policy to address potential geotechnical and environmental impacts in the long-term erosion and stability of cut slopes through varying rock and soil types. Once the fragile alpine topsoil has been stripped away and the toe of a natural slope is removed, instability can result in the oversteepened cut slope. Further erosion of the weathered rock mass occurs and rock and soil debris continually fall from the cut slopes and collects in the narrow ditch at the base of the slope, often into the

roadway itself. Continued maintenance is then required to dispose of the material.

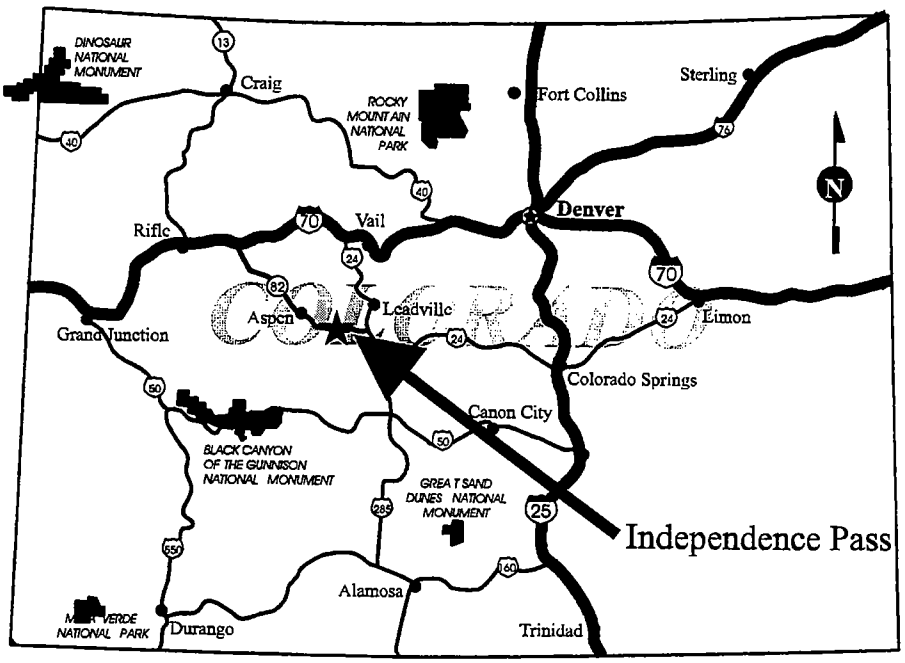


Figure 1. Project site location.

The severe winter seasons and danger of avalanches forces Colorado Department of Transportation (CDOT) to annually close the mountainous portion of the highway from October until the Memorial Day Holiday weekend. Because of its seasonal status and non-use by commuters, Highway 82 rates a lower priority for CDOT in the region. Many locals in the Aspen area felt that the roadway corridor was being neglected. The independence Pass Foundation (IPF) was formed in 1989 by the Environmental Research Group, an Aspen area nonprofit citizen's group dedicated to mitigating human impacts on high mountain ecosystems. The Highway 82 corridor from Aspen to the Pass crosses five different ecosystems ranging from the montane zone near Aspen, through the spruce-fir forest, the subalpine meadows, the delicate Krumholz zone at timberline, and the very sensitive alpine tundra. The IPF was created to focus on major restoration work along the Independence Pass corridor, and to coordinate restoration activities between the many government entities involved in Independence Pass management. In 1990 the IPF formed the Independence Pass Restoration Team, consisting of the IPF, Pitkin County, the City of Aspen, CDOT, United States Forest Service (USFS), Colorado Department of Natural Resources, Colorado Geological Survey (CGS), and other agencies. The earlier work completed from 1989 to 1995 was revegetation and modest slope restoration work at lower elevations by volunteers. (White and Fuller, 1998)

In 1994 the IPF turned its attention to the 'Top Cut.' Some early attempts were made in 1995 in revegetation by hydrofertilizing and hydroseeding but were essentially failures. The Restoration Team recognized that this area had complex problems, both geotechnical and environmental, that needed study before any additional work was to be undertaken. An engineering geologic report, an avalanche susceptibility review, and hydrologic study have been completed for the Top Cut area. Included with these studies was the commissioning of precise photogrammetric maps and digital elevation models (DEM) at two foot contour accuracies, and the generation of GIS geologic and topographic, and slope classification maps. Based on these studies, and the availability of modest additional funding through the CDOT Statewide Rockfall Protection Program, slope restoration prototype and rockfall mitigation sites were selected and construction completed during the Fall seasons of 1996 and 1997.

GEOLOGIC, GEOMORPHIC, and ENGINEERING GEOLOGIC SETTING OF THE TOP CUT

The project area lies at the core of the rugged Sawatch Mountain Range in Pitkin County, 17 miles east of Aspen and 0.4 miles west of the Continental Divide. The Continental Divide marks the border of Pitkin and Lake County and is the southeastern boundary of the Mt. Massive Wilderness Area. The USFS through the White River National Forest and its District Office in Aspen administer the land, except the highway right-of-way. Independence Pass stands on the crest of the massive Sawatch uplift. This mountain building occurred during the Late Cretaceous-Early Tertiary Laramide Orogeny (72-40 m.y.a.) and was completed during periodic block faulting and regional uplift in the Late Miocene and Pliocene Epochs(25-5 m.y.a). During and since the mountain building, millions of years of erosion have stripped away thousands of feet of sedimentary rocks, leaving a core of Precambrian-aged granites, metamorphic rocks, and small discontinuous Tertiary-aged intrusives within the study area. Periodic glacial activity during the Pleistocene age (2 m.y.a. to 12,000 years ago) shaped the mountains and the valleys to what is seen today. (Van Loenen, 1985)

The Independence Pass Top Cut is a mile of road cuts on Highway 82, from milepost 59.7 to 60.7. It passes through the tree line, from 11,500 to 12,000 feet in elevation. The alignment of the roadway leaves the bottom of the Upper Roaring Fork Valley, curves up onto the eastern flank of the valley, and begins to level off as it approaches the Continental Divide. See Figure 2 map. The valley is glacial in origin and has the classic U-shape cross-sectional form. This morphology has resulted in very steep natural slopes through the middle elevations of the valley. In many areas they approach 1:1 (45°) and remain naturally vegetated. The geomorphic system was in a state of equilibrium because the topsoil formation and vegetation could follow the retreating ice as the glacier diminished and emptied from the valley some 12,000 years ago. Due to the steepness of the natural slope, substantial cuts and fills were required to facilitate a two-lane highway through the naturally steep valley flank. Dis-equilibrium occurred when the roadway was excavated through the upper valley.

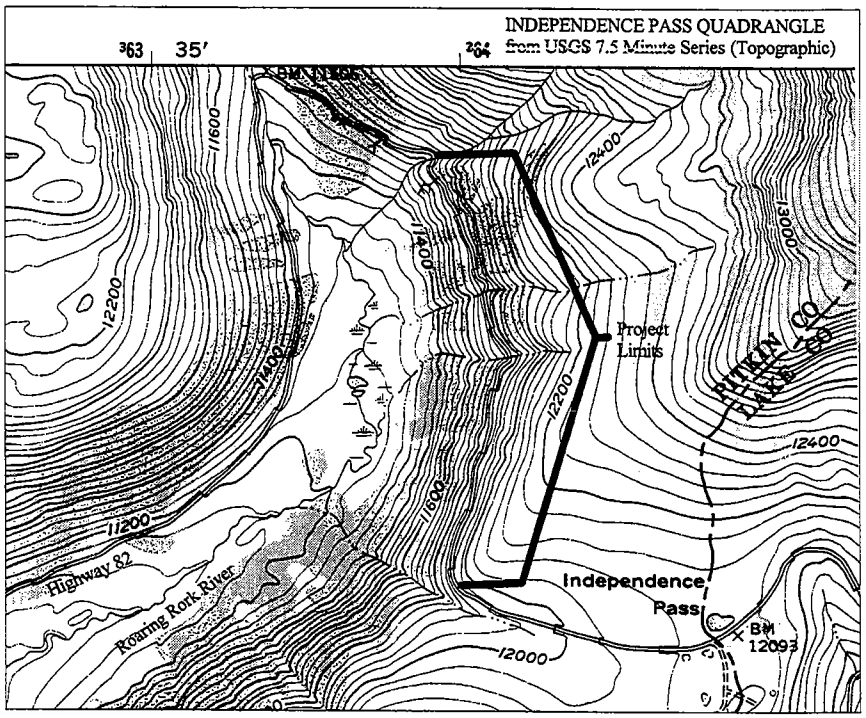


Figure 2. Project location. Scale 1 inch = 2000 feet.

Road construction encountered variable rock and soil materials along this alignment that, over the years, have reacted and weathered differentially upon exposure to the alpine climate. In this highway alignment, cuts were excavated in materials that ranged from very hard bedrock, generally resistant to weathering, to highly disintegrated rock and soil that erode easily.

The cut slopes of Highway 82 at the Top Cut expose granitic and metamorphic rocks. These cut slope exposures were mapped using a modified weathered rock classification. Due to the high elevation, mechanical weathering is the primary erosive agent. While chemical weathering loosens crystalline grains and alters mica mineral constituents, very little actual chemical decomposition of the crystalline rocks was noted, except along isolated, well delineated, shear zones. The bulk of the weathering is in degrees of granular disintegration. Soil gradation tests of samples collected from the base of the most eroded of the cut slopes, not including the larger rocky material, revealed gravelly sand with less than 10% passing the #200 sieve (Wright Water Engineers, 1997). Listed are some general descriptions of the mapped rock units (White, 1996):

Type 1. Hard Rock. Very hard and resistant to erosion. Does not easily backwaste or erode in a rock cut slope. Rock is fresh but certain zones can be highly jointed and fractured. Those discontinuities are clean without in-filling. There can be some grussy weathering along prominent joints. Internal rock strength can hold a vertical cut slope except where unfavorably orientated discontinuities exist. Rock "rings" when struck with a rock hammer. Modes of failure include planar, wedge, and toppling.

Type 2. Slightly Weathered Rock. Similar to Type 1 but certain zones are highly fractured and jointed and there is commonly granular disintegration (grussy) zones along prominent joints. Weathered zones still retain tight rock fabric and are still resistant to raveling or erosion in cut slopes. Slow granular disintegration of rock creates small grus fans at the base of slopes. Rock "thuds" when struck with a rock hammer. Modes of failure include planar, wedge, toppling, and raveling (granular disintegration and surface particle detachment).

Type 3. Weathered Fragmented Rock. Highly fractured and weathered. Cut slopes are still obvious bedrock but are beginning to crumble and fail, losing rock structure and strength. The more resistant rock zones are heavily fractured. Commonly, zones have disintegrated into grus where 'Core stones' are prevalent. The rock cannot maintain the original excavated cut slope angle and continually erode back. Debris fans annually occur at base of slopes. Rock is slightly penetrated when struck with a rock hammer. Modes of failure include toppling, raveling, and very shallow and irregular circular failures.

Type 4. Disintegrated Rock and Soil. Highly fractured and very weathered with most rock structure lost. Small areas of relic rock structure remain mostly jumbled and indistinct. Cut slopes continuously erode back to an angle of repose that approximates colluvial rocky soils (34°). Slopes steepen where protected by vegetal cover at brow of cut slope. Debris fans at base of slopes require periodic removal to retain road ditch width. Rock is easily penetrated when struck with rock hammer. Modes of failures include raveling and shallow circular failures near scarp of cutslope.

The mapping of the cut exposures along the 'Top Cut' highway alignment has revealed a close correlation of weak rock zones to advancing erosion into the natural tundra slope, which was expectable. Once the top 6 to 12 inches of topsoil and binding vegetation has been stripped away, erosion quickly removes the remaining thin veneer of rocky colluvial soil. All of the slopes that exhibit the most pronounced erosion and advancement of the brow upwards have had all the topsoil removed and are now eroding into the broken, heavily weathered, bedrock. Unfortunately, this condition results in a rocky soil mantle that is barren of organic material and constitutes a poor growth medium for revegetation efforts.

Generally, the early designed highway cut slopes exceeded the angle where vegetation can recur naturally and no attempt was made to revegetate the slope after excavation. At the design cut angle, and without any vegetation cover or slope stabilization techniques, the surface of very weathered disintegrated rock and residual soil could only support the original cut slope temporarily. Through the years the cuts in the weaker material erode back as the slope attempts to achieve stability or equilibrium at a reduced grade, the angle of repose. The original slope grade is stable where vegetation binds and protects the rock and soil. Backwasting of the denuded cut slope creates an oversteepening situation below the vegetated brow. In some circumstances the oversteepening just below the brow can be vertical to even overhanging. The eventual undermining of the brow of the cut slope will result in shallow slump failures and further headward erosion into the above natural vegetation. Figures 3 through 5 illustrate the advancing erosion common of cut slopes at the Top Cut. (White, 1996)

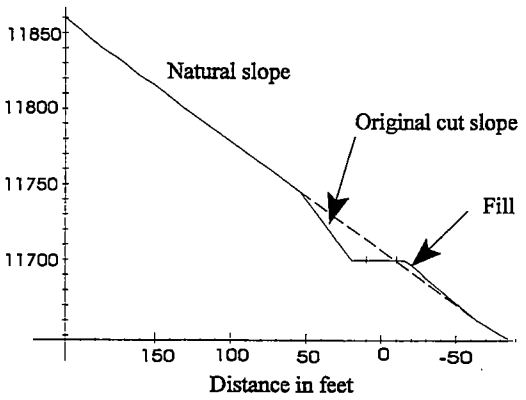


Figure 3.

Fortunately, the rock is granitic or metamorphic. Even those slopes that are in Type 4 rock and soils are still in granular material with high frictional strength. Overall, the global stability of these slopes is good and the possibility of traditional deeper-seated rotational failure occurring is improbable. Certain areas of Top Cut, in more competent rock, are prone to planar failures where jointing dips toward the roadway and are day-lighted by the cut slope. The more weathered slope materials are granular and angular in characteristic, and so have high frictional strength but little cohesiveness. The mode of failure of these slopes is by the process of *raveling*. Raveling of cut slopes in granular soils is the separation, or particle by particle removal, of soil constituents (ranging from sand to boulder-sized rock fragments) from the soil or weathered rock slope surface. An oversteepened exposure is usually required to cause raveling from normal weathering processes (i.e., rain, snow, wind, freeze-thaw cycles, etc.). Under these conditions soil particles detach from surrounding particles and gravity causes them to fall away.

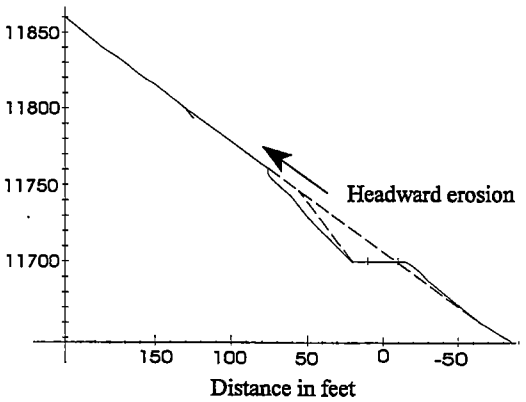


Figure 4.

As expected, the cut slopes mapped as Type 3 and Type 4 slopes, Weathered Fragmented Rock and Decomposed Rock and Soil, exhibited the worst raveling and shallow failures. The cut slopes mapped as Hard Rock and Slightly Weathered Rock are holding the original cut slope more successfully and retain, for the most part, a stable brow of rock where the overlying tundra is not receding. All the slopes, at different rates, erode and fill the ditch below with debris, or enter the

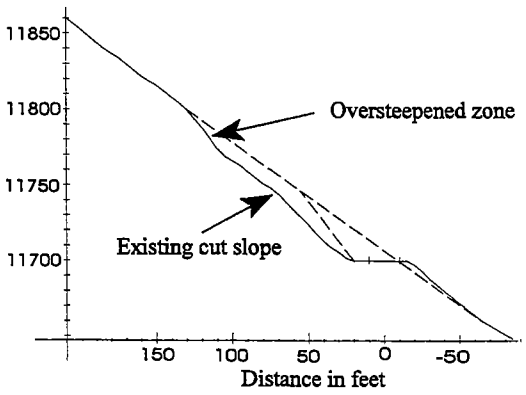


Figure 5.

roadway. Until recently, the CDOT maintenance method of disposal was to simply cast this material over the side onto the fill slope below the road. This activity, with snow removal also disposed over the side, has created accelerated erosion zones below the roadway. Rills and gullies in the Stripped Vegetation Zone end as small debris fans at the valley bottom near, or into, the Roaring Fork River. These accelerated erosion or stripped vegetation zones below the roadway are the most visually obtrusive aspect of the Top Cut as seen from across the valley and have created the worst environmental degradation hazard to the Roaring Fork River.

The Independence Pass Top Cut has been recognized for some years as an active rockfall hazard zone. Five slopes have been delineated that have been rated in CDOT's Colorado Statewide Rockfall Project. Two slopes rated sufficiently in severity to be included within the top 10 rockfall hazard slopes in CDOT Engineering Region 3/Maintenance Section 2, one of which rated No. 11 overall, statewide. (Andrew, 1994) This hazard will worsen every year as traffic volumes continually increase and vehicle impact probabilities rise.

Avalanche hazards exists at the Top Cut. While not usually a hazard to the traveling public since the road is closed during the winter, they do occur during late snow storms after the road officially opens for Memorial Day weekend. Trim lines, above and below the roadway, in the trees show that large avalanches have occurred at the site. Several portions of Highway 82 at the Top Cut have been mapped as high hazard zones. (Mears, 1979) The concern with avalanches is the potential shear stresses they impart on any remedial work that is contemplated for the project.

THE INDEPENDENCE PASS TOP CUT PROJECT

As stated above, to understand the complexity of the geology, soils, hydrology, and climate of the Top Cut area, engineering geology, avalanche, and hydrologic studies of the area were conducted. In addition, research was conducted on available anchored revegetation systems on the market and existing CDOT research and construction projects in high altitudes at Interstate 70, west of Eisenhower Tunnel, and Highway 40 on the west side of Berthoud Pass. (Goff and Rosener, oral comm., 1996)

The biggest problems perceived at Independence Pass Top Cut was the continual erosion of (1) the existing cut slopes, and (2) the damaged slopes below the road stripped of vegetation, either through erosion or burial, and the environmental impact by siltation and sediment loading of the Roaring Fork River. Rates of erosion varied, dependant on the quality of the rock mass. Table 1 shows the mapped rock units from the engineering geology study, their areal extent calculated from the GIS map, and calculated sediment yields using the modified universal soil-loss equation, from the hydrologic study. The sediment yeilds for the decomposed Rock and Soil mapped unit and the Stripped Vegetation Zones are quite high. The estimated total sediment loads are somewhat misleading because they do not represent rock material gravel and larger, indicative of debris fans and rockfall from the cutslopes. CDOT Maintenance crews estimate they remove 750 tons of rock, gravel, and sand from the cut slope ditches every year.

Cut slope Rock Units	Acres	Sediment Yield (Tons/acre/year)	Estimated Sediment Load (tons/acre/year)	
Hard Rock	0.89	0	0	
Slightly Weathered Rock	1.03	2.84	2.9	
Weathered Fragmented Rock	1.93	5.68	11.0	
Decomposed Rock and Soil	2.26	32.65	73.8	
Natural vegetated slope	0.18	1.66	0.30	
				88.0*
Basins above Cut slopes	574	2.76 (avg.)		**1,483**
Slopes below Highway				
Stripped Vegetation Zones	20.05	71.4	1,432	
Exposed Bedrock	5.78	0	0	
Vegetated Zones	145.17	3.3	479	
				1,911
			Total	**3,482**

*Note: Sediment yield and load figures do not include the larger rocks or boulders eroding from the slopes. CDOT estimates they remove **750 tons of rock** annually during ditchline maintenance.

from Wright Water, Engineers, Inc., 1998

Table 1. Sediment yields based on mapped zone areas calculated from Engineering Geologic GIS map.

Prototype project planning began for slope reconstruction, stabilization, and revegetation in the winter of 1995/1996. The priorities outlined by the IPF were those slopes dangerous to the public (rockfall), those slopes with the highest sediment yields and erosion potential, and costs. Out of fiscal necessity, planning centered around costs. The Foundation totally relies on contributions, donations, volunteerism, and assistance by local, state, and federal government agencies. During the planning period, a new funding source was found in CDOT's Statewide Rockfall Program. Limited funding was being set aside for Highway Safety Projects for Rockfall Control at each of the CDOT Region's top 10 rated slopes. Two slopes at Top Cut fell into that category, both of which were being considered as prototype project sites for IPF. The CGS helped CDOT in defining a scope of work for rockfall control at the Top Cut rated slopes that included scaling, anchored wire mesh, rockbolts, and anchored revegetation matting. After months of lobbying, in September 1996 a CDOT Purchase Order of $95,000 was signed for Rockfall Mitigation at Independence Pass to a contractor that specializes in rock stabilization, on a Time and Material (T&M) basis. The availability of the CDOT rockfall mitigation funding was very fortunate. It enabled IPF to devote the bulk of their resources to slope restoration and revegetation. The prototype site selected is called the Twin Gullies. This cut slope site has two small gullies that mark the weaker formational contact of granite, gneiss, and pegmatite. Gullying has occurred at these near vertical rock contacts, and has advanced up the steep hillside. The right ravine has advanced more than 130 vertical feet up into the natural slope. Small fans of rocky debris annually accumulate in the ditch below these gullies that includes detached slabs of tundra topsoil. Several options for mitigation and stabilization of the Twin Gullies were examined. The differing techniques were weighed regarding costs, natural appearance, and ability to revegetate. Table 2 illustrates the advantages and disadvantages of six design options considered for the Twin Gullies.

Design Options:	Advantages:	Disadvantages:
#1 ➤Geotextile Reinforced Soil (GRS) modular block retaining wall - 15 feet high. ➤Slope reconstructed at 1½:1 above wall with anchored revegetation matting.	➤Would prevent additional erosion of brow ➤Some of the earth work could be done by inmate labor.	➤Large amounts of fill required. ➤Would require anchored high strength slope revegetation matting. ➤Would have an obvious constructed wall appearance. ➤Contains expensive specialized work done by contractor. ➤High cost depending on modular block selection.
#2 ➤Tiered GRS Retaining Wall to top of cut slope.	➤Would completely cover cut slope and stop further erosion ➤Wide variety of wall facades available. ➤Can include planting pockets on each tier step.	➤Large amounts of fill required. ➤Would have an unnatural stair stepped, constructed wall, appearance. ➤Specialized contractor needed. ➤Increased exposure to avalanche shearing forces. ➤High to very high costs depending on wall facade selection.
#3 **➤Mechanically stabilized rock buttressed wall 15 feet high (geotextile reinforced).** **➤Slope reconstructed at 1½:1 above wall with anchored revegetation matting.**	**➤Cut slope erosion would cease.** **➤Labor intensive, much of the work could be done by inmate labor.** **➤Reconstructed slope would revegetate.** **➤Good local boulder availability.** **➤Cheapest option.**	**➤Large amounts of fill required.** **➤Would require anchored high strength slope revegetation matting.**
#4 ➤Geotextile Reinforced Oversteepened Slope	➤Would completely cover cut slope and stop further erosion. ➤There would be no revegetation costs of the oversteepened slope. ➤Labor intensive, work could be done by inmate labor.	➤Large amounts of fill required. ➤Flat massive monolithic wall would have unnatural appearance. ➤No, or very little, revegetation of slope. ➤High cost
#5 ➤Reinforce slope with shotcrete using crane man basket. ➤Short GRS retaining wall at ditch-line.	➤Would completely cover cut slope and stop further erosion ➤Expensive techniques exists to sculpt and color shotcrete slopes. ➤Moderate to high cost. ➤No additional fill needed.	➤No revegetation would be possible. ➤Contains specialized work done by a contractor. ➤Batch plant distance difficulties. ➤Cold weather concerns. ➤Flat massive monolithic slope would have unnatural appearance
#6 ➤Draped wire mesh to ditch line	➤Would 100% mitigate rockfall. ➤Tundra brow at scarp would be stabilized. ➤No influence by weather. ➤No additional fill needed. ➤Cheapest alternative.	➤Slope would remain unchanged. No revegetation would occur. ➤All work would have to be done by specialized contractor. ➤Erosion of fine sediments would still occur.

Table 2.

The IPF Board Members felt, being primarily an environmental organization working on USFS property, that they should try to restore and revegetate the slopes if possible. That policy should remain consistent through the entire Top Cut Project site. Looking at the list of options, it was felt that to achieve as much a natural appearance as possible, and give every possibility for revegetation, Option #3, shown in bold in the table, would be the best design. A 15-foot high rock

wall would enable a reconstruction of a 1½:1 slope above it to match the top of the cut slope for the left gully. This design provided a method to 'heal' the cut slope. Amoco woven geotextile 2044 (wide width tensile strength-4800 lbs/ft. ASTM-D-4595) was used in 12-inch compacted lifts of minus 6-inch fill. Boulders from three to six feet in diameter are to be used as the facing. A schematic is shown in Figure 6. The reconstructed slope profile would almost match the natural slope above. The reconstruction would have no additional vertical exposure above the existing native ground surface, so avalanche shear and normal stresses would not increase. (Mears, oral comm.) Upon consultation with the CDOT Region 3 geologist and the CDOT Geotechnical Unit, this design was selected and approved.

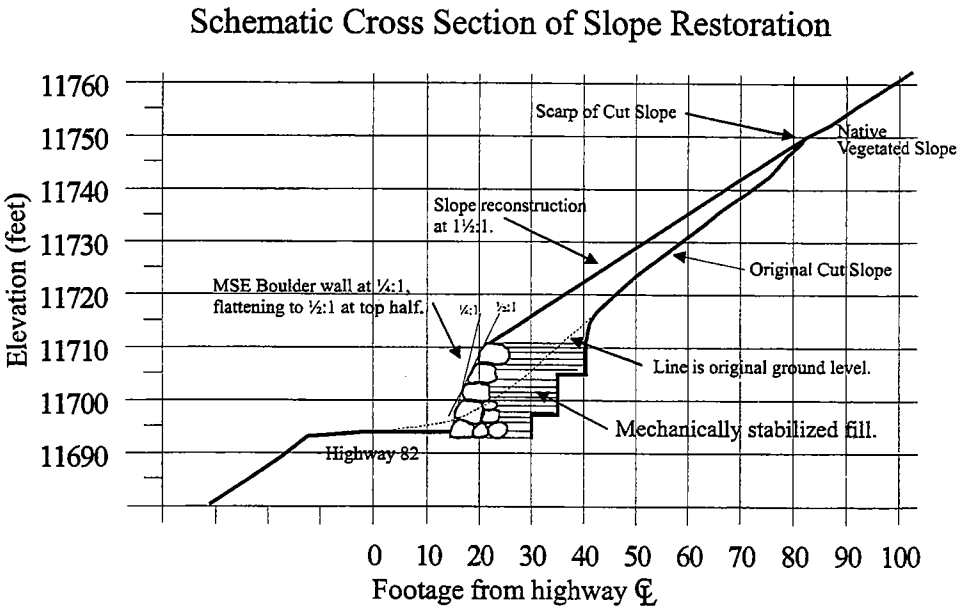


Figure 6. Schematic of Reinforced Rock Wall and slope reconstruction.

Numerous revegetation matting types are available on the market. Most are organic based stitched rolls of straw, coconut fiber, or shredded wood. Stronger and more expensive polypropylene, nylon, and polyester mats are also exist. Selection factors at the Top Cut are harsh climate, poor growth medium, reduced growing season, a west slope aspect, and future long term fertilizing, seeding, and planting to revegetate a reconstructed steep 1½:1 slope. Also, static and dynamic snow loading had to be factored. An aggressive, very strong, soil anchorage for vegetating steep barren slopes was required. Based in part on CDOT's experiences at Berthoud Pass (Goff and Rosener, oral comm.), the IPF selected a matting product called Enkamat S 20. The material is a three-dimensional geomatrix of heavy nylon monofilaments thermally bonded to a strong (1560 lbs/ft, ASTM D4595) polyester geogrid. The material is anchored using fabric pins or rebar hooks. Then a thin layer of soil is raked over, and worked into, the 3-D matrix of the matting. Protected from UV exposure, the matting will last many years, continually reinforcing the slope soil, as vegetation slowly comes in. The material is not cheap, though, current prices, depending on quantity, are around $10 per square yard.

In the Fall of 1996 and 1997, the IPF and CDOT implemented and shared costs in a rockfall mitigation project, and a prototype slope reconstruction, restoration, and revegetation project at the Top Cut. The CDOT Maintenance Section also planned working at the site to clean out ditches. The combining of projects worked well because savings was incurred by sharing mobilization costs and use of CDOT earth moving equipment. The material generated from scaling operations of the Rockfall Control Project and cleaning debris out of the ditch lines was used as fill for the reinforced

rock walls and the slope reconstruction. The fill derived from the disintegrated crystalline rock, minus the large rocks, is excellent granular material (USCS SP-SM to SW and AASHTO A-1-a to A-1-b). Less than 10% passed the #200 sieve. (Wright Water, Engineering, Inc., 1997)

The CDOT Rockfall Control Project was done at standard rates of T&M and costs were consistent with this type of slope and rock stabilization work for CDOT highways. Significant savings was realized, through, by the availability of CDOT Maintenance heavy equipment and operators to haul material at the project site at no additional cost to IPF or the Rockfall Control Project. In addition, the project occurred after Highway 82 was closed for the season, so no traffic control costs were necessary the first year.

The Rockfall Control Project in the fall of 1996 and 1997 completed the following:

1. Highway cut slopes were scaled of loose rock through the entire project site. CDOT maintenance crews moved boulders and rocky fill during scaling and ditch clean-out operations. The boulders and fill were instrumental in IPF's Twin Gullies slope reconstruction project.

2. A 120 foot by 50 foot swath of wire mesh was anchored, using 60 grade #10 bars, to the top rated rock slope that had continuously rained rocks onto the road, and annually filled the ditch width. This mitigation prevents further erosion, stabilized the tundra brow, and prevents rockfall from the roadway. The wire mesh was PVC-coated a dark brown color. The dark wire mesh blends with the irregular reddish-brown rock face and is almost invisible. This was specialized work that requiring a crane and manned drilling basket.

3. Rock reinforcement, with rockbolts using a crane and manned drill basket, was completed at one of the more dangerous rock features in a CDOT rockfall hazard rated slope.

4. A rock-faced, geotextile reinforced slope, similar to the schematic shown on Figure 6, was constructed in a small erosional amphitheatre in very weak decomposed rock near a large shear zone. As with the Twin Gullies reconstruction, Enkamat S20 revegetation matting was also installed. This work was high on the slope and required a crane for mobilization of the rock and fill by bucket. The dangerous high slopes required technical rock climbing so contract labor at high-scaling rates was required. Total costs were $75 per square face foot for the wall, and $30 per square feet for slope reconstruction including installation of revegetation matting.

IPF's slope reconstruction and revegetation project realized substantial savings by entering into an agreement with the Buena Vista Correctional Facility to provide work crews for this project. Besides the construction and restoration work, the inmates also spent several days planting trees and shrubs on the slopes below the road. Pitkin County Roads and Bridge Division also supplied small crews and equipment to the project during the afternoons. The IPF prototype project completed the following:

1. Reconstruction of slopes at the Twin Gullies Prototype site. Fill material and boulders were provided, at no costs to IPF, from the scaling operation and collections from ditch clean-outs at other areas of Highway 82 between Aspen and the Pass. CDOT maintenance crews hauled the material to the staging area at the site. The wall abutments were built tight against stronger rock exposures on the sides of the gullies. That, and a lack of surplus fill, prevented the construction of a ramp to

reach the slope above the walls. The rocky fill for the slope reconstruction above the rock walls had to be placed by crane bucket and smoothed and compacted by hand. The final slope was prepared with top soil donated by the Village of Snowmass and hauled by CDOT. Installation and anchorage of the Enkamat S20 followed, according to design and specifications provided by the manufacturer. To account for the passive and possible dynamic snow loading every winter the matting was anchored with 18 inch, #4 rebar J-hooks, on 5 foot centers. A one to two inch layer of screened (<3 inch) fines were then placed atop the Enkamat and worked into the three-dimensional matrix of the matting. A seed mix designed for this altitude was broadcast over the restored slope, plants were installed, and the slope hydromulched. Almost the entire labor pool for the Twin Gullies reconstruction was by the inmates from the Buena Vista Correctional Facility, and to a lesser extent Pitkin County. CDOT Maintenance provided all haulage of material. The only expense to IPF was the woven geotextile and the contractor's back-hoe, loader, and shaker screen used to build the walls. **Cost to IPF for the rock walls was only $5 per square face foot.** The slope reconstruction was more costly because a crane was required to place the fill.

2. At the head of the ravine of the right Twin Gully, ten 6' by 9' by 6" thick gabion mattresses were installed and anchored to #10 threaded bars drilled and grouted into the underlying rock. Three to six inch diameter rock was screened from the fill stockpiled by CDOT and transported by crane basket to the gabion mattresses. The mattresses' lids were closed and wired and 6" by 6" plates were bolted down on the bars sticking through the mattresses. The anchored mattresses will prevent further headward erosion of the ravine. This work was high on the slope and dangerous so inmate labor could not be used.

3. Revegetation test plots were selected within the stripped vegetation zone below the roadway and installed with different revegatation matting donated to the IPF. Test planting terraces were also constructed and lined with matting in the stripped vegetation zone (with the highest sediment yield) to control further erosion. Dept. of Corrections inmates did this work with donated materials and cost the IPF nothing except supervision costs.

FUTURE ACTIONS

In 1997, IPF secured funding for $100,000 to support erosion-control activities in the Independence Pass Top Cut area. This funding was secured from the Intermodal Surface Transportation Efficiency Act (ISTEA) through CDOT and Pitkin County. CDOT sponsored the funding request at the federal level and Pitkin County is acting as the Local Agency. IPF will do the project planning and contracting associated with expenditure of these funds. IPF will also provide the 20% match ($25,000) required to secure the $100,000 grant. This funding will be spent on roadway curb, ditch, and culvert projects to assist stabilization and rehabitation work below the highway. A large part of stabilization and revegetation of the stripped vegetation zones below the roadway will be controlling storm flows that sheet over the roadway onto the embankment slopes. Also, each year, additional plantings and anchored test section sites of donated revegetation matting are planned for volunteers and inmates from the Buena Vista Correctional Facility.

Work will also continue with the cut slopes above the highway at the Top Cut. The next project is a $400,000 project to restore slopes at what we call the 'Big Cut'. The first phase of this project will begin in October 1998. Already the IPF has received a donation from the CDOT Dillion Residency of rolls of Enkamat S20 and humate, surplus from their projects on Berthoud Pass. Costs

will be even cheaper than the Twin Gullies because cranes will not be needed for the slope reconstruction. The Big Cut is the longest and highest of the eroding cut slopes along the Highway 82 'Top Cut' area. The cut length is 600 feet long and the highest part 100 vertical feet high. Cut slope bedrock was mapped predominantly as Type 4, Decomposed Rock and Soil. This slope has also been rated in CDOT's Statewide Rockfall Hazard Rating Project. (Andrew, 1994) Slope grades at the site range from the angle of repose (34°), to overhanging at the brow (scarp) and are completely barren.

To keep the restoration consistent with the Twin Gullies site the IPF proposes additional rock wall construction, almost 500 feet long, using geotextile reinforcement of the backfill. Unfortunately, the height of the cut slope will not permit slope reconstruction from the top of the rock wall to meet the brow of the cut slope at the 1½:1 angle. To prevent further erosion at the oversteepened tundra brow above, slope stabilization is warranted. Soil nail-anchored wire mesh, underlined by Enkamat revegetation matting, would contain further loss of the tundra brow and eliminate the serious rockfall hazard at this site. To stabilize further slope material loss, and foster partial revegetation, the wire mesh will be draped on the cutslope from the anchorage to where the reconstructed slope begins. See schematic in Figure 7. This rock wall and slope reconstruction is designed for staged, or phased, construction as funding and/or material becomes available.

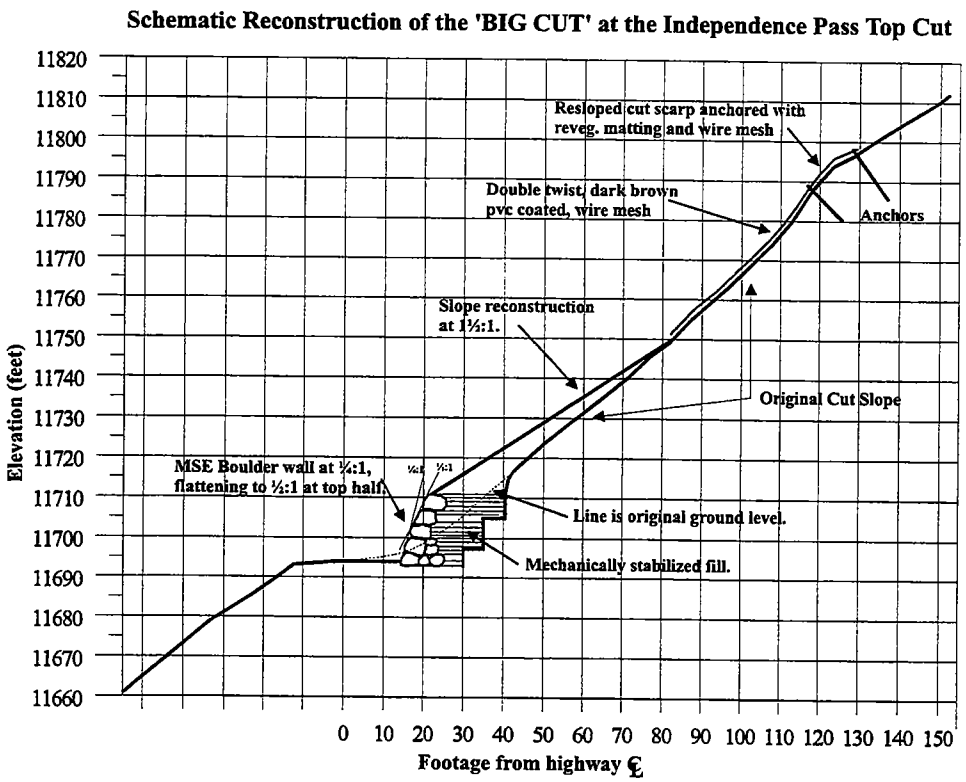


Figure 7. Schematic of Big Cut restoration.

CONCLUSIONS

Rock but-tressed slope reconstructions have shown to be effective in stabilizing cut slopes in very weathered rock. The design is very flexible and has a more natural appearance than an artificial facade. Where this remedial work has been done the ditchwidths of the roadway remain clear and future maintenance costs are reduced. In addition, rockfall hazards that result from the erosion of these slopes has been reduced or eliminated. Costs were kept down by the IPF restoration team approach. These reconstructions can be planned and designed from accurate GIS digital

elevation models of the project site. To further defray costs to the IPF, CDOT Maintenance provided haul trucks, heavy equipment, and personnel while contractors and vendors provided portions of their services or materials as tax deductible donations to the Foundation. Buena Vista Correctional Facility provided the primary labor force. Revegetation test plots and prototype restorations will be continuously monitored and evaluated. The apparent success of this project has led to the planning of larger, more ambitious, projects for the future.

Excellent digital elevation modeling (DEM) has been the key to substantial cost savings for Top Cut projects. The accuracy of the digital elevation data and ability to generate 2 foot contours for the area gives us the ability to forego the costs of preliminary and construction surveying. The DEM allowed the generation of cross sections at any stationing we choose. All material quantities and height and slope grade specifications were calculated using only DEM data. This would not be possible with more traditional 'concrete and steel' construction. In addition, the slope classification map, using engineering slope designs increments (e.g., 3:1, 2:1, 1½:1, 1¼:1, 1:1), has been useful to find possible areas within the stripped vegetation zones below the roadway where cheaper, less aggressive, revegetation matting test plots maybe successful. The slope classification map was generated with 5' by 5' pixels of equal slope on a 1 inch =200 feet base.

The Restoration Team Approach

Participants in the Restoration Team for the Top Cut projects include several entities contributing time, effort, and/or material. CDOT Maintenance Section 2 and Pitkin County provide equipment and work crews. The Reinforced Soil Research Center at the University of Colorado at Denver is providing additional engineering expertise in geotextile applications. CDOT Geotechnical Unit is providing engineering review and approval. Contractors and vendors entering this partnership have either donated or provided material at cost. The White River National Forest has provided equipment at subsidized costs and very important borrow areas for boulders and fill. Several outdoor groups and local schools have provided volunteers to help in revegetation, seed collection, and planting work. An invaluable cost-saving resource is the inmate labor pool made available by the State Correctional Facility in Buena Vista, Colorado. We also cannot forget the expertise of those individuals, experts in their fields, that attended the annual restoration meetings providing invaluable insight and recommendations. While most of these contributions are modest individually, the total is substantial and notable.

ACKNOWLEDGEMENT

This paper and presentation has been modified from an earlier version presented to the biannual 1998 ASCE Geotechnical Seminar proceedings on April 17, 1998 in Denver, Colorado. (White, 1998)

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400,000 CUBIC YARD SLOPE FAILURE ON INTERSTATE I-26

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Soils and Foundation Engineer
North Carolina Department of Transportation

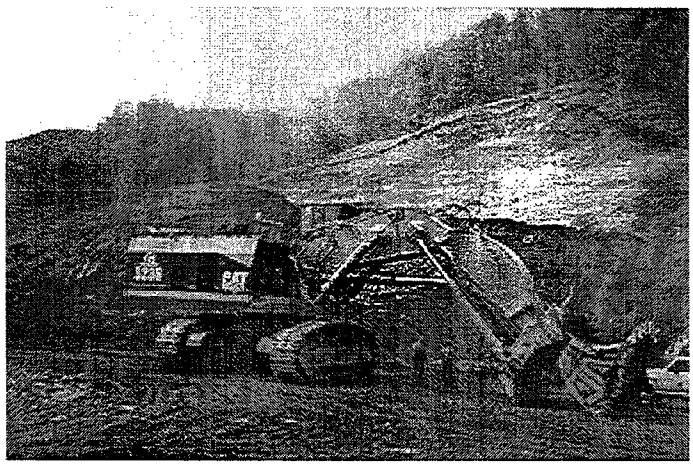
Abstract

A classic block failure occurred in May 1997 during construction of Interstate-26 in Madison County, North Carolina. The failure encompassed an area 600 ft.X 700 ft.X 30 ft. deep at a location that required a 2.5:1 (H:V) cut slope design. This area contained colluvium material that is inherently unstable due to its formation from old landslides over thousands of years. In order to continue with construction, a field investigation, an analysis, and a solution had to be devised. This paper will discuss details of these requirements.

Introduction

The third section of Interstate 26 (A10-C) is currently under construction. This 5.908 mile section is another leg in the corridor that extends from Asheville, North Carolina to Erwin, Tennessee. Gilbert Southern has the contract for $105,646,973.35, and is currently working to move 25,800,000 cubic yards of material. The four large mountain cuts which the contractor is removing makes way for the interstate 450 feet below. This project currently represents the largest dollar amount the state has awarded in a single contract. Gilbert Southern is to complete this section by December 31, 2001.

The contractor initiated work during the winter of 1996 by mobilizing some of the largest equipment ever used on a North Carolina DOT project. Twenty-two tractor trailers transported the components of a Caterpillar 5230 Hydraulic Shovel/Backhoe to the site, where it took three months for assembly. The shovel began the cut excavations at grade elevation and loaded each Caterpillar 785B truck with 150 tons of rock and soil.



Caterpillar 5230 Hydraulic Shovel/Backhoe with 26 Cubic Yard Bucket

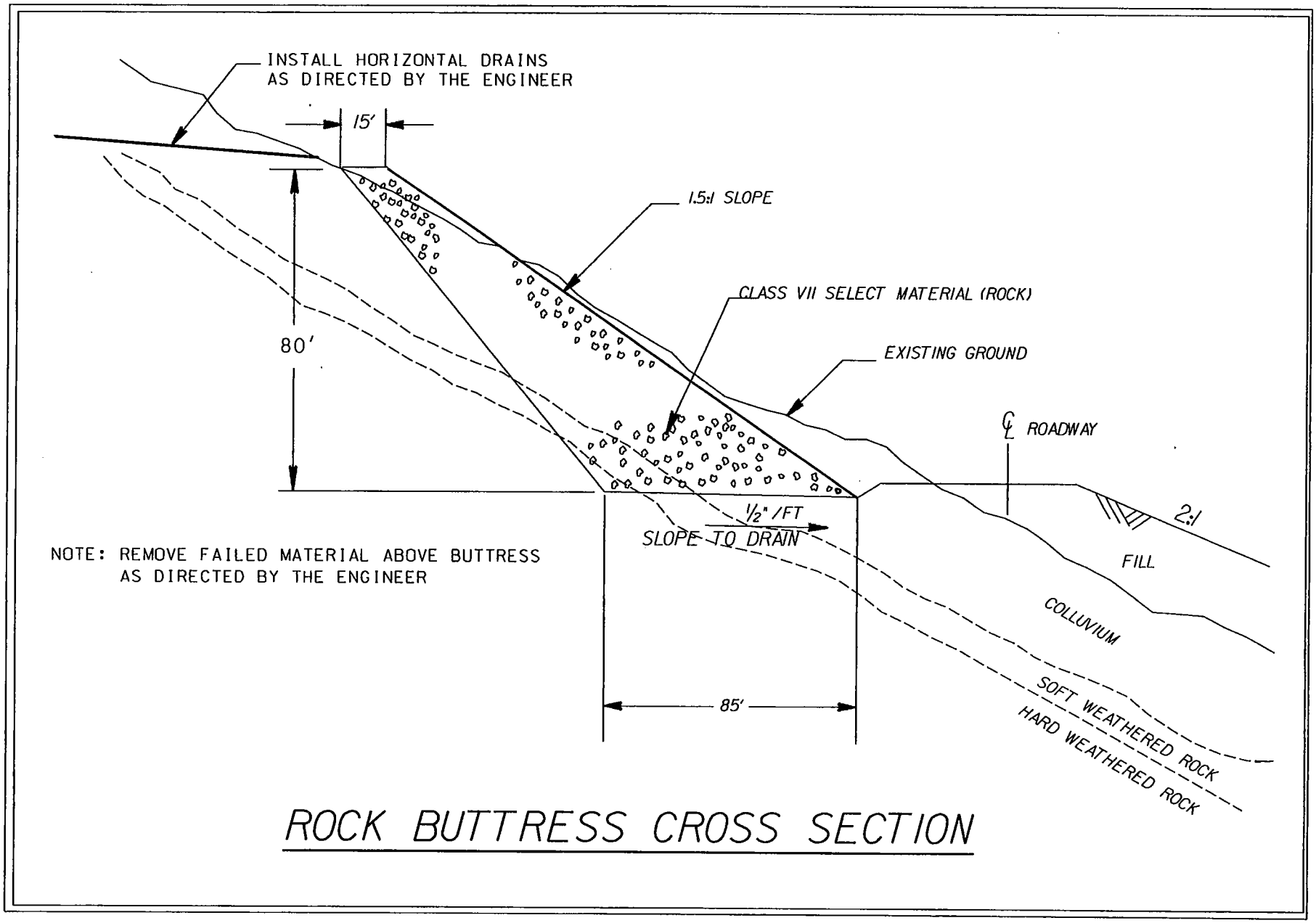
Failure

In May 1997, one of the 2.5:1 (H:V) excavations required two D-10 Bulldozers up slope to push material downhill to the backhoe. Earth material piled up on the lower third of the slope allowing the shovel to efficiently dig at its maximum reach of 48 feet. The existing water table had saturated the slope, while water flowed out of the hillside. The contractor had a vertical cut standing 40 feet high at the backhoe, when a tension crack developed 600 feet up slope. Construction terminated at this location, and the contractor decided to move his 3 million dollar backhoe away from the slide.

Tension Cracks Developed 600 Feet Up Slope

Site Investigation

The state initiated a site investigation and installed slope inclinometers. The borings showed colluvium material up to 40 ft. deep. In some areas, the material appeared to have some residual structure. This may be explained if one theorizes that large chunks of residual soil broke off and slid down the mountain intact. Two hundred feet above the failure is a 50 ft. high vertical rock cliff that may be the remnants of previous block failures. Water pressure caused some tension cracks to open to depths of 15 ft. and kept the slope in a saturated condition. Bore holes indicated water levels within 15 to 40 ft. of the surface. Slope inclinometers recorded movements at depths from 25 to 40 ft. which followed the bottom of the colluvium layer. Movement continued until the failure had grown to a surface area 700 feet by 600 feet. Movement near the tension crack at the top of the slope exposed a near vertical face of 40 ft.



ROCK BUTTRESS CROSS SECTION

Movement Continues

Work Road Moved 25 Feet Downhill From Inspectors

Original Design

During the original slope design done in 1995, field mapping showed the presence of colluvium, and two borings indicated the depth of colluvium varied from 10 to 22 ft. deep. Based on research done by P.C.Lambe and A.H.Riad (1991) at other colluvium locations in North Carolina, the following soil parameters were used for design: phi=28 degrees, cohesion=50 psf, and a pore pressure ratio Ru=.3. These parameters yielded a slope design of 2.5:1 (H:V).

Additional measures to increase stability included the installation of horizontal drains above the slope stakes. These drains were installed and allowed to flow for three months before excavation started.

Analysis

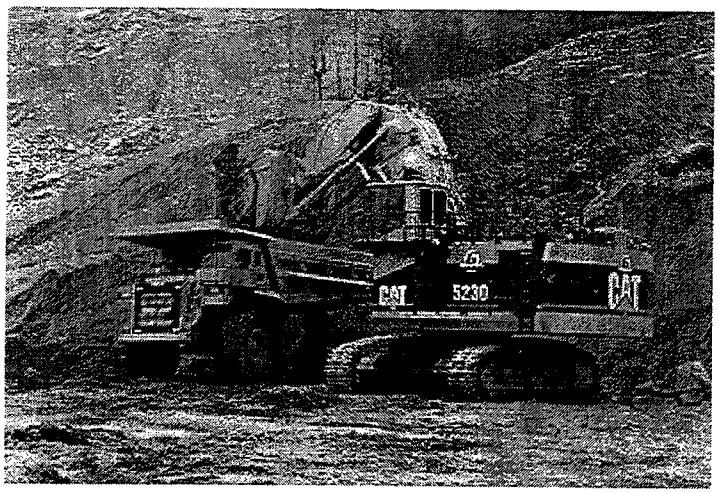
Cross-sections and slope inclinometer data delineated the geometry of the slide. A back analysis using the X-STABL slope stability program helped determine the shear strength of the soil at failure. The following soil parameters developed a factor of safety of one: phi=20, cohesion=0, and water 15 ft. below ground. These shear strengths aided in the design of a rock buttress located at the toe of the slide. By adding this large counterweight at the toe of the slide, the factor of safety increased to 1.3. Excavation for the buttress will involve cutting back into the hillside 85 ft. and then proceeding up on a 1.5:1 slope until a vertical height of 80 ft. is reached. This area will then be filled in with large blasted rock from 3 ft. in diameter down to 2 inches.

Construction

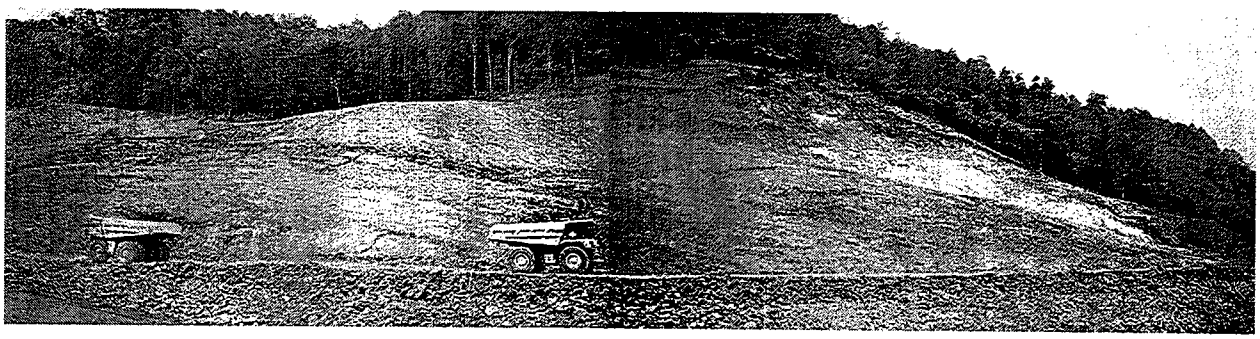
Construction started in August 1997 with a drilling machine mobilized to install horizontal drains. Drains would provide an outlet for the groundwater that flowed along the bottom of the colluvium layer. One row of drains placed at grade produced heavy flow. Three other rows placed at various elevations on the slope produced mixed results. About 40% of the drains flowed at one time or another. Monitoring of the failure took place during the wet winter months as more tension cracks opened and the movement continued.

An excavation for a 240,000 cubic yard rock buttress started in May 1998 with the large Caterpillar 5230 backhoe removing the toe of the slope. Two D-10 dozers pushed material down the slope as the backhoe scooped up 26 cubic yards bites. An additional 15 ft. of colluvial material above the buttress is being removed to bring the total excavation to 300,000 cubic yards. After removing the wet colluvium material at the toe, large Caterpillar 785B trucks hauled in blasted rock and began ramping up the buttress. The trucks were only able to operate on grades less than 12 percent so this limited the placement of the entire buttress. All of the north end was placed while the south end simply contained the access ramp. The contractor left this south area without any buttress material and returned to work at other cuts. He made plans to return in November 1998 to mobilize the smaller Caterpillar D400 trucks and create a road around the top.

However, this southern area with the toe undercut and no buttress to stabilize it, has currently begun to move again. New tension cracks have opened 700 ft.up slope while water flows down the slope keeping it in a saturated condition. Three horizontal drains, flowing into a down slope pipe have leaked water into the hillside. A slope inclinometer located in the slide area sheared off 26 ft. below ground. The current plan is to monitor the slide until November, then remove additional failed material, place the remaining buttress rock, and install horizontal drains above the buttress. Most of the construction decisions and time tables involve supplemental agreements with the contractor and are based on economics and availability of equipment. In this case, it would be best to repair all of the slide at one time, eliminate the movement, and remove the ground water.



Backhoe Removing Material at Buttress Location



Trucks Hauling Blast Rock for Buttress at Toe of Failed Area

Conclusion

Colluvium material is inherently unstable and requires flat, dry, well constructed slopes to maintain stability. The original design using soil parameters (phi=28 degrees, cohesion=50psf, and Ru=.3) proved to be inadequate for this wet site. This analysis showed that a 2.5:1 cut slope

would be stable. A back analysis done after the slope failure revealed the following parameters: phi=20 degrees, cohesion=0 psf, with water 15 ft. below the surface.

Construction practices may lead to instability. With the contractor excavating a 40 ft. vertical cut at the base of the slope he is removing the material that acts as a resisting weight to the block failure. This action along with the high ground water may have been the mechanism that triggered the landslide.

A rock buttress is to be designed and constructed at the toe of the slope to act as a counter balance to the block failure.This buttress will be 85 ft. wide and 80 ft. high and contain 240,000 cubic yards of blasted rock. The estimated amount of material to be removed will be 400,000 cubic yards at a cost of $4.00 per cubic yard. The cost for placement of the blast rock will be $1.00 per cubic yard, bringing the total cost of repair to $1,840,000.

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WOLF MOUNTAIN LANDSLIDE
MILEPOST 135.67
SNAKE RIVER CANYON
TETON COUNTY, WYOMING

Mark Falk, P.E., P.G.
G. Michael Hager, P.G
WYDOT, Cheyenne, Wyoming

On May 18, 1997, a large snowmelt-triggered landslide occurred on a steep slope approximately 1500 feet vertically above U.S. Highway 26/89 through Snake River Canyon in northwestern Wyoming. The result of this landslide was a debris flow consisting of rock, mud, and trees which covered a 300 foot section of the highway with a 15-foot thick deposit. The debris flow continued to be active for approximately two weeks, during which time up to 100,000 cubic yards of material flowed down the slope.

The closure of the road as a result of the slide had an immediate negative economic impact on the surrounding area. The highway through Snake River Canyon is the main route for supplies and tourists coming from the west to Jackson, Wyoming. In addition, there are approximately 400-500 commuters who travel from the Star Valley area at the west end of the canyon to work in Jackson. The alternate route was a 90 mile detour over two mountain passes, which during peak traffic could take over two hours compared with the normal 25 minute drive up the canyon. The closure of the road took a heavy toll on the rafting businesses based in Jackson, because the most popular rafting areas were downstream from the slide. The only access to these areas involved the 90 mile detour, which made most of the rafting trips cost-prohibitive.

The area was declared a disaster by the Governor, and the Wyoming National Guard was mobilized to help clear the roadway. A permit was obtained from the Army Corps of Engineers to place the material that was cleared from the road onto the delta in the Snake River which had been formed by the debris flow. Within three weeks, the road was opened to twice-a-day commuter traffic, and within one month, traffic was restored during daylight hours. Six weeks after the initial slide, the highway was opened to full time traffic.

For the protection of the highway, two small basins were constructed above the roadway to catch small scale mud flows, and a 48" diameter culvert was installed under the road to carry the water. Due to the limited size of the catchment basins and continual small mud flows, regular cleaning of the basins was required throughout the summer.

An extensive geologic mapping and seismic investigation of the slide area was performed to determine the potential for similar failures in the future. This investigation concluded that there was little potential in this particular drainage for a similar large scale failure which would require major, expensive remediation such as a tunnel-like structure to protect the roadway or shifting the alignment to the other side of the river. Based on the mapping, it was determined that the most cost effective solution would be to expand the catch basins from the existing capacity of approximately 500 cubic yards to 6000+ cubic yards.

INTRODUCTION

U.S. Highway 26/89 through Snake River Canyon is the main route from the west for supplies and tourist traffic into Jackson, Wyoming, which is the most popular destination resort community in the state. Traffic counts during summer months of 5,000-6,000 vehicles per day make this one of the most heavily traveled two-lane primary highways in the state. In addition, there are 400-500 commuters who travel from the Star Valley area at the west end of the canyon to work in Jackson year-round. This road provides year-round access to all recreational activities in the Snake River Canyon, including, rafting, fishing, hunting, hiking and kayaking. This road is truly the economic lifeline of the area.

The alternate route into Jackson from the west is a narrow two-lane road with grades of 10%, which crosses an 8400 foot high mountain pass. Due to the steep grades, trucks are not allowed over this mountain pass. From the Star Valley area, it takes approximately two hours to reach Jackson using the route over Teton Pass, compared with 25-30 minutes through Snake River Canyon.

THE EVENT

In the early morning hours of May 18, 1997, it was reported to WYDOT maintenance personnel in Jackson that muddy water was flowing across the road at milepost 135.67 on US 26/89 in Snake River Canyon. A crew was immediately dispatched to clear the road and close it, if necessary. The muddy water flow continued until dawn, at which time the water flow turned into a viscous mixture of mud and rocks which began to accumulate on the road. At this time, the source of the material was still unknown. By 9:00 a.m., several large trees (in excess of 40' tall) had been carried by the mud and covered the road. At approximately noon, the WYDOT Resident Engineer from Jackson hiked up the mountain to determine the source and/or cause of the mudflow. The thick timber, very steep terrain, and snow cover made it very difficult to observe the mudflow along the lower portion of the mountain. In a clearing approximately 1300' above the roadway, water and mud was observed flowing from an isolated point in a small drainage. Above this point, large semi-circular tension cracks were observed in the snow.

The next morning, WYDOT Geology personnel flew to Jackson to observe the mudflow. The group took a helicopter tour of the ridge to get an aerial view of the slide. By this time, a large rotational landslide had failed, exposing a large cirque-like headscarp. The slump block which had formed below the scarp was approximately 400' wide at the face and 450' from the back scarp to the face of the block. The headscarp above the back of the slump block was approximately 100' high. At the base of the slump block, a near vertical 50' high face had formed. At this time, it was possible to visually observe the movement of slide. It was a glacier-like movement, which caused large masses of the material from the face of the slump block to cave off into the water stream which was emanating from the base of the block. It was estimated that this water was in excess of 200 gallons per minute. When the material from the face was mixed with the water flow, it formed a concrete-like consistency material which flowed down the slope in the form of a debris flow. Approximately 450' vertically below the slump block, the channel narrowed from 400' to 80' wide. At this point, the trees and rocks in the debris flow would form small dams that would block the flow until enough material accumulated that the dam would be breached. This resulted in a series of surges of material

being transported down the channel, which ultimately covered the roadway.

By the end of the day on May 19, a 300' section of the highway was covered with a 15' thick deposit of mud, trees and rocks. The delta which had formed below the roadway extended approximately 250' out into the Snake River. The river valley in this area is very wide, so the addition of the material into the river did not significantly affect the flow characteristics of the river. At this time, WYDOT Geology personnel advised district personnel that there was nothing that could be done to prevent further debris flow and the slide would have to "run its course". The district personnel made plans for a long term road closure and informed the public that the highway would be closed until the slide stabilized (see figure 1).

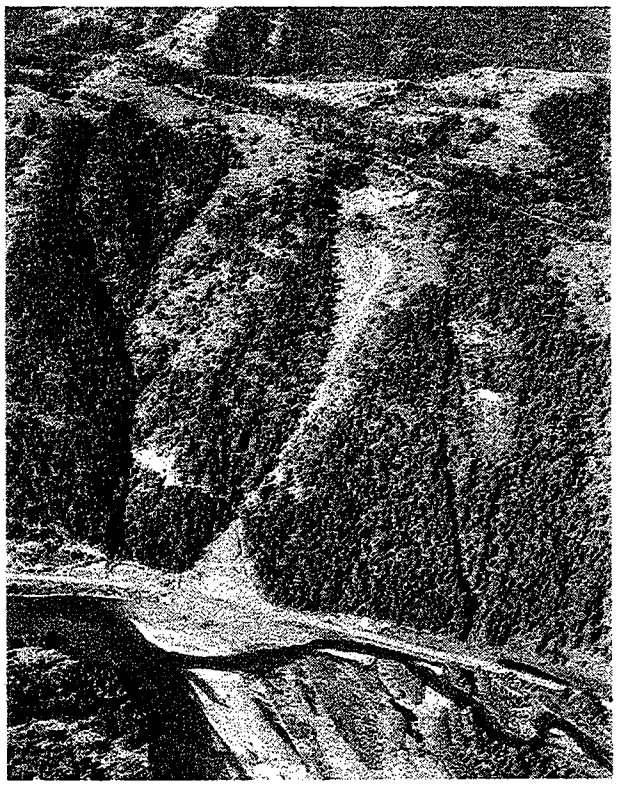

Figure 1. Oblique view of the Wolf Mountain Slide. Notice failed rotational block near the top of the ridge and delta of slide debris in the Snake River.

For the following six days, significant amounts of debris continued to flow down the slope and accumulate on the roadway. By May 27, the volume of material that was flowing onto the roadway began to decrease, and it was apparent that the slide was starting to dry out and beginning to become more stable. The primary concern at this point was to get the roadway open so that traffic could once again safely pass through this area. This became more critical after Memorial Day, which is the official begining of the tourist season and the time when traffic volumes in the canyon increase dramatically.

On May 27, due to the economic impact the road closure was having on the businesses and commuters in the area, Governor Jim Geringer declared the site a disaster area. As a result of this proclamation, it was possible for the Governor to mobilize a National Guard Engineering Unit to clear the debris from the roadway. There were several other regulatory hurdles that had to be cleared

before this operation could commence. The first problem was what to do with the material. The Guard unit did not have the capability to truck-haul the material to an off-site waste area. It was estimated that it would cost approximately $350,000 to haul the material to another location to be wasted. An application was made to the U.S. Army Corps of Engineers for an emergency permit so that material removed from the roadway could be placed on the "delta" of slide debris which had formed in the channel of the Snake River. The emergency permit was issued on May 29 to allow up to 20,000 yd^3 of material to be placed on the delta in the river.

This site is within a designated critical eagle habitat area, in which any activity with construction equipment is prohibited during this time period to avoid disturbance of the recently hatched bald eagle chicks. An agreement was made with the U.S. Fish and Wildlife Service to relax these restrictions. According to this agreement, the USFWS would monitor the behavior of the eagles to determine if there was any negative impact to the eagles from the slide removal operation. WYDOT and the National Guard would try to limit activity and noise, when possible, during early morning and late afternoon feeding times. A special use permit was also required from the U.S. Forest Service to build some access trails outside the highway right-of-way as part of the cleanup operation. Given the tight time frame for all of these agreements, there was very good cooperation among all of the agencies involved.

The actual removal of the slide debris began on Sunday, June 1. In the begining, the removal of material was very slow because of the number of trees in the debris, and they had to be removed one at a time. As part of the agreement with the Forest Service, all trees removed from the debris were stockpiled to be sold as salvage timber. As a safety measure, whenever equipment was working at the base of the slide, there were three spotters placed at various positions along the slide path to watch for any dangerous movement of material. These spotters were in constant radio contact, so if there was a significant debris flow event, the personnel at the base of the slide could have adequate warning.

By June 4, the majority of the material had been removed from the road and equipment could pass from one side of the slide to the other. There was still approximately 100 gallons per minute of muddy water flowing down the slide chute at this time. This was a major consideration, because prior to the slide there had not been a culvert under the road at this site, and the roadway could not be opened until the water flow was controlled. A 48" diameter culvert was installed under the road, and the water flow was diverted to this culvert. The National Guard spent the remainder of this two week exercise removing slide debris from the backslope above the roadway. A private contractor completed the backslope work and constructed two retention ponds to contain small amounts of material carried down the mountain during small rainfall events. In addition, an access trail was constructed to allow the retention ponds to be cleaned.

Begining on June 9, the road was opened for commuter traffic. In the mornings northbound traffic was allowed through for one hour, and in the evenings southbound traffic was allowed through for one hour. On June 17, the road was opened for all traffic during daylight hours. During this period, flaggers were present, as well as spotters on the mountain to monitor any slide movement. By July 3, the slide had stabilized enough that the road was opened to unrestricted two-way traffic 24 hours a day.

THE INVESTIGATION

After the road was re-opened, WYDOT concentrated on developing a long-term plan for dealing with the slide. Two days after the slide occurred, a value engineering team composed of members of different programs within WYDOT was assembled to do some "brainstorming" and come up with a list of possible solutions. The list of options ranged from a permanent closure of the road through the canyon, to re-channeling the Snake River to the opposite side of the river valley. From this wide and varied list, three alternatives were proposed to the executive staff of WYDOT. These alternatives included relocating the highway to the other side of the river, construction of a tunnel-like structure across the slide path to allow future slides to flow over the road, and building a bridge which would span the slide path and allow future slides to pass under the bridge and allow the river to naturally carry away the material. The options were based on two basic assumptions, the first being that in the future, the landslide which created the debris flow would continue to move and provide large amounts of material for additional debris flows every year. The second assumption was that since there was now a well-defined channel where there was previously a densely vegetated slope, snow avalanches in the winter would be a constant maintenance problem.

Immediately after failure, there was concern that the approximately 500,000 yd^3 slump block was very unstable and had the potential for a catastrophic failure. At that time, it was thought that the slide failed in a thick sequence of colluvium that had been deposited in the bowl-shaped valley. It was unknown whether the water flow from the base of the slump block would dry up as snowmelt was completed, or if the slide had interrupted the natural groundwater flow and this would become a perennial high-volume spring. Given these facts and all of the unknowns associated with this slide, a conservative approach was taken by the Geology Program, and concern about the potential stability along the entire ridge between Dog Creek and Cabin Creek was raised.

Before any long-term plan for dealing with the slide could be implemented, it was necessary to have a good understanding of the slide that failed, and how it was affected by such things as the geology, topography, presence of water on the slope, vegetation and potential triggering mechanisms. Understanding these factors was the key to analyzing the potential for other failures along this ridge. The WYDOT Geology Program undertook a thorough mapping project of the entire ridge to collect all of this information. The ridge is a proposed wilderness area, and this designation limited access.

The ridge on which the Wolf Mountain Slide (WMS) occurred is a north-south trending ridge approximately three miles long, which parallels the Snake River from Dog Creek on the north to Cabin Creek on the south end. From the river elevation to the crest of the ridge there is a maximum of approximately 1600' of vertical relief. The geologic formations along the ridge dip to the west into the ridge at approximately 25-30 degrees (Love and Christiansen, 1985 and Albee, 1968). A complete geologic section from the Lower Cretaceous age Bear River Formation which forms the top of the ridge, is exposed down section through the Middle Jurrasic age Twin Creek Limestone which outcrops at the river (see figure 2 for Geologic Cross-section).

The WMS occurred within the shales and interbedded friable sandstones and siltstones of the lower

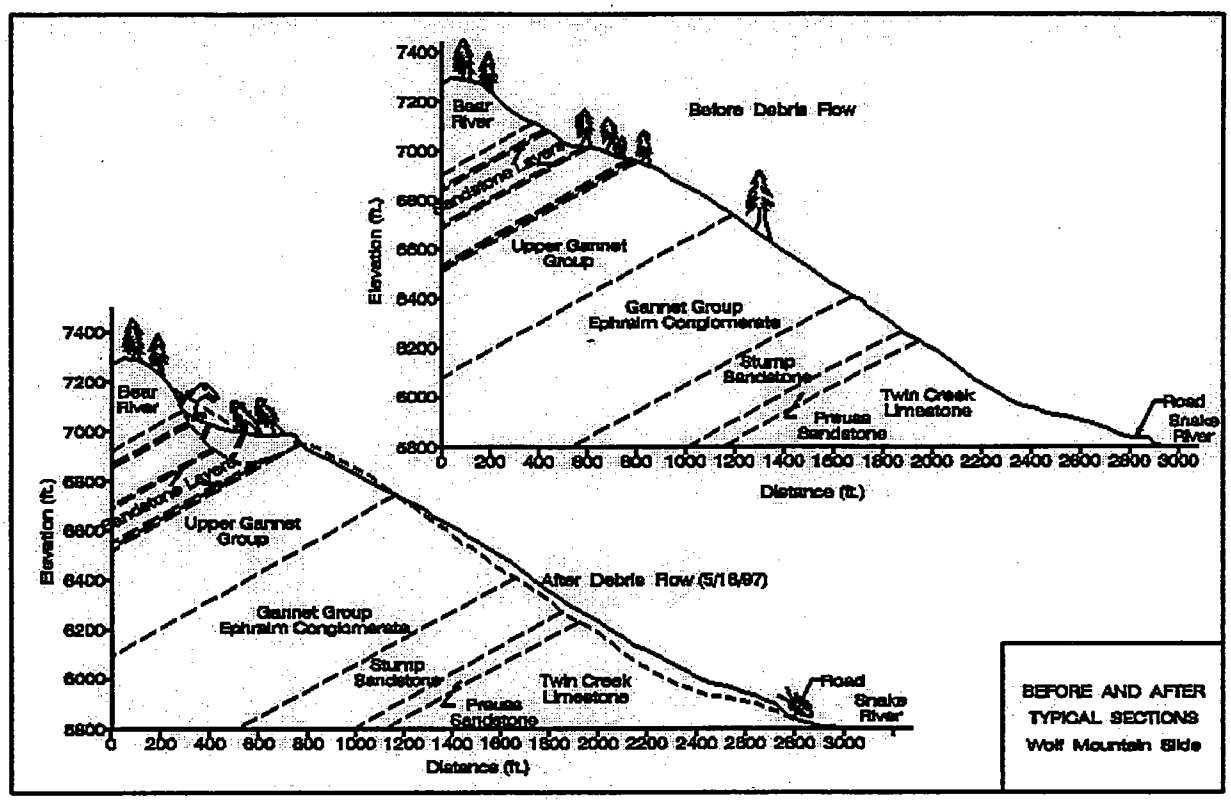

Figure 2. Before and after cross-sections of the Wolf Mountain Slide showing geologic formations.

portion of the Lower Cretaceous Bear River Formation. The base of the slide terminated at the top of the limestone which forms the top of the Cretaceous Gannett Group. Since the formations along this ridge dip toward the west at 25-30 degrees, the truncated ends of the formations on the east facing slope are subject to erosion. The hard resistant cross-bedded sandstone which forms the ledge rock is well exposed above the failure and becomes less prominent along the ridge in both directions. A 10'-15' thick layer of colluvium overlying the failed shale bedrock is evident along the north and west side of the slide. Along the south side of the slide, a contact between the colluvium and bedrock is not apparent. A seismic traverse run along this edge indicates that the colluvium along this side is in excess of 30' thick.

A cross-section was made from the topographic map which was created before the slide failed, and it was determined that before failure, the slope angle within the shales of the Bear River Formation was 32 degrees. In addition, aerial photos indicate there were fewer trees in this bowl compared to some of the adjacent drainages. An area of vegetation that is indicative of high moisture content soils and some slight water seeps was noted at the southeast corner of the slump block.

In the drainage immediately north of the WMS there is a large old failure which is nearly identical in size and shape to the WMS. The cross-bedded sandstone ridge above this failure is very prominent, and the scarp geometry of the circular failure in the shale is identical to the WMS. The slump block has moved down so that the toe of the slide is directly on top of upper limestone of the

Gannett Group as identified in the WMS discussion. This large slide is divided into a north and south slump, with the south slump being the most recently active. Cores were taken from trees in this portion of the slump block, and they indicate that these trees are approximately 80 years old. Although no cores were taken from the trees in the north and oldest portion of the slide, based on the diameter of these trees, it is estimated they could be on the order of 130-140 years old. Looking at this slide relative to the river, it is theorized that this large slide and the resulting debris flow deposited a large amount of material in the Snake River at some time before the highway was built. This would explain the presence of the gravel bar on the outside bend of the river, which is not a typical depositional pattern for this type of river.

Given the fact that this old slide has a slump block nearly identical in shape and size to the WMS slump block, it was concluded that the potential for further large scale catastrophic movement of the WMS slump block is low.

Approximately 2000' south of the WMS at the highest point along the entire ridge, erosion has truncated the shale layers within the Lower Bear River Formation. From this point southward to Cabin Creek, there is no shale exposed on the easterly facing slope. From this, it is concluded that along this portion of the ridge there is very little possibility of a failure similar to the WMS.

This investigation identified only one area that appears to have the potential to experience a catastrophic failure similar to the WMS. This area is located in the drainage immediately to the south of the WMS. This area is a fairly deep bowl with relatively little vegetation. The cross-bedded, sandstone ridge above this bowl is quite prominent, and the shale in the Bear River Formation is standing at approximately 27 degrees. There is no evidence that any failure has taken place in this bowl. There are very few trees on the ridge above the bowl, and it appears that large amounts of snow collect in this deep drainage. In addition, there is a fairly well-defined drainage channel from this bowl to the roadway.

It is believed that a major factor in this bowl remaining stable is the existing slope angle of 27 degrees within the shale, compared to the 32 degree slope angle which was present within the shale in the WMS before it failed. Another reason it is believed the potential for this bowl to fail is low is that it remained stable throughout back-to-back years of near-record precipitation, and if weather patterns return to somewhat normal for a year or two, this material will probably become more stable.

In addition to the geologic and topographic information gathered, some research was done into the effect temperature has on snowmelt-triggered landslides. A study done on 20 historical slides in the central Rocky Mountain region by the USGS (Chleborad, et al, 1997 and Chleborad, 1997) indicates a strong relationship between landslide events and rising temperatures. This study concluded that the highest percentage of snowmelt-triggered slides occur within 18 days after the first yearly occurrence of a six-day running average of daily maximum temperature greater than or equal to 58 degrees at the site. The WMS took place five days after the six-day running average reached 58 degrees (Personal communication, Alan Chleborad, USGS, 1997 and University of Wyoming, 1997).

THE SOLUTION

Based on the information obtained during the mapping investigation, it was reported to the WYDOT executive staff that the potential for further catastrophic failures along this road section was low enough that expensive alternatives such as moving the highway to other side of the river, building a tunnel or "mud shed", or building a bridge to span the slide path, were not warranted. It was recommended that remediation efforts be concentrated on stabilizing and controlling the material which was generated from the failure of the slide. These efforts included enlarging the retention pond, and seeding the slopes to help prevent small scale surface erosion.

It was decided that enlarging the retention pond was the most cost effective option. In order to minimize addditional disturbance, the retention pond was designed to be within the limits of the slope that had been affected by the slide. In plan view, this resulted in a triangular excavation which had a relatively flat bottom and 1:1 side slopes up to 180' high. Parallel to the roadway, a berm with 1.5:1 (H:V) side slopes was constructed to a height approximately 10' above the bottom of the retention pond. This retention pond was designed to have a capacity of 7800 yd^3 when material is level with the top of the berm. To obtain this capacity, it was necessary to excavate approximately 117,000 yd^3 of the hillside. See Figure 3.

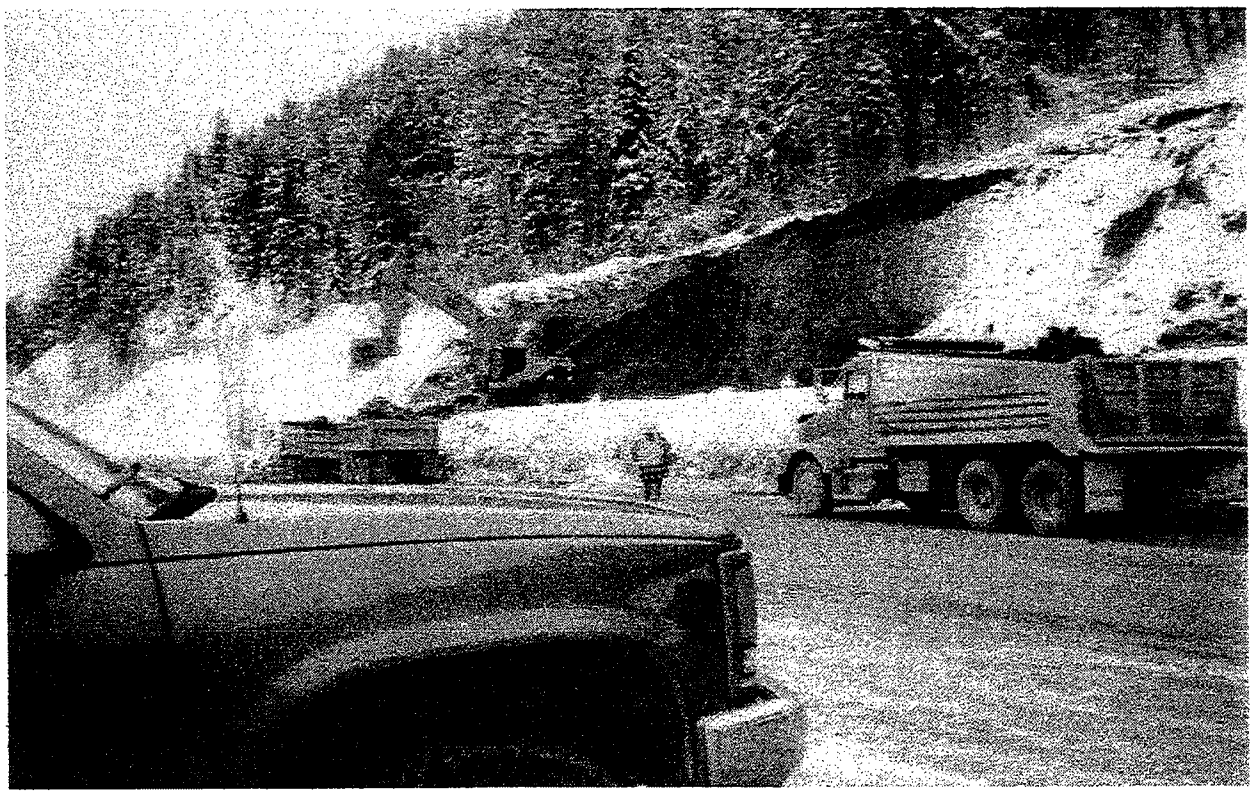

Figure 3. Excavation of the catch basin between the road and the debris flow chute. The trackhoe is sitting on the planned berm that keeps the landslide debris from covering the road and directs the mud to a 48" pipe located in front of the truck on the left side of photo.

Construction of this $1.24 million dollar project began on January 3, 1998. The project had to be completed by February 15, 1998, because it was within the critical eagle habitat area, and since it was no longer an emergency situation, all regular USFWS restrictions applied. Before construction began, small charges were dropped within the chute above where the retention pond was to be constructed to relieve the potential for snow avalanches. The snow showed very little avalanche response to the bombing, so from this fact and the observed slope angles within the slide path, it was concluded that avalanches were not going to be a significant problem at this site. All of the material which was removed to create the retention pond had to be truck-hauled to waste sites. Half of the material was taken to a site 11 miles from the slide, and the other half was taken to reclaim an old gravel pit 18 miles from the site. As part of this contract, it was recommended to replace the 48" diameter culvert which is the outlet for the pond with a larger structure, possibly a concrete box culvert, to allow the mud to flow under the roadway more easily. Given the extremely tight time schedule and very difficult winter working conditions, it was determined that it wasn't possible to install this structure under this contract. Even though five days were lost to bad weather and approximately six feet of snow accumulated during construction, the project was completed two days ahead of schedule. (And the eagles were not disturbed!)

THE RESULTS

In mid-March, 1998, three days of temperatures in the low 50's triggered some activity at the slide. The flow of water increased from the trickle that was present during construction, to 30-40 gallons per minute. Mud and rocks were begining to accumulate in the retention pond, and it was feared that the pond would quickly fill. Geology personnel inspected the site on March 31 and determined that most of the material had come from within the slide chute on the bottom half of the mountain. The weather returned to a more normal pattern for the next four weeks, with lower temperatures, very little further snow accumulation and relatively slow snowmelt. As a result of this, there was little additional accumulation of material in the retention pond, and the water flowing through the pond was removing material in a self-cleaning action.

On April 29, 1998, WYDOT Geology personnel inspected the pond at 5:00 p.m. A week of weather in the 50's-60's had activated the slide again. A water flow estimated to be 100 gallons per minute was carrying mud and rocks, some of which were 2'-3' diameter, into the pond. At this time, the mud was flowing directly from the slide chute to the 48" culvert and passing under the roadway with very little accumulation of material in the pond. During that night, several large trees and root balls came down and plugged the culvert. By 7:00 a.m. on April 30, the pond was completely full, level with top of the berm. At this time, equipment was mobilized to clean the inlet of the pipe and begin removing material from the pond to allow storage of more debris. The slide remained very active for the next 24 hours, with most of the material originating from the area where the slide path below the slump block narrows from 300' wide to approximately 80' wide. After this 24 hour period, the majority of the snow was gone from slide chute and significant movement of mud had ceased. The flow of water emanating from the base of the slump block had increased to approximately 200 gallons per minute, and this increased water flow was begining to clean significant amounts of mud and rock out of the retention pond. Between the natural cleansing action of the water and the equipment removing material from the pond, approximately one-third of the material was removed

by May 5. After this event, a temperature study determined that on April 29, the six-day running average temperature of 58 degrees was reached (University of Wyoming, 1998). See figure 4.

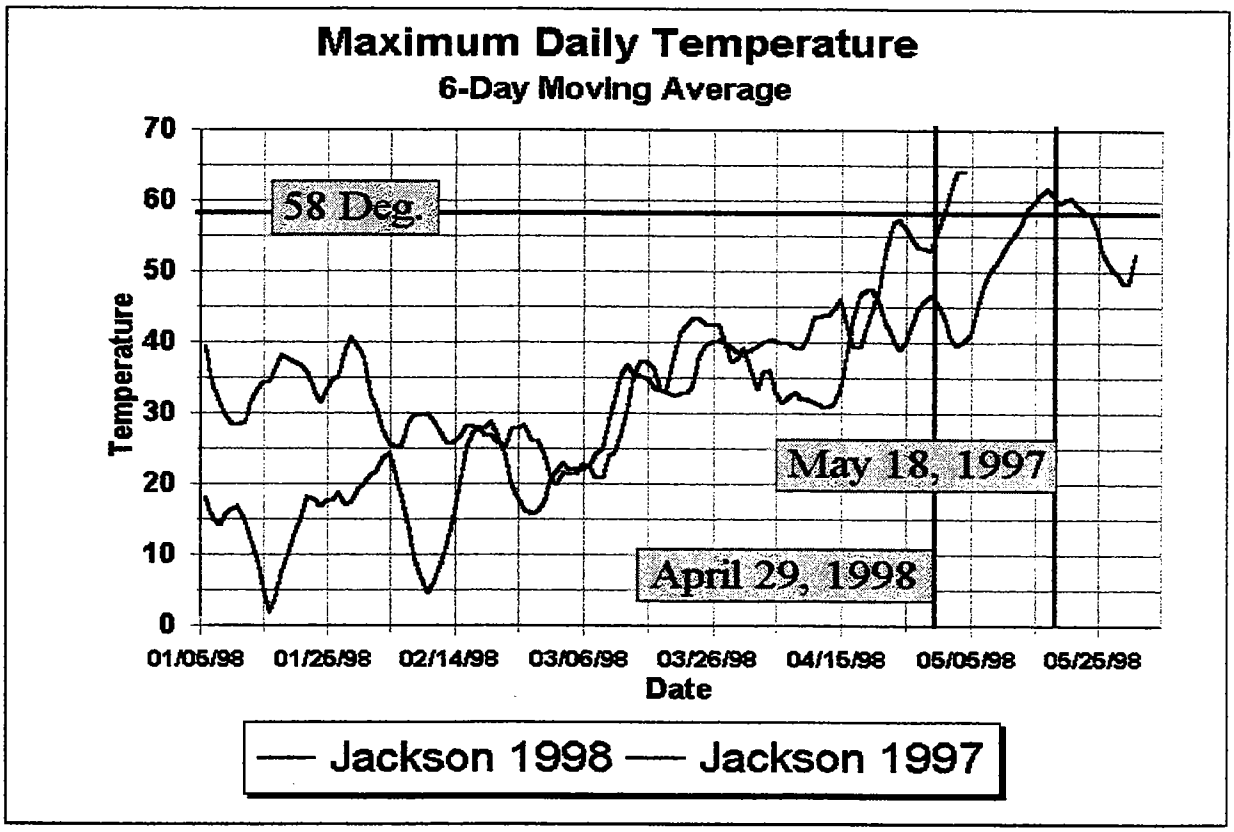

Figure 4. Graph of the 6-day moving average of the maximum daily temperature.

The next phase of the slide remediation was the aerial seeding of the slide debris within the chute. It is believed that if some vegetation can be established within the slide debris, the amount of material that will reach the retention pond during snowmelt and extended rainfall events can be significantly reduced. This seeding was done in mid-May, 1998.

CONCLUSIONS

It appears the most critical debris flow events at the Wolf Mountain Slide are snowmelt-related and can be expected to occur yearly. In the future, running temperature records will be kept to help determine when the most critical time for debris flows will be. It is anticipated that the amount of material which comes down will decrease each year, as vegetation begins to grow in the slide chute and the loose material in the chute flattens close to its natural angle of repose. It is imperative that WYDOT clean the retention pond each fall so that in the spring there is sufficient storage capacity to hold the debris flows. Although this natural action of soil, water, and gravity cannot be prevented, it is believed that WYDOT has developed an effective method to manage the results of these debris flows with minimum inconvenience to the traveling public.

ACKNOWLEDGEMENTS

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Micropile and Tieback Reinforced Walls for Landslide Stabilization, Snake River Canyon, WY

John Turner[1], Robert Hasenkamp[2], and John Wolosick[3]

Abstract

Stabilization of the Blue Trail Landslide on U.S. Highway 26/89 was accomplished by constructing three reinforced earth retaining walls, each consisting of battered micropiles (also referred to as minipiles) and tieback anchors fixed to a cap beam. The landslide and walls are the subject of an instrumentation and monitoring program. Objectives are to determine the mechanisms of load transfer from the sliding mass to the structural elements of the wall and to evaluate the effectiveness of this type of wall in controlling ground movements. Instrumentation to monitor load transfer consists of strain gages on micropiles and load cells on tieback anchors. Ground and wall movement instrumentation includes tiltmeters, slope inclinometer casings, and ground surveying. Piezometers are being used to monitor variations in pore water pressures with time. Monitoring results to date show that the tieback anchors are providing most of the resisting force, because they were installed and prestressed after installation of the micropiles. The design load-resisting capacity of the micropiles will be mobilized only if additional ground movement occurs. Measured ground movements since construction of the walls have been minimal, indicating this is a highly effective technique for stabilizing highway slopes.

The Blue Trail Landslide

U.S. Highway 26/89 between Alpine Junction and Hoback Junction in northwestern Wyoming follows the scenic Snake River Canyon, which cuts through Paleozoic and Mesozoic sedimentary rocks of the Wyoming-Idaho Overthrust Belt. This physiographic region is characterized by northward trending low-angle thrust faults along which thousands of feet of marine sedimentary rocks were displaced eastward about 50-150 million years ago. The Blue Trail Landslide is located at M.P. 127, approximately 15 km east of Alpine Junction. Seasonal slide movements created ongoing maintenance and safety problems from the time the current roadway was constructed, between 1955 and 1960, until construction of the walls in 1997. The slide has been monitored since 1969, when WYDOT first conducted a site investigation of the slide area.

Geological and geotechnical aspects of the slide are described by Dahill (1995) and Hasenkamp et al. (1996). The slide trends N70°W (35° Rt. Skew) and stretches from the edge of the Snake River, 61 m below the road, to approximately 300 m above the existing

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roadway. The headward zone is active approximately 6 m upslope of the roadway and the mass extends vertically beneath the highway to depths of 10 to 12 m. The portion of the slide above the roadway and active headward zone is currently stable. Figure 1 shows the slide area as viewed from the west.

The slide mass at Blue Trail consists of a heterogeneous mixture of detrital sandstone, siltstone, shale, and limestone with sand, silt, and clay. Slide debris is predominantly brecciated rock that has deteriorated to the consistency of clayey gravel, and varies in thickness from 3.5 m to 15 m with various perched water tables confined to permeable zones. Water levels appear to vary seasonally. The landslide debris is underlain by the Lower Cretaceous Bear River Formation, which consists of interbedded shales, siltstone, and thin limestone at a strike of N67°W and dip of 27°W. The eastern edge of the slide is very near the contact between the Bear River and underlying Gannet Formation. The Lower Cretaceous Gannet Group consists of siltstone, sandstone, and cross-bedded quartzite.

The Blue Trail Landslide is a block-type failure with movement occurring along a relatively weak layer at the soil-rock interface. Observations indicate that movement is initiated through a combination of wet conditions in the slide and high water levels in the Snake River, both of which occur during maximum runoff in late spring or early summer. When the slide debris is at a high water content, driving forces are increased (higher weight) and resistance is decreased due to higher pore water pressure and lower effective

Figure 1. Blue Trail Landslide site, April 1997.

Normal stress on the failure surface. High water in the river erodes the toe of the slide, further reducing resistance. The combination of increased driving forces and decreased resistance is believed to be the mechanism for slide initiation.

Additional site investigations were conducted more recently by Chen-Northern (1989) and WYDOT (1995) for the purpose of selecting and designing remedial measures to control movement of the Blue Trail Slide. These investigations were used to identify the slide surface, establish depth to bedrock, and obtain samples for laboratory testing. Slope stability analyses were then conducted using XSTABL. Assuming that the slide was close to a state of limit equilibrium (FOS = 1.0) stability analyses were used to back-calculate average effective stress strength parameters for the failure surface. Assuming there is no effective stress cohesion (c = 0), an average residual friction angle ϕ = 14° yields a global FOS = 1. These strength parameters were used in subsequent analyses to assess various means of increasing stability of the slide.

Summary of Design

After considering a variety of approaches for stabilizing the Blue Trail Landslide (Hasenkamp et al., 1996), Wyoming DOT accepted a design-build proposal submitted by Nicholson Construction Company to construct three of its trademark INSERT (**IN** Situ **E**arth **R**einforcement **T**echnique) walls. Each wall consists of a row of battered micropiles installed through and attached to a concrete cap beam, as illustrated in Figure 2. For the Blue Trail Landslide, potential seismic loading required the addition of tieback anchors, also installed through and attached to the cap beam.

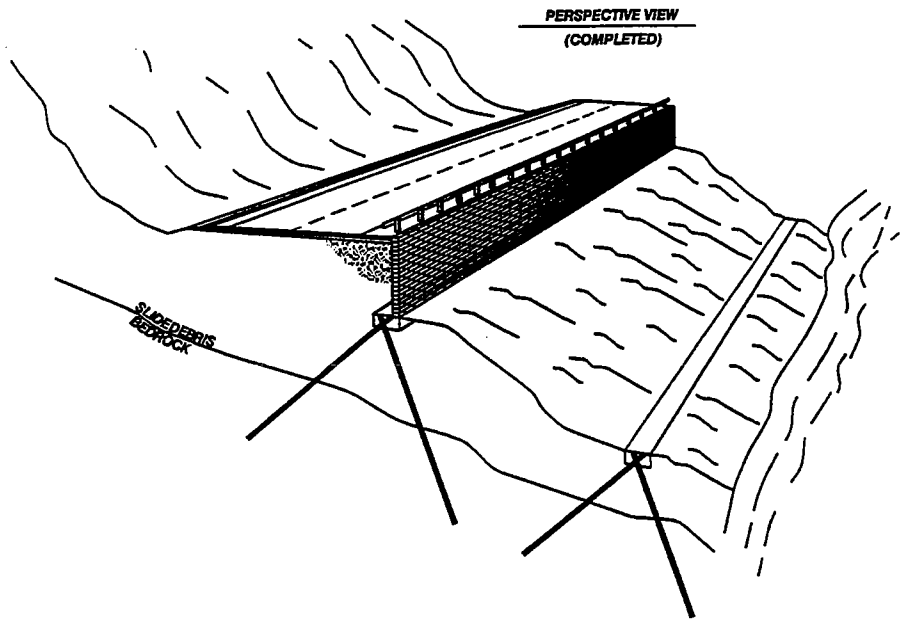

Figure 2. Conceptual drawing of micropile-reinforced INSERT wall.

The general steps in the design of a micropile-reinforced earth wall for slope stabilization are as follows (Pearlman et al., 1992):

1. Conduct slope stability analyses to determine the additional resisting force on the failure plane required to achieve the desired factor of safety.
2. Check the potential for structural failure of the micropiles due to loading caused by the moving soil.
3. Check the potential for failure by plastic flow of the soil around the micropiles.

For the Blue Trail Slide, slope stability analyses (Step 1) were conducted to estimate the driving and resisting forces for the existing conditions, for which it was assumed that the slope was in or close to a state of limit equilibrium (FOS = 1.0). For the static case (i.e., no seismic) these analyses yield an approximate driving (and resisting) force of 5.6 MN/m (383 k/ft). For the seismic case, assuming a pseudo-static seismic coefficient of 0.11 g, the driving force increases to approximately 7.86 MN/m (538 k/ft). Design criteria for the global stability of the slope and wall system were established as follows: static FOS $\geq$ 1.3 and seismic FOS > 1.0.

Resistance against sliding provided by INSERT walls is based on analysis of the structural capacity of the micropiles (Step 2) and the pullout capacity of the tieback anchors. Each micropile consists of a high strength steel pipe, 114 mm diameter and 7.6 mm wall thickness, grouted into a 152 mm diameter drilled hole. Estimates of micropile capacity are based on the assumption that landslide driving forces are transferred to the micropiles as axial and lateral loads. The ultimate lateral resistance provided by a single micropile is estimated using the *p-y* method of analysis, in which the micropile is assumed to act as a free-headed pile at the slide surface. The program COM624 (Reese and Sullivan, 1983) was used for the lateral load analyses. Additional resistance derived from battering the piles with respect to the direction of movement is then estimated based on an elasticity solution given by Poulos and Davis (1980). The amount of axial load transferred to the micropiles is limited by the ultimate side resistance that can develop between the pile perimeter and the soil in the sliding mass. Resistance to sliding derived from the bending and axial capacities of micropiles which are battered alternately upslope and downslope are then combined and micropile spacing is selected to provide the desired additional resistance to meet the minimum factor of safety.

The length of the active zone of sliding at Blue Trail is too great to permit a single location of support to control the several possible modes of failure. A single wall at the center of the slide would not necessarily prevent further settlement of the roadway, while a wall directly beneath the roadway would not control slope failures downslope from the wall. The most efficient solution was found to be the use of two INSERT walls, the first located 3m below the roadway ("upper") and the second located approximately midway between the road and the toe of the slide ("lower"). A third INSERT wall ("middle") was added at the western edge of the project, 12 m below the roadway, to increase the factor of safety of the slope where it was being surcharged by the MSE wall used for roadway widening. Figure 3 is a plan view of the site showing the final wall locations.

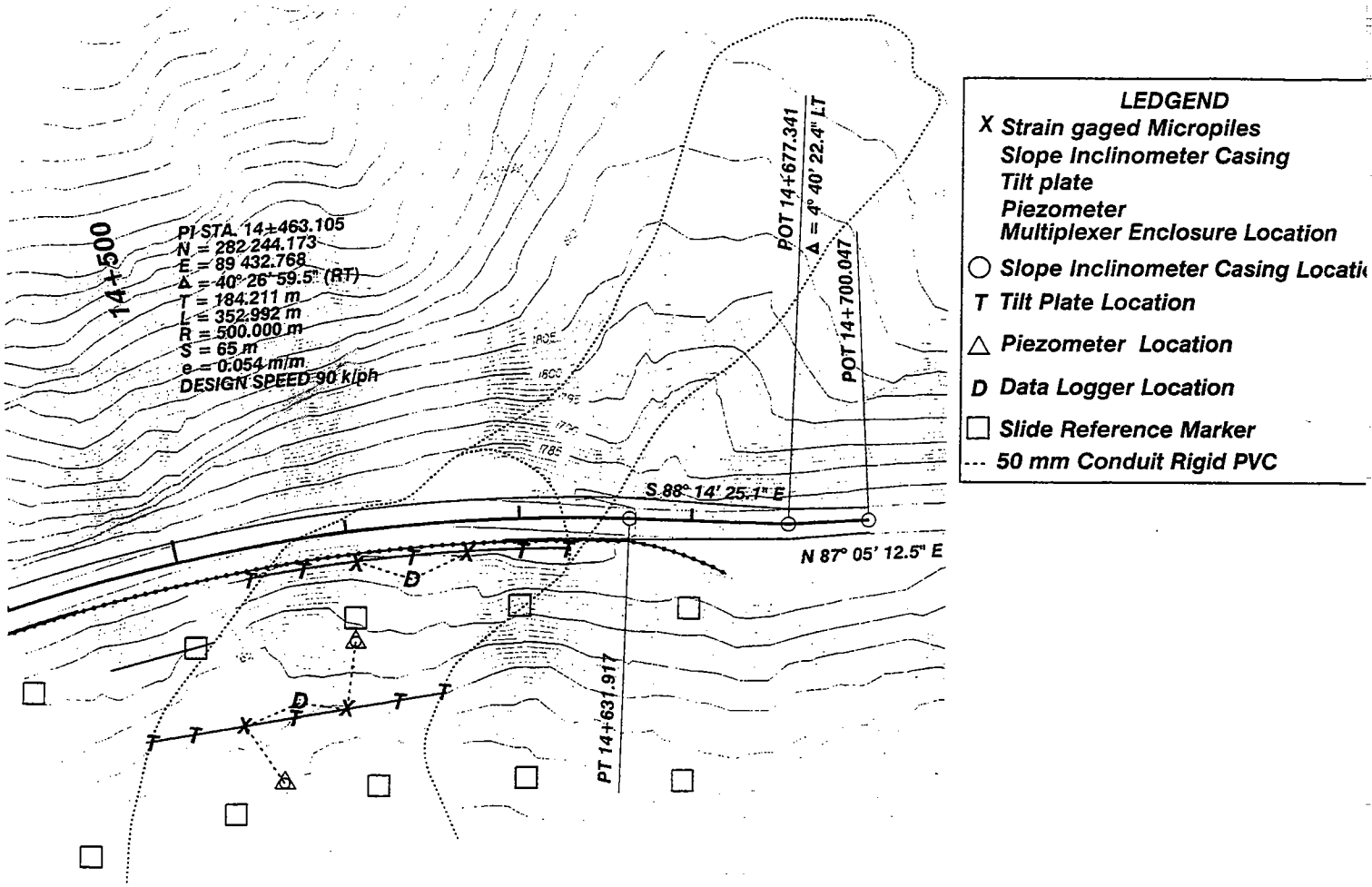

Figure 3. Plan of site showing slide, walls, and instrumentation locations.

Total resistance to sliding provided by two micropile walls does not satisfy the minimum required factor of safety against sliding for the seismic loading case. Tieback anchors provide the necessary additional resistance and also act to reduce post-construction ground movements to minimal levels. Tieback anchors in the upper wall were designed for a capacity of 1.56 MN (350 kips) at a spacing of 7.6 m (25 ft), providing 205 kN/m (14 k/ft), while the anchors in the lower wall have a design capacity of 2.82 MN (630 kips) at a spacing of 3.8 m (12.5 ft), providing 740 kN/m (50 k/ft). Tieback anchors are battered 30° from horizontal. In the slope stability analyses, effects of tieback loads on both normal force acting on the failure plane and the tangential component of force are taken into account.

Final design specifications for the two walls (upper and lower) are summarized in Table 1. Total resistance provided by the two walls yields a static FOS of approximately 1.6 and a seismic FOS of approximately 1.1. The three walls combined included 478 micropiles and 40 high-capacity tieback anchors. Figure 4 shows a typical section.

Table 1. Summary of Design Parameters for INSERT Walls

Wall	Length m (ft)	Micropile Spacing m (ft)	Micropile Resistance kN/m (k/ft)	Tieback Capacities MN (kip)	Tieback Spacing m (ft)
Upper	94 (308)	0.51 (1.67)	742.5 (51)	1.56 (350)	7.6 (25)
Lower	88 (289)	0.38 (1.25)	837.4 (57)	2.82 (630)	3.8 (12.5)

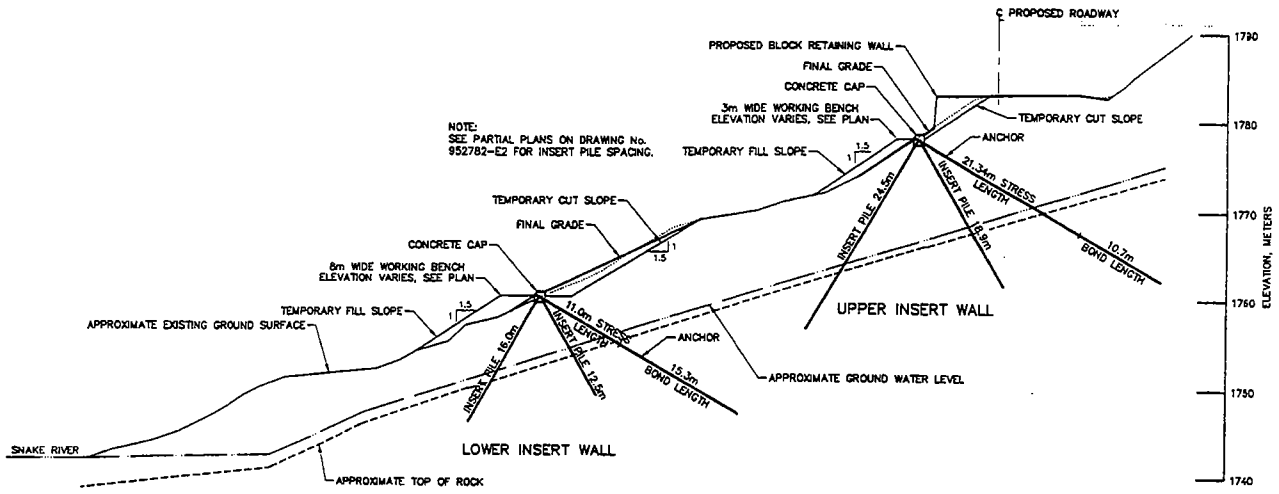

Figure 4. Typical section of micropile and tieback reinforced walls, Blue Trail Landslide.

Construction

The walls at Blue Trail Landslide were constructed by Hayward Baker, Inc. Construction began in April 1997 with excavation of access roads and working benches for the cap walls (Figure 1). Figure 5 shows the completed lower cap wall, which is a heavily reinforced concrete beam 1.2 m x 1.2 m in section. Blockouts consisting of 152 mm diameter corrugated plastic pipe were installed at 30 degree angles (alternating between upslope and downslope) to provide guides through which micropiles were drilled and grouted. Figure 6 shows the drilling operation, in which a 152 mm diameter hole is excavated using a crane-suspended, hydraulic powered rotary drilling rig. Drilling techniques included tri-cone roller bits and/or down-hole-hammer drilling tools. Typically, compressed air was used to flush drill cuttings. Drilling was often difficult

where boulder fill and talus materials were encountered. Micropiles were installed by using the high strength pipe piles as drill tooling. The piles were then withdrawn, the drill bit removed, and the pile placed to the bottom of the hole. Neat cement grout, consisting of portland cement and water (w/c ratio of 0.45) was injected from the bottom of the hole, filling the drill hole and the inside of the pipe. The final stage in wall construction was installation of the tieback anchors. Drilling and setting of steel casings was first done through the talus and overburden down to bedrock. Rock sockets, 12-17 m in length were then hammered through the casing. The strand anchors were placed down the hole and tremie grouted using neat portland cement. 10-strand anchors were used in the upper and middle walls (7-wire strands), while those in the lower wall have 18 strands. Figure 7 shows the lower cap wall and tieback anchor strands. The roadway above the upper wall is supported on a MSE wall consisting of geogrid-reinforced backfill with keystone block facing.

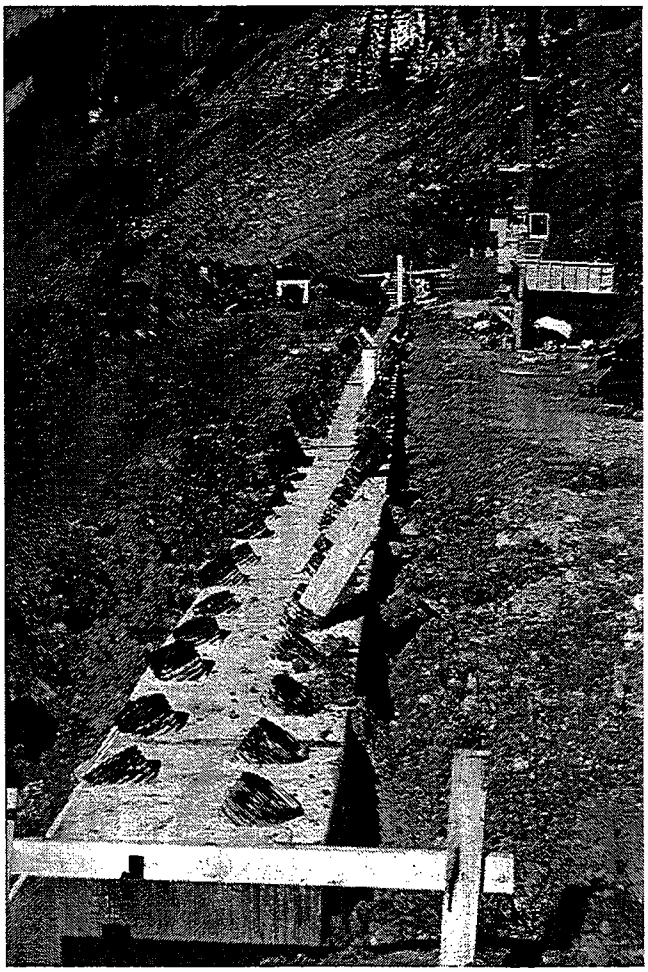

Figure 5. Lower cap wall before installation of micropiles and tieback anchors.

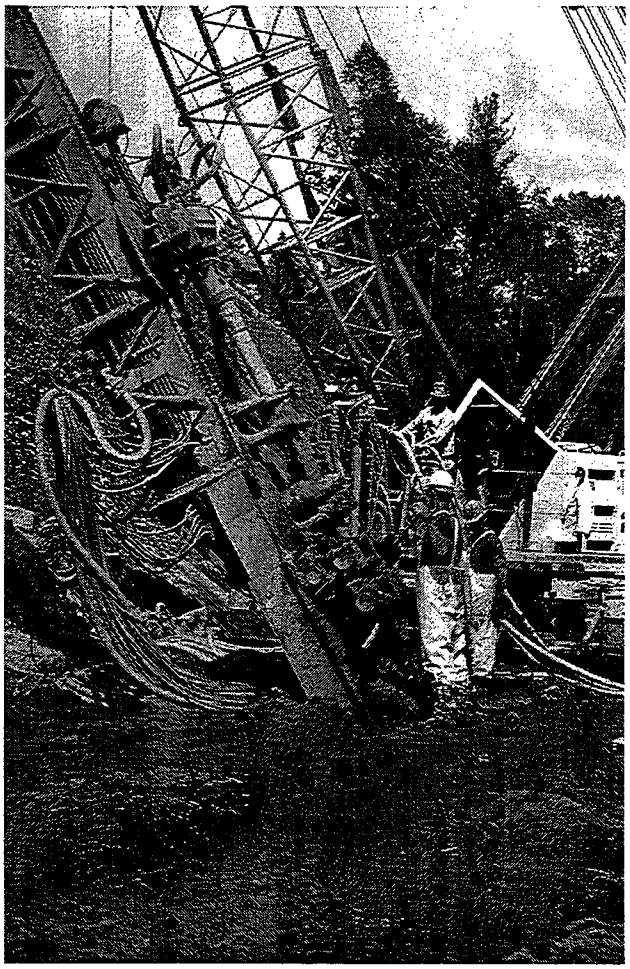

Figure 6. Drilling machine for micropile installation.

Figure 7. Lower cap wall and 630 kip tieback anchors.

Instrumentation and Monitoring Program

The INSERT walls at Blue Trail are instrumented as part of a monitoring program being conducted by Wyoming DOT and the University of Wyoming. The principal objectives are to assess the effectiveness of this type of landslide stabilization method and to investigate the mechanisms of load transfer to the structural components (micropiles and tieback anchors) of the walls. Instrumentation includes strain gages on micropiles and load cells on tieback anchors for the purpose of monitoring loads, and slope inclinometers and tiltmeters for monitoring wall and ground movements. In addition, several piezometers were installed to monitor pore water pressures close to the assumed slide plane. Figure 3 shows the instrumentation locations. Four micropiles in each wall (upper and lower) are instrumented with vibrating wire strain gages. Strain gage depths were selected in an attempt to have two monitoring points above the assumed slide plane and two monitoring points below as shown in Figure 8. The four instrumented micropiles in each wall include two adjacent piles (one upslope and one downslope) at distances of approximately one-third the wall length from the east and west ends of the walls, respectively (Figure 3). At each monitoring depth, three strain gages were attached at a circumferential spacing of 120 degrees (Figure 8), providing strain readings needed to determine principal stresses and their orientation, which are then used to calculate axial load and bending moment. Permanent load cells were installed to monitor tieback anchor forces in two anchors of each wall (four instrumented anchors total). The instrumented tiebacks are also located at distances of approximately one-third of the wall length from the east and west ends of the two walls (Figure 3).

Slope inclinometer casings are placed at various locations, including several which extend through the cap walls as well as locations above and below the walls (Figures 3,9). Casing deformations have been monitored on a regular basis since the start of construction through the present (June 1998). Several of the casings underwent excessive shear deformations and were replaced during the construction period, before the wall provided sufficient resistance to limit further slope movements. Tilt plates which can be monitored with a tiltmeter are located on the top surface of the cap walls at fourteen locations (seven on each wall) to monitor wall rotations. Readings were made beginning in late July 1997, before installation of the tieback anchors, and during September 1997, following installation and prestressing of the tieback anchors.

Results and Interpretation

Wall and Ground Movements

The slide was active during the initial stages of wall construction. Figure 10 shows inclinometer readings at inclinometer well No. 3, located in the center of the lower wall. The casing was installed on May 27, immediately after which movements occurred in the downslope direction. As the micropiles were being installed, the rate of movement decreased. As tieback anchors were installed, the direction of movement in the upper 3 to 4 meters below the wall began to reverse. When the tieback anchor closest to this

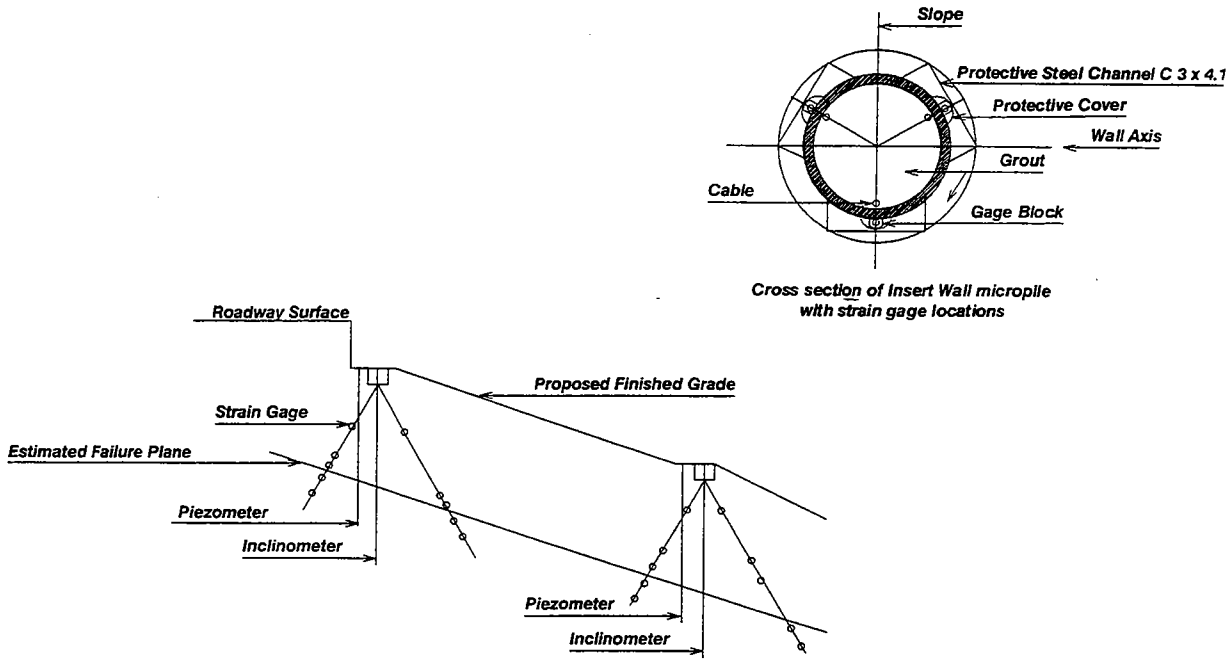

Figure 8. Details of strain-gaged micropiles.

Figure 9. Taking slope inclinometer readings, lower cap wall.

inclinometer casing was installed and prestressed (Aug 28), the cap wall and inclinometer casing moved in the upslope direction. This behavior is typical of inclinometer readings in the lower wall and is a direct result of the high prestress loads applied to the tieback anchors, which were 2.82 MN (630 k). Inclinometer readings in the upper wall also showed decreasing ground movements as the micropiles were installed, but did not show a reversal in direction with installation of tieback anchors. Prestress loads in the tieback anchors of the upper wall were 1.56 MN (350 k). The force per unit length of wall for the tiebacks in the upper wall is 205 kN/m, compared to 740 kN/m in the lower wall.

Wall and ground movements since construction of the walls have been minimal, based on tiltmeter, slope inclinometer, and ground surveying measurements, indicating that the INSERT walls have been highly effective in controlling the Blue Trail Landslide.

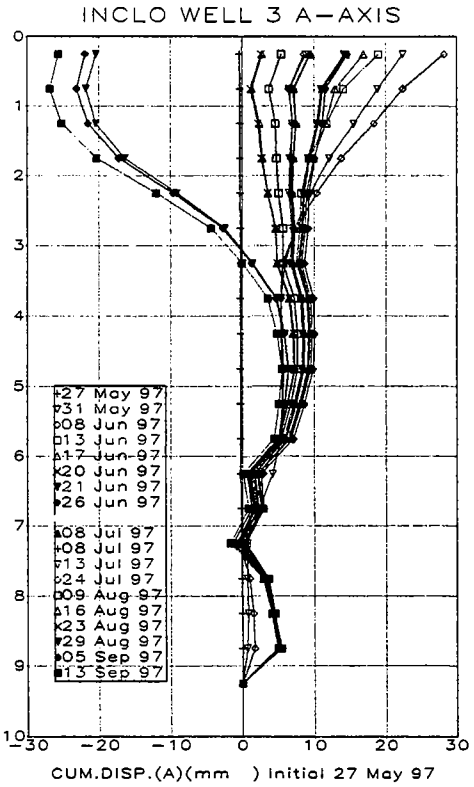

Figure 10. Slope inclinometer readings, center of lower cap wall.

Load Transfer

Figure 11 illustrates axial load transfer in micropiles of the lower wall. Micropile 326 is battered in the downslope direction. After micropile installation, but before installation of the tieback anchor closest to this micropile (9/7/97 reading), axial load transfer in the upper portion of the micropile is minimal, while axial load transfer close to the assumed slide plane shows the pile to be in compression. This is the expected mode of axial load

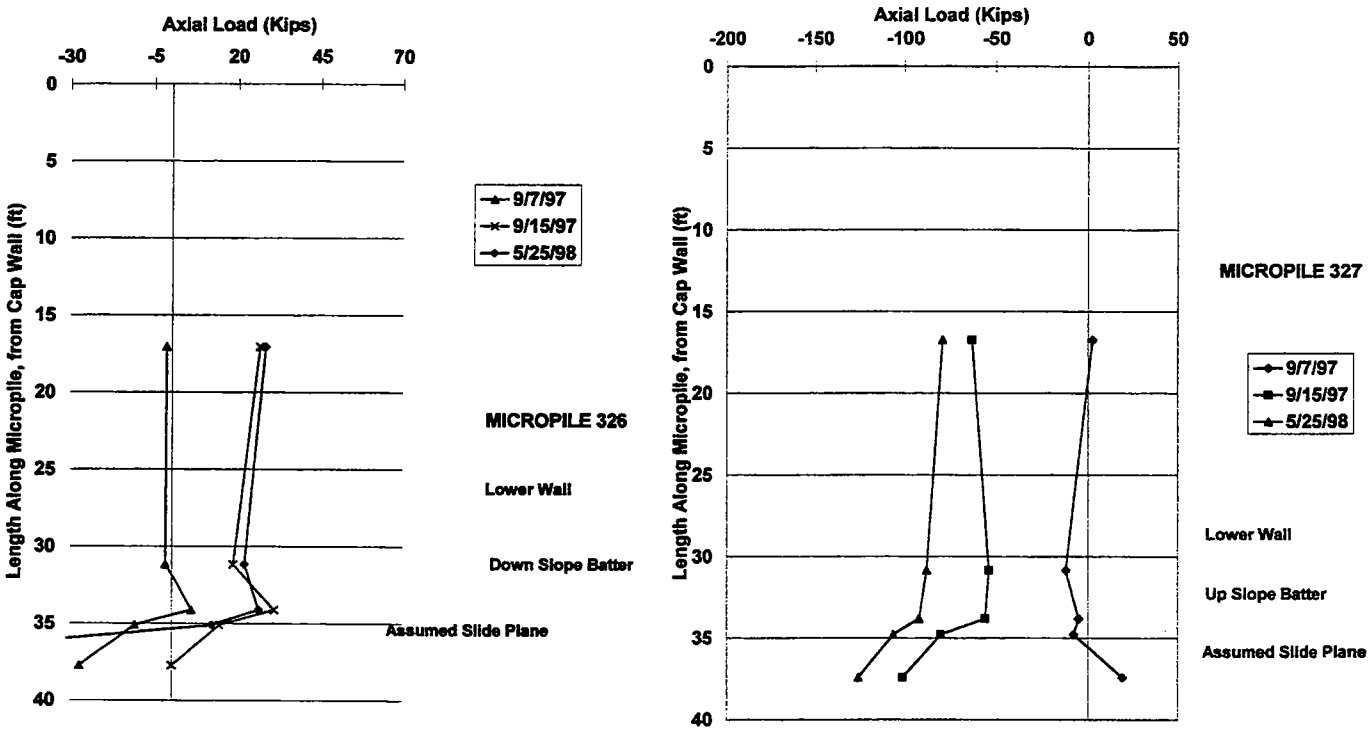

Figure 11. Typical axial load transfer for micropiles of lower wall.

transfer in piles with a downslope batter resisting landslide driving forces. However, when the tieback anchor closest to this micropile was installed and prestressed (9/15/97 reading), the pile went into tension. Application of prestress load to the anchor applies an eccentric load to the head of the micropile at the cap wall. Similarly, the adjacent micropile which is battered upslope (Micropile 327) goes into compression. Axial load in Micropile 326 (downslope batter) has not changed significantly since installation of the tieback anchors, while the magnitude of compression loading in Micropile 327 has increased slightly.

Discussion

Reinforced earth walls consisting of closely spaced battered micropiles and tieback anchors attached to cap beams were used successfully to control ground movements at the Blue Trail Landslide. Monitoring of wall and ground movements show that this a highly effective method for controlling ground deformations which had been causing roadway damage for thirty years.

Assumptions made in designing walls that utilize both micropiles and prestressed tieback anchors may lead to satisfactory performance, but field monitoring of axial load transfer

and bending in the micropiles shows that the direction of loading may be opposite of that assumed in design. This is a direct result of the construction sequence, in which micropiles are installed first, followed by installation and post-tensioning of tieback anchors. Micropiles are passive reinforcing elements requiring downslope ground movements to develop their resistance to further sliding. When tieback anchors are prestressed, the cap beam is loaded in the upslope direction, transferring axial load and bending moment to the micropiles in the direction opposite to that assumed in design, resulting in compression in upslope battered micropiles and tension in downslope battered micropiles. For the Blue Trail Landslide, prestress loads in tieback anchors of the lower wall were high enough (740 kN/m) to cause this load reversal in the micropiles, while prestress loads in the upper wall (205 kN/m) did not cause load reversal. These observations suggest that research is needed to determine the optimal balance between resistance to sliding provided by the micropiles and that provided by tieback anchors when both are used in one system. The most efficient and cost-effective design is one that utilizes both the micropiles and tieback anchors working in unison to resist sliding.

Acknowledgments

The authors wish to thank the Wyoming Department of Transportation for their support of the instrumentation and monitoring of the walls at Blue Trail Landslide. Mike Hager and Jim Dahill of the WYDOT Geology Program provided invaluable assistance to the first two authors in obtaining field measurements, providing reports and other data on the site, and sharing their knowledge and expertise. Brad Campbell of D'Appolonia Engineers provided valuable information about design of the INSERT walls, and John King of Seismic Imaging installed much of the instrumentation.

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Three Interesting Drilled Shaft Foundation Projects in Arizona

Robert E. Schock[1]

ABSTRACT

Drilled shafts provide excellent foundation support for bridge structures, especially those subject to extreme events such as flood and earthquake. This paper discusses three recent bridge projects in Arizona where designers had to deal with difficult subsurface conditions and removal of streambed soil by deep scour, or landslide. Each project highlights the inherent efficiency when designers can place large, single drilled shafts that transition into pier columns. Attention to details of the construction joint between columns and shafts, and the cooperative involvement of the drilled shaft installer are key elements to a successful installation. Each of the three projects involved extensive preconstruction dialogue between designers and contractors that resulted in changes to resolve constructability issues and save construction time.

INTRODUCTION

Drilled shaft foundations have been in use in Arizona since the early 1950's. However, it was not until the major Salt River Flood Years of 1978-80 that bridge designers began to consider large-diameter, deep drilled shafts for the new and replacement structures that had to withstand much greater peak design flows and riverbed scour much deeper than earlier predictions. Some of the early projects suffered from construction problems, largely due to design and construction inexperience. Later projects attracted national foundation specialists with heavy drilling equipment and experience in slurry-displacement construction. By the mid 1980's, the State's bridge-building effort was in full swing and the drilled shaft program was an unqualified success, culminating in the mile-long Salt River bridge in the early1990's.

CASE STUDIES

1. Kitt Peak Observatory

The Kitt Peak Observatory is located in the Quinlan Mountains, 38 miles west of Tucson, Arizona. The observatory houses the world's largest collection of optical telescopes, and twenty-two universities perform research there. State Route 386 is the only road access to the top of the peak. At the 5,300 ft elevation, the road passes over a filled ravine. Over a nine-year period, the embankment had exhibited continued signs of instability and gradual slope failure. During this time, two attempts by Arizona Department of Transportation (ADOT) to restore and stabilize the slope had been unsuccessful.

To ensure a permanent solution and maintain uninterrupted access to this critical facility, ADOT elected to construct a 2-span bridge over the ravine. Because the road is perched on the side of

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the mountain, there was no space for conventional bridge construction. Instead, the general contractor constructed a one-lane bypass detour against the mountainside, then built the bridge from roadway grades by top/down methods.

Bridge Design

The geotechnical investigation revealed that the roadway fill and overburden were potentially unstable, but good quality rock was present at approximately 40 feet below the surface – too deep for spread footings.

The 170-foot long bridge was designed as a 35-foot wide, precast concrete girder structure, spanning the ravine by means of a single, central pier. The pier is supported on a seven-foot diameter drilled shaft, installed into granite bedrock at a depth of 40 feet through a variable soil matrix of natural talus and fill that included boulders and large rock blocks, placed in the past to support the embankment.

Drilled Shaft Construction

In order to penetrate the roadway fill and advance casings through the boulders and rock blocks, the drilled shaft contractor mobilized a large, crane-mounted, twin engine drill rig to the mountainside, as shown in Figure 1 below.

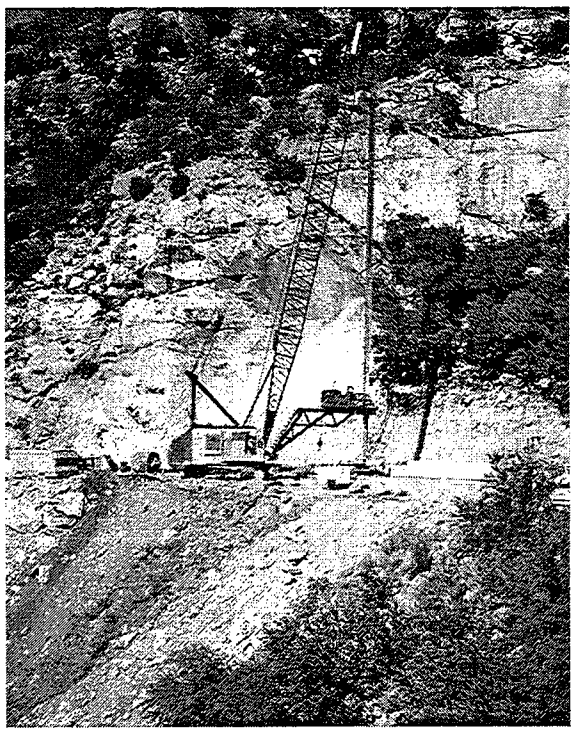

Figure 1. Crane Mounted Drill Rig On Site

Slope stability analyses had verified that the roadway could safely support the 200-ton machine. As drilling progressed, temporary steel casing in three telescoping stages was set and

advanced through the boulder fill to the top of bedrock (Figure 2). A seven-foot diameter rock socket was then cored 25 feet deep into the solid Kitt granite (Figure 3). Following completion of the rock socket, a smooth steel casing was installed from the top of the socket up to grade (Figure 4).

Figure 2. Open Shaft Showing Telescoping Casing

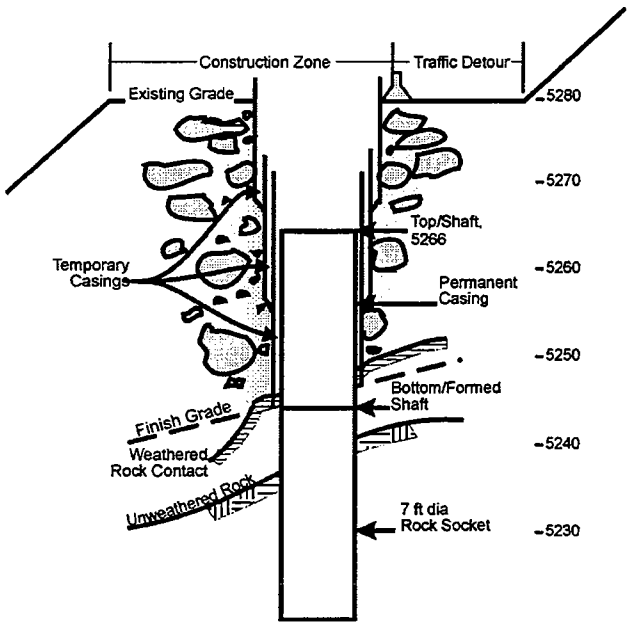

Figure 3. Cross Section of Shaft Installation

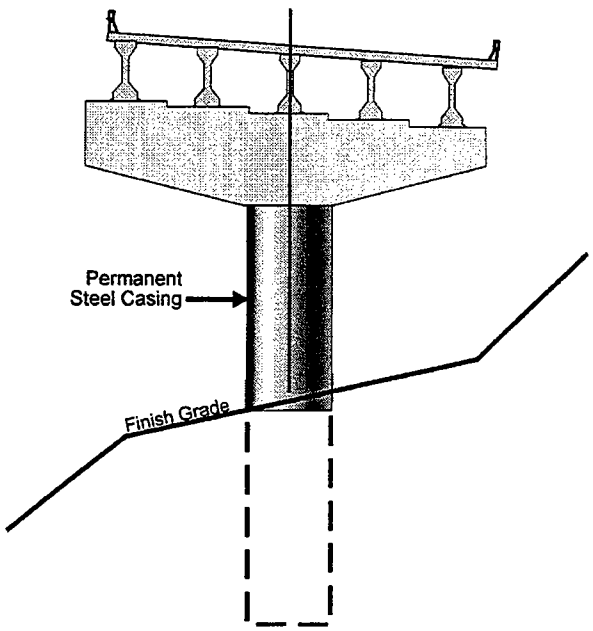

Figure 4. Cross Section of Completed Pier

A full length reinforcing cage was trucked to the site and placed in the shaft. Then, 85 cubic yards of concrete was placed by free-fall to complete the drilled shaft foundation. Figure 5 shows the pier cap being formed on the completed shaft.

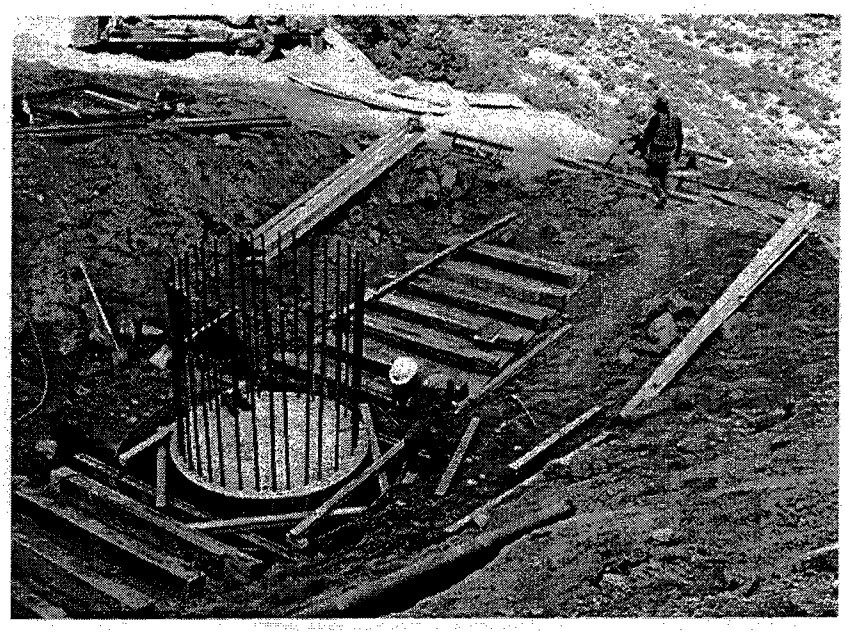

Figure 5. Construction of Hammerhead Pier Cap on Completed Shaft

With the supporting pier completed, the general contractor then built the bridge on grade and excavated the roadway fill out from underneath the bridge. The smooth-cased portion of the drilled shaft now forms the single column under the completed bridge (Figure 6).

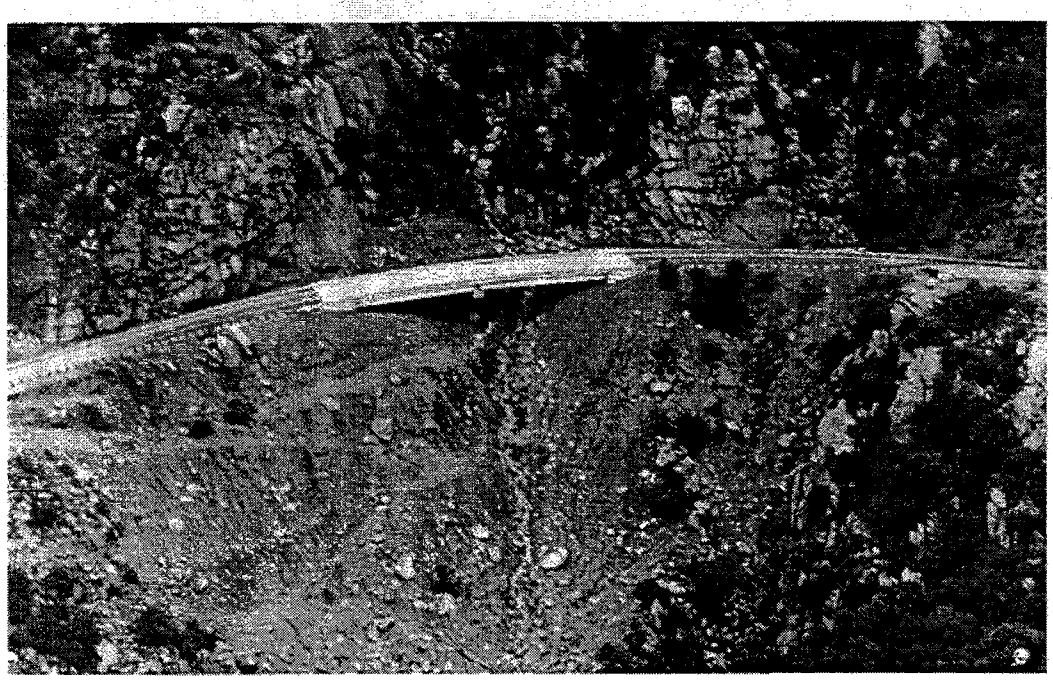

Figure 6. Completed Bridge Spanning Excavated Ravine

2. Verde River Bridge

The Verde River Bridge is located in Dead Horse Ranch State Park, near Cottonwood, Arizona. Constructed by ADOT in 1993, the 4-span, 550-foot long bridge was designed with a precast AASHTO girder superstructure supported by rectangular columns atop drilled shaft foundations.

Drilled shaft Design

The subsurface profile consisted of 20 feet of sand, gravel, cobble and boulder riverbed alluvium overlying claystone, mudstone and marl of the Verde formation. Design scour analysis for the bridge is based on a 500-year event of 106,000 cfs flow, with scour to 19 feet below riverbed.

A very efficient foundation design was possible due to relatively high capacity in the bedrock; the entire bridge was supported on seven drilled shaft foundations, as follows:

- A single, 11-foot diameter shaft for each of the three piers
- Twin, six-foot diameter shafts under each abutment

Figure 7 shows the foundation design for pier support.

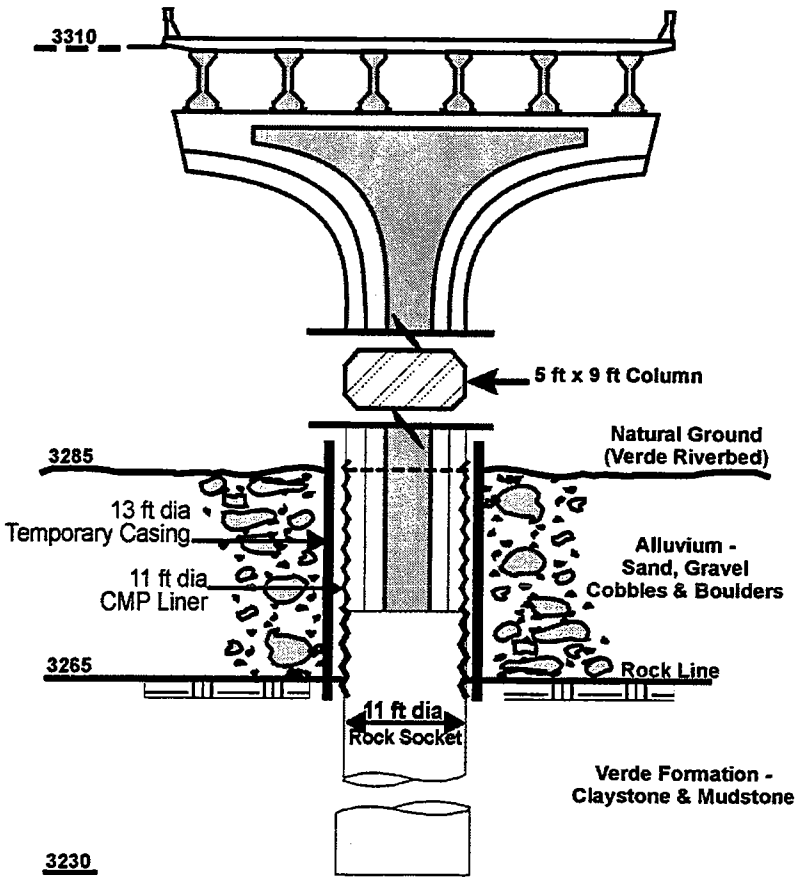

Figure 7. Typical Pier Section

Bridge Construction

The shafts were installed in a 4-week schedule, using a large, twin engine crane attachment which was able to seal temporary casings into the rock and excavate the rock sockets by augering in the dry (Figure 8). The full-diameter, 11-foot cut in the pier sockets required coring.

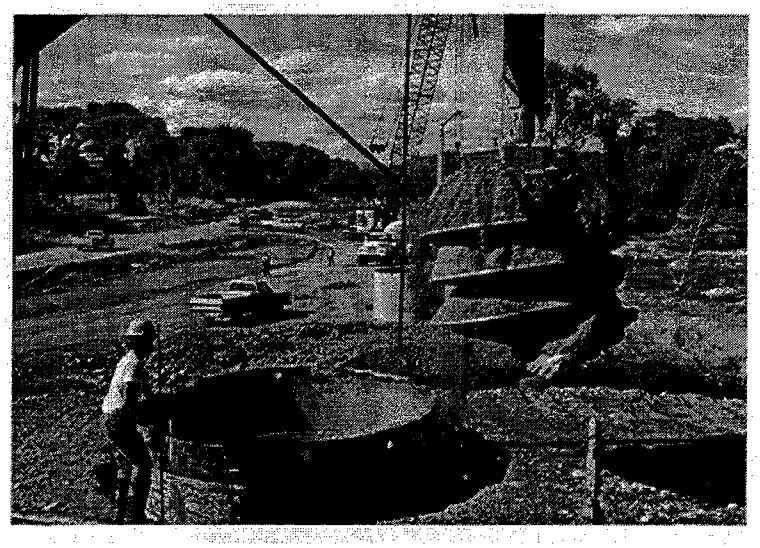

Figure 8. Augering Six-foot Abutment Shaft in Verde Formation Claystone

CMP liners were installed at each pier (Figure 9) to allow construction of the transition from shaft to rectangular column, 10 to 15 feet below river level. Concrete was placed in dry shafts by free-fall and in wet shafts by pumping (Figure 10)

Figure 9. Pier shaft Ready For Concreteing With 13-ft Temporary Casing and 11-foot CMP permanent Casing in Place

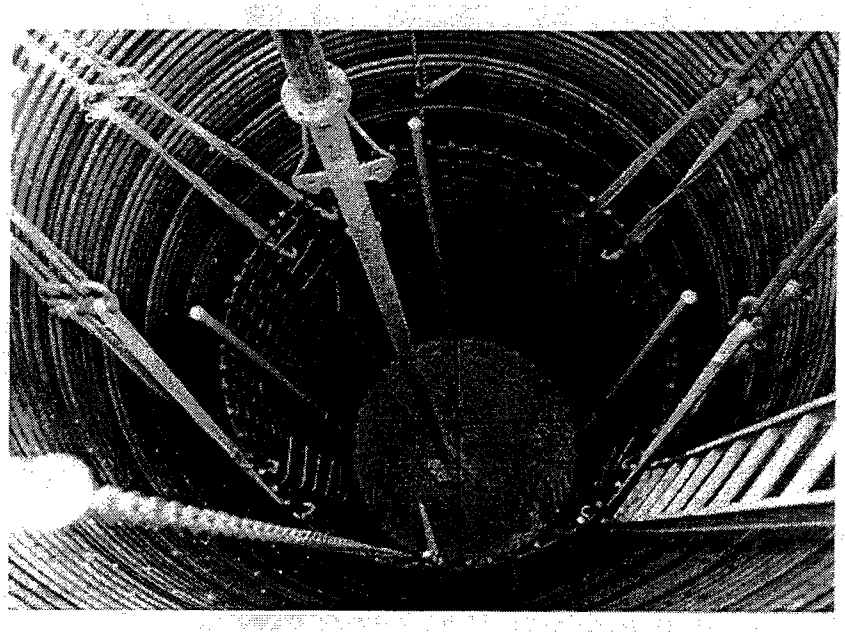

Figure 10. Concreting 11 Foot diameter Pier Shaft

After the drilled shaft was concreted, the temporary casing was extracted, leaving the 11-foot CMP liner in place as a temporary cofferdam. Column reinforcing was then placed and anchored with a 5-foot deep transition cap pour. The hammerhead pier was then formed and poured. After stripping, the CMP liner was cut and removed and the pier backfilled.

3. Salt River Bridge

The Salt River watershed drains 13,000 square miles of central Arizona. Between 1911 and 1939, six water storage dams were constructed upstream (east) of Phoenix to provide municipal and irrigation water to the valley. Since the dams were constructed, the Salt River has been dry except for short-term flows during the winter and spring rainy season.

During winter storms in the years 1978-80, storage capacities of the dams were exceeded and large releases causes 14 bridge failures and much destruction downstream. Since that time, most of the replacement and new bridge structures have been designed with deep drilled shaft foundations extending below predicted scour elevations to obtain flood proof support for the structure. Designers must typically consider peak flows of 250,000 cubic feet per second and up to 60 ft of scour.

Bridge Alignment and Design

Arizona Department of Transportation began planing the Red Mountain Freeway in 1985, when Phoenix area voters passed a sales tax referendum to finance a local freeway system over a 20-year program. The freeway begins near central Phoenix and extends east into the communities of Scottsdale, Tempe and Mesa. For several miles, it runs along the north bank of the Salt River until it approaches the Pima-Maricopa Indian Reservation, where a river crossing was planned.

In addition to providing flood-proof support, ADOT faced two additional problems in selecting the alignment for the new bridges to carry the freeway over the Salt River:

- The north bank of the river was on reservation land, which cannot be acquired by the process of eminent domain.

- The south bank of the river, with several old landfills containing potentially hazardous materials, is part of the EPA-designated Indian Bend Wash Superfund site.

ADOT's solution was 5,500-ft long twin bridges running in the Salt River channel for almost a mile, before angling in and out of the channel at either end. Bridge design called for AASHTO type 6 modified precast concrete girders on 140-ft spans.

Load Test Program

There was another problem with the site that lay beneath the surface and was not subject to negotiation with Government or landowners. At a depth of about 90 feet, a thick clay layer was found instead of the dense sand, gravel and cobble (SGC) river alluvium which usually extends hundreds of feet below the Salt River bed. With design scour levels of 50 to 60 feet, the shafts would derive much of their support in this clay layer, which proved difficult for the geotechnical engineers to characterize in terms of the strength and consistency of the deposit.

To provide better information, a load test program was conducted in summer, 1990 that included two tests to failure on 36-inch shafts in skin friction within the clay. In order to isolate the load test shaft from the upper SGC layer, a double telescoping casing system was used. An outer 60-inch casing, 80 feet long was sealed into the clay, with a 42-inch inside forming the load test shaft up to grade (Figure 11). Below the 42-inch casing, a 36-inch shaft extended 40 feet into the clay.

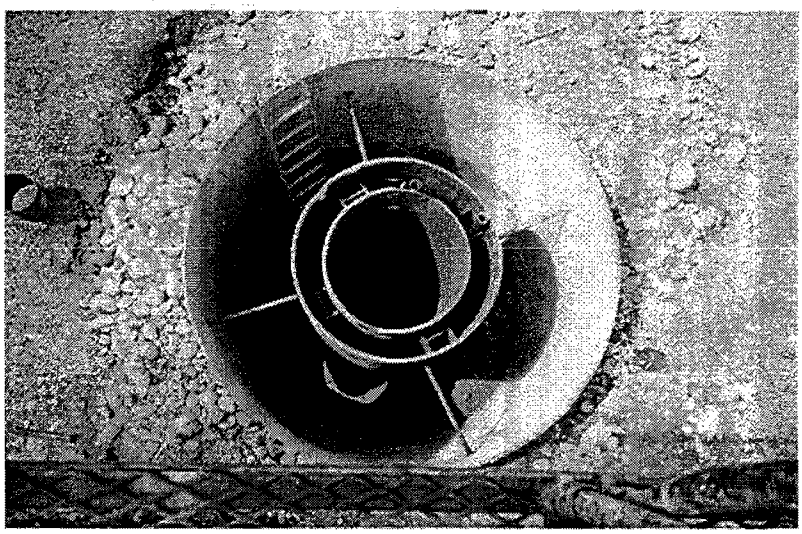

Figure 11. Double Telescoping Casing

In order to eliminate end bearing, the shaft was overdrilled and a 5-ft layer of sand was placed under slurry at the shaft bottom. After the test shaft was concreted, the sand was jetted out by high pressure air and water through tubing that was installed within the reinforcing cage (Figure 12). The resulting void space at the shaft tip allowed testing of side resistance only.

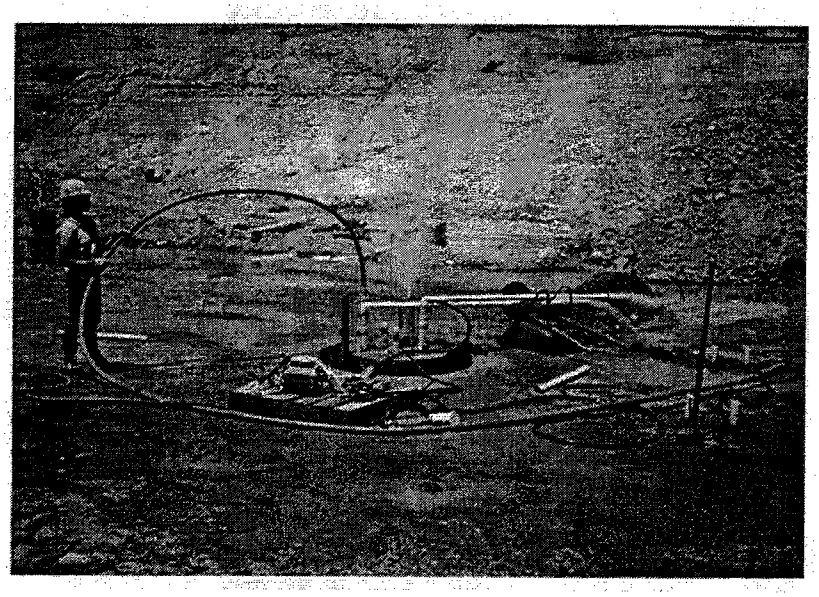

Figure 12. Jetting Out of Sand with High Pressure Air and Water

The resulting test values were significantly above previous estimates based on laboratory results, leading to a considerable foundation cost savings. The final design shafts were shortened from an average of 175 feet to aboout 130 feet, saving an estimated $4.5M in construction costs. Total cost of the load test program was about $750,000.

Bridge Construction

Foundations for the bridge are single, drilled shafts supporting each column. Columns are arranged in groups of four, one pair under each twin bridge. Shafts are 9 and 10 ft diameter at the piers, and 6 ft diameter shafts support both abutments. Shaft depths ranged between 100 and 156 feet (Figure 13).

The shafts were cased through the surface layers of the caving sand, gravel and cobble (SGC) riverbed deposit to depths of up to 60 ft. Groundwater was encountered within 5 ft below the riverbed in the channel, meaning that the shafts were installed entirely in the wet. Installing large diameter shafts in the Salt River required the largest available drilling equipment to penetrate the SGC deposit and advance the oversize casing. Excavation was generally by heavy duty augers in the extremely abrasive SGC, and enclosed drilling buckets were also used.

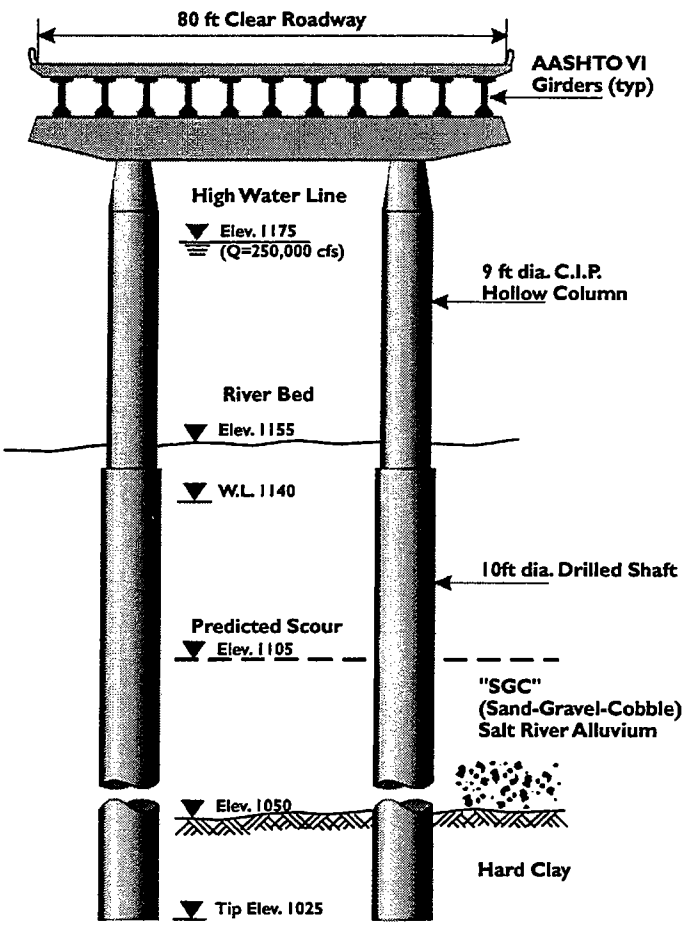

Figure 13. Cross Section Showing Drilled Shaft Foundation

Reinforcing cages were upended and placed as single units, up to 164 ft long and weighing up to 30 tons, as shown in Figure 14. Each shaft has a permanent CMP steel liner placed inside the oversized casing. Concrete was placed through a gravity flow tremie pipe. Concrete pours ranged to nearly 500 cubic yards for the largest shafts. A total of over 19,000 lineal feet of both 9 and 10 ft diameter drilled shaft and 70,000 cubic yards of concrete was installed during the 9 month construction schedule. Figure 15 shows a pair of shafts nearing completion.

Quality Assurance

Each shaft was measured for verticality by slope indicators prior to placement of concrete. Integrity testing was performed by the ADOT, utilizing the Gamma Ray backscatter method. Eight test conduits were cast into each shaft by tying 2.5 inch PVC pipes to the reinforcing cage. The gamma ray probe was the passed through each tube, top to bottom, and concrete density readings were plotted for each shaft. Tests showed continuous, sound concrete in all shafts. The completed superstructure is shown in Figure 16.

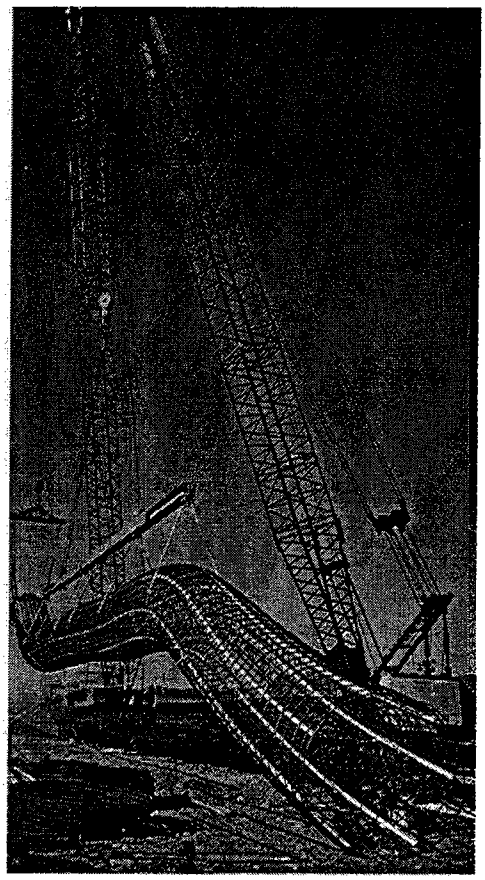

Figure 14. Reinforcing Cage Being Upended For Placement

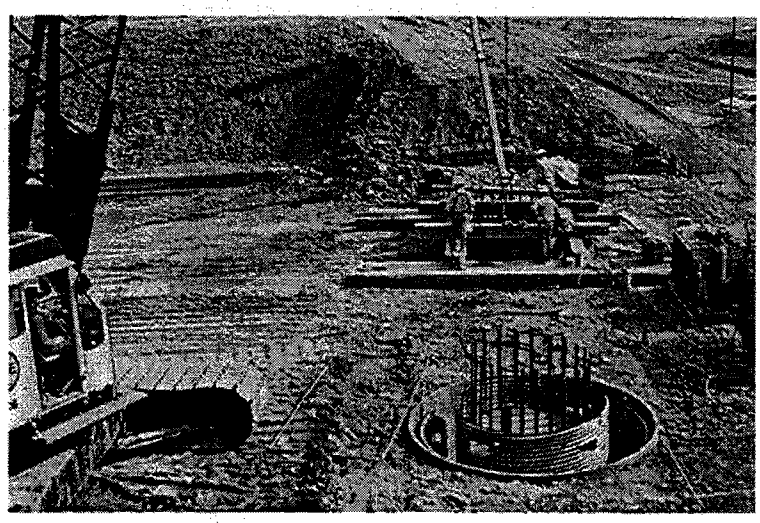

Figure 15. Completed shaft in Foreground. Preparing to Pour Adjacent Shaft in Background

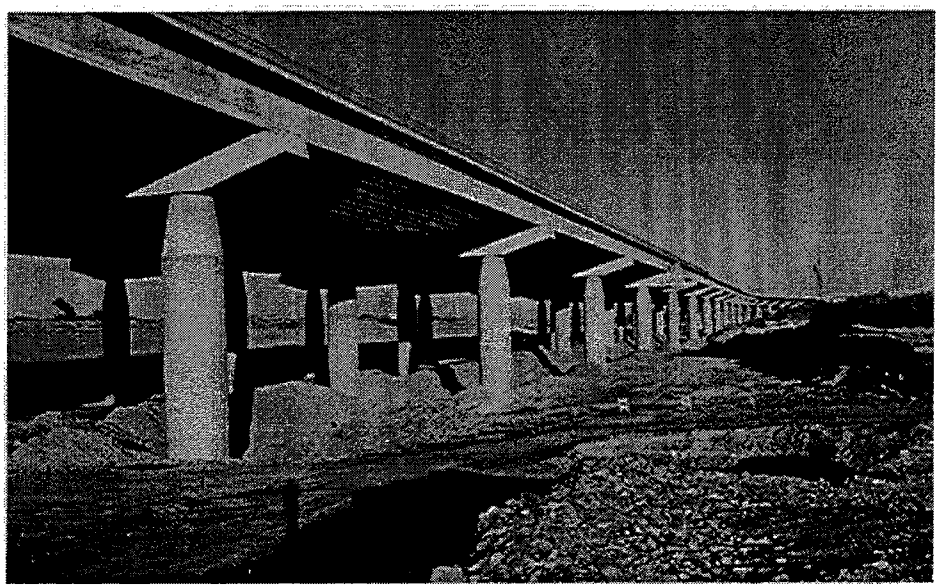

Figure 16. Completed Bridge Superstructure

SUMMARY

These projects are a showcase for the ability of drilled shafts to provide efficient, cost-effective foundations in hostile bridge environments. Over the last 15 or so years, design confidence has grown to the point that drilled shafts have become ADOT's standard design for most of their bridge projects, including sites with good subsurface conditions.

Key to this success has been the open, communicative attitude on the part of ADOT and its design consultants, which has sparked continuous discussion and improvements in design details, specifications, and construction quality in this specialty line of work.

ACKNOWLEDGEMENTS

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	Geotechnical Engineers	Huntington Engineering
		Agra Earth & Environmental

Statnamic and Static Lateral Load Analysis of a Battered Pile Group vs. a Drilled Shaft Group

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Abstract

The Mississippi Department of Transportation conducted a load test project in the East Pascagoula River for the design of a foundation for a high rise bridge on U.S. Highway 90 in Pascagoula, Mississippi. The load test project tested the axial capacity of 48" and 84" drilled shafts, 24" and 30" pre-stressed concrete piles, and 36", 54", and 66" concrete cylinder piles. In addition, at the water line caps were constructed on two 84" drilled shafts and on six - 30" pre-stressed concrete piles. These caps were load tested laterally by jacking the two caps apart and by applying dynamic loads using the Statnamic device. The pile cap consisted of a 2 X 3 group of 30" pre-stressed concrete piles with the 4 corner piles on a 3:12 batter. The static and Statnamic lateral load testing of the two caps were completed in order to determine the effect of static and dynamic loading on structures. The Statnamic lateral load test was designed to imitate barge impact loading. Static lateral loads were incrementally applied to the two caps until a load of 440 tons was obtained. The testing was halted at that point to prevent further damage to the piles and the shafts. Statnamic lateral loads were applied to each cap in the weak and strong directions. The loads were gradually increased with each test. A Statnamic lateral load of 650 tons was applied to the pile cap in the strong direction of the cap. A world record lateral load with a Statnamic device was achieved by applying load of 825 tons to the drilled shaft cap.

Introduction

The Mississippi Department of Transportation needed to replace a 45 year old Basque span on U.S. Highway 90 over the East Pascagoula River. The existing structure was in need of repairs and was prone to hang up and to cause traffic delays of up to 10 hours. The existing bridge also served to bottle neck traffic from 6 lanes to 4 lanes. The bridge was located at the entrance for the Pascagoula Navy Home Port and Ingles Shipyard where approximately 10,000 people work. An eight lane, 90 feet high bridge was needed to replace the existing bridge. The 3,579 feet long new bridge was going to be constructed using a 5% grade in order to get up and down within the confines the city and the entrance to Ingles.

The East Pascagoula River is located in the extreme southeast corner of Mississippi, just a few miles upriver from the Gulf of Mexico. The drainage area of the Pascagoula River includes most of the southeastern 1/4 of the state and encompasses approximately 9,500 square miles. This drainage is split between the East and West Pascagoula Rivers as it enters the brackish marshes of the coastal region. Approximately 32 percent of the flow (137,197 cfs) enters the East Pascagoula River.

The geotechnical investigation into the site indicated that the river channel contained soft, organic rich silts and clays, complexly interbedded with some loose sands. Below an elevation of -60 feet, non formational coastal deposits of dense sands interbedded with firm sandy, silty clays were encountered to an elevation of -130 feet, the depth of the deepest boring. These soil layers were laterally extensive across the entire site.

The consulting engineering firm working on the design of the bridge initially recommended utilizing 259 steel "H" piles to support the footing that would be constructed inside of a coffer dam. The bottom of the seal concrete was to be at elevation of -95 feet. This put the footing 20 feet below the calculated scour (-45 feet elevation) for the bridge as per FHWA guidelines. At this point, an alternate foundation design was sought. In consideration of the 18 feet water depth and the 90 feet high pier, a 'pile cap' at the water line was considered to be the best alternate.

Preliminary design loads and moments were high for the main channel piers. The design load or 'p-load' was in excess of 6,500 tons for each of the main piers. Ship impact loads were in excess of 1060 tons. Numerous foundation types were investigated to determine what foundation type would best support the bridge. Using the conventional MDOT design methods, drilled shafts, pre-stressed concrete piles, steel "H" piles, concrete cylinder (Raymond) piles could not be utilized to support the structure with a safety factor of more than 1. The calculated scour was at an elevation of -45 feet, which resulted in moments that could not be designed for using conventional design methods.

The MDOT approved a plan to conduct a load test program that tested the axial capacity of 48" and 84" drilled shafts, 24" and 30" pre-stressed concrete piles, and 36", 54", and 66" concrete cylinder piles. In addition, at the water line caps were to be constructed on two 84" drilled shafts and on six - 30" pre-stressed concrete piles. These caps were to be load tested laterally by jacking the two caps apart and by applying dynamic loads using the Statnamic device.

The plan was designed by Dr. David Crapps, P.E. of Schmertmann & Crapps, Inc. in Gainesville, Florida in conjunction with Dr. Dan Brown, of Auburn University. The contractor for the Project was Mr. Bob Hodges of Warren Paving, Inc. of Hattiesburg, MS. Dr. Crapps administered the contract between the MDOT and Warren Paving, Inc..

It was the intent of the project to accurately determine the bearing capacities of the different foundation types as well as to test the static and dynamic lateral load capacities of the two caps. The primary objective of the static lateral load test was to generate soil design parameters for the shafts and the piles, and to compare the response of the static lateral load test to the dynamic lateral load tests. The Statnamic lateral load tests were to imitate barge impact conditions on both foundation types. It was also the intent of the project to design the two structures to respond equally during the lateral load test program. The structures were designed using the Florida-Pier Program (Department of Civil Engineering, University of Florida, (1997)). In addition to using the data for the design of the bridge foundation, the data from the project was to be used by the University of Florida to modify and improve the Florida-Pier Program. The University of Florida, in turn and in conjunction with the project consultant, supplied additional soils information and in situ testing.

Geology

A total of 21 borings were completed by the MDOT as part of the initial geotechnical investigation for the design of the new bridge. Six of these borings were completed in the water along the centerline of the proposed alignment for the bridge. The borings were terminated at elevations of -120 to -130 feet. The water depth varied from 9 feet to in excess of 22 feet across the channel; however, it was deeper in the main channel. Between the mud line and an elevation of -60 feet, approximately 40 feet of complexly interbedded, sands, silts, and clays were encountered. These alluvial soils contained very soft to stiff clays and silty clays which contained with many logs and other organic debris. Several layers of organic silt were also encountered. The sands intermixed with the silts and clays were loose to medium dense and often contained clay layers intermixed with clayey sand. These complexly interbedded soils represent an array of depositional environments associated with the formation of the Pascagoula River estuary. Some of the more massively bedded sand layers in this upper alluvial layer represents deposition in the main channel as part of the bed load of the Pascagoula River. This is evidenced by the presence of gravel at the base of the sand units. The abundance of wood encountered in the borings are believed to have originated from a time when logs were floated down river to saw mills on the Pascagoula River.

Beneath this sequence of alluvial deposits, a thick bedded deposit of very dense, very fine to coarse sand was encountered. This layer was underlain by a clayey, silty sand. The borings were terminated in a very dense, slightly silty, very fine to medium sand. These three layers were encountered between elevations -60 and -130, where the borings were terminated. Interbedded with these three layers were some thinner, discontinuous layers of organic, clayey silt and organic sands. This thick sequence of soils were laterally extensive across the site. They appear to be part of a thick coastal deposit of Holocene Age. These coastal deposits cover a narrow band that extends across the coastal counties and it includes the barrier islands. These deposits also extend for a considerable distance up the flood plains of the Pearl and Pascagoula Rivers. For the load test program, the drilled shafts and the piles were tipped in the bottom sand layer.

Test Site

The proposed high rise bridge will be located approximately 200 feet north of the existing bridge on U.S. Highway 90 in Pascagoula Mississippi (Figure 1). The location of the test site was a 100 by 150 feet site that was located approximately 150 north and east of the proposed east main river pier (Figure 2). Additional, detailed, soil borings were completed by the consultant at pre-determined locations that correlated with the locations of the foundations that were being tested. SPT, Dilatometer, and Pressuremeter tests were also completed at specified locations by the University of Florida. This additional soils information was obtained in order to verify the soil types at the test sites and to aid with the interpretation of the load test data. Following the completion of the soil testing, shaft and pile lengths were set with a tip elevation of -107 feet for all foundation types tested on the project.

Drilled Shaft Group

An 84" drilled shaft was constructed at the site without an Osterberg load cell. A second 84" drilled shaft was constructed to the west of the first shaft (Figure 2) and at 3 diameters spacing (21 feet on center). This shaft contained one Osterberg load cell at the base of the shaft and one Osterberg load cell located near the middle of primary bearing strata (the Coastal deposits).

Both drilled shafts were fully instrumented with strain gauges that were monitored during the load testing. Slope Indicator pipes were attached to the rebar cages of the drilled shafts. These pipes were later used to monitor the movement of the drilled shafts during the lateral load tests.

The Osterberg load cells were loaded individually beginning with the bottom cell. The shaft was loaded to 991 tons where it failed in shear. The net shear was 789 tons. The shaft did not fail in end bearing. The upper cell was tested second while the bottom load cell was kept open (without pressure). The upper section of the shaft failed in shear at a load of 408 tons. The bottom Osterberg cell was then allowed to close and the upper cell was tested again in order to move the soil between the two cells. These staged tests, in addition to the strain gauges, allowed accurate measurements of the shear forces to be obtained for the different soil layers.

A 30.5 by 10 by 8 feet tall concrete cap was constructed on top of the two 84" drilled shafts. All of the wiring for the instrumentation, the slope indicator pipes, and the hydraulic hose for the Osterberg load cell were extended through the cap. The hydraulic hoses were later extended through the drilled shaft cap in order for the drilled shafts to be tested again after the cap was constructed and the static lateral load tests was completed.

30" Pre-stressed Concrete Pile Group

Six 30" voided pre-stressed concrete piles fully instrumented with strain gages were driven in a 2 X 3 group at the site. The piles were driven open ended with a vent tube near the top. All of the piles were jetted to an elevation of -70 feet. The two piles in the middle of each row were driven as vertical piles. The four corner piles were driven on

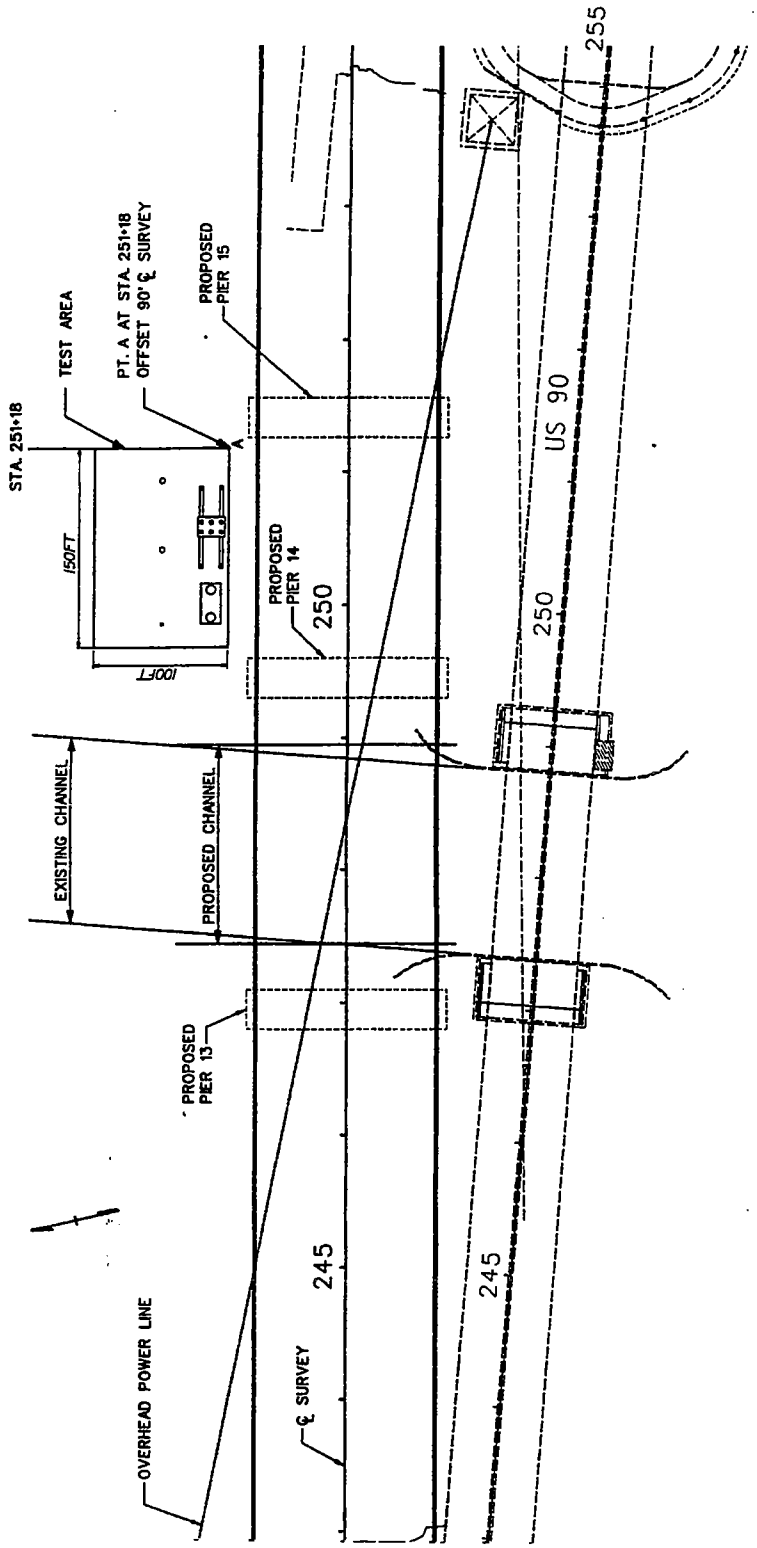

Figure 1 Location of the load test project for U.S. Hwy. 90 over the Pascagoula River, Pascagoula, Mississippi.

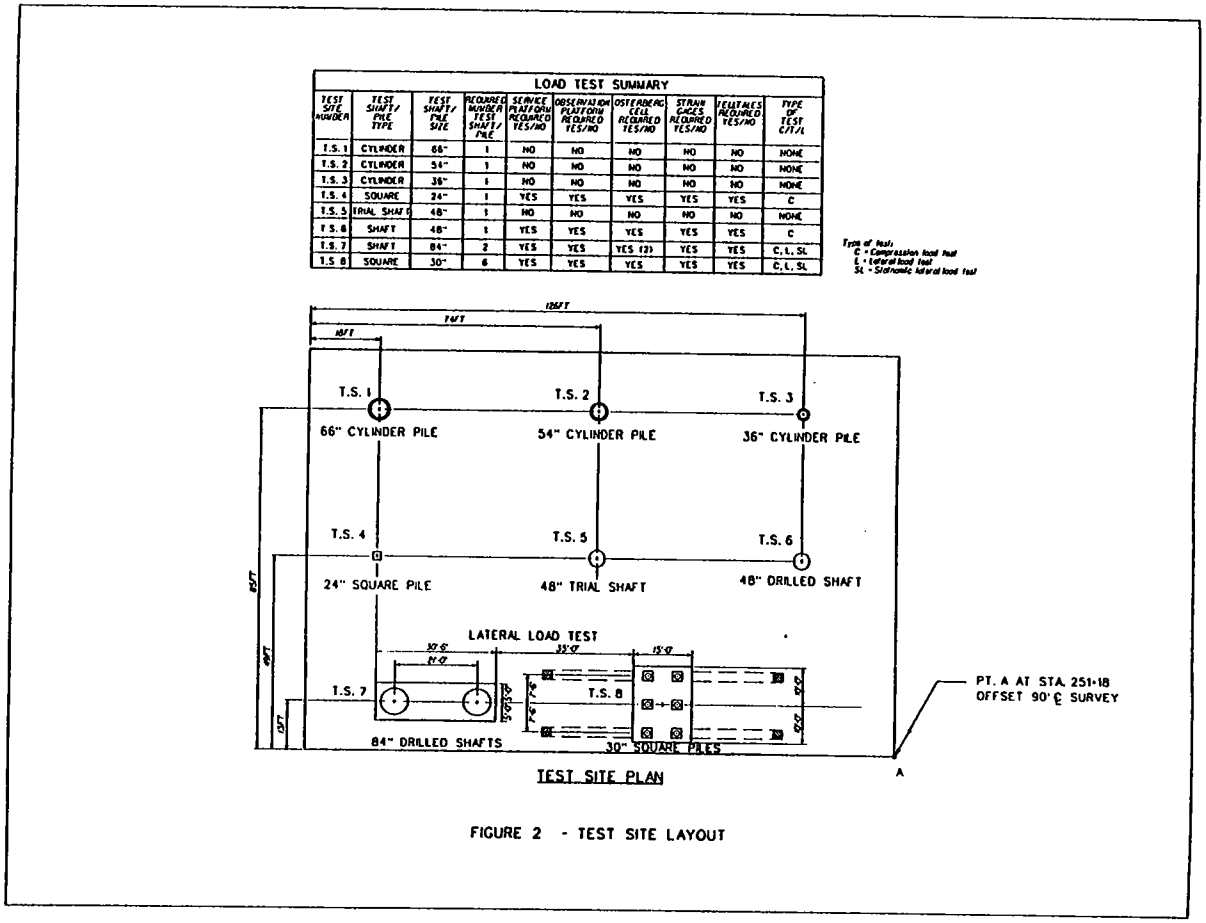

FIGURE 2 - TEST SITE LAYOUT

Figure 2 Layout of the Pascagoula River load test project.

a 3:12 batter. This formed a trestle bent with the two middle piles being vertical (Figure 2). The two vertical piles were driven first. One of the vertical piles contained an Osterberg load cell at the bottom of the pile. After the two vertical piles were driven, the pile with the Osterberg load cell was tested to a load of 324 tons when the pile failed in shear. After the load test was completed, the remaining piles were driven on a 3:12 batter.

Slope Indicator pipes were inserted into the vertical piles to an elevation of -80 feet and the voids in the piles were grouted. These pipes were later used to monitor the movement of the piles during the lateral load tests. The corrugated metal pipe liner in the battered piles were plugged at a depth of 25 feet. A rebar cage was inserted into the voids of all the piles. The rebar cages extended above the tops of the piles in order to make a true moment connection between the piles and the cap that was to be constructed on the piles. A 15 by 20 by 8 feet tall cap was constructed on the pile group. All of the wiring for the instrumentation, the slope indicator pipes, and the hydraulic hose for the Osterberg load cell were extended through the cap. The hydraulic hoses were later extended through the pile cap in order for the load cell to be tested again after the cap was constructed and the lateral load tests were completed.

Static Lateral Load Test

After the drilled shaft and pile caps were constructed, a support frame for the static lateral load test was constructed between the two caps. The support frame rested on roller bearings on the top of the two caps. A 600 ton hydraulic jack with a 24" throw was held in place by the support frame and installed against a hemispherical bearing plate located at the center on the pile cap. A 30 inch diameter concrete-filled steel pipe was used to transfer the load from the jack to a hemispherical bearing plate at the center of the end of the drilled shaft cap. This set up allowed the two caps to be jacked apart in their strong direction.

One set of Slope Indicator probes connected in series were installed in each of the Slope Indicator pipes closest to the support frame (on the trailing edge of the caps) on both the drilled shafts and in the pile cap. These probes were continually monitored during the lateral load tests. The Slope Indicator pipes on the leading edge of the caps were read after each load was applied by using conventional Slope Indicator probes.

Reference frames installed in front of the leading edges of the caps were utilized to monitor translation, rotation, and twisting (about a vertical axis) of the caps during each of the lateral load tests. All of the 106 strain gauges in the drilled shafts and in the piles were continuously recorded during each of the lateral load tests.

The static lateral load test was taken to a load of 440 tons using 40 ton increments. The movement of the caps and the strain levels in the piles and shafts were monitored continuously during the test to insure that the piles and shafts were not excessively damaged and that sufficient data was obtained to determine static lateral load capacities. The static lateral load test was terminated at a lateral load of 440 tons when the deflection of the drilled shaft cap reached approximately 2". At that point the pile cap had moved approximately 1.5". It was at this point that the strain gauges indicated that the piles and the shafts were cracking near the connection with the caps.

Displacement of the two caps was primarily translational. There was some rotational movement of the caps. The leading edge of the pile cap was forced up approximately 0.1" (Figure 3). The leading edge of the shaft cap was forced down approximately 0.3" (Figure 4). These opposite directions of rotation (shown on Figures 3 and 4 as positive and negative movement) can be contributed to the battered piles in the pile cap. The data clearly showed that there was very little rotational movement of the two caps about the horizontal axes. There was also very little twisting about the vertical axes. This indicates that the lateral load that applied to the two caps was correctly centered on both caps.

During the static lateral load test, the drilled shaft cap proved to be more rigid under low lateral loads. At 160 tons, the drilled shaft cap had moved only 0.275 inches as compared with the pile cap which had moved .350 inches (Figures 3 and 4). As loads increased to 440 tons, the displacement of the shaft cap outpaced the pile cap. This is illustrated in a comparison of Figures 3 and 4. Total displacement of the drilled shaft cap was 1.9 inches and the

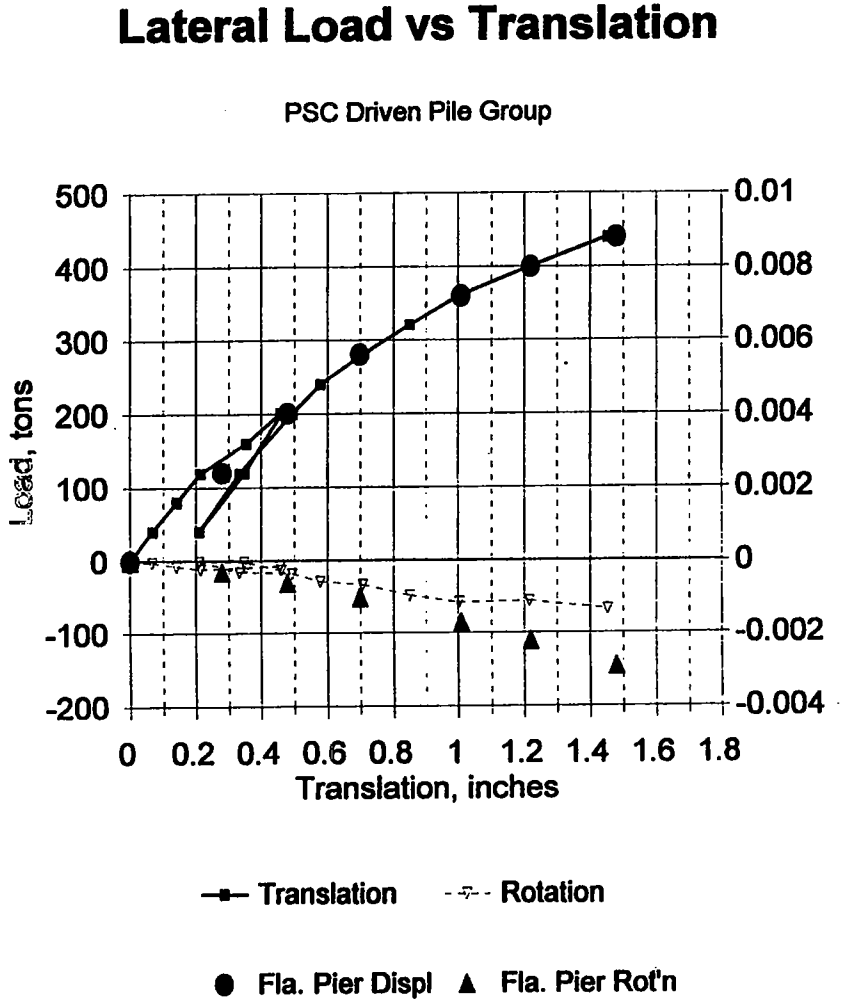

Figure 3 Comparison of predictions of lateral load vs. translation and rotation from Florida-Pier to measured values during the static lateral load test.

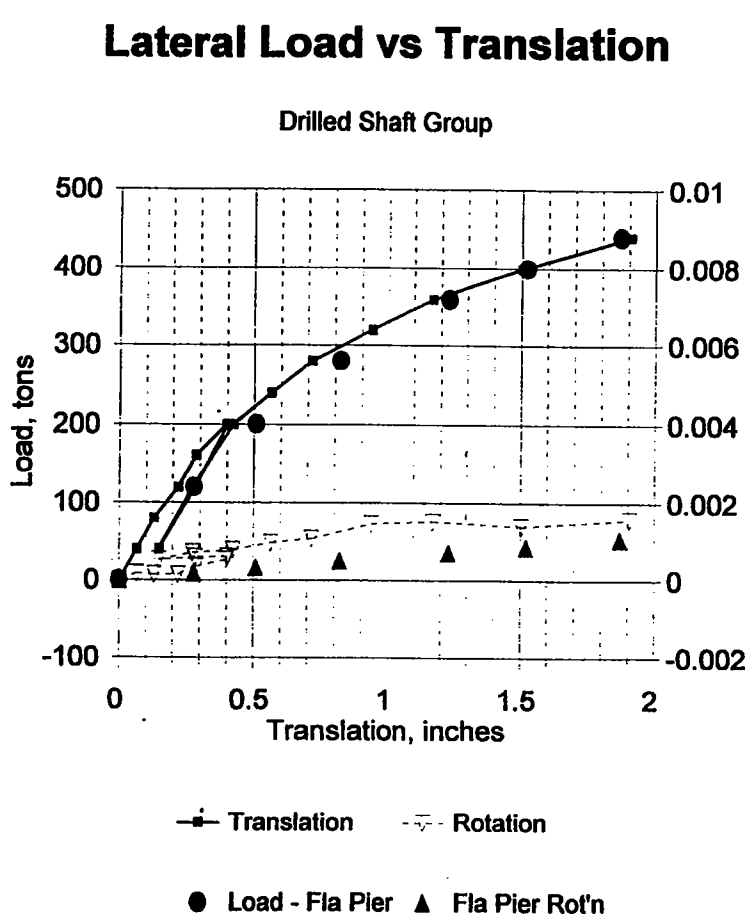

Figure 4 Comparison of predictions of lateral load vs. translation and rotation from Flordia-Pier to measured values during the static lateral load test.

pile cap was 1.45 inches at a lateral load of 440 tons. Figures 3 and 4 clearly illustrate how closely the Florida-Pier Program predicted the translational and rotational movements of the two caps.

The strain gauge data from the static lateral load test was converted into axial forces for each of the piles in the group. The data clearly illustrates in Figure 5 that the battered piles on the leading edge of the pile cap were in compression and that the vertical pile on the leading edge was in tension during the static lateral load test. On the trailing edge of the pile cap the reverse was true. The battered piles were in tension and the vertical pile was in compression. As strange as this may seem to be, it was this reaction by the piles that kept the pile cap from rotating during the test. This reaction was predicted by the Florida-Pier Program. The axial forces predicted by the program were only slightly lower than the measured values (Figure 6).

The strain gauges and the Slope Indicator data indicated that the point of fixity for the piles was at an elevation of -45 feet. This is 30 feet below the mud line and it is at the top of the first dense sand layer in the upper alluvial soils. The point of fixity was the same for the drilled shafts.

After the static lateral load test reached 440 tons, the load was reduced to 400 tons. At this point the Osterberg load cell in the drilled shaft was reloaded in an effort to fail the drilled shaft in end bearing. It was thought that the additional 300 ton weight of the cap, the moment connection between the two shafts and the cap, and the 400 tons of lateral load would provide the needed resistance to fail the shaft in end bearing. It did. The 84" drilled shaft was failed in end bearing at a load of 1,300 tons. During this Osterberg load test, the drilled shaft cap was pushed up about 0.25 inches, and an additional 25 tons of lateral load was transferred to the pile cap.

Statnamic Lateral Load Tests

Axial Statnamic load tests have become an accepted method of determining axial loading on piles; however, using the Stanamic device to conduct lateral load tests is a new field of endeavor. At the time that this load test project was started, this was to be the 4th time that the Statnamic device had been used for a lateral load test. It was the 2nd time a lateral Statnamic load test had been used on a real project that was going to be built. It was the first time the device had been used over water. The first load on the initial Statnamic test on this project was going to be the largest lateral load that had been applied using the device.

The idea behind the lateral use of the Statnamic device was to try to imitate a barge impact on a structure and to determine how the structure would respond to a dynamic load. This data would be utilized in conjunction with the static lateral load test to determine how the two types of loading would affect the structure.

A 1574 ton (14 MN) Statnamic device, weighing approximately 63 tons was used for the tests. The device consisted of a hemispherical bearing plate, a cylindrical piston which contained the fuel, a center post for holding the concrete filled steel rings that served as the reaction mass, and a sled. The sled was placed near the end of the barge between two 14 inch steel "H" piles welded to the deck of the barge. The center post and the weights were placed horizontally on the sled. The piston was supported by a steel frame mounted on top of the caps. The piston was fueled just before each test. The sled, center post, and the reaction mass were moved (ever so painfully) to the end of the barge where they were mated with the piston. After everything was ready for testing, the barge was cleared, and the Statnamic device was fired. The Statnamic device was sent sliding down the barge a distance of 8 to 45 feet depending on the size of the load test.

Five Statnamic lateral load tests were completed on the strong axis of the pile cap and 2 loads were completed on the weak axis. The loads applied along the strong axis were in the opposite direction of the static load test. The loads applied to the pile cap and the resulting translation are illustrated on Figure 7. These values were plotted with the values obtained from the static lateral load test. Figure 7 clearly illustrates that the pile cap moved less under higher Statnamic loads than static loads. The Statnamic loading of the pile cap (Figure 7) indicates that beyond the initial load, the load-displacement is (for all practical purposes) linear. The initial load was low enough to be in the

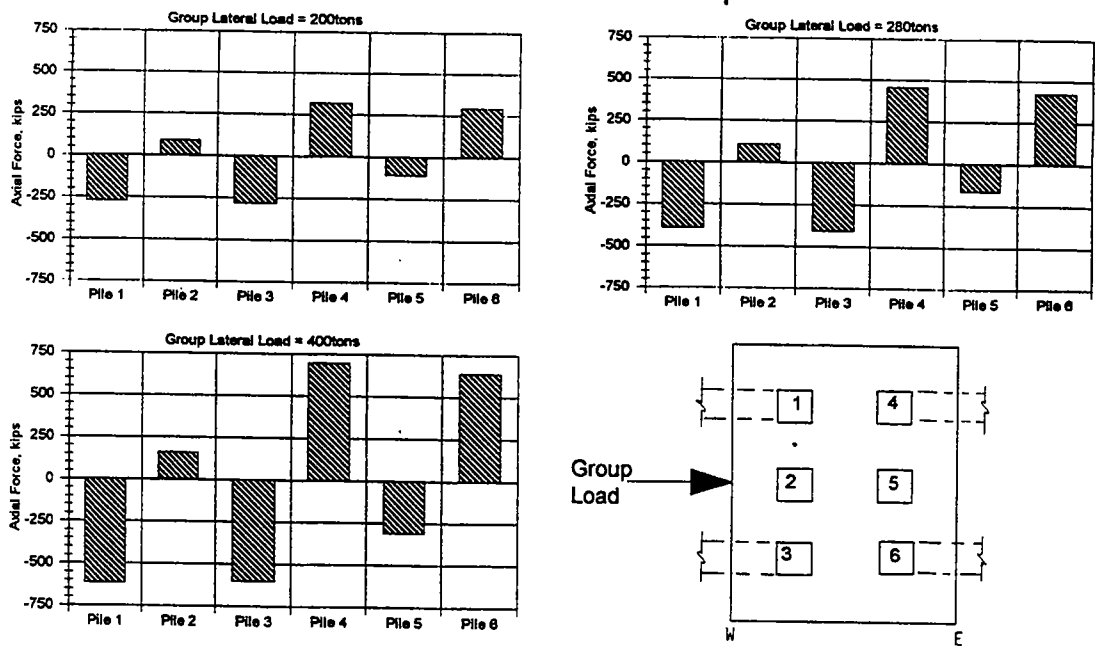

Figure 5 Summary of pile axial forces below pile cap during the static lateral load test.

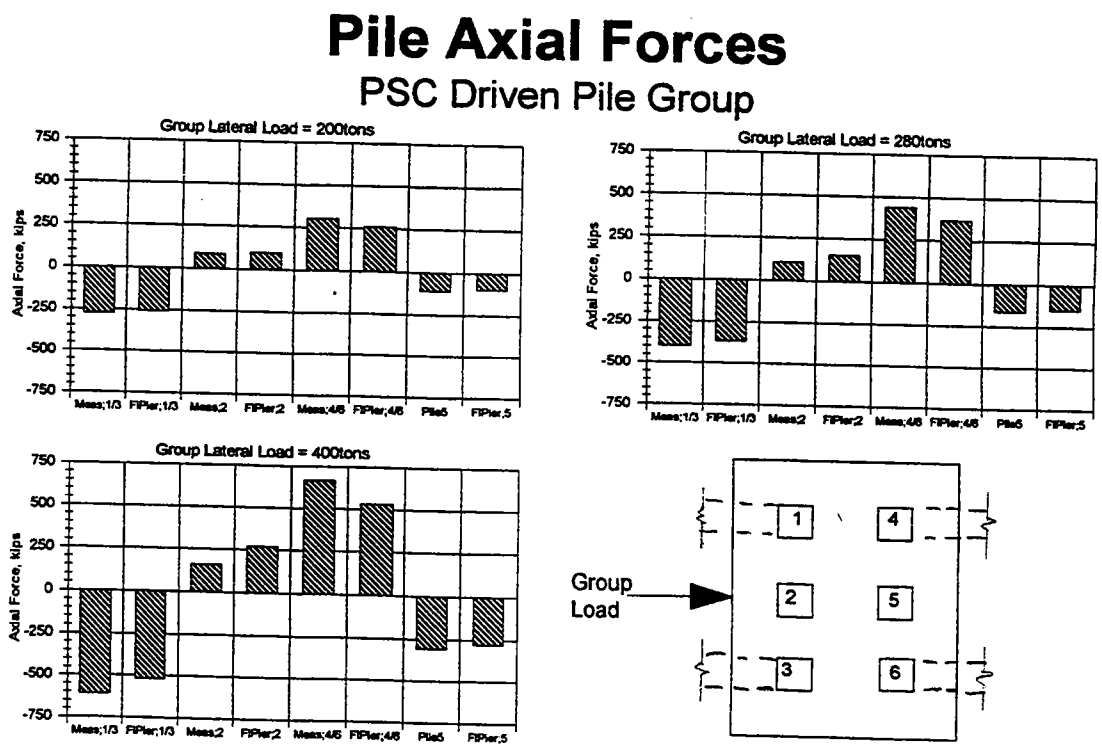

Figure 6 Summary of comparisons of predicted axial forces from Florida-Pier to measured values during the static lateral load test.

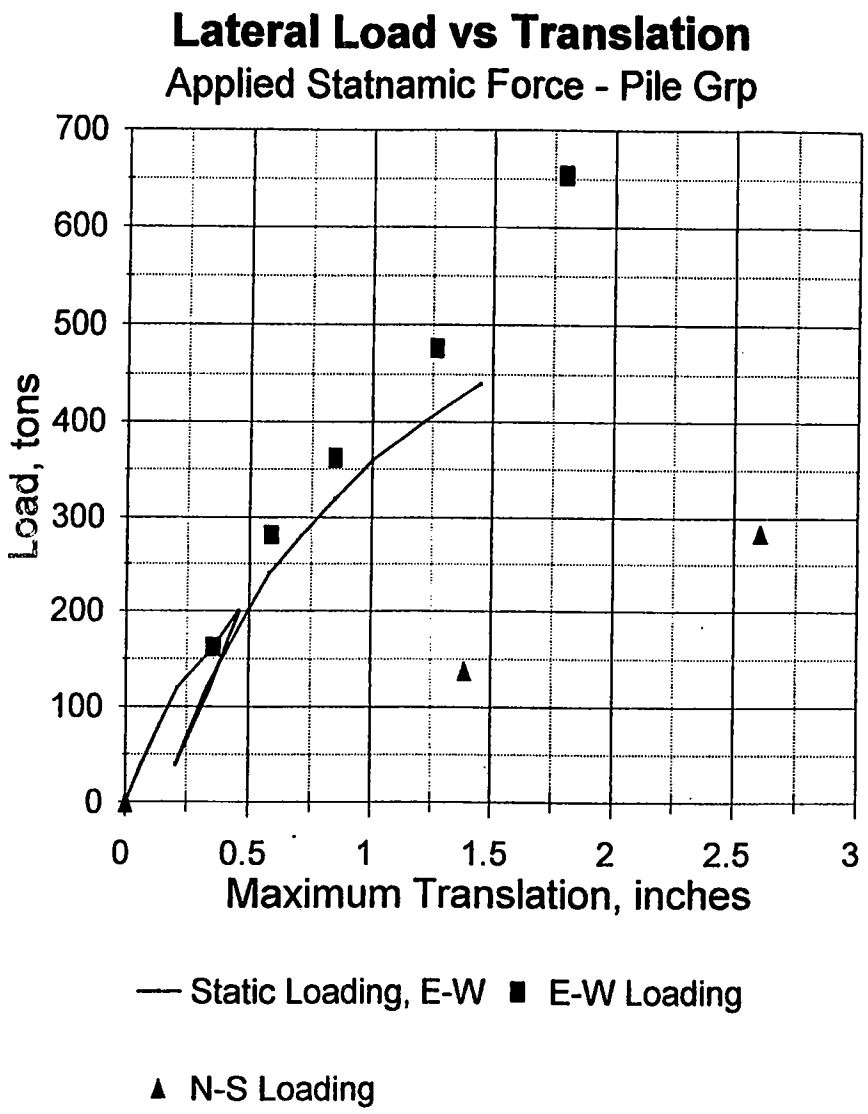

Figure 7 Comparison of maximum applied Statnamic forces to static forces vs. translation for the pile cap. Results are unique to test site conditions.

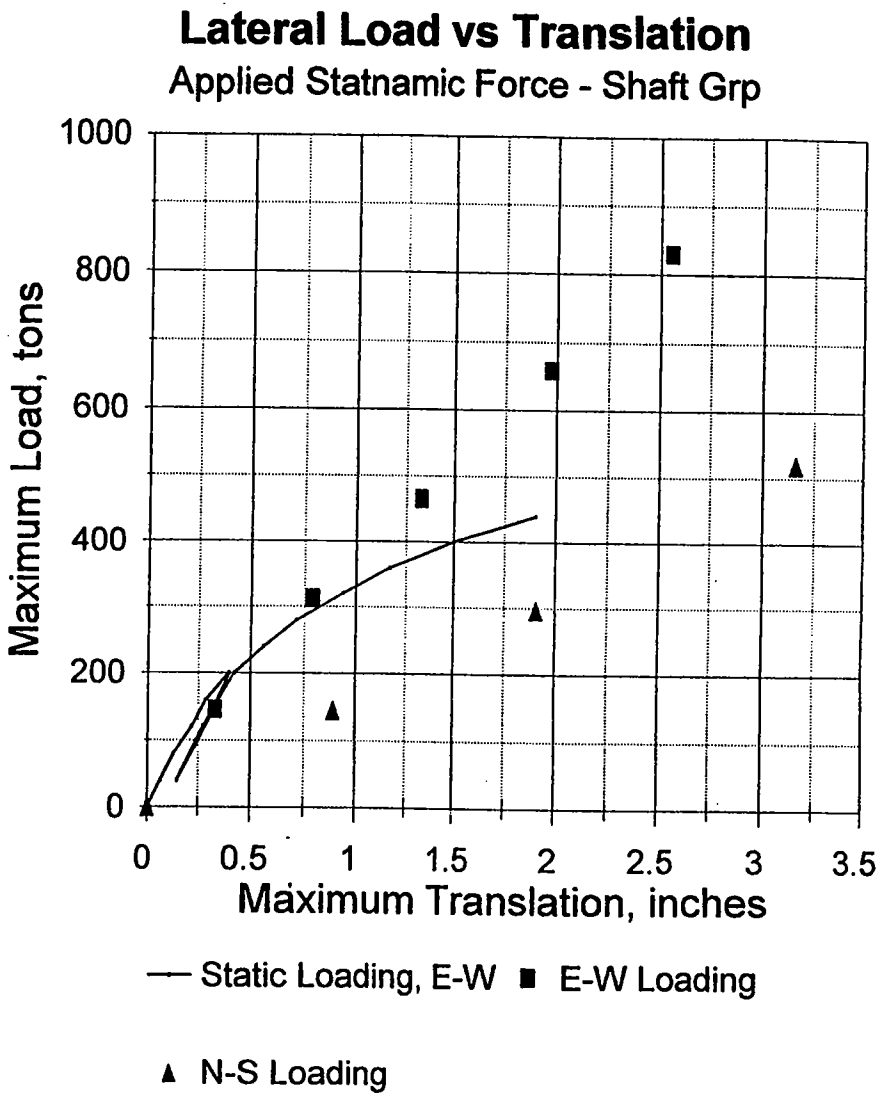

Figure 8 Comparison of maximum applied Statnamic forces to static forces vs. translation for the drilled shaft cap. Results are unique to test site conditions.

initial stiff zone indicated on Figure 3. Figure 7 also illustrates that the pile cap had much more movement along the weak axis than along the strong axis.

Five Statnamic lateral load tests were completed on the strong axis of the shaft cap and 3 loads were completed on the weak axis. The loads applied along the strong axis were in the opposite direction of the static load test. The loads applied to the shaft cap and the resulting translation are illustrated on Figure 8. These values were plotted with the values obtained from the static lateral load test. Figure 8 clearly illustrates that the shaft cap moved less under higher Statnamic loads than static loads. The Statnamic loading of the shaft cap (Figure 8) indicates that beyond the initial load, the load-displacement is (for all practical purposes) linear in both directions. The initial load was low enough to be in the initial stiff zone indicated on Figure 4. Figure 8 also illustrates that the shaft cap had much more movement along the weak axis than along the strong axis.

A comparison of Figures 7 and 8 clearly indicates that both the pile and the shaft groups respond better to dynamic loads that they do to static loads. This is especially true at the higher loads where the Statnamic and the static curves diverge. This divergence of the curves also shows that the pile group is much more capable of withstanding static loading under higher lateral loads than the drilled shaft group.

The fifth load of 825 tons stands as a world record for a lateral load by a Statnamic device. This test sent the 63 ton Statnamic device 45 feet across the barge. Figure 9 is the record of that test. Figure 9 illustrates that the load was applied to the drilled shaft cap in less than 0.1 seconds. The shaft group cap accelerated at a rate of 1.75 g's.

Figure 10 is a record of the 650 ton Statnamic lateral load applied to the pile group. Again, the load was applied in approximately 0.1 seconds with an acceleration of 2.0 g's for the cap.

During the Statnamic load tests, the empty slope indicator tubes on the trailing edge of the caps were loaded with custom made accelerometers that recorded the acceleration of the caps, piles and shafts at various points. This allowed the acceleration of the caps, piles, and shafts to be measured during the Statnamic load tests.

During the Statnamic lateral load tests, the pressure in the Osterberg load cells was monitored as a means of determining the applied load at the bottom of the 84" shaft and the 30" pile. The telltales and the strain gauges served as a method of verifying the measured loads. Figure 11 indicates that axial load transferred to the ends of the vertical pile was almost the same for both Statnamic and static load tests.

Figure 12 is a plot of the moments that were measured in the vertical pile of the pile group during both the Statnamic and lateral load tests. The measured moments are opposite in direction because the loads were applied in opposite directions. Figure 12 shows that the Statnamic load test generated much higher moments in the pile just beneath the cap than those moments generated by the static lateral load test under similar loads.

Conclusions

The load test project was a complete success in many ways. The information is being used to design the foundation for the new bridge. The changes in the design of the foundation for the bridge will pay for the cost of the load test project many times over. The information obtained from the entire load test project is currently being used in the design of two other bridges on the Mississippi Gulf Coast.

The load test project proved that the Florida-Pier Program can be utilized to integrate soil design parameters with foundation and super structure design to generate a cost effective design that is safe for both the builders of the bridge and for the users of the bridge.

Much more information will be extracted from the data as time allows. The data extracted to this point, proves that current, conventional design methods are too conservative. Innovative design practices should be used in conjunction with testing.

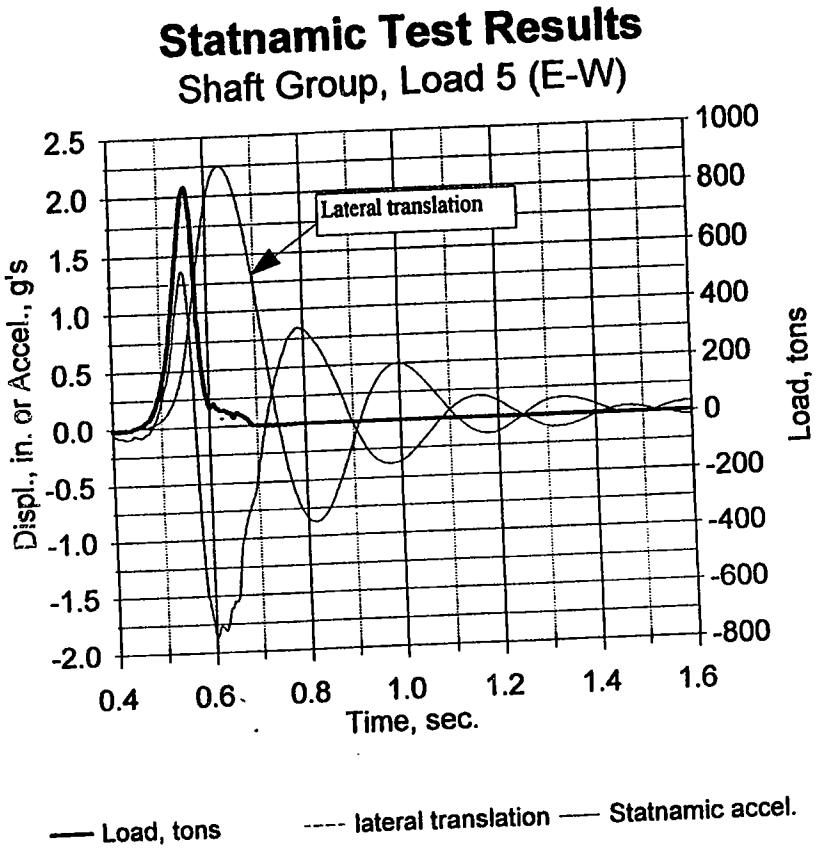

Figure 9 Time history for applied Statnamic load of 825 tons, lateral translation, and acceleration for the drilled shaft group. (load placed along the strong axis - in the east-west direction)

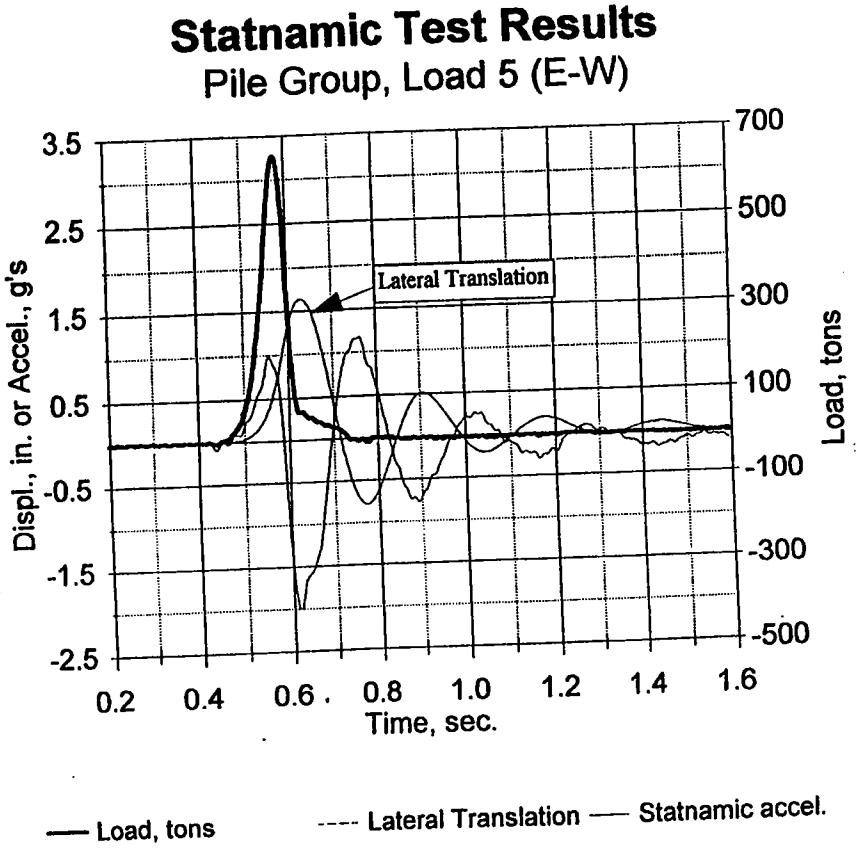

Figure 10 Time history for applied Statnamic load of 650 tons, lateral translation, and acceleration for the pile group. (load placed along the strong axis in the east-west direction)

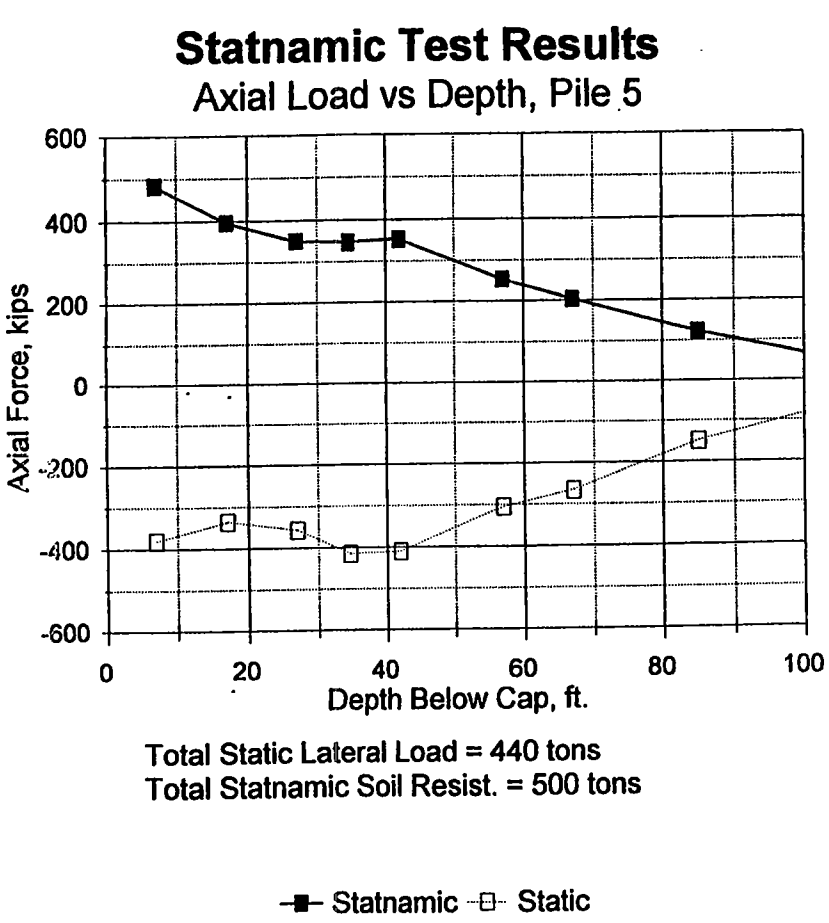

Figure 11 Comparison of axial load distribution below the pile cap for static and dynamic loads on pile 5 using data from the static and Statnamic lateral load tests.

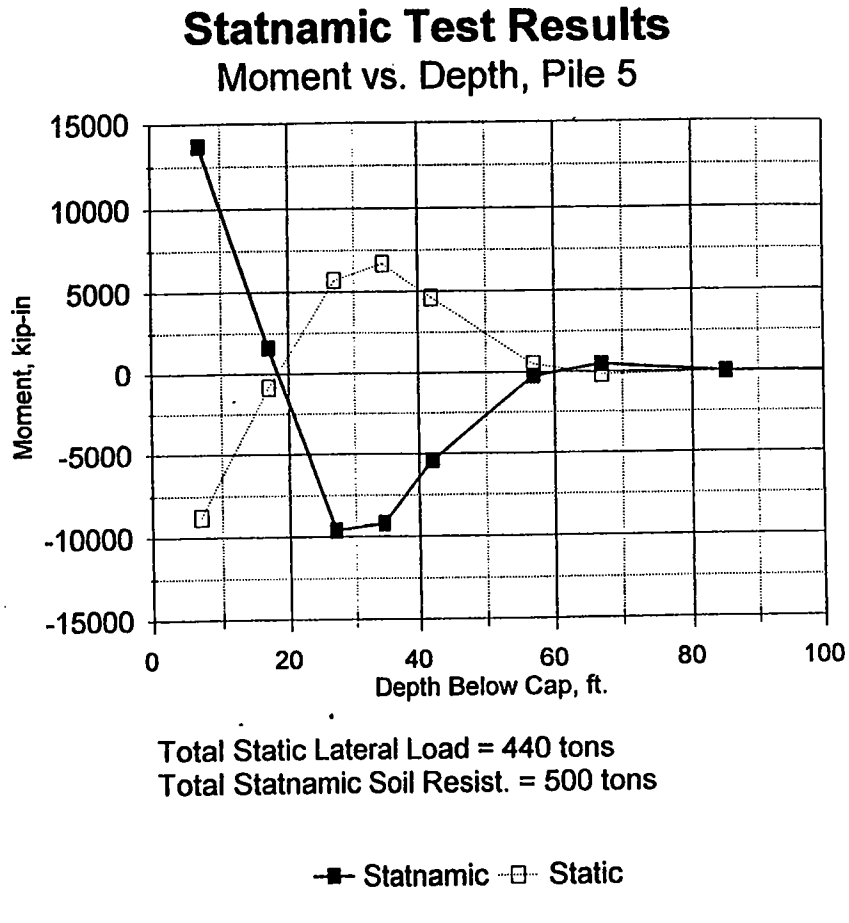

Figure 12 Comparison of static bending moments below the pile cap and dynamic bending moments for pile 5 using data from the static and Statnamic lateral load tests.

Acknowledgments

Dr. David Crapps, P.E., of Schmertmann and Crapps in Gainesville, Florida deserves the credit for a completely successful design of the project, for his diligence in administering the contract, and for his pursuit of excellence in his work. Dr. Crapps clearly put priority on getting the job done with quality first. Dr. Dan Brown, P.E., of Auburn University, worked as a consultant to Dr. Crapps. Dr. Brown's expertise in the field of design and analysis of pile groups was invaluable to the project. Dr. Crapps and Dr. Brown worked together to analyze the mountains of data and to put together the final report. Mr. Bob Hodges, of Warren Paving, deserves credit for his excellent work as contractor on the project. His desire to ensure a quality job helped to make the project a complete success. Thanks is extended to the FHWA and their support of the project. Their understanding of the need for this project resulted in the savings of millions of tax payer dollars.

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TESTING AND APPLICATION OF FLEXIBLE WIRE ROPE NETTING BARRIERS TO RETAIN MUD & DEBRIS FLOW

BY

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&

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INTRODUCTION

Briefly, debris flows are water-saturated masses of rock, soil and organic matter that rush down mountain slopes. Consequently, debris flows can denude slopes, damage structures, drastically alter streams and occasionally cause loss of human life. They are a common phenomenon in mountainous areas as well as coastal ranges, worldwide.

During the summer of 1994, a 50,000 acre wildfire swept through San Luis Obispo County, California. Most of the fire spread through steep mountainous terrain in areas where several state highways are located. In the aftermath, Caltrans' geologists reviewed the area and found several locations as having a potential for rockfall and mud and debris flows. These slopes consist of colluvium with occasional bedrock outcroppings and are as much as 500 meters high. Entire slopes have been denuded, exposing a multitude of loose rocks, small drain-ways and swales.

reinforced concrete channel and is 95 eters long, 2 meters wide and 1.2 meters ep. The uppermost 88 meters of the flume d lie at an angle of 31°, while the lower 7 eters gradually flattens to a run-out slope 3°. Massive 1.25 meter wide concrete nels were placed on the concrete run-out d in order to provide an additional 5 eters of channelized flow into the barrier. to 20 m^3 of saturated sediment can be scharged consecutively with an average locity of about 10 m/s into the retaining tting system with each test.

Considering these conditions, Caltrans decided to install an experimental wire rope netting barrier at grade level at one of the danger zones (see Figure 1). During the following winter a series of debris flows impacted the storm event where several high intensity rainfall events would occur, several separate debris flows were initiated. An estimated sixty cubic meters (60 m^3) of debris flow material accumulated behind the barrier as the result of three to six individual flows. Estimated impact energies ranged between 80 and 130 kilo-joules. Most of the material was stopped. This

Figure 1: Caltrans Experimental Wire Rope Net Barrier

is the first time a flexible wire rope net barrier was recorded to have stopped mud and debris flows. Common wisdom up to this point was that the fine grained material would pass through the wire rope net grid and could not be retained.

TESTING OF THE MUD AND DEBRIS FLOW SYSTEMS

These rather encouraging results emanating from the Caltrans barrier have led to further investigation dealing with how to test retaining systems for the retention of debris and mudslides by Geobrugg in Santa Fe, New Mexico. Subsequently, a controlled field testing program was undertaken in order to view the response of such flexible systems under full scale mud and debris flow loading. The testing was accomplished with the aid of the United States Geologic Survey (USGS) Debris Flow Flume, which is located within the H. J. Andrews Experimental Forest near Blue River, Oregon (see Figure 2).

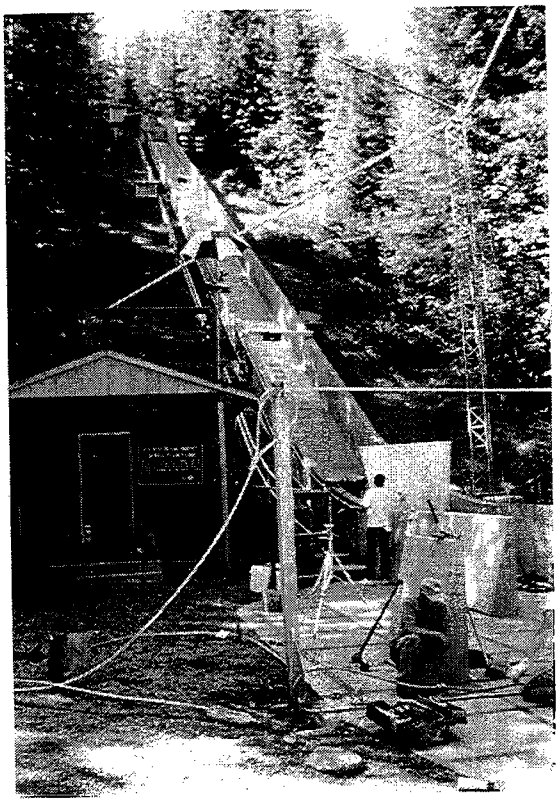

Figure 2: USGS Debris Flume

The testing of our retaining systems was conducted by the California Polytechnic State University (CALPOLY) team and was assisted by the USGS, Caltrans and Geobrugg personnel. The flume consists of a reinforced concrete channel and is 95 meters long, 2 meters wide and 1.2 meters deep. The uppermost 88 meters of the flume bed lie at an angle of 31°, while the lower 7 meters gradually flattens to a run-out slope of 3°. Massive 1.25 meter wide concrete panels were placed on the concrete run-out pad in order to provide an additional 5 meters of channelized flow into the barrier. Up to 20 m^3 of saturated sediment can be discharged consecutively with an average velocity of about 10 m/s into the retaining netting system with each test.

SELECTION OF DEBRIS

The selection of appropriate material to be used for these tests is an important step. Initial laboratory investigations focused on three possible soils:

1.) A poorly graded, gravel type sand mixture previously used by the USGS for debris flow studies.
2.) A poorly graded, gravel type mixture that had been deposited along the Caltrans San Luis Obispo site in 1994-95.
3.) A poorly graded, medium to fine sand that is routinely deposited along the Pacific Coast Highway near the community of Malibu.

Bulk samples of the three soils were brought to CALPOLY for geotechnical laboratory testing. It was discovered that the USGS debris sample mix was remarkably similar to those collected from the sites in San Luis Obispo and Malibu. It was decided that the USGS mix be used for testing of the mudslide barriers.

BEHAVIOR OF THE BARRIERS

The water saturated debris mass is deposited into the uppermost section of the flume channel and once the containment is released, if runs down into the chute toward the wire rope barrier for containment. A total of six tests were conducted with four different barrier designs. All retaining systems had a height of 2.44 meters (8') and a length of 9.14 meters (30'). This was sufficient to completely span the flume's concrete run-out pad (See Figure 3).

The intention was to begin our testing with the weaker or more economically manufactured systems. The so-called lesser of these systems as shown on the following tables A & B, Test #1 thru #3 stopped the mud and debris flows effectively, but much repair to the system was needed which realistically speaking would not be cost-effective.

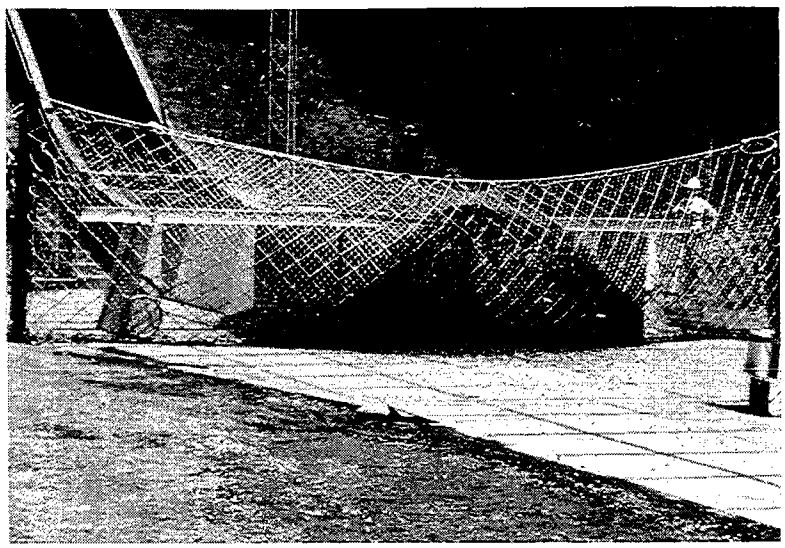

Figure 3: Wire Rope Net Test Barrier

TABLE: A

Comparison of Barrier Effectiveness In Terms of Debris Containment

Characteristic	Units	Test 1	Test 2	Test 3	Test 4	Test 5	Test 6
Initial Debris Volume	m^3	9.8	9.6	10.3	10.4	10.4	10.1
Volume Passing Beneath or Thru Net	m^3	SC	0.46	SC	0.12	.046	.0046
Percentage Passing	%	---	4.8	---	1.2	0.44	0.05

Note: SC = System Collapsed

It should be noted that the primary objective of the June 1996 field-testing program was to establish the degree to which a relatively open and porous flexible barrier could stop and contain rapidly moving, staged debris flows.

TABLE: B

Comparison of Debris Volume, Flow Velocity, And Net Deflection Data

Characteristic	Units	Test 1	Test 2	Test 3	Test 4	Test 5	Test 6
Debris Volume	m^3	9.8	9.6	10.3	10.4	10.4	10.1
Time to Arrival of Initial Gravel Spray	sec	18.0	17.0	18.0	17.4	18.9	16.4
Velocity of Gravel Spray	m/sec	---	12.5	---	10.0	---	10.0
Initial Velocity of Debris	m/sec	---	9.0	6.5	8.0	6.0	5.0
Recorded Maximum Outward Deflection	m	---	1.46	0.30	1.93	0.40	1.50
Recorded Post-Flow Outward Deflection	m	---	1.35	0.10	1.41	0.30	1.47

The goal of full containment was finally realized in test #4 and #5 and especially in test #6, where less than 0.05% of the sediment passed beneath and through the net system. With regard to tests #4, 5 and 6, a much stronger design was applied for the retaining barriers, resulting in no damage to the system when impacted by the mudflows. The actual design data pertaining to the different barriers tested will not be discussed in this paper but are available from the September 30, 1996 CALPOLY Test Report.

TEST SUMMARY

The field-testing program allowed the observation of four different mudflow mitigation systems. In conclusion one could say that the system used for test #4, 5 and 6 can be used for stopping and containing mud and debris flows of up to 11 m^3 (See Figure 4).

Another point to be made is that the data acquired during the field testing program reconfirmed the value of the patented Geobrugg braking element (commonly used with the rockfall retaining system) and the reason for locating them near the corners of the barrier support structure. Without these braking elements the mud and debris flow barriers could not work and much damage to the system would occur. As with rockfall barriers, flexibility is the key for a successful mud and debris flow system. In addition, it was the first use of load cells in the field testing program from which the data acquired will be quite helpful to the geotechnical and structural engineers who are responsible for the design of the netting retaining systems.

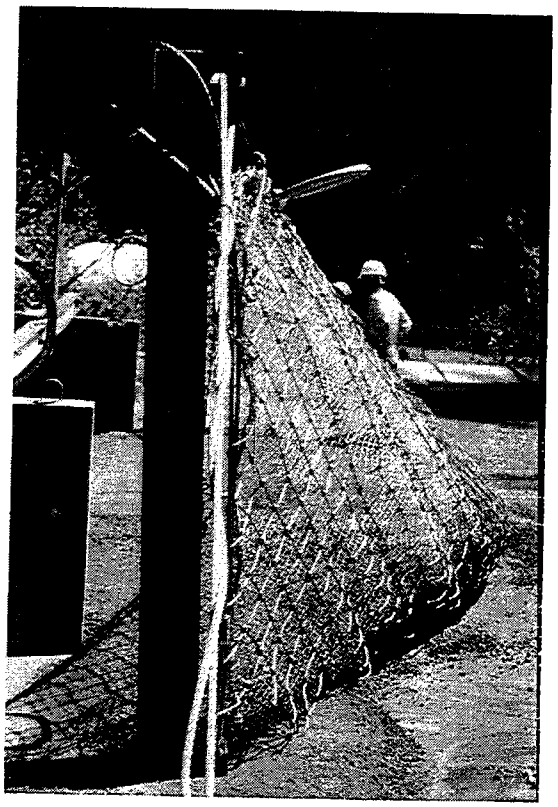

Figure 4: Side View of Test Wire Rope Net Barrier

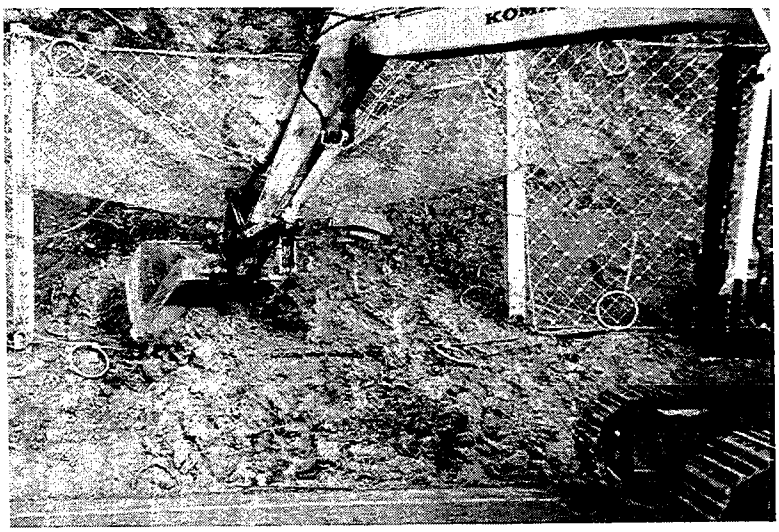

Figure 5: Mud & Debris Removal With Front End Loader

It should be noted that in some instances, maintenance was required such as replacing braking elements and torn wire mesh and/or silt material. Almost as important is the removal of debris that has accumulated in the barriers during testing. This can easily be removed by using a front-end loader (See Figure 5).

CASE STUDIES

Optimistically, by viewing the rather successful test results, our first commercially designed mud and debris flow system was installed at the "Stella Vicinity Slide" for the Washington State Department of Transportation (See Figure 7). By November of 1997, this system had already stopped a 4-5 m^3 debris and mudflow. Only a minor amount of fines passed through the netting (See Figure 6).

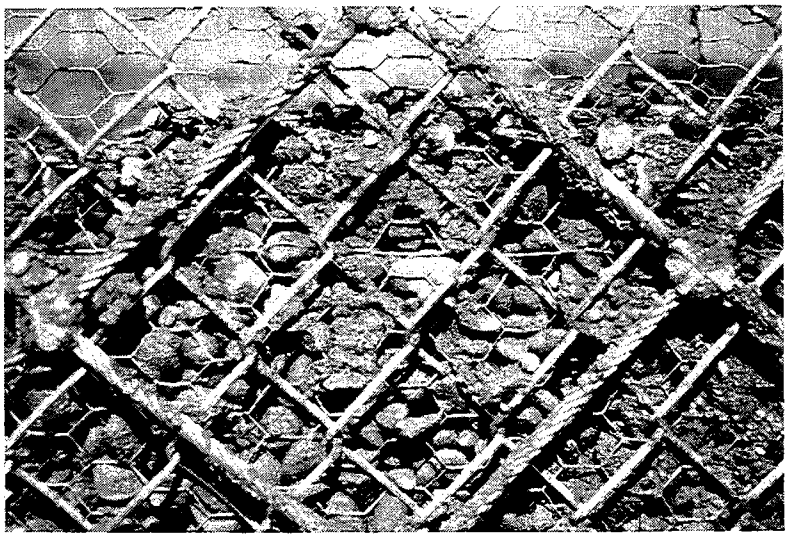

Figure 6: Close-Up of Fines
Contained in the Wire Rope
& Mesh Netting

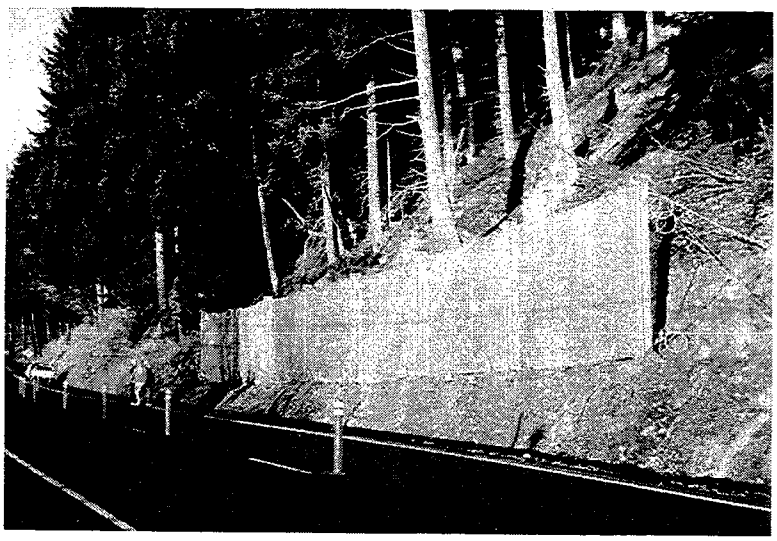

Figure 7: Washington DOT "Stella
Vicinity Slide" Barrier

The capability of these systems for mitigating slope instability, beyond individual rockfall, was furthered by events experienced during the 1997 winter's "El Nino" storms. At Gaviota Pass on California State Highway 1, one particular debris flow moved 30 meters downslope into a wire rope netting system. An estimated 60 yd^3 of material was stopped including fine-grained soil, rocks and trees (See Figure 8).No damage to the system occurred.

At the area known as Waddell Bluffs also located along California State Highway 1 which also was hit heavily by intense rains, the rock retaining nets which were installed only in late 1997 were severely impacted by debris flows. In one particular event, approximately 60 meters of slope material consisting of rock, mud and trees, etc., were moved downhill and upon impact with wire rope barriers, and estimated 400 m^3 of material was stopped with no material passing through the system.

Inasmuch as this particular system was not designed for such heavy kinetic energy, minor damage did occur that was easily repaired immediately after the occurrence of the mud and debris event.

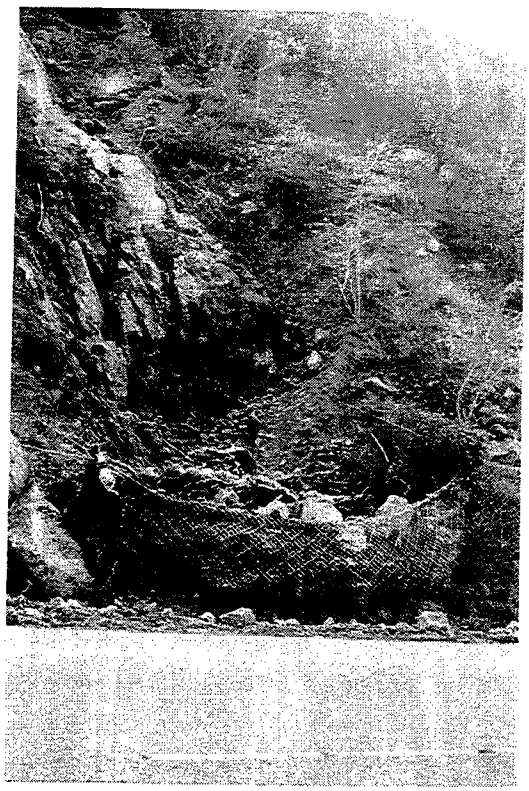

Figure 8: Gaviota Pass Debris Flow

CONCLUSION

The results obtained during the testing and the system installed by the Washington DOT and Caltrans, were most encouraging and added significantly to the existing body of knowledge. In time, more testing of these systems will be necessary and furthermore, one needs to learn in practice how these systems will behave and can be utilized in the safest and most economical way with a minimum amount of maintenance.

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Soil Nailing

Importance of Geology and Construction Considerations as They Relate to Field Inspection

by

Harry L. Siebert[1] and Barry D. Siel, P.E.[2]

Introduction

Soil nailing is a relatively new technique introduced in France in 1972. Acceptance can be a slow process and as a technique is used it changes and improvements occur. For instance, it took 20 years for pre-splitting to be accepted and today, although changes have occurred, the failure mechanism to generate a fracture between boreholes has not been determined.

It is imperative to the success of a soil nail project that field inspectors be properly trained in the construction of soil nail walls. The Federal Highway Administration has published the "Soil Nailing Field Inspectors Manual" which is an excellent guide that details the inspection process. This publication is also used by experienced superintendents. Experienced contractors will use the flash coat technique to prevent shallow surface failures in marginal soils and to protect the excavation during inclement weather. Soil nailing is commonly used in place of traditional shoring for building excavations. Improvements often occur from suggestions of field inspectors. The influence of geology on the success of a soil nail project is critical. On a number of projects, geotechnical reports have indicated that the site was not suitable for soil nailing due to adverse rock conditions or soil types. Subsurface information is best obtained and analyzed by experienced staff who understand the influence of the geological and engineering properties of the materials on the applicability, design, constructability of a soil nail wall.

Construction of soil nail walls below ground water elevation is not recommended. If perched water is encountered during construction drainage of the water is necessary and shallow weep holes may not be adequate. Long term hydrostatic conditions must be considered during design taking into account regional hydrological conditions.

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Dealing With Difficult Ground and General Problem Solving

Field experience suggests that the degree of difficulty relative to soil and rock slope design creates the opportunity to propose remedial actions. Where communication between the owner's representative and the contractor should allow problem resolution to a few options consistent with the contract specifications. Even though soil nail walls are may be new to a user, there is extensive technical expertise available. When properly designed and constructed, failure during construction of a soil nail wall is not common.

Changes in conditions such as materials encountered differing from those anticipated, excess ground water, and intersecting previously undetected utilities require the field inspector to contact the designer to determine of any adjustments to the design are appropriate. In these instances, it is critical that the designer get back to the inspector with a solution to the problem as quickly as possible so as not to delay the project. On a project where a soil nail wall was being constructed to support a cut in an existing highway embankment the investigation failed to reveal that the embankment consisted of flood debris with trees, metal and voids. The wall began to fail during installation of the first row of nails and it was determined that the ground conditions were inappropriate for soil nailing technology. The project was redesigned using a caisson pile wall.

There are soils that can be appropriate for soil nailing but offer special construction challenges, such as coarse granular materials (boulders and cobbles). Soft, fine-grained soils need to be evaluated for long term creep potential. Valley fill deposits and glacial lake sediments can have saturated sandy gravel lenses, nests of cobbles and boulders, silt and soft clay layers. If these materials are delineated during the subsurface investigation the soil nail wall can be properly designed and appropriate construction methods can be anticipated by a qualified contractor.

Valley fills and coarse gravels generally contain boulders and may require the contractor to use specialized equipment to advance the borehole to the required length, such as a rotary-percussion drill rig. When the subsurface investigation indicates ground conditions that may preclude open hole drilling a cased hole drilling method such as drilling with casing and flushing the cuttings or using a rotary-duplex system may be required. A qualified contractor will review the soil conditions and bring the appropriate drilling equipment to the project, and the bid price will reflect the difficulty of the drilling. In the case of changed conditions when the contractor does not own the proper equipment or it is not available for the project, the owner and the contractor often have to cooperate and work together to develop an appropriate course of action.

Corrosion Considerations

Long term performance of soil nails depends on their ability to withstand corrosive attack from the environment. Corrosion protection is an integral and extremely important aspect of soil nail construction and inspection. There are three levels of corrosion protection. The corrosion protection required for a particular project depends on the aggressiveness of the soil and groundwater (as defined below) and the purpose of the soil nail wall. The highest level of corrosion protection is encapsulation of the tendon with a corrugated PVC or HDPE tube and the annular space filled with a neat cement grout. This is often referred to as "double corrosion

protection." This level of corrosion protection is required for permanent soil nail walls installed in aggressive ground. A lesser level of corrosion protection is a minimum 0.3mm thick fusion-bonded epoxy coating and provides adequate corrosion protection for permanent soil nails in non-aggressive ground or temporary soil nails in aggressive ground. For temporary soil nail walls in non-aggressive ground, the grout surrounding the tendon within the drill hole provides adequate corrosion protection. A temporary soil nail wall is considered to have a service life of no more than 3 years.

There are four criteria by which aggressive or non-aggressive ground is defined: electrical resistivity, pH, sulfates and chlorides. The ASTM and AASHTO test methods and critical values for each criteria can be found in the FHWA "Manual for Design & Construction Monitoring of Soil Nail Walls" and are the same criteria that are recommended by the Post Tensioning Institute for soil and rock anchors and the FHWA "Permanent Ground Anchors" manual. Low pH levels and the presence of sulfates will also affect the grout design.

The owner's inspector should know the corrosion requirements called out in the design and ensure that the soil nails installed on the project meet those requirements. The inspector should also be familiar with how any changed conditions may effect the corrosion requirements so that any necessary changes can be negotiated with the contractor.

Materials that may require careful corrosion protection considerations include mine or solid wastes, and soils or rocks with high sulfate such as black shales. When subsurface water is encountered, either during the subsurface investigation or during construction of the wall, water or wet soil samples should be obtained and pH measurements made. If the pH is outside the acceptable range the designer may need to redesign the grout and/or modify the corrosion protection specifications.

When nail tendons arrive on the project, the inspector should observe the unloading and storage operation to insure that the corrosion protection is not damaged. Further inspection should be done to insure that the epoxy coating and/or encapsulation is in conformance with the specifications and determine what if any field repairs can be effected.

Owner - Contractor Relations

A key factor for the owner to establish a good relationship with the contractor is have a trained, experienced, full time inspector on the project. Soil nail inspection is not an easy task to perform. Inspectors are required to make astute and timely field decisions based upon their knowledge, experience, and understanding of the project requirements. Experience has shown that soil nail wall construction is best accomplished when the contractor and inspector perform as a team and maintain a strong relationship in which each mutually respects and openly acknowledges the experience, intelligence, and opinions of the other. A situation which can complicate the owner - contractor relationship is when one or more subcontractors are performing parts of the soil nail wall work such as excavation, drilling or shotcreting. The soil nail contractor's superintendent and the owner's inspector must control the excavation to see that the proper construction sequence is followed. There have been cases where the excavation

subcontractor has taken the cut for the soil nail wall down to grade before the soil nailing subcontractor has arrived on the project. Overexcavation of the wall face will require backfilling with either shotcrete or cast-in-place concrete.

On small projects (less than $50,000) the owner may need to develop design parameters that allow local (less qualified) contractors to bid on the project and then require the contractor to retain a qualified soil nailing consultant. This situation will require extra diligence on the part of the inspector and should not be used where difficult ground conditions are anticipated, regardless of the size of the project.

Conclusions

This presentation is not intended to review the entire soil nail wall construction process but to highlight some concerns. If communication between the inspector and the soil nail wall construction manager is established prior to mobilization on site, many of the concerns discussed herein will be greatly reduced.

It is recommended that the term "soil nail" be used to refer to a nail that is placed in a bore hole that is drilled into soil (unconsolidated material). The term "ground nail" should be used where more than one type of material is present, such as soil, glacial till, weathered rock and rock.

With the increased utilization of permanent and temporary soil nail walls a data repository for completed projects should be developed.

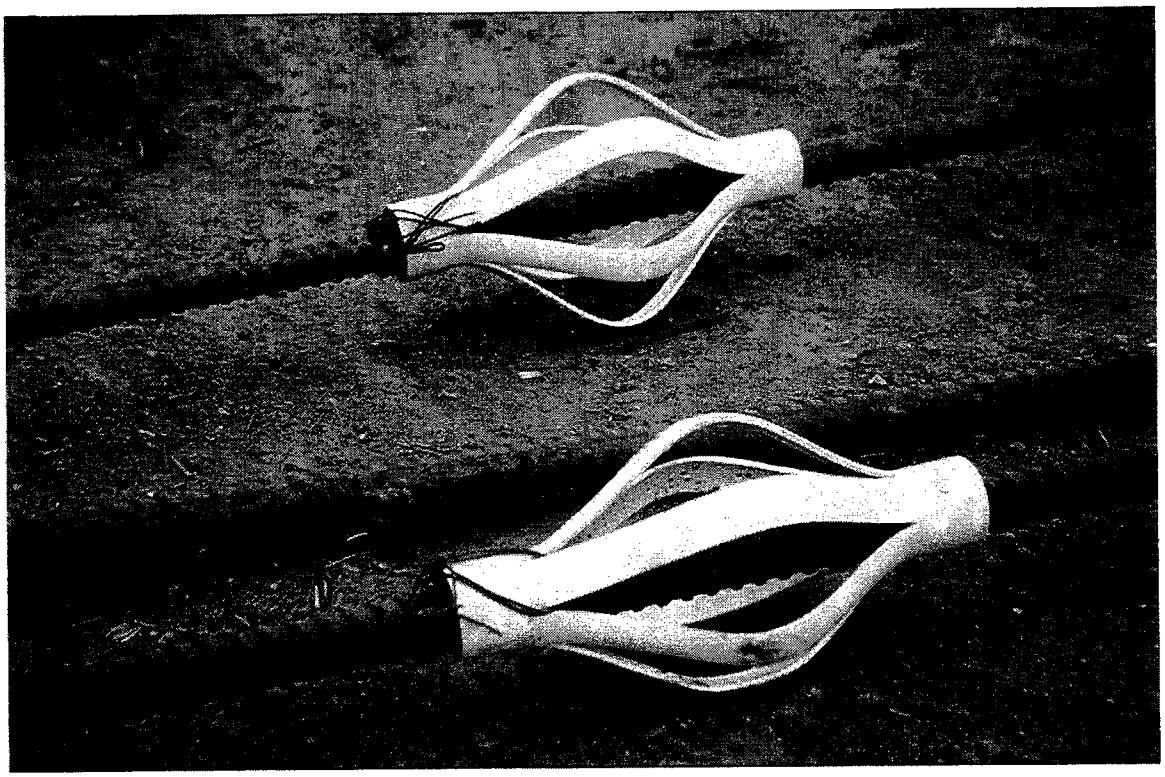

Figure 1. PVC centralizer cover wire with tape to avoid difficulty inserting tremie or dislodging centralizer during installation

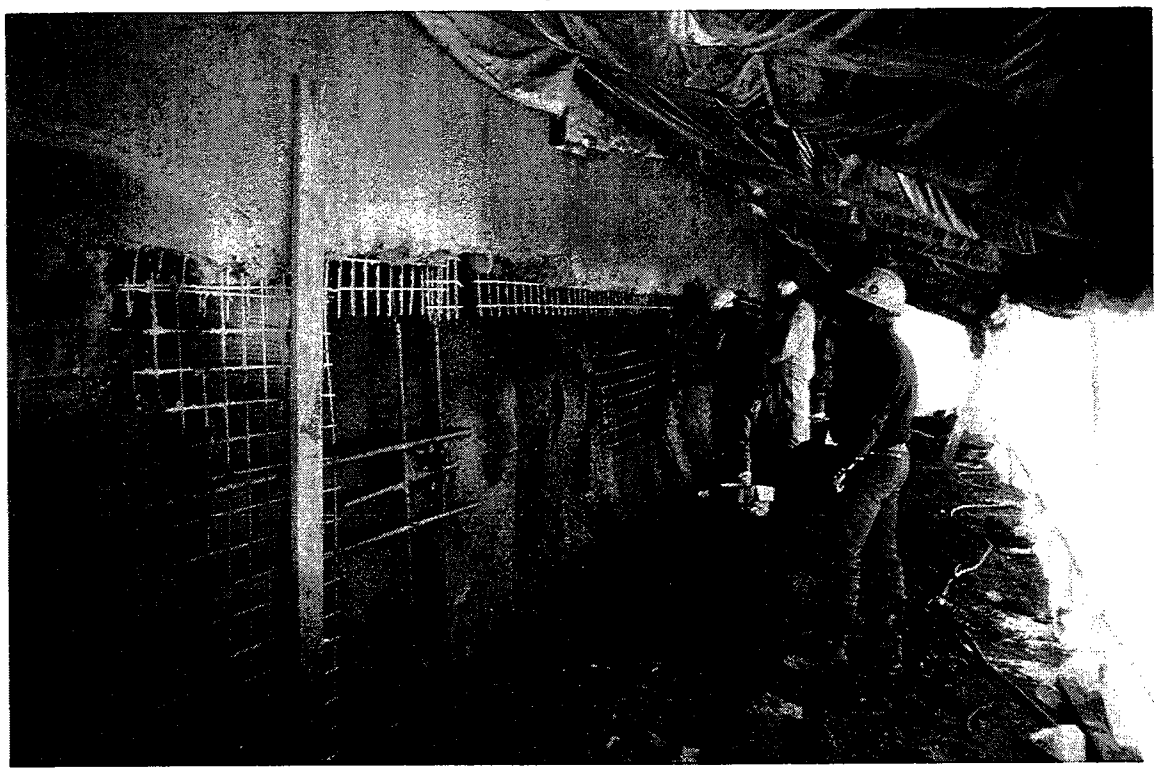

Figure 2. Inside a shelter for winter soil nail installation, heat as necessary and note adequate work area.

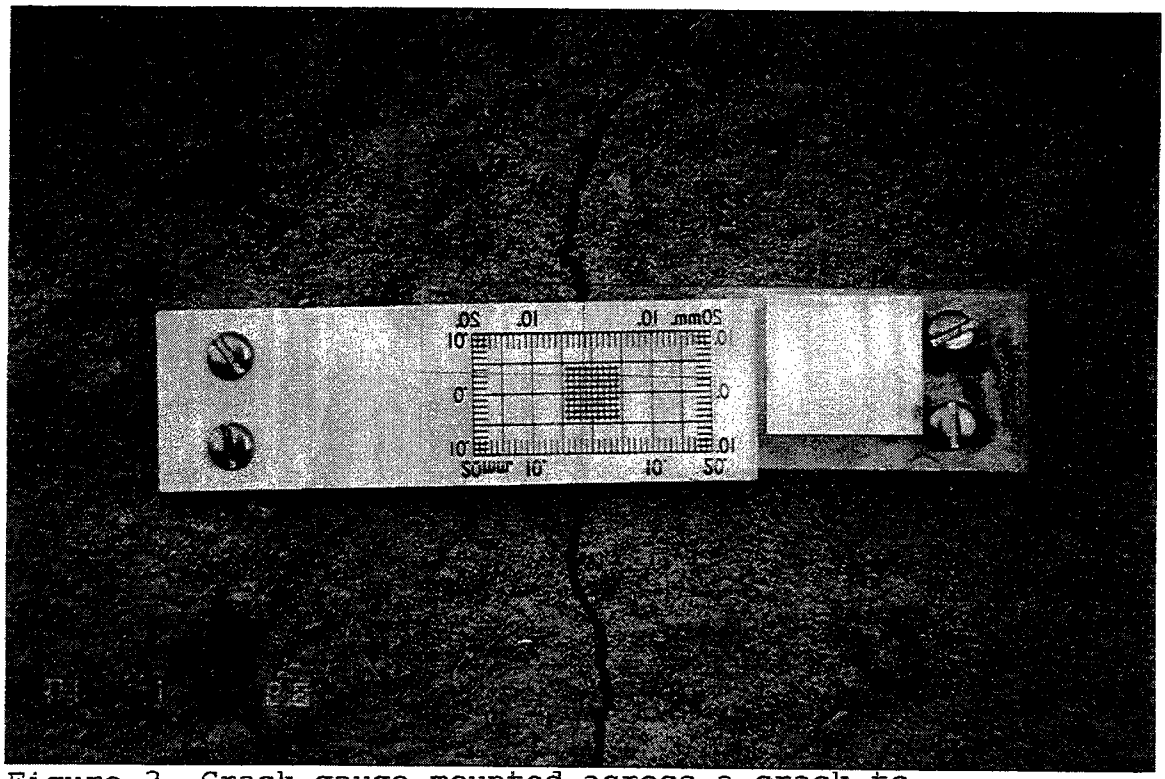

Figure 3. Crack gauge mounted across a crack to determine movement during and after construction.

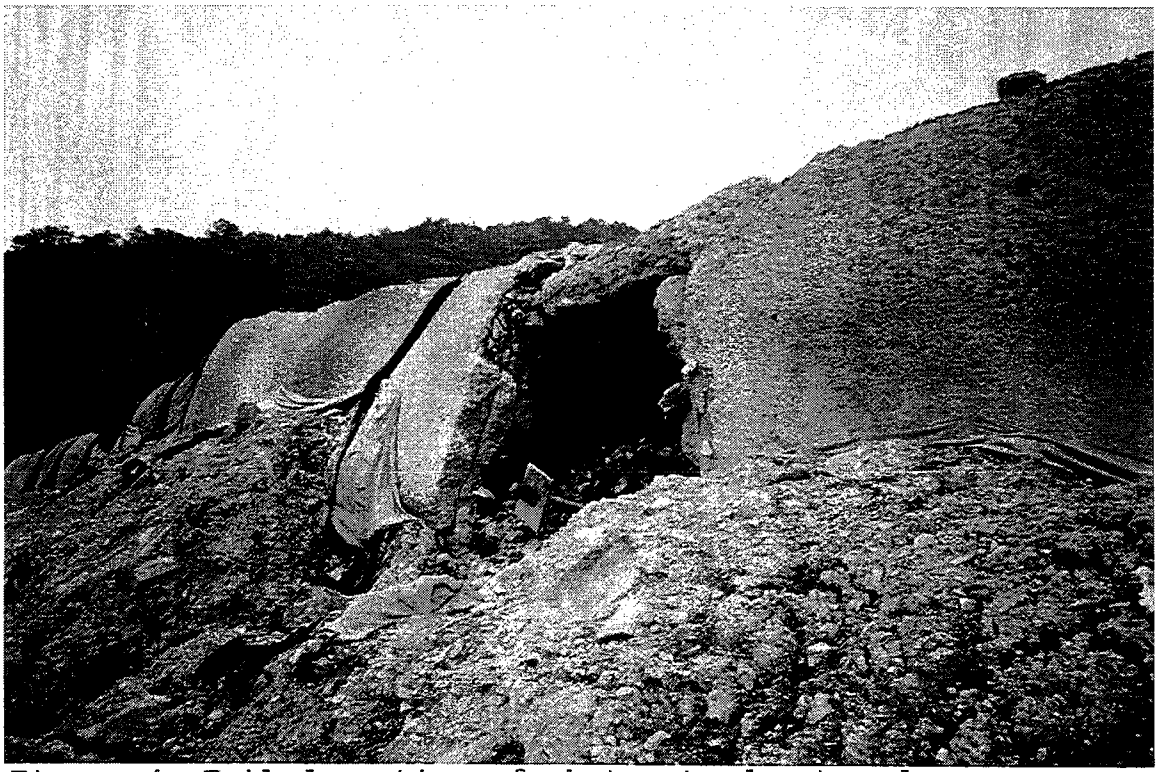

Figure 4. Failed section of shotcrete due to adverse geology (bedding dipping out of the face interbedded sandstone/siltstone/claystone)

CALL OF THE CANYON
MECHANICALLY STABILIZED EARTH STRUCTURE
WITH SIMULATED ROCK FACING

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Douglas E. Alexander, P.E., Geotechnical Engineer, Arizona Dept of Transportation

ABSTRACT

Rock slope instabilities are one of the most potentially disastrous and troublesome highway maintenance problems encountered by DOT's. Causing not only extensive property damage, but in extreme cases, disability and death, rockfalls can vary from a few cobbles on the pavement to massive rock slides able to crush vehicles and close roadways. The search for effective stabilization processes had lead to some creative used of construction technology. Everything from conventional concrete retaining walls to deep, tieback cable anchor systems have been used. Occasionally, stabilization isn't practical and the designer resorts to simply channeling rockfall into the cut ditch with wire mesh or even to building rock sheds over the highway and allowing rockfall to continue at it's natural pace.

Complicating matters, many of the most active rockfall areas are located in rural environments often controlled by entities such as the U.S. Forest Service, Bureau of Land Management, National Parks Service, and other environmentally sensitive organizations. Frequently, the most practical solution is not acceptable since it is not aesthetically pleasing.

Such was the situation ADOT found itself in when devising a solution for the Call of the Canyon. Compelled by the Forest Service to develop a project that would not offend the general publics sense of natural beauty, ADOT conceived a retaining system that would not simply prevent rockfall from occurring, but also enhance the natural beauty of the local environment. The ultimate solution involved constructing tiered MSE retaining structures, then facing them with simulated rock that would match adjacent natural formations. Judging by public reaction and the Forest Service satisfaction, we believe we succeeded.

Arizona State Route 89A, which begins just north of Prescott and ends at the Arizona/Utah state line, passes through the Town of Sedona and continues on through Oak Creek Canyon heading for Flagstaff. Sedona, at the mouth of Oak Creek Canyon, is at an elevation of 4300 feet and Oak Creek Canyon tops out at almost 7000 feet elevation. Refer to Fig (1) for location. Oak Creek Canyon, although of smaller scale, rivals the Grand Canyon for scenic beauty and is its equal in tourist volume. It is an international travel destination and often referred to as a national treasure.

The canyon depth ranges from 1500 feet at the head of the canyon increasing to 2500 feet near Sedona. The lower elevations of the canyon begin in Redwall limestone, moving upward through the Supai Group-Hermit shale sequence, into Coconino sandstone, terminating in Kaibab limestone, all very colorful.

State Route 89A was constructed in 1933 as Forest Road 7-D, a 22 foot wide track carved out of the canyon formed by Oak Creek, with near vertical rock cuts and steep colluvial slopes that haven't changed much since original construction. Some road widening has occurred, but shoulders have remained narrow or non-existent. When rock fall occurs, as it occasionally does, there is no where but on the road for the rock to fall. Through the years there have been numerous rock falls ranging from cobbles on the road to full closures of the highway. More than one death and numerous injuries have been attributed to these rock falls.

In 1992, a study of rockfall problem sites was completed by deLeuw Cather, with the assistance of Mr Ken Euge, throughout the eighteen mile length of the canyon. Twenty six sites were identified as having significant potential for rock fall hazard. Three sites have been completely remediated, one site is under construction, and one project was terminated due to constructability issues part way through. The second site remediated was Call of the Canyon, the topic of this presentation. It is site no 16 on Fig (2). The name is derived from a Forest Service parking lot across the highway from this site. It is at the trailhead of the West Fork Trail.

Oak Creek, which flows the length of the canyon, has a major impact on construction activity. Not only are there environmental considerations, but since Oak Creek Canyon is very narrow, work space and staging areas are severally limited. An even greater impact comes from traffic. The ADT, average daily traffic, is approximately 7000. If the ADT is averaged between winter and summer traffic then the traffic load during our construction season is probably much higher that that. To illustrate the difference in tourist load refer to Fig (3) for winter and Fig (4) for summer. This is Slide Rock, a very popular Arizona swimming hole. In fact, it's so popular that the County Health Department closes it occasionally due to high coliform count.

ADOT, with the help of the Forest Service, developed a single Environmental Impact Statement for all sites. A preferred alternative for remediation was selected and approved for construction in each case. Normally this process would be done for each site as part of project development. In theory, more would be known about each site as solutions were presented for public consideration and the best approach would be identified if more time was available to study each site. In this case the decision was made to only go through this painful process once. Environmentally, Oak Creek Canyon is one of the most sensitive locations in the State. The selected remedy for this site required construction of a tiered retaining wall. This could not be just any retaining wall, but had to be a thing of natural beauty. This was an imposition made by the Forest Service. In all fairness, the previous project developed a changed condition which

required a pile and lagging wall to correct investigation deficiencies which partially resulted from Forest Service restrictions. In that case, we were only allowed to perform jack hammer borings, which we supplemented with a seismic survey. In construction we discovered that the bedrock anticipated were really room size boulders. Right of way limits were already set and irrevocable so the pile and lagging wall was constructed. The Forest Service still hears about that from disenchanted residents.

Figure (5) shows the site prior to construction. It is a road cut in colluvium approximately 60 feet high at its highest point and stands at a slope of roughly ¾:1. District Maintenance reported frequent rockfall at this site during wet weather as well as frozen water on the highway at the south end of the project, just as you enter a 19 degree curve. As you would suspect from figure 5, cobbles and boulders were the common size material spalled from this slope. With little to no cut ditch available to catch rockfall, most of it landed on the highway.

Vowing not to repeat the same mistakes made at the first project, we directed our geotechnical consultant to retain Salisbury and Associates to perform soil borings using their portable Winkie drill. We were able to obtain good boring logs and some rock cores to perform stability analyses and design from. Slope stability analyses indicated a stable slope, even under construction conditions so an MSE wall type was selected. A simulated rock face was specified to be attached to the MSE wall.

At that time, ADOT had pre-approved three MSE type wall suppliers, VSL, Reinforced Earth, and Hilfiker. The contractor could choose from these three. Prior to start of construction, the contractor came in with a value engineer(VE) proposal. A subcontractor had assured him that they could soil nail the slope for less than the cost of installing an MSE wall. ADOT spent several weeks reviewing and commenting on the soil nail design proposal. Ultimately, the sub contractor admitted they'd overlooked a few things and the soil nail wall would be more expensive. The contractors short construction season just got shorter. The contractor finally submitted plans for an MSE wall for our review.

The project was awarded to Ames Construction of Phoenix in February 1996. Ames subcontracted the material supply of the tiered retaining wall structure to Hilfiker Retaining Walls of Eureka, California, who proposed their welded wire wall with simulated rock facing. Design of the retaining structure was prepared by MacIntosh Engineering of Auburn, California.

The welded wire wall was first introduced in the 1970s and is one of several types of mechanically stabilized embankment (MSE) structures in use today. Since its inception, there have been more than 4,000,000 square feet of structure installed in over 6,000 structures worldwide. Many welded wire walls have exceeded 80 feet in height. The highest welded wire wall known to this author is 104 feet, at a gold mine in northern British Columbia, Canada.

The specifications called for AASHTO design criteria for Earth Retaining Systems, 1992 Edition, with 1993 Interims. Design parameters were: a design life of 75 years, random and foundation soils with a unit weight of 125 pcf and $f = 32^{\circ}$, select backfill with a unit weight of 125° and $f = 34^{\circ}$, and a seismic acceleration of 0.15g.

The wall height varied from 10 feet to a maximum of 60 feet, supporting a 2:1 slope. The typical wall configuration consisted of 2, 3, or 4 tiers, with each tier at varying heights and varying

foundation soils with a unit weight of 125 pcf and f = 32°, select backfill with a unit weight of 125° and f = 34°, and a seismic acceleration of 0.15g.

The wall height varied from 10 feet to a maximum of 60 feet, supporting a 2:1 slope. The typical wall configuration consisted of 2, 3, or 4 tiers, with each tier at varying heights and varying horizontal setbacks. All walls had a 1:10 batter on the face. The idea of the structure was to simulate the rock facing to blend into the surrounding environment as naturally as possible. Figure (6) shows a typical tier configuration for the wire wall, and Figure (7) shows the tiered configuration itself at the highest point.

The design of the welded wire wall was in accordance with AASHTO Standard Specifications for Highway Bridges, Fifteenth Edition, 1992, with 1993 Interims, and the National Concrete Masonry Association Design Manual for Segmental Retaining Walls, Section 2.5.4, Tiered Walls.

Construction of the welded wire wall begins by grading a level surface equal to the length of the welded wire mats. The mats are placed and aligned. Backfill is placed and compacted in lifts to the top of the mat, leaving a void immediately behind the wire facing. Backing mats and hardware cloth are placed behind the facing. The next mat is placed, and the void behind the facing is filled and compacted with light mechanical equipment. The procedure is repeated to the top of the wall.

The length of soil reinforcing mats varied from 8 feet for the top tier to as much as 22 feet in some portions of the lowest tier. The welded wire mats had wire spacing of 6 inches for longitudinal wires and 12 inches for transverse wires. Wire size varied from W4.5 to W9.5. Drainage was provided by double-sided drainage composite placed along the cut slope and tied into 6-inch diameter perforated pipe at the toe.

The design of the synthetic rock facing was provided by Caruso, Turley Scott, Inc. of Tucson. The system consists of prefabricated glass-fibre reinforced concrete panels, attached to the welded wire wall with nylon straps. The approximate 3-inch spacing between the wall face and the panels were finished with 3000 psi grout.

Construction of the welded wire wall proceeded smoothly. After the welded wire wall was completed, construction of the rock facing began. The wall facing subcontractor, The Larson Company of Tucson, started at the top tier and worked down.

When the contractor had finished the top two walls and begun the third, a couple in the parking lot were overheard discussing the construction. The wife asked the husband what they were doing and he replied that it was obvious, they were building a retaining wall to support that rock above it.

Most public reaction to the project was positive, in spite of an editorial which referred to the wall as "an ugly beast," and that it is not possible to duplicate nature.

The overriding public sentiment for the project was expressed in an editorial referring to the wall as "an image of beauty corresponding with the surrounding environment," and thanking ADOT for "the great work done so we can all enjoy the beauty of the Oak Creek Canyon as it really is."

Construction was completed in the Summer of 1996, and the resulting structure is an aesthetically pleasing one.

ARIZONA STATE HIGHWAY SYSTEM

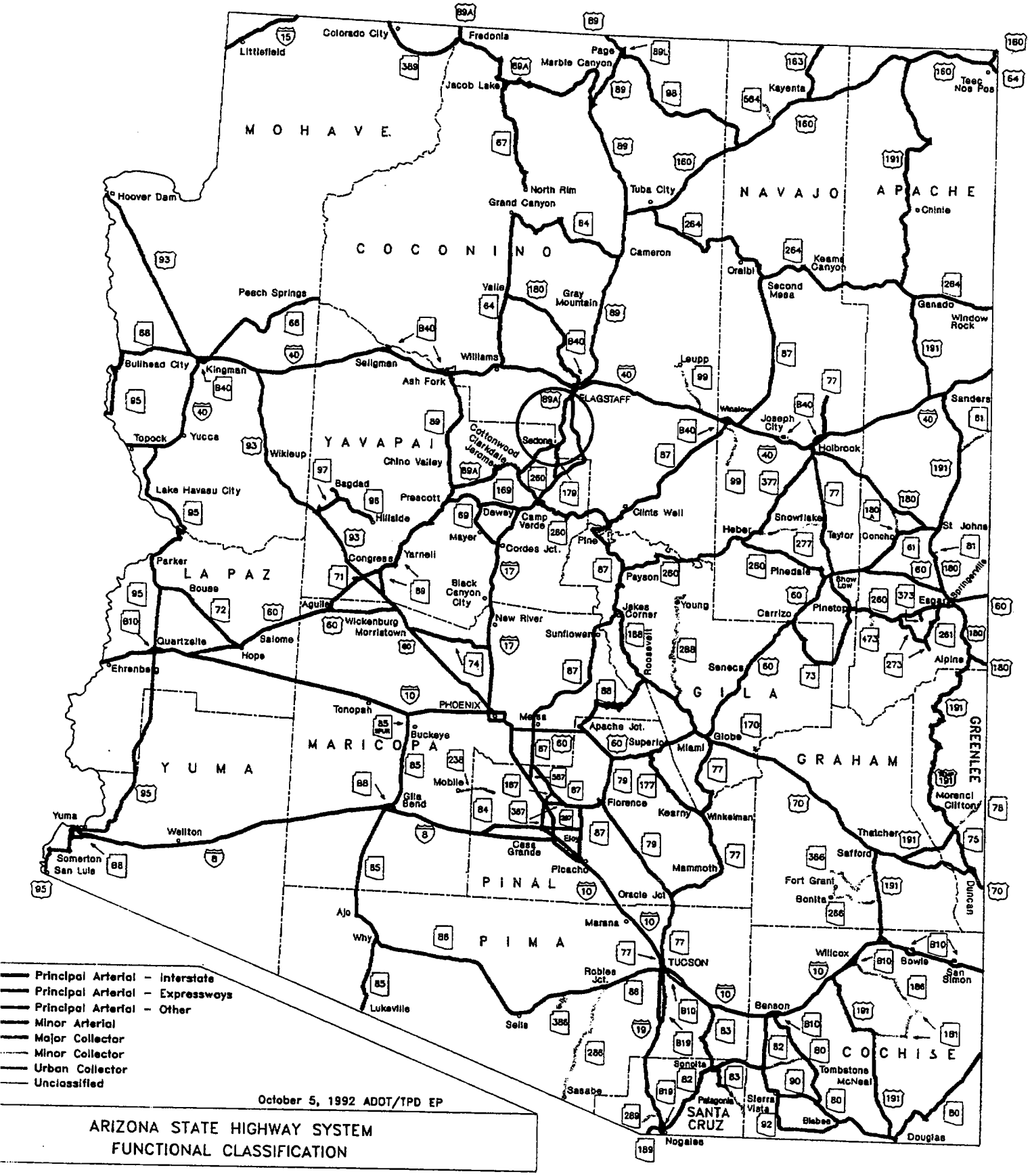
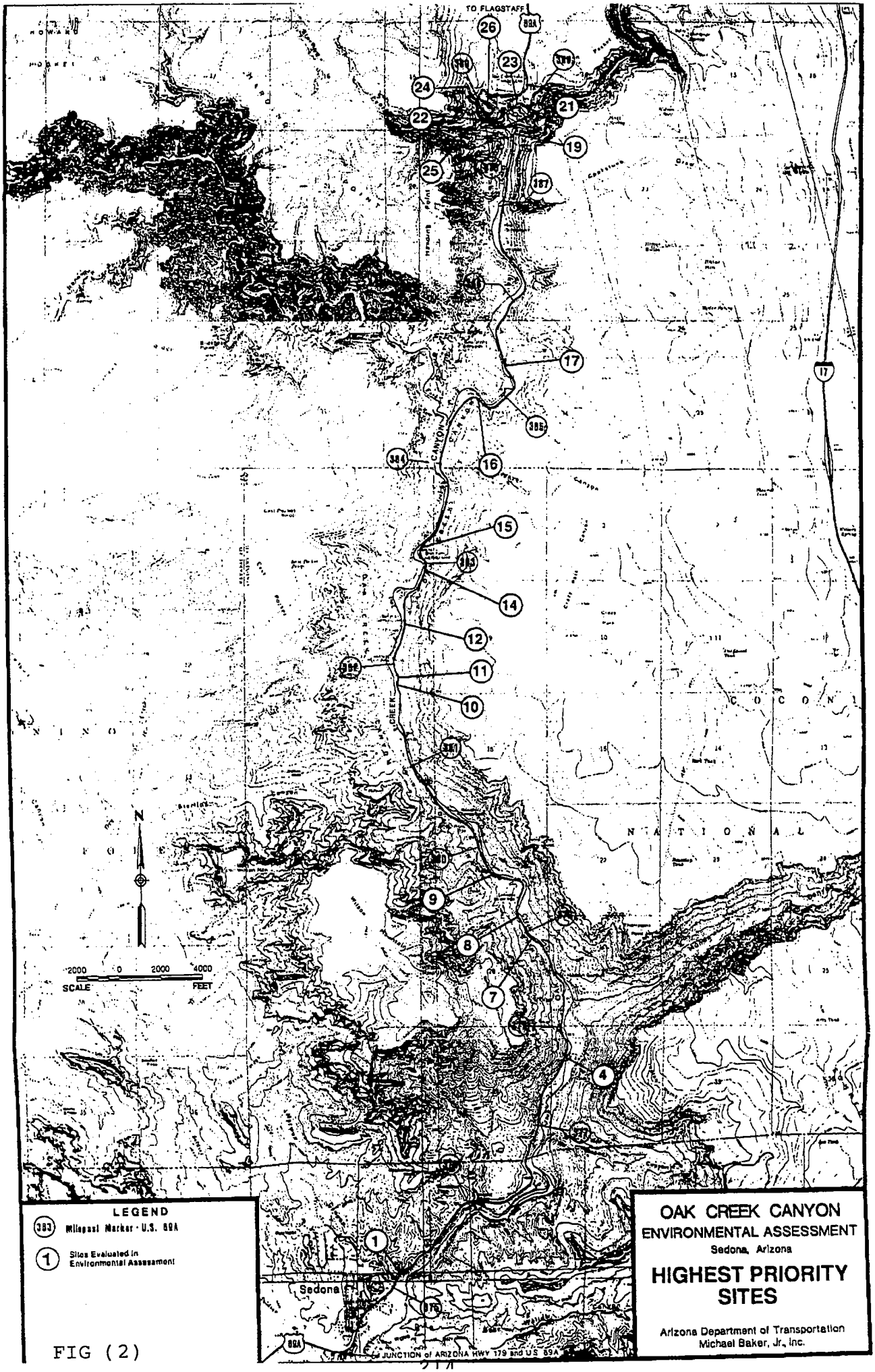


FIG. (1)


TO FLAGSTAFF
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1 Sites Evaluated in Environmental Assessment
OAK CREEK CANYON
ENVIRONMENTAL ASSESSMENT
Sedona, Arizona
HIGHEST PRIORITY SITES
Arizona Department of Transportation
Michael Baker, Jr., Inc.
JUNCTION of ARIZONA HWY 179 and U.S. 89A

FIG (2)

FIGURE (3)

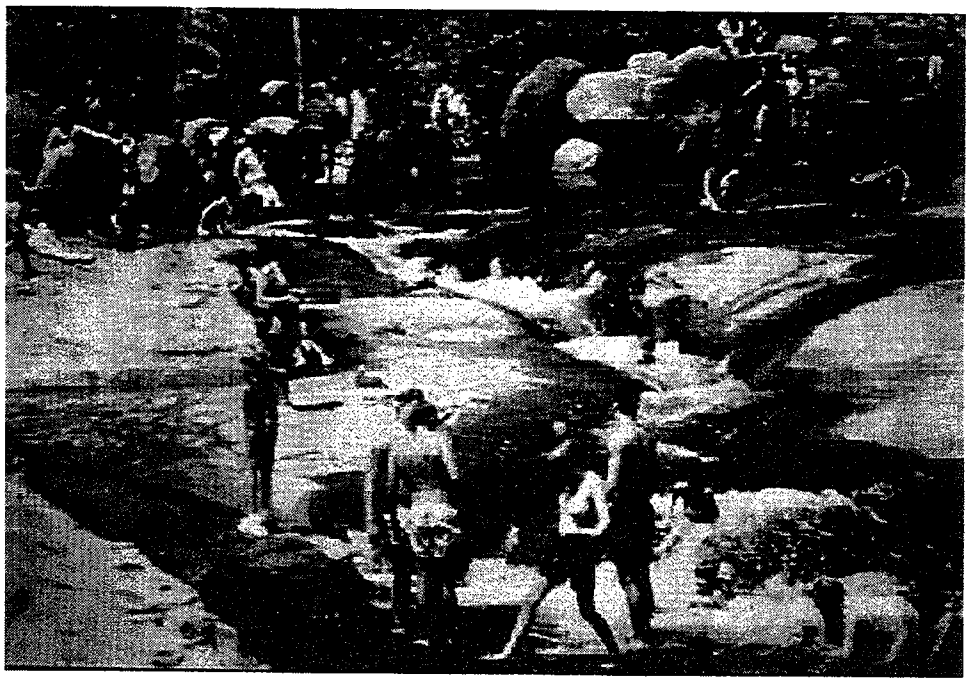
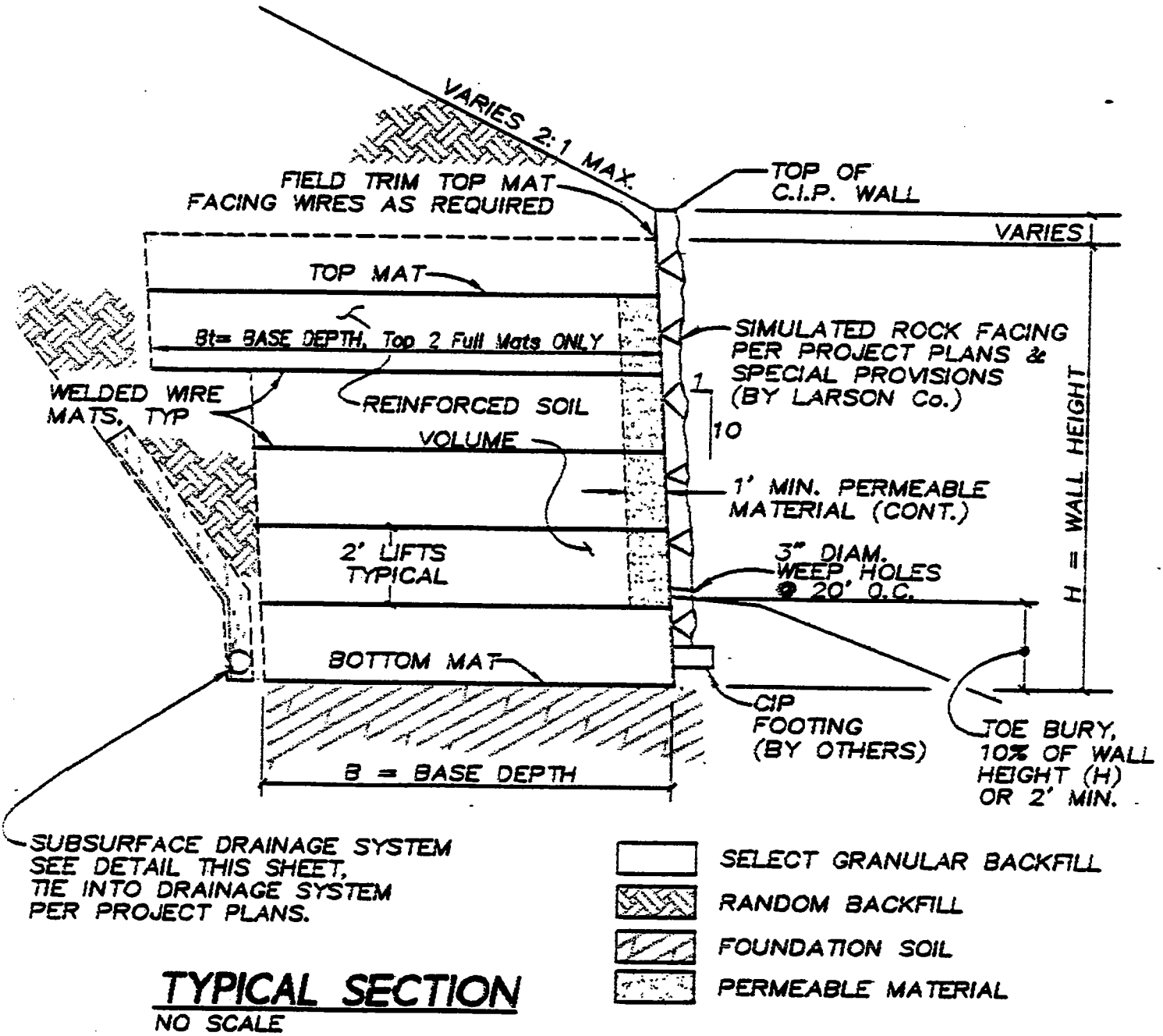


FIGURE (4)

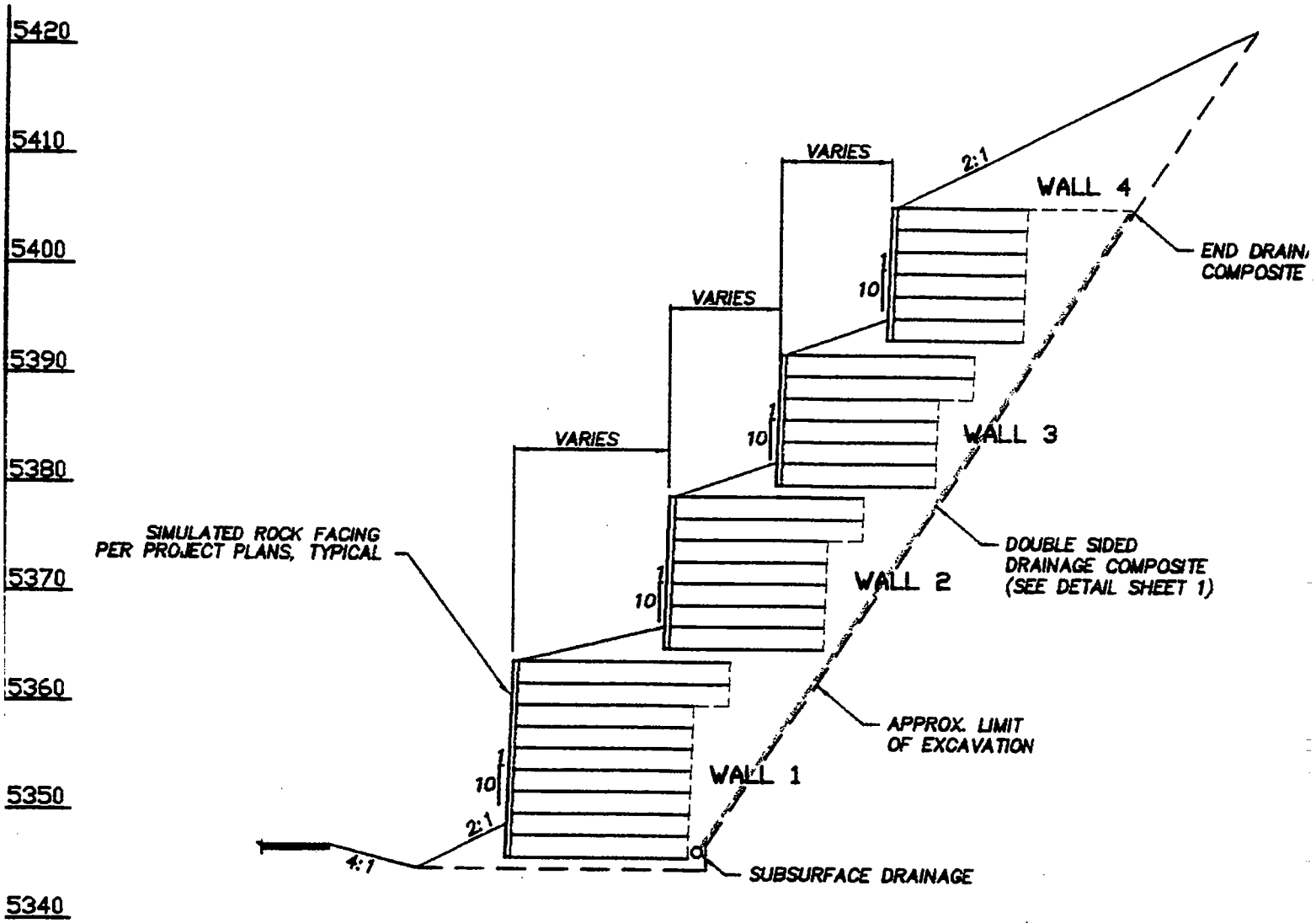
FIGURE (5)



TYPICAL SECTION

NO SCALE

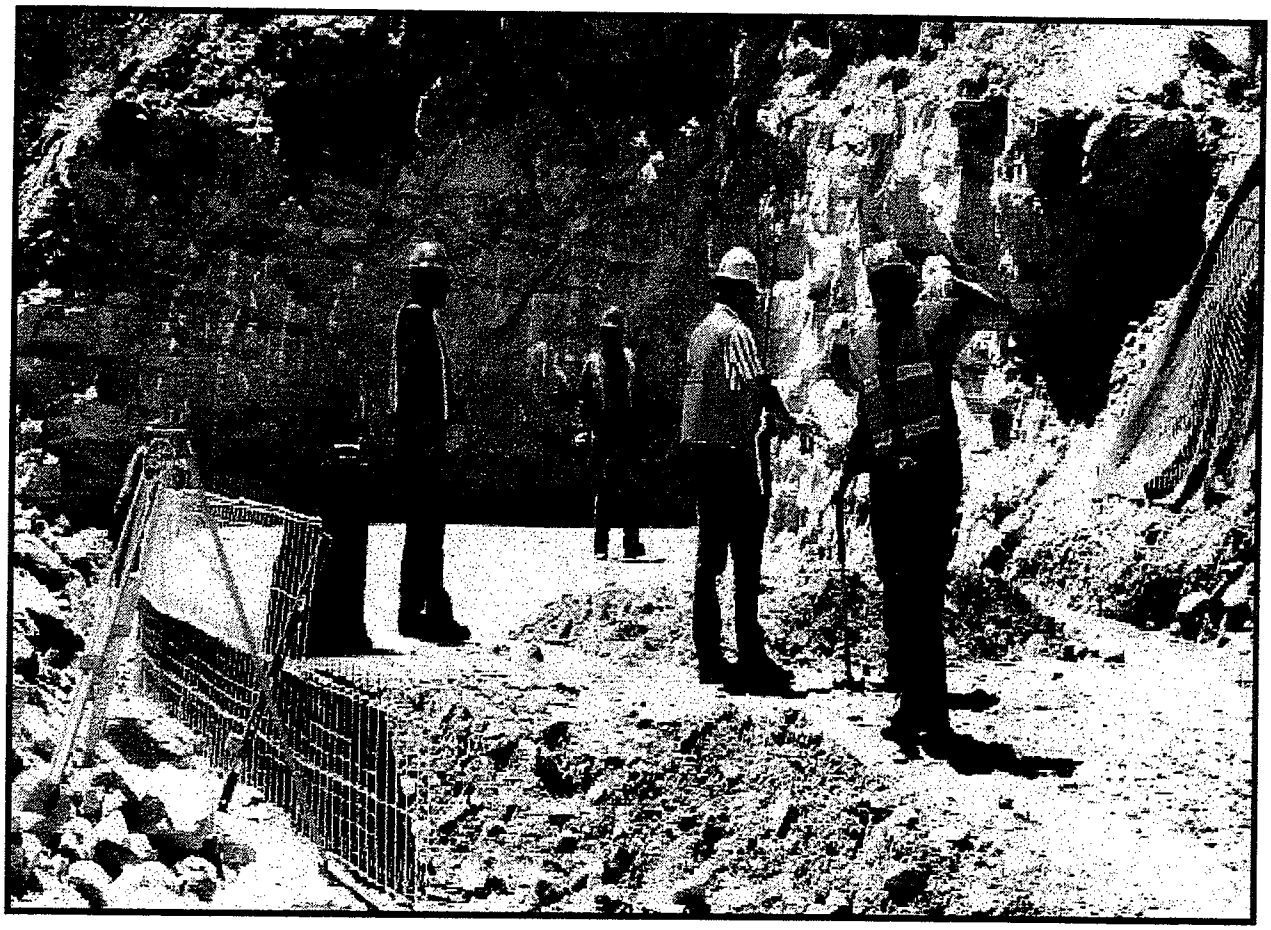
FIG (6)

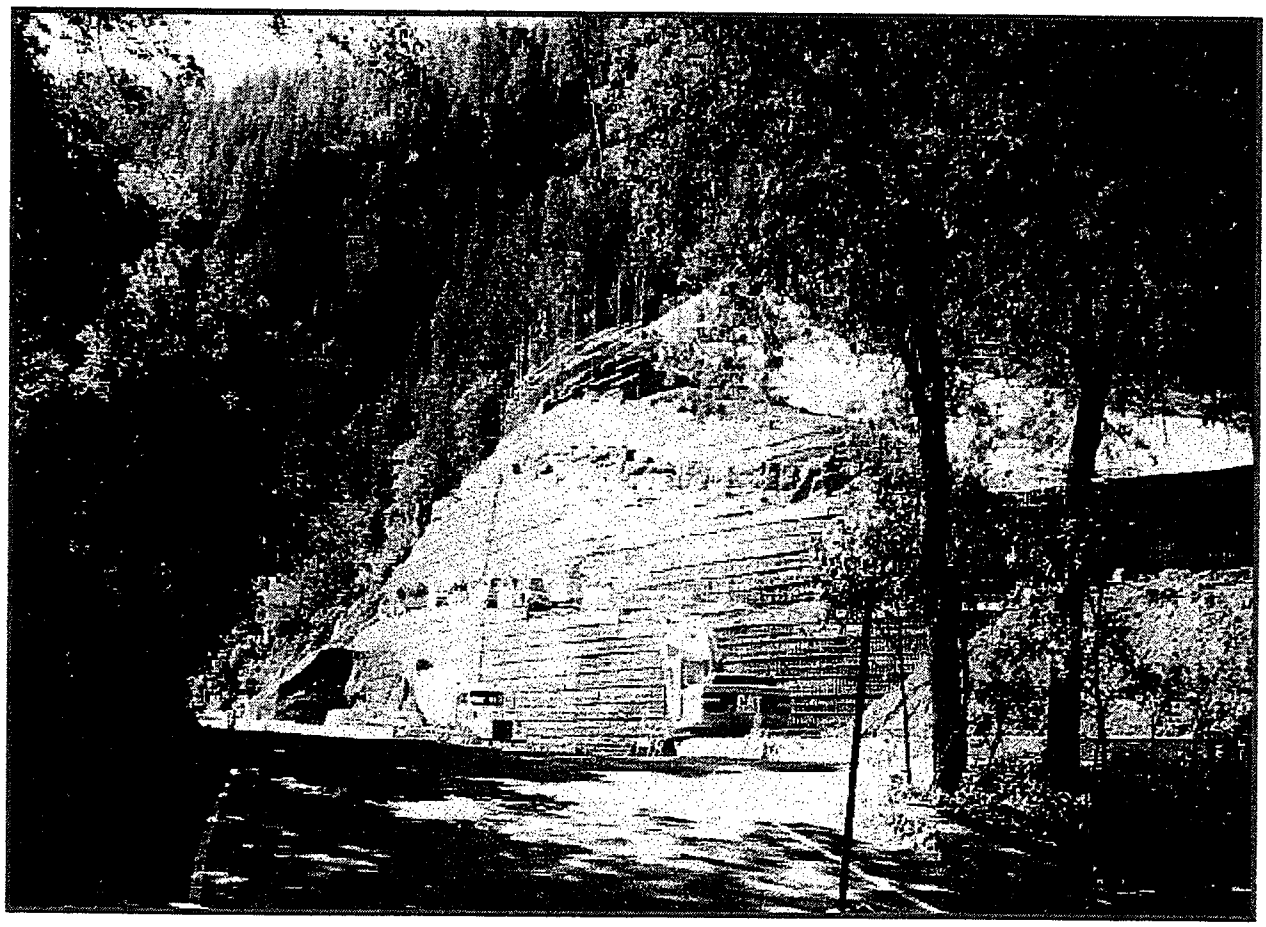


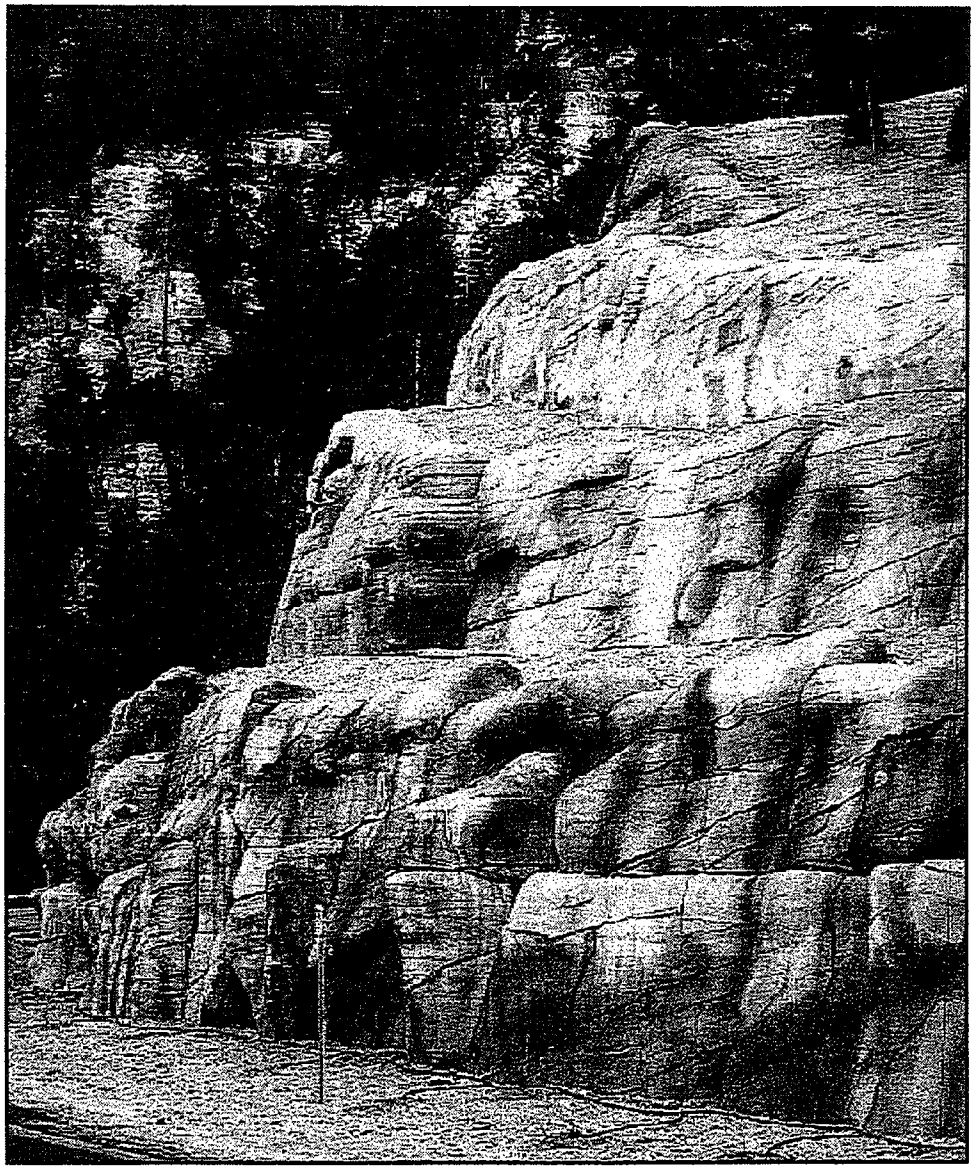
SECTION A-A

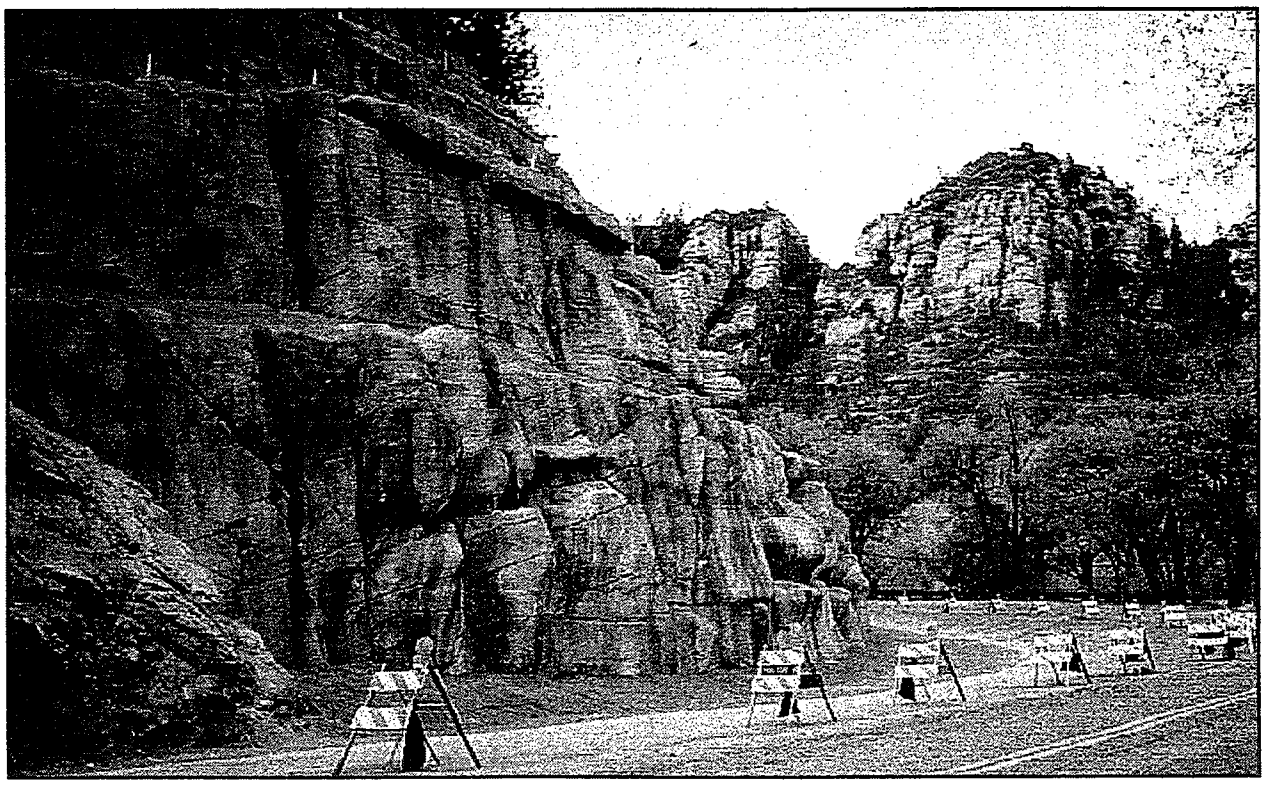
NOT TO SCALE

FIG (7)









Compaction Grouting of a Deep Embankment Fill along Interstate I-15, Virgin River Gorge, Arizona

Gary E. Taylor[1], Glen A. Gorski[2] and Douglas E. Alexander[3]

ABSTRACT

A portion of Interstate I-15, at the mouth of the picturesque Virgin River Gorge in northwestern Arizona, was constructed during the mid 1960's and early 1970's on shot-rock and coarse granular fills placed approximately 75 feet high within a deep ravine. The ravine is bounded on both sides by high-angle faults which intersect within the area of distress. These faults, containing thick clay gouge zones, provide a preferred flow path for hot springs flowing from the underlying Callville Limestone, Redwall Limestone, and Quaternary conglomerate through the embankment fill to the adjacent Virgin River channel.

Since completion of the interstate, up to several feet of embankment fill settlement has resulted in a continual paving maintenance problem. Indications are that these settlements were primarily due to ground water percolating through the fill and flushing out the finer grained material within the coarse (cobble-boulder) granular fill. As a result, voids and subsequent settlement occurred within the embankment. The embankment was potentially unstable due to the loose and vuggy nature of the granular fill, the presence of flowing groundwater, and potential local seismicity.

A rigorous ground modification program was designed to both densify and stabilize the embankment fill with a unique subsurface structural design consisting of arching compaction grout columns. During the compaction grouting program, abnormally high grout takes were recorded at several primary grout locations. Since placement of large volumes of grout, under pressure, in individual locations could lead to potential slope failure, the design grouting program was immediately re-evaluated. An on-site test section was performed, leading to modifications to the grouting program that addressed potential slide concerns while still meeting project objectives.

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[3] Foundation and Geologic Investigation Engineer, Arizona Dept. of Transportation

This paper discusses selection of compaction grouting, original and modified grouting designs, execution of the grouting program, and performance since the installation.

INTRODUCTION

A portion of U.S. Interstate Highway 15 (I-15) passes through the northwestern corner of Arizona between Nevada and Utah, entering the picturesque narrows of the Virgin River Gorge (Figure 1). Grading in the area began in the early 1960's, and paving was completed in late 1974 to early 1975. The highway consists of a four lane asphalt roadway, with two, twelve-foot lanes, a ten-foot outside shoulder and a two-foot inside shoulder in each direction. Northbound and southbound lanes are separated by a median Jersey barrier.

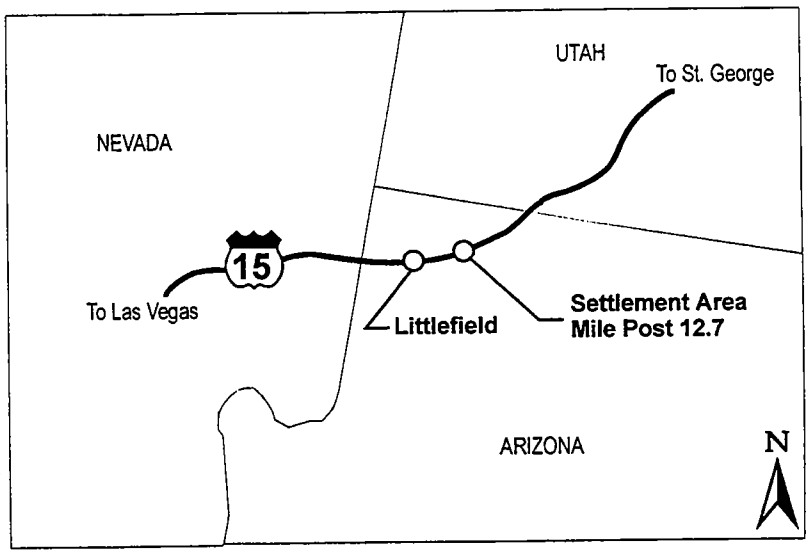


Figure 1. Site Location Map

At milepost 12.7, the highway had experienced over 17 inches of settlement, evidenced by observations of the profile of the Jersey barrier and thickness of the asphaltic concrete recorded. Although the settlements in the area did not present an emergency situation, they were considered to be a significant maintenance problem that would continue for many years if not corrected. To provide a permanent solution to the settlements and minimize future maintenance costs, Arizona Department of Transportation evaluated several remediation options, ultimately selecting compaction grouting to densify and stabilize the embankment fill.

GEOLOGIC AND GEOTECHNICAL CONSIDERATIONS

The site is located near the border of the Mountain Region of the Basin and Range and Transition Zone Physiographic Province. The settlement area coincides with a natural drainage trough which flowed through a faulted zone of the Calville Limestone, a late Pennsylvanian to early Permian marine limestone. This formation also includes some shale and mudstone beds that were formed from continental sedimentary deposition.

The maximum topographic relief in the area is approximately 6,000 ft. from the Virgin River elevation to Mount Bangs Summit elevation (±8,000 ft.), which lies in the Virgin River Mountains located about 6 miles south of the site. Mount Bangs marks the local divide between the north and west drainage into the Virgin River and the drainage south into upper Lake Mead.

Cutting through the south end of the Virgin River Gorge, Interstate 15 meanders through a series of complex, thrust-faulted and folded Mississippian to early Permian marine limestone formations dipping 35 to 45 degrees west.

At the west end of the settlement area, a north-south trending, high-angle fault (Redmont Fault), between Quaternary conglomerate and the Calville Limestone, is exposed in the road cut. In addition, a north-west trending high-angle fault is exposed in the road cut at the east end of the site. These two faults intersect in the area of the roadway settlement, which provides the preferred drainage trough for surface water flow and also the likely subsurface pathway for thermal springs. Limestone, gravel, cobbles, and boulders used to fill the original drainage trough had a low percentage of silts and sand. There is evidence that spring water has soaked through the porous rock fill, adding to the settlement problem.

SELECTION OF REPAIR DESIGN

Alternative repair designs considered by Arizona DOT were:

1. Compaction grouting to full depth.
2. Compaction grouting below 30 feet followed by dynamic compaction of the upper 30 feet.
3. Dynamic compaction of the upper 30 feet only.
4. Removal and replacement of fill material.
5. Cement slurry grouting.
6. Installation of a bridge structure.

Compaction grouting to full depth was selected on the basis that this technique would have the least impact on traffic flow, would not create a groundwater barrier which may increase hydrostatic pressures within the embankment fill, would be cost-effective, and could meet project schedule requirements.

COMPACTION GROUTING TECHNOLOGY

Compaction grouting is an in-situ method of improving ground conditions by displacement. As the very viscous, low-slump grout is pumped in stages, homogeneous grout bulbs are formed that infill voids and displace, densify and thus strengthen the surrounding soil. By sequencing

the grouting work from primary to secondary, and, if required, tertiary locations, significant improvement can be achieved. Compaction grouting is effective in all granular soil matrix conditions, provided the clay content is low.

Originally developed in the 1950's for correcting building settlements, compaction grouting technology has since evolved to treat a wide range of subsurface conditions for both new construction and remedial work.

COMPACTION GROUTING PROGRAM DESIGN

Traffic control design during the repairs required that a minimum of two lanes in both the northbound and southbound directions remain open. To meet these requirements, the shoulder was temporarily extended and paved in order to widen the area to accept three separate work areas, each approximately thirty feet in width, for equipment and material access. Figure 2 shows a photograph of the jobsite layout and traffic control.

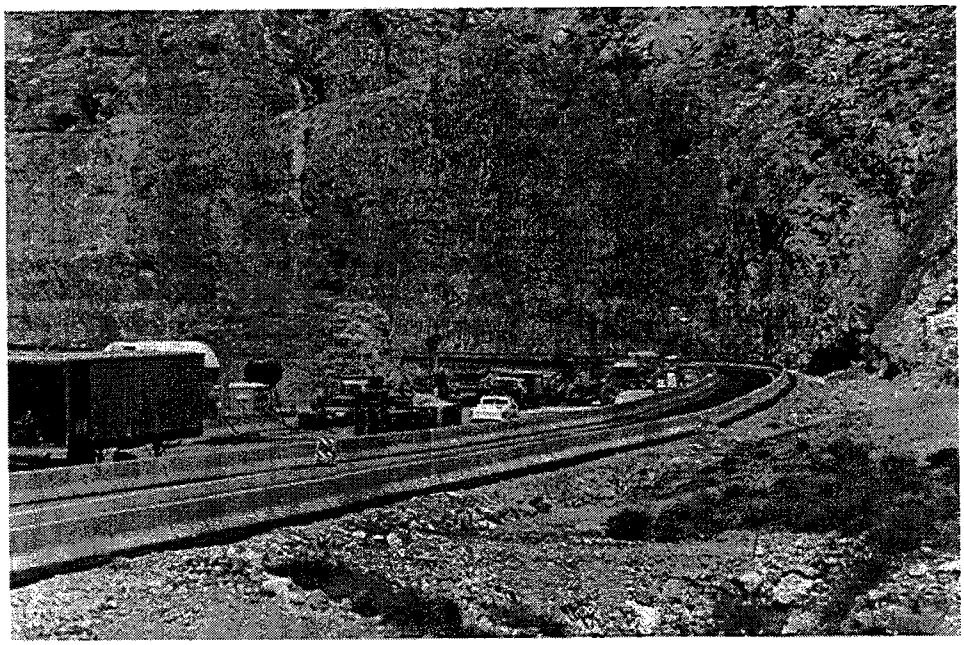
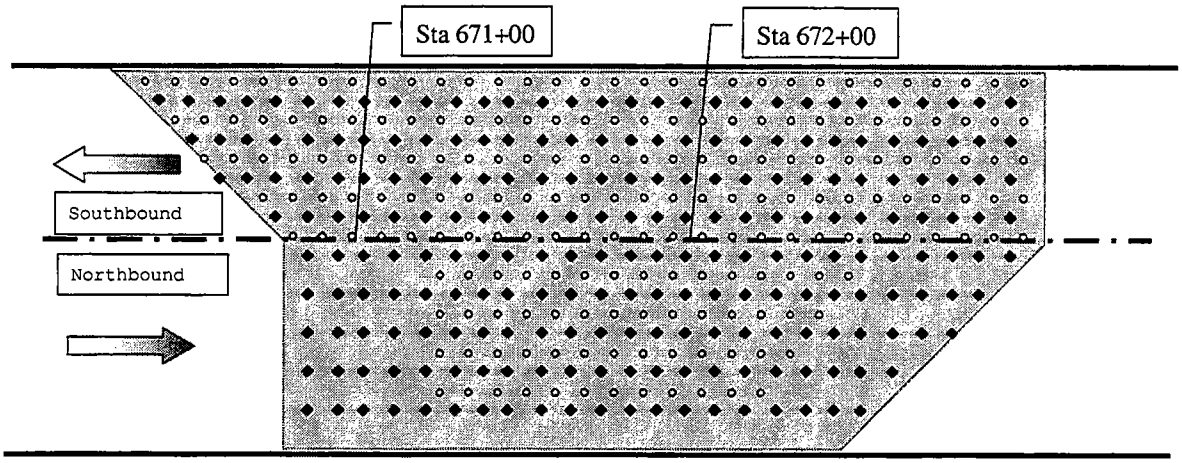


Figure 2. Jobsite Layout and Traffic Control

The drilling and grouting program consists of a primary ten foot square grid pattern, with secondary grout locations at the centroid of the primary grid for a net grout pipe spacing of 7.1ft. (Figure 3).

Depths of grout casings varied from 10 to 97 feet across the site in a trough shaped geometry approximating the natural ravine which existed prior to the emplacement of the embankment fill. The depths were dictated by founding the bottom elevation of the compaction grout column within bedrock (Figure 4).



◆ -Primary Injection Point

° -Secondary Injection Point

Figure 3. Layout of Primary and Secondary Grout Injection Points

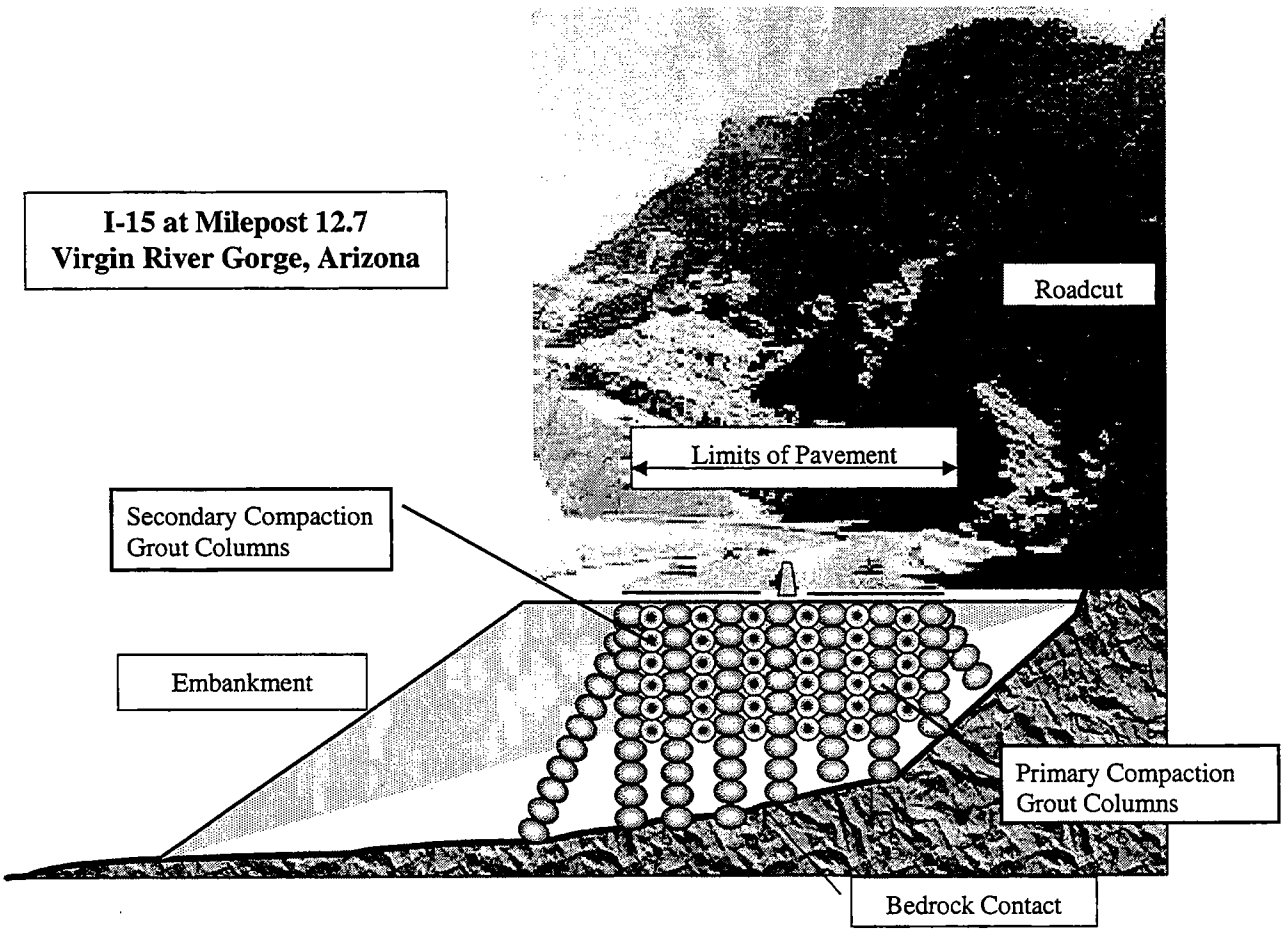


Figure 4. Idealized Cross Section Through Deep Portion of Compaction Grouting Zone

Grouting was typically done in 2.5 foot stages, with a maximum primary pumping rate of three cubic feet per minute and secondary pumping rate of three cubic feet or less per minute. The typical grouting sequence began in the deeper fill sections near the river and progressed toward the road cut adjacent to the northbound lanes.

Drilling

The drilling equipment selected for this project consisted of two, air-powered rotary percussive track drills which utilized an Odex/Tubex overburden drilling system (rotary percussive concentric duplex). The drill string uses three inch inside diameter casing with a concentric drill bit capable of drilling through diverse soil conditions to depths up to 200 feet. Casing lengths varied from five to ten feet. Drill cuttings were flushed from the borehole using air and dense drill foam.

Drilling through the vuggy limestone boulders and cobbles was difficult. Due to the porous nature of the fill, return of the drill cuttings often times caused plugging within the casing. Thick concentrations of drill foam suspended the cuttings and provided the proper medium to force the spoils to the surface.

In addition, some boulders were encountered that were as much as 6 feet in cross-section. This made the driller's determination as to whether the grout column was founded on bedrock difficult. To provide proper seating, drill depths of previously installed adjacent primary locations were studied and determinations to continue the drill advancement were made in the field. Depths encountered during the installation of secondary grout locations was less critical. It was determined that the secondary compaction grout locations would bridge between the adjacent primary grout column to form a compaction grout "arch". The bottom of the secondary injection points were left hanging above the bedrock contact (see Figure 3) as the fill below had been partially densified during the primary injection. The spacing of the injection points was close enough to result in the transfer of load from the secondary columns to the primary columns, creating the arching effect. This methodology was also thought to be beneficial in maintaining flow paths for the ground water that likely flows along the fill / bedrock contact.

Compaction Grouting

Compaction grouting methodology consisted of a typical procedure of extracting the casing by utilizing hydraulic rams and breaking the casing by conventional hand tools. Grout injection via the grout header was monitored by observations of fluid-isolated pressure gauges, located in both the header and the grout pump.

Grout slump was generally maintained at less than one inch. As with most compaction grouting projects, the acquisition of suitable unblended sand with the proper gradation was difficult. Local geology either provides natural soils that are high in clay content or high in sand content. A well-graded, silty, fine-grained sand that is necessary to allow pumping under high pressures without water bleed and subsequent 'sand blockage" during hot temperatures was not readily available. Therefore, the grout material selected consisted of "reject" asphalt fines that contained, on average, the gradation shown in Table 1.

As Built		As Specified
Percent passing	*U.S. Sieve No.*	*Percent passing*
100	0.5	100
100	0.375	
97 - 98	4	85 Min.
79 - 84	8	
65 - 71	16	
55 - 59	30	
48 - 52	40	
37 - 39	50	
16 - 22	100	
9 - 15	200	20 Max.

Table 1. Asphalt Fines Gradation

Cement content varied from 12 to 15 percent by weight to maintain the specified unconfined compressive strength of more than 240 psi at 7 days. Typical grout strengths of over 500 psi at 7 days were common with the "reject" asphalt fines material.

Compaction grouting was performed typically in 2.5 foot stages until one of the following occurred:

1) Grout pressures exceeded 600 psi. as measured at the grout header.

2) Injected grout volume of 75 cubic feet per 2.5 foot stage

3) A sudden drop in grout pressure, indicating soil shear.

4) A grout pumping rate of 0.75 cu. ft. / min. at 400 psi was reached.

5) Surface heave exceeded one inch.

After completion of approximately 30 percent of the primary locations in the section of southbound lanes (embankment portion), several primary grout locations achieved the grout quantity refusal criteria of 75 cubic feet in the 2.5 lineal foot stage. As a consequence of placing large grout volumes in a single stage, significant deformation and separation failures of adjacent, previously installed, grout casing occurred. In addition, placement of large volumes under pressure in a single location may lead to uncontrolled lateral movements and possible slope failure. In consideration of the very loose and vuggy nature of the soils, it appeared that the embankment was potentially unstable, especially given the groundwater conditions and the local seismicity. Grouting was suspended while the owner and the specialty grouting contractor evaluated the design program.

RE-EVALUATION OF GROUTING DESIGN

In order to have a better understanding of the fill material, a test section was performed. Primary locations along the first two parallel rows adjacent to the slope face were grouted in a square closure pattern until the specified refusal criteria was achieved. Following the primary injections, the intermediate secondary locations were drilled and grouted. If secondary grout takes were reduced significantly from the primary locations, recommendations were to reduce the grout quantities by fifty percent to provide more uniform densification.

Concurrent with and following the completion of the test section two decisions were made in order to continue the grouting program and maintain project schedule:

1. First, that primary grout locations would be injected on 20 foot spacings rather than ten foot spacings to minimize casing deformation and prevent large concentrations of unsolidified grout surcharging the fill.

2. Secondly, and less frequently, that grout volumes introduced into the primary locations would be reduced by fifty percent, thus allowing the more uniform densification discussed above.

Implementation of Modified Grouting Program

Following resumption of grouting, typical grouting records for primary locations near the deepest section of the embankment fill indicated high pressure refusal (600+ psi.) occurring randomly through the fill. In cases where maximum volume criteria was achieved, back pressure and injection pressures were typically 100 to 200 psi. and 100 to 600 psi. respectively. Higher secondary pressures typically indicate grout closure.

Secondary locations showed significantly less cases where maximum volume refusal criteria was achieved. Refusal pressure criteria was commonly achieved at 70 to 80 percent of the grout stages and in some cases 100 to 200 psi. injection pressure drops were observed within the upper 20 to 30 feet. Surface heave (less than 1/16 inch) typically occurred at each 2.5 foot stage within the upper 20 feet at both primary and secondary locations.

A total of approximately 13,000 lineal feet of drilling required approximately 4,100 cu. yds of compaction grout. Following the compaction grouting treatment, the upper five feet of the work area was removed, recompacted, storm drains replaced, and surface repaved.

CONSTRUCTION MONITORING AND VERIFICATION TESTING

During construction, horizontal rotating lasers were used to monitor surface movement and minimize heave. Review of the secondary grout takes and pressures indicated that closure and significant densification had been achieved. Since completion of the project, in 1993, Arizona DOT had carried out periodic, visual inspections. The area has not shown any significant signs of settlement.

SUMMARY

The use of compaction grouting in this loose, porous, rocky fill provided the necessary improvement to reduce maintenance and alleviate potential failure of the embankment. By densifying the in-situ fill and grouting the voids, the problems potentially associated with local seismicity were also reduced. This procedure also provided the least disruption to traffic flow and was flexible enough to overcome difficult drilling conditions and large variations in the embankment fill material.

PALEOSEISMOLOGY AND SEISMIC HAZARD ASSESSMENT IN ARIZONA

by Philip A. Pearthree
Research Geologist
Arizona Geological Survey

Abstract

Paleoseismology involves the use of geologic and geomorphologic techniques to study large prehistoric earthquakes that have ruptured the ground surface. Paleoseismologic investigations may involve reconnaissance investigations of suspected young faults, detailed geologic mapping of faults, interpretation of trenches excavated across faults, analysis of subsurface information, and dating of geologic units. Such investigations are a critical component of seismic hazard assessments because the historical record is very short relative to the time between large earthquakes on most faults. Investigations of individual faults can provide information on the age of youngest fault rupture, the typical displacement across the fault during large earthquakes, the length of fault rupture, and the long-term rate of slip across the fault. These data can then be used to estimate magnitudes of earthquakes generated by the fault and how frequently these earthquakes occur.

During the past few years, geologists of the Arizona Geological Survey have conducted paleoseismologic investigations of young faults and have integrated this information into seismic hazard assessments. We investigated the Sugarloaf fault, in the mountainous Transition Zone northeast of Phoenix, for the Arizona Department of Transportation. We found evidence that the fault ruptured and generated a moderately large earthquake (M ~6.5 to 6.7) between 5,000 and 20,000 years ago. We concluded, however, that the long-term slip rate on the fault is very low, and that average time between large earthquakes is at least 50,000 years. In a probabilistic hazard analysis, the impact of this fault is not great. We have also studied the many young faults in the Flagstaff area of northern Arizona. In that region, we concluded that all of the faults have very long recurrence intervals like the Sugarloaf fault. Because there are as many as 50 potentially active faults in the region, however, their cumulative impact on probabilistic hazard assessments is significant.

The current state of knowledge of potentially active faults in Arizona is summarized in a new database and map that we have compiled in cooperation with a U.S. Geological Survey program to map young faults around the world. Quaternary faults have been documented throughout most of Arizona, with the exception of the interior of the Colorado Plateau in the northeastern section of the state. This implies that large, surface-rupturing earthquakes could occur throughout most of the state. Arizona faults are not very active by world-wide standards, however, with slip rates ranging from about 0.01 to 0.2 mm/year (2 to 3 orders of magnitude less than the San Andreas fault), so the probability of a large earthquake occurring in Arizona in 50 or 100 years is fairly low.

Introduction

Geologic and geomorphologic investigations of known or suspected young faults (paleoseismology) provide critical data for seismic hazard assessments. The historical record of earthquakes varies from decades to centuries in different portions of the earth, and the instrumental record of earthquakes typically is much shorter. These records are very short relative to the time between large earthquakes on most faults, so the historical record long enough to permit scientists to delineate all potentially active faults or to confidently assess the rate of occurrence of large earthquakes on individual faults. Paleoseismologic investigations focus on documenting evidence for the occurrence of prehistoric earthquakes that were large enough to rupture the ground surface. These investigations provide no information about small and moderate earthquakes, but they can provide a record of the occurrence of large earthquakes that is hundreds of years to hundreds of thousands of years long. Because large earthquakes have the potential to cause tremendous damage and loss of life, they are the earthquakes that are most important in seismic hazard assessments. The paleoseismic record typically includes multiple large earthquakes on individual faults, so it provides information with which to estimate recurrence intervals between large earthquakes. These data can in turn be incorporated into probabilistic seismic hazards assessments that attempt to quantify acceleration levels that a given site might experience over 50, 100, or 250 years.

Paleoseismologic investigations may be conducted at reconnaissance or detailed levels, depending on the scope of the investigation and the amount of funding available. Relatively low-cost regional reconnaissance studies serve to identify young faults that could potentially generate large earthquakes. Methods that are commonly employed include interpretation of aerial photographs, aerial reconnaissance in light planes or helicopters, evaluation of topographic maps, and reconnaissance field studies to briefly examine relationships between faults and geologic units of various ages. From these studies, we may be able to assess whether a fault has been active during the past 10,000 years (the Holocene), the past 130,000 years (the late Pleistocene and the Holocene), or the past 1.6 million years (the Quaternary). We might also be able to roughly estimate how frequently the fault ruptures, based on the amount of displacement of geologic units of increasing age. Detailed studies of individual faults provide much more information about fault behavior at substantially greater cost. Detailed studies focus on confirming that the fault is active and estimating the ages of young fault ruptures, the amount of slip on the fault during large earthquakes, the length of fault rupture, and the long-term rate of slip across the fault. Investigations may involve detailed geologic mapping of faults, excavation and interpretation of trenches across faults, topographic profiling of faulted landforms and analysis of subsurface information to measure fault displacement, and dating of faulted and unfaulted geologic units.

Paleoseismologic investigations that have been conducted in Arizona during the past two decades have provided valuable data regarding the potential for large earthquakes in Arizona. Examples of recent detailed investigations of individual faults and regional reconnaissance studies in Arizona are briefly summarized in the next section of this paper. The seismic-hazard implications of these paleoseismologic investigations are considered as well. The final section of this paper is an overview of young faulting and seismic hazard across Arizona based on a recently compiled Quaternary fault database for the state.

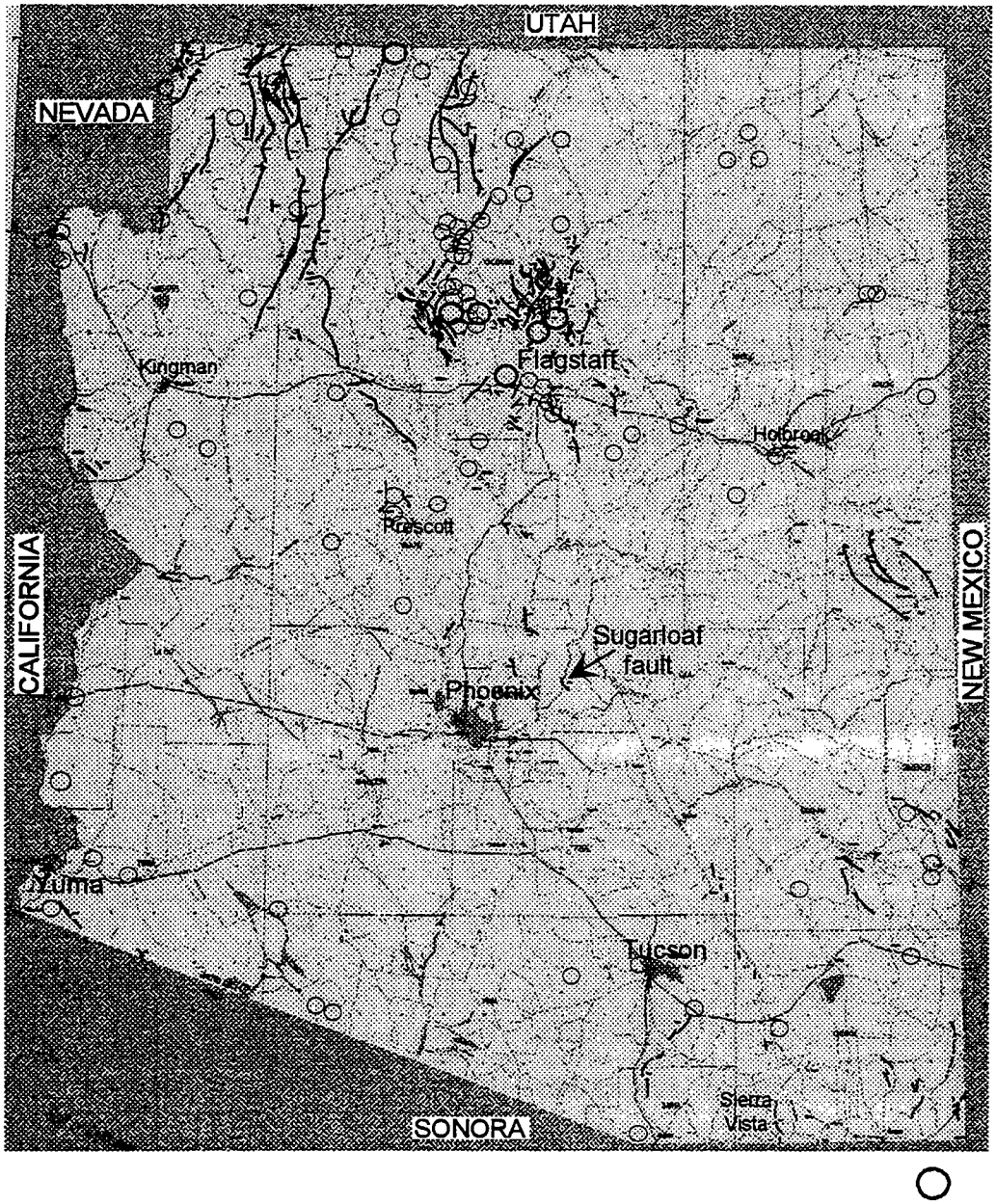


Figure 1. Map of known Quaternary faults and historical epicenters in Arizona. Faults are depicted with bold lines. Light, small circles depict earthquakes between magnitude 3 and 5. Medium, bolder circles depict earthquakes between magnitude 5 and 7. Largest circles are earthquakes larger than 7. The fault map was generated from digital data provided by R. Dart and L.A. Bradley of the U.S. Geological Survey.

Case Studies

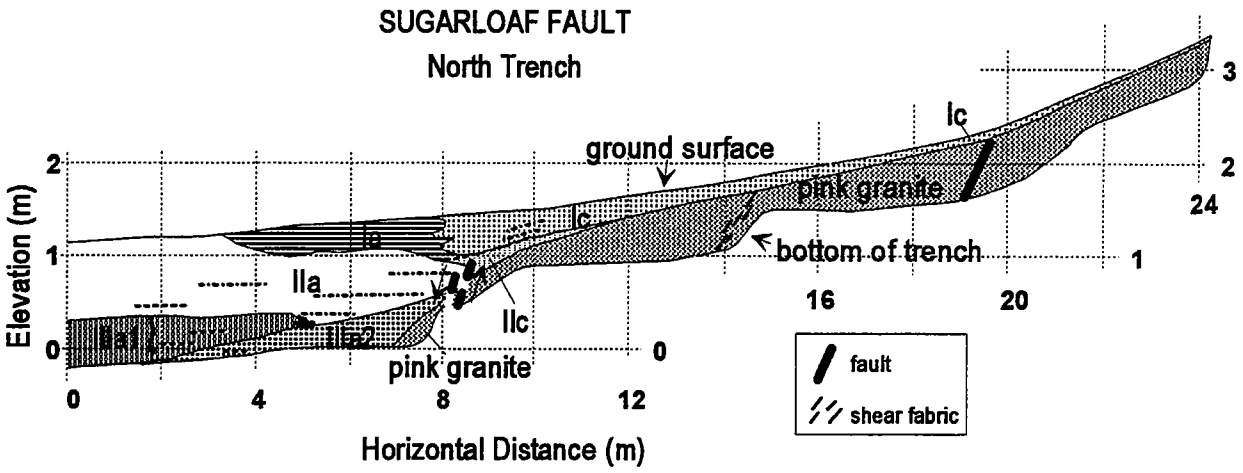
This section summarizes the results of two recent seismic hazard studies conducted in Arizona that rely heavily on paleoseismic data. In each case, geologic investigations were conducted to gain information regarding fault activity, paleoearthquake magnitudes, and recurrence intervals between earthquakes on the faults. The hazard implications of the paleoseismic data gained from these studies were evaluated by integrating the data into new probabilistic seismic hazard assessments.

Sugarloaf fault in central Arizona. Heavily travelled State Route (SR) 87, which traverses the rugged topography of central Arizona between Phoenix and Payson, is currently being upgraded from two to four lanes. The highway crosses the Sugarloaf fault, a 12-mile (20 km) long, northwest- to north-trending normal fault with known Quaternary activity, and several new bridges were planned in the vicinity of the fault (see Figure 1 for location). With the support of the Arizona Department of Transportation, we conducted a multi-faceted paleoseismologic investigation of the Sugarloaf fault in order to assess the seismic hazard it might pose to the new SR 87 bridges (Pearthree and others, 1995).

The first facet of our investigation involved surficial geologic mapping and development of a chronologic sequence of the deposits along the Sugarloaf fault that could be used to evaluate the late Quaternary rupture history of the fault. We identified late Holocene stream deposits (alluvium), the upper portions of which locally contain artifacts from the Hohokam culture (600 to 1,200 years old); late Holocene hillslope deposits (colluvium) that interfinger with the late Holocene alluvium; latest Pleistocene colluvium and alluvium (estimated age 10,000-20,000 years based on soil development); and late Pleistocene colluvium and alluvium (estimated age 50,000 to 100,000 years based on soil development and geomorphology); dissected Tertiary alluvium; Tertiary basalt; and Precambrian granite.

The second facet of our investigation involved interpretation of two excavated trenches across the fault and several natural exposures (see Figure 2). Stratigraphic exposures reveal that late Holocene alluvium and colluvium are not faulted. A minimum constraint on the age of youngest faulting is provided by this unfaulted late Holocene alluvium, which was evidently deposited in the past few thousand years. Latest Pleistocene colluvium and alluvium are faulted, so the fault ruptured sometime between about 5,000 and 20,000 years ago. Both the latest Pleistocene deposits and late Pleistocene deposits are displaced the same amount, about 0.5 to 1 m. Thus, the fault evidently has hand only one large earthquake in about the past 50,000 to 100,000 years. This long recurrence interval is consistent with other faults in central and southern Arizona that have been studied in detail (for example, Pearthree and Calvo, 1987; Piety and Anderson, 1991).

The magnitude of surface-rupturing earthquakes generated by the Sugarloaf fault is another important parameter for seismic hazard assessment. Magnitudes of paleoearthquakes may be estimated using empirical regressions of magnitude vs. surface rupture length or maximum displacement developed from historical earthquakes (Wells and Coppersmith, 1994). The minimum rupture length used for the Sugarloaf fault encompasses the central 2.7 miles (4.3 km) of the fault, which has the sharpest geomorphic expression and clear evidence of late Quaternary



Unit Ic - late Holocene colluvium
Unit Ia - late Holocene alluvium
Unit IIc - latest Pleistocene colluvium
Unit IIa - latest Pleistocene alluvium
Unit IIIc - late Pleistocene colluvium
Unit IIIa - late Pleistocene alluvium

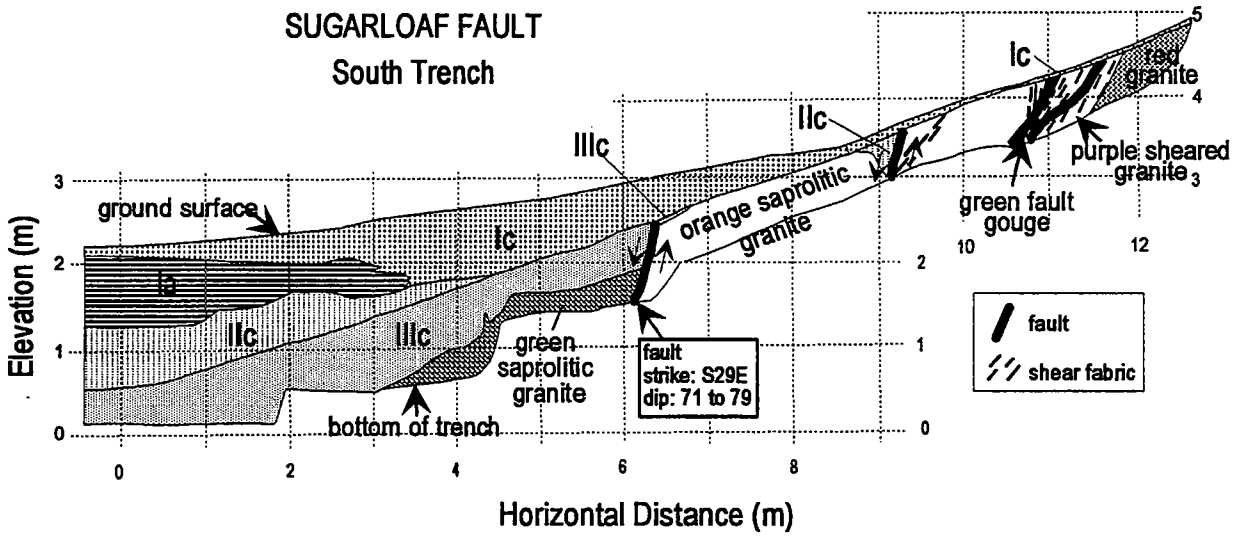


Figure 2. Logs of trenches excavated across the Sugarloaf fault. Alluvium and colluvium of late and latest Pleistocene age (units II and III) is faulted. Holocene deposits (unit I) are not faulted. View is from the north, horizontal distance is from east to west.

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movement. The southeastern 2.5 miles (4 km) of the fault is associated with a prominent bedrock scarp that diminishes to the southeast; it may have had late Quaternary activity. We found no definitive evidence of late Quaternary movement on the northern 7.5 miles (12 km) of the fault, but included it in our most conservative estimate of fault rupture length. The earthquake magnitude estimated from the minimum fault length of 2.7 miles is about 5.8; if the whole 12 miles of the fault ruptured in one earthquake, the estimated magnitude is about 6.6. The surface displacement associated with the youngest fault rupture was estimated from displacement of late and latest Pleistocene deposits in the gully and trench exposures. Vertical displacement of late and latest Pleistocene colluvium is about 1.6 to 2.3 ft (0.5 to 0.7 m). We consider 3.3 ft (1 m) of vertical displacement to be a fairly conservative maximum. Earthquake magnitudes estimated from 0.5 to 1 m of displacement range from 6.3 to 6.7.

We estimated probabilistic seismic accelerations associated with the Sugarloaf fault using the SEISRISK III program (Bender and Perkins, 1987). The Sugarloaf fault was not treated as a discrete earthquake source in previous state-wide probabilistic analyses (see Euge and others (1992). We therefore added the acceleration values associated with the Sugarloaf fault to the acceleration value for this area of 0.045 g developed by Euge and others (1992). Six combinations of maximum magnitude and recurrence interval were used to accommodate the substantial uncertainty in our estimates of paleoearthquake magnitude and the frequency of surface ruptures on the fault. The most conservative assumption, with a maximum magnitude of 6.7 and a recurrence of 50,000 years, yields an increase in the horizontal acceleration value with a 0.1 chance of occurring in 50 years (the 50-year acceleration value) of 0.05 g around the fault. This results in a 50-year acceleration value for the closest proposed highway bridge of slightly less than 0.1 g. All other scenarios produce smaller acceleration values. This increase in the 50-year acceleration for the bridge sites was not large enough to cause any design changes. Using the most conservative scenario, the horizontal acceleration with a 0.1 chance of occurring in 250 years (the 250-year acceleration value) associated with the Sugarloaf fault is as high as 0.11 g. Adding these to the background acceleration of 0.125 g from Euge and others (1992) results in an acceleration of about 0.24 g. This indicates that faults with fairly long recurrence intervals may pose a significant hazard to structures with long projected lifetimes.

Young faults and seismic hazard of the Flagstaff area, northern Arizona. Historical seismic activity and the presence of abundant young faults both suggest that there is significant seismic hazard in the rapidly growing Flagstaff area in north-central Arizona. Six earthquakes of magnitude 5 or greater have occurred in this area during the past 110 years; the largest of these was a magnitude 6.2 earthquake that occurred west of Flagstaff in 1912. No surface ruptures have been documented for these earthquakes, but many faults have been mapped in the Flagstaff area that cut Quaternary rocks of the San Francisco volcanic field. The fact that these faults cut fairly young rocks suggests that they are potential sources of large earthquakes. Previous probabilistic seismic hazard assessments that cover this area have yielded different acceleration values. In particular, the study conducted by Euge and others (1992) reported substantially higher acceleration values for this region than have other studies (see Table 1).

We provided another perspective on seismic hazard in the Flagstaff area based primarily on geologic assessments of the potentially active faults in the area (Pearthree and others, 1996).

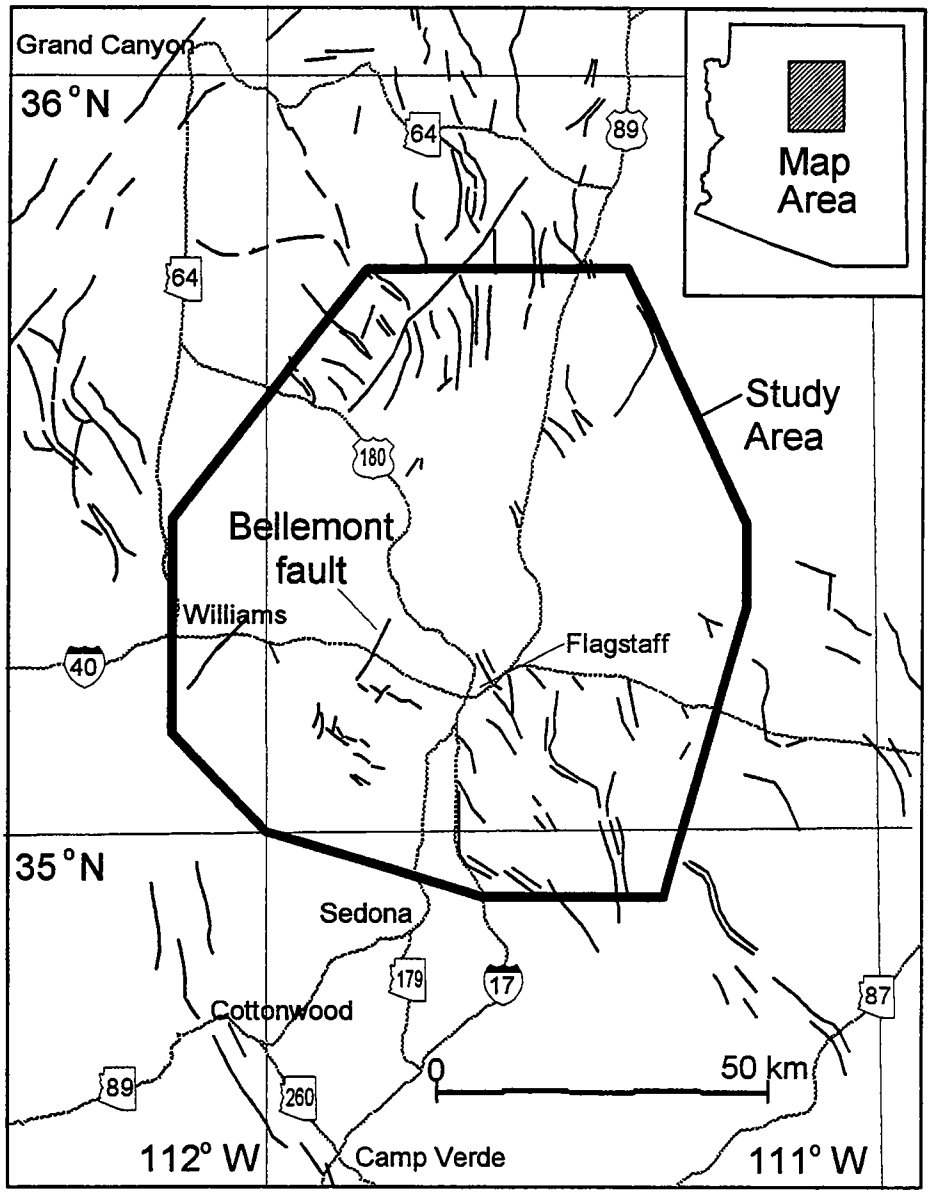

Figure 3. Young faults of the north-central Arizona. Map is modified from Euge and others (1992).

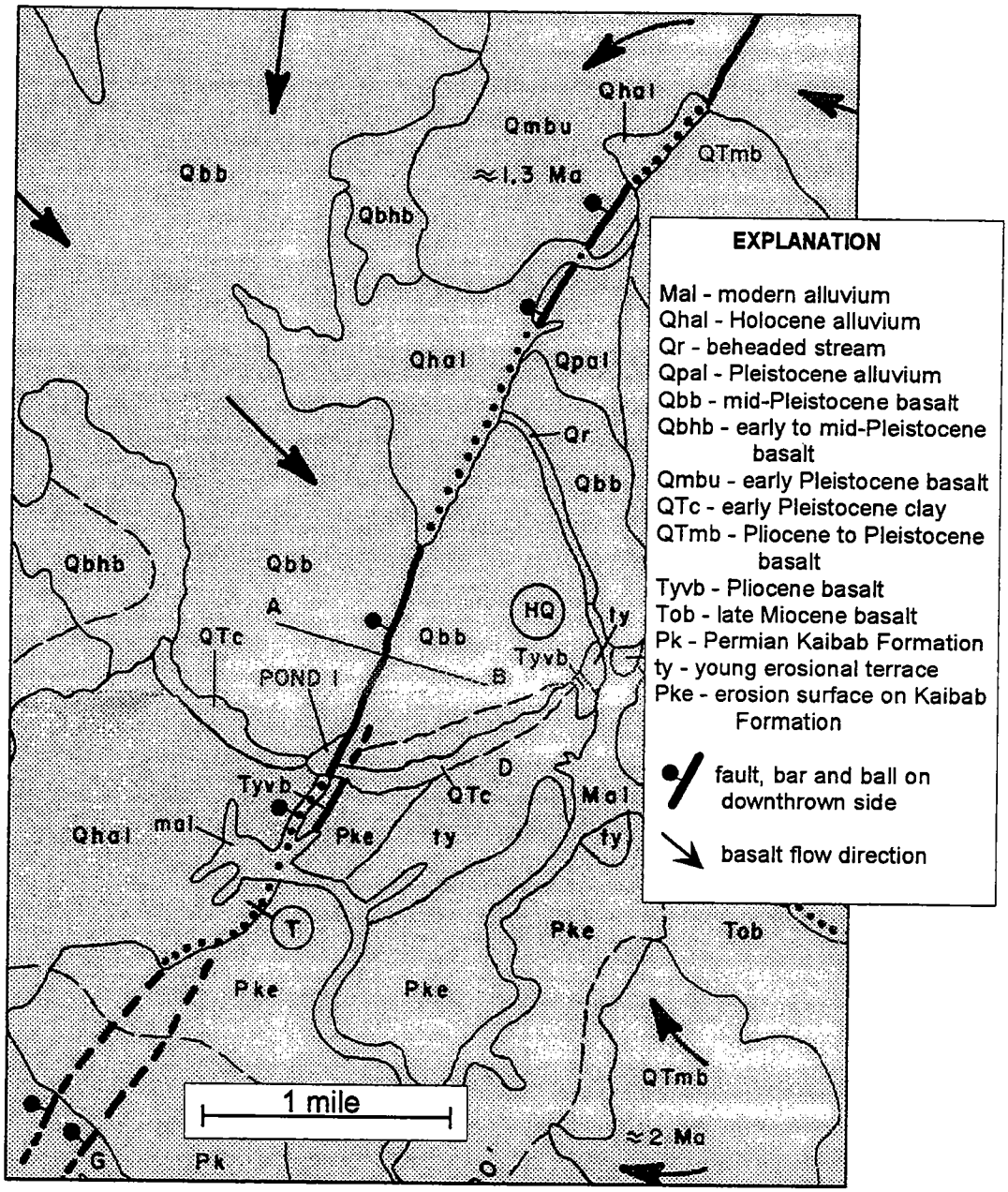

Figure 4a. Geologic map of part of the Bellemont fault west of Flagstaff. Pliocene to middle Pleistocene basalt flows and a Pleistocene alluvial fan are faulted; Holocene deposits are not faulted. The approximate location of the cross section shown in Figure 4b is shown by the line labeled A-B. Ages of basalt flows are given in millions of years (Ma).

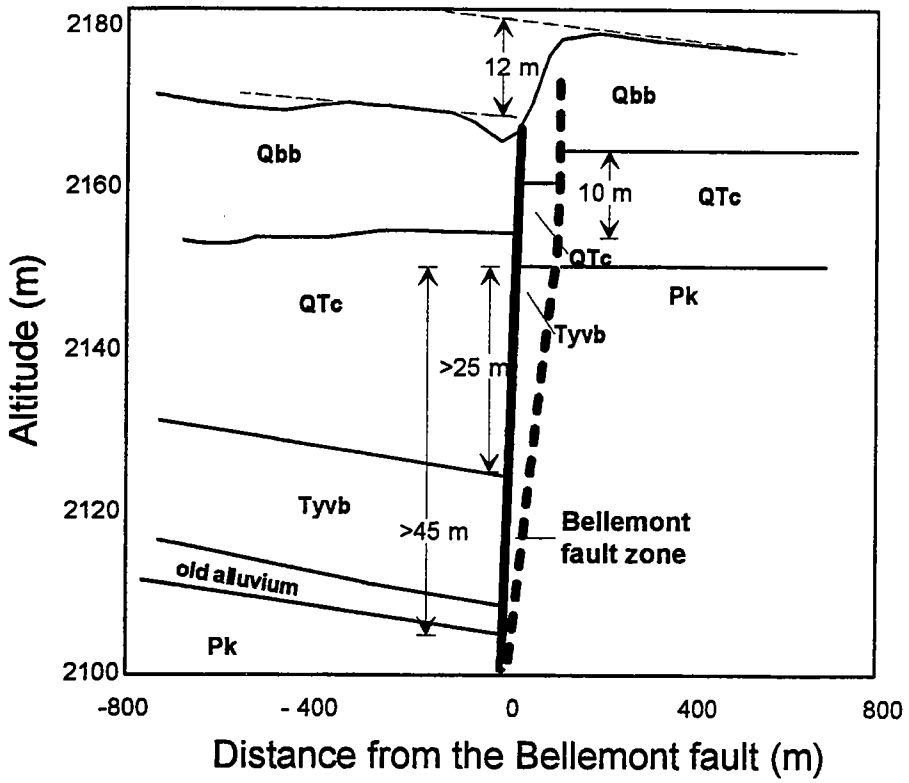

Figure 4b. Composite geologic cross section across the middle part of the Bellemont fault. The youngest faulted unit depicted, the middle Pleistocene Qbb basalt flow, is displaced 10 to 12 m. The Pliocene basalt flow Tyvb is 25 m below the surface on the downthrown side of the fault and it is exposed between two strands of the fault, so it is displaced by at least 25 m. The Tertiary erosion surface cut onto Permian Kaibab Formation is displaced at least 45 m. The Bellemont fault has been active throughout at least the past 5 million years, and 10 to 12 m of slip has occurred during the past 500,000 years or so.

Because none of the faults in this area had been studied in detail, much of our effort was devoted to a detailed paleoseismologic investigation of the Bellemont fault west of Flagstaff (see Figure 3 for location). Regional fault behavior was assessed with reconnaissance evaluations of the activity of other young faults in the area. Using fault and paleoearthquake data gleaned from these studies, several alternative probabilistic assessments of seismic hazard in the Flagstaff area were developed and compared with previous probabilistic assessments.

We conducted detailed geologic mapping (Figure 4a), trenching, topographic profiling of faulted landforms, and analysis of subsurface information from well logs (Figure 4b) along the Bellemont fault. These investigations indicate that the fault has been active at least through the past 5 million years, because older geologic units are displaced more than younger units (Figure 4b). The average slip rate during the past 300,000 to 700,000 years has been between 0.015 and 0.04 mm/yr. Maximum fault slip in the most recent paleoearthquake was 5 to 10 ft (1.5 to 3 m), and the magnitude of that earthquake was probably 6.6 to 6.9. The average time between large earthquakes on the fault is between 40,000 and 130,000 years, and at least 10,000 years have passed since the most recent earthquake. From these data, we concluded that the Bellemont fault generates large earthquakes, but very infrequently.

Using existing geologic map information and field reconnaissance investigations, about 50 faults in the Flagstaff area were identified that have been active in the past 5 million years and might be potential sources of large earthquakes (Figure 3). High concentrations of faults exist south of Flagstaff and north and east of the San Francisco Mountains; relatively few young faults are found immediately adjacent to the mountains. Long-term slip rates on all of these faults are low, ranging from 0.04 mm/yr to less than 0.01 mm/yr. During the past 750,000 years or so, 17 faults have likely ruptured at least once and 15 faults evidently have not ruptured; there is no definitive evidence for the remaining faults. Based on the low slip rates, typical intervals between surface ruptures on individual faults must be at least 100,000 years. Because there are so many potentially active faults, however, we infer that on average large earthquakes occur in the region every 2,000 to 5,000 years.

We developed new probabilistic acceleration maps for the Flagstaff area based on our investigations of young faults. Our analyses differ from previous analyses in that we used young faults as discrete sources for the large earthquakes in the Flagstaff area. We used several alternative assumptions about the number of potentially active faults, maximum earthquake magnitudes on the faults, and background seismicity to develop various probabilistic hazard scenarios (Figure 5). Acceleration values vary a reasonable amount across the region, reflecting the relative density of potentially active faults. The 50-year and 250-year acceleration values we obtained in these analyses are consistent with most previous analyses, but are substantially less than the values obtained by Euge and others (1992) (Table 1). They imply that seismic hazard in the Flagstaff area is moderate. Large earthquakes occur in this region, but not frequently enough to result in high probabilistic seismic accelerations over short exposure periods.

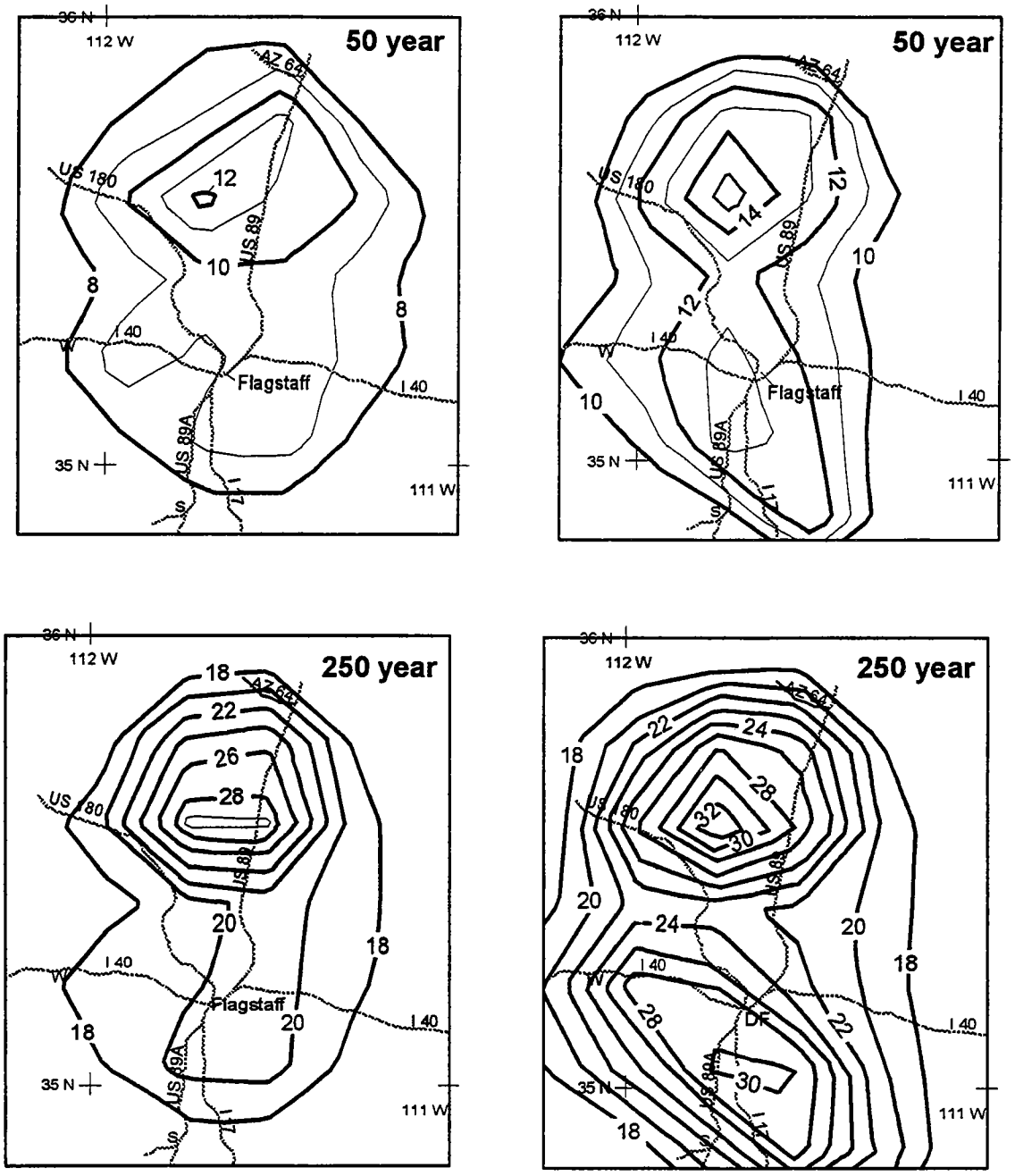

Figure 5. Examples of 50- and 250-year, 90 percent non-exceedance accelerations values obtained using faults as sources for large earthquakes. Contours show horizontal accelerations as a percent of 1 g. Maps on the left were obtained using fewer faults and lower maximum earthquake magnitudes for the faults. Maps on the right were obtained using all possible faults and higher maximum earthquake magnitudes.

Source	50 year interval	100-year interval	250-year interval
Algermisson, 1982	0.09	n.a.	0.23
Algermisson, 1990	0.08	n.a.	0.23
Euge and others, 1992	0.2	n.a.	0.42
Bausch and Brumbaugh, 1994	0.11	0.17	0.26
U.S. Geological Survey, 1996	0.08	n.a.	0.22
Pearthree and others, 1996	0.07-0.13	0.1-0.18	0.17-0.26

Table 1. Comparison of seismic acceleration values for downtown Flagstaff determined by various studies. All accelerations are expressed as a fraction of 1 g. Ranges of values shown in the last row are from the least conservative (small number of faults, smaller maximum magnitudes) and most conservative (all faults, larger maximum magnitudes) scenarios.

Summary of Quaternary Faulting and Seismic Hazard in Arizona

The current state of knowledge of potentially active faults in Arizona is summarized in a new database and map that has been compiled (Pearthree, in review) in cooperation with a U.S. Geological Survey program to map young faults around the world. Available published information on Quaternary faults is entered into standard data forms (Microsoft Word files). Data include the geologic setting of the fault, fault characteristics, the geomorphic expression of the fault, the age of the youngest paleoearthquake, and the fault slip rate. Fault traces have been compiled and digitized at 1:250,000 scale, and are represented on a map of Arizona at 1:750,000 scale. This new fault compilation, combined with recent revisions and updates of the catalog of historical seismicity in Arizona (AEIC, 1993; Bausch and Brumbaugh, 1997), provides an occasion to briefly assess seismic hazards across the state.

Areas in Arizona with the greatest seismic hazard are the southwesternmost corner of the state (the Yuma area) and the northwest quarter of the state. Seismic hazard in the Yuma area is high primarily because of its proximity to the very active faults of the San Andreas system in southern California and northern Mexico. Within Arizona, the greatest number of Quaternary faults and the most active faults are found along the margin of the Colorado Plateau in north-central and northwestern Arizona. Many of these faults have low slip rates, but a few faults in this region are significantly more active, with slip rates as high as 0.2 mm/yr (Jackson, 1990; Stenner and others, 1998). This part of Arizona has had persistent, moderate seismic activity throughout the historical record, with earthquakes as large as magnitude 6.2. Both the Quaternary fault characteristics and historical seismicity imply that seismic hazard in the northwestern quarter of Arizona is moderate.

Seismic hazard is substantially lower in central, east-central, and southeastern Arizona. Young faults are much less common, slip rates on these faults are very low (typically less than 0.02 mm/yr), and recurrence intervals between large earthquakes on individual faults have been estimated at 100,000 years or more. Historical seismicity through these areas has generally been

very low, with intermittent bursts of moderate activity in small areas (AEIC, 1993). The major exception to this pattern was the magnitude 7 ¼ Sonoran earthquake of 1887, which resulted in a 45 mile (75 km) long surface rupture in the Mexican state of Sonora, just south of the Arizona-New Mexico border. This earthquake demonstrates that large earthquakes can occur in regions with relatively few Quaternary faults and low levels of seismic activity, even though the probability of them occurring during a 50-year time frame is very low.

Seismic hazard is very low in the interior of the Colorado Plateau in northeastern Arizona and in most of southwestern Arizona. Historical seismic activity has been low in both of these regions. There are no recognized Quaternary faults in northeastern Arizona, and a handful of Quaternary faults have been documented in southwestern Arizona. These data indicate that the probability of a large, surface-rupturing earthquake occurring in either of these areas is exceedingly low.

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Former Manufactured Gas Plant (FMGP) Sites; Nemesis of Transportation Planning

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ABSTRACT

Transportation projects have the well-known site characterization burden of striking out across the land like an arrow. We do know where the arrow will fall, however it is what the arrow encounters that counts in the question of coal tar contaminants. America may contain as many as 43,000 former industrial sites that made gas or byproducts from coal. Five major processes were utilized for the purpose, all involving some form of the incomplete combustion (pyrolysis; absence of oxygen) of coal and, later, refined, and, eventually, crude oils. The product, loosely known as coal tars, contains a nearly unlimited number of toxic compounds that varied by the feedstock, process and operational variables used to make the gas or to capture and refine the tar, light-oil, ammonia, as well as cyanide and sulfur. Economics guided the establishment and operation of the manufactured gas plants, coke ovens, by-product plants, tar distilleries, and wood treatment plants that were associated with gas making and its residues. Regulators today track about three dozen of these toxic compounds, yet the range is at least 500 to 3000 separate non-halogenated (without chlorine) compounds injurious to human health and the environment.

Former manufactured gas plants (FMGPs) were the most numerous of these facilities, and these were located, on a rule of thumb, just where transportation planners are most likely to respond to improvement of infrastructure needs for the 21st century:

- Near as possible to the Central Business District (lowest investment in gas piping);
- On the immediate "wrong side" of the railroad tracks; to indicate most-common mode of coal/feedstock supply;
- Next to a waste-discharge creek, stream, river or wetland (ease of waste discharge);
- In the topographically lowest area (gas rises uphill), and;
- In towns as small as 1000, but more typically > 6000, as of 1890 (necessary cash flow).

These plants typically used once-through process water, and accumulated solid and fluid wastes at a rate that generally outdistanced the manager's capacity for prudent disposal options. The gas plants were not systematically recorded and careful detection is required to identify them and to respond in transportation planning (Hatheway, 1999, in press). Construction discoveries are to be avoided whenever possible, because of loss of options for dealing with their hazardous wastes.

INTRODUCTION

Manufactured gas was developed as a usable product in 1792 by the Scottish practical engineer William Murdock, as his home in Redruth, Cornwall, where served as the resident engineer for installing and servicing Boulton & Watt (Birmingham, England) steam engines to power the tin-mine dewatering pumps. From this point forward, the development of manufactured gas is perhaps the most interesting of all of the many entwined tales of the industrial revolution. Suffice it to say that the technology jumped the Atlantic Ocean to North America by 1810 and was fully employed at Baltimore, in providing fuel for gas lights by 1821.

Strictly speaking, illuminating and fuel gas can be made from all organic matter and during the 160-odd years of the manufactured gas industry (finally succumbing about 1960 in the U.S. and 1970 in Britain), the tar-spewing plants (Fig. 1) were found almost everywhere people clustered. And where those people then clustered, so go we now to improve their descendants' means of transportation.

TYPICAL GAS-GENERATING PROCESSES

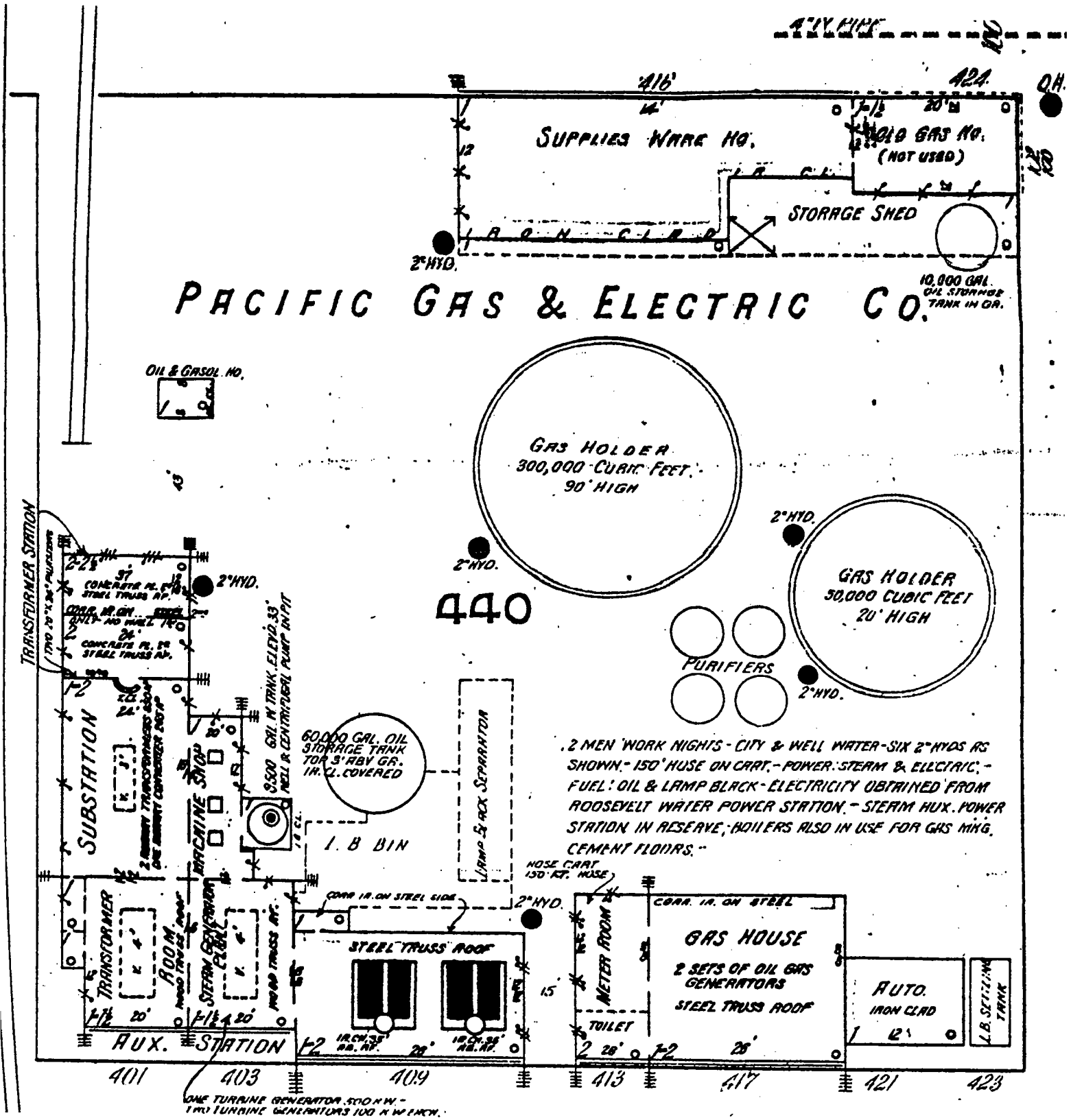
The literature of manufactured gas and associated technologies is vast and informative. Variations in process and technique were complicated and gas works bore enough similarity so that one knowledgeable with the processes can scan a Sanborn Fire Insurance Map or visit any site with remnant structures, and conjure up at least a flavor of what went on there in the past. Available space, topographic relief and the selected gas-manufacturing process set the stage for practical development of the physical layout and arrangement of components and buildings and their umbilical plumbing trenches (Table 1).

GENERALIZED GAS MANUFACTURING PROCESS

Gas, as freshly manufactured, is not suitable for storage or distribution. Process variables release a brew of strongly temperature-dependent, form as cyclical aromatic compounds (Table 2) built of and around the benzene ring and rise to chains of up to six such clusters. In the final analysis, these VOCs and SVOCs represent the bottom-line threats of FMGPs and related sites. There is no way in which the absolute degree or concentration could, even today, be predicted, even if all of the independent and dependent variables were known. We can say, however, predict the probable main chemical compounds as representatives of each level of increasing complexity, single–ring through six-ring compounds. We can also specify where to look for these contaminants and how to search for and categorize such compounds as *source areas* (volumes) and *migrated contamination* (*contaminant transport*).

Source areas automatically suspect are the Gas House (retort house/generator house), Condenser Building or portion of Gas House; Purification House or outdoor Purifier Boxes, and the Gas Holders (Relief Holder, Storage Holder, Distribution Holder), along with all of the subsurface pipe transfer trenches on the plant site. Relief Holders had different roles between coal-gas and carburetted water gas plants; the former generally was the first or oldest holder at the plant and later was used to take raw gas before completion of the purification treatment. The latter type were required as a

Figure 1: Sanborn Company Fire Insurance Map of the second manufactured gas plant at Phoenix, Arizona in its 1915 state, as modified from its 1887 origin and operated by its second owner. Phoenix had a population of 25,000 at the time. This plant shows as an oil-gas plant and was converted from a Lowe Process carburetted water-gas plant, probably located at the second site, in the upper right corner of this view. The plant was located on the NW corner of S. 2nd Avenue and West Buchanan. Even with this information at hand, we can surmise that this site has a complicated waste management and disposal history. The large circles, representing gas holders, are nearly unique in graphic forms on these maps and extremely helpful to the eye when scanning map folios or microform map copies in the search stage. (Courtesy U.S. Library of Congress, through the Chadwyck-Healy Company).



pressure equalization mechanism during the cyclic nature of carburetted water gas plant operation. Other prime source areas are all subsurface tanks, such as the basin of sub-grade-type gas holders, as well as the tar wells, tar cisterns, tar storage "tanks", and ammoniacal liquor collection tanks.

Gas was prepared for storage and distribution by a sequence of steps (Table 3), breaking down mainly into the *condensation* chain, responsible for removing particles of

Table 1
Generic Manufactured Gas Processes

Process	General Purpose	Waste-Stream Pollutants
Coal Gas Horizontal Retort Inclined Retort Vertical Retort	Illuminating gas @ > 450 Btu, generally at the power of 14-18 candles (candle power = cp).	Coke most valuable by-product; Used for Water-Gas; Tars highest quality & urban-marketable; CN; Ammoniacal liquors as main tar-bearing pollutant.
Coke-Oven Gas Merchant Plant Utility Plant By-product Plant Steel Mills	Gas is the main by-product Company sells to steel industry Gas company sells coke; Uses gas Uses gas, sells coke & by-product Use both coke and gas.	Ammoniacal liquors as process wastes and also from coke quenching; Heavy volumes of tars produced.
Water Gas (Blue = Producer Gas) At Individual Factories Mines and Mills	Fuel gas, devoid of essential gas lighting qualities > 450 Btu/cf.	Reduced unit amounts of tar and ammoniacal liquors; but often large quantities generated.
Enriched Water Gas Town Gas Institutional Gas Machines Mansions & Estates	Dual use, for illumination and fuel; Some sold as compressed gas in rural areas.	Enrichment led to additional tars and ammoniacal liquors.
Carburetted Water Gas Coke as reactor bed Anthracite as reactor bed Bituminous coal as reactor bed Sub-Bituminous coal as reactor	Labor-saving, high capacity Original specification feedstock Desirable alternative feedstock Undesirable alternative feedstock Highly undesirable feedstock.	Illuiminosity enriched with light oils for greatest brilliance. As feedstock quality decreased nearly valueless tar-water emulsions became major contaminant.
Light-Oil Gas Town Gas Institutional Gas Machines Mansions & Estates	Used for illumination, relatively high candle power and Btu value.	Lampblack, tar and the various cyclical aromatic compounds; Progressive companies in west made fuel briquettes.
Crude-Oil Gas Pacific Coast East Coast	Lighting and heating; Residues vary by patented process and quality of oils.	Typified by lampblack sorbed with PAHs and related toxics.
Pintsch Oil Gas (Railway Yards) Compressed gas tanks on cars	Railway lighting for coaches, dining, baggage and mail cars	Lampblacks and tars.
Acetylene Gas; Rural use in homes, military posts and institutions.	High-illumination; Usually generated at site of use: Some railway usage.	Sludge, directly from generator.

Compiled by the author.

impurities and the *purification* process, responsible for removing ever-smaller trace-element impurities carried in and released from the feedstock coal or oil.

It is important to note that the aromatic hydrocarbons are referred to as *nonaqueous* in terms of their chemical characteristics, but this is a term of art to organic chemistry and denotes that the nonaqueous compounds are those that are not full

soluble in water. These MAH, DAH and PAH compounds all are environmentally threatening because of their ability to dissolve into groundwater to significant concentrations both of carcinogenic and noncarcinogenic concern.

Table 2
Hazardous Manufactured Gas Wastes

Type Waste	Origin	Environmental Character
Cyclic Aromatic Hydrocarbons Monocyclical: MAHs; BTEX Duocyclical: DAHs (Tar Acids) Phenols, Cresols, Naphthalene Polycyclical: PAHs (Tar when miscibly associated)	Created from volatile content of feedstock coal or from enrichment and carburettion oils as released and formed under heating in absence of oxygen and reformed in condensation cooling.	MAHs are LNAPLs and VOCs DAHs are NAPLs and VOCs PAHs are DNAPLs and SVOCs All of above are water soluble to environmental significance and leach continually to groundwater; can enter fractured bedrock.
Tar: Sum total of cyclic aromatic hydrocarbons plus water, carbon, and trace elements/compounds.	Composite tar "oil" formed on temperature variation upon release from feedstocks.	Form "source" bodies below gas-plant components and leach into site soil and water.
Ammonia; ammoniacal liquor plus cyclic aromatics.	Fouled process water.	Ammonia part of carrier fluid, now DNAPL tar PAHs.
Cyanide and cyanogens.	Mainly in purifier box wastes.	Water soluble; can release pure CN in low pH surroundings.
Sulfides.	Mainly in purifier box wastes.	In various sludge compounds.

Compiled by the author. Terms of art are: MAH (monocyclic aromatic hydrocarbon); DAH (duocyclic aromatic hydrocarbon), PAH (polycyclic aromatic hydrocarbon); NAPL (nonaqueous-phase liquid); LNAPL (light nonaqueous-phase liquid); DNAPLS (dense, nonaqueous-phase liquid); VOC (volatile organic compound), and SVOC (semi-volatile organic compound); CN (cyanide).

Another complication is that all of the cyclic aromatic hydrocarbons are co-miscible and tend to be found as the "tars" that contaminate soil masses constituting *source areas* at the site. Source areas, by definition are contained as the interstitial (pore) fillings of contaminated soil of the vadose zone. Furthermore, at the edge of each of the source-area masses, there is a continual, slow, but release of the lighter-density fraction of compounds until the mobility of the source area reaches a positional equilibrium of expansion. As a rule, most of this should have occurred by the present era and continual releases now occur by solubilization into such groundwater as is not in contact.

The PAHs also are, by definition SVOCs (*semivolatile organic compounds*) and will be found to "hang" below the various previously-mentioned components of the gas plant, the MAHs floating mainly on the groundwater surface, but also carried along as miscible components of the sinking tars. The DAHs likewise will co-miscibly associate in the tar masses and the PAHs, being dense, will sink, with time, to and into the groundwater surface and may move into the top-of-rock and enter geologic discontinuities in the site bedrock, and thereby continue to contaminate groundwater held there.

Table 3
Generic Sequence of Gas Manufacturing

General Step	Purpose	Typical Residues
Note: There were many variations in the relative sequence of these steps, at least after condensation	Note: Interpretation of steps requires accurate understanding of plant history	Additional waste leakage typically occurred all around and between the plant components.
Prepare the gas machine Heat and charge; Seal from the ambient atmosphere; Exclude the presence of oxygen	Release the volatile content of coal or other feedstock, in the new form of gas	Virtually none in coal gas; Other processes have low-value to valueless carbon-based wastes as waste streams.
Gather the rising gas; Transfer such from the gas machine	*Exhauster(s)* commonly located at various places in system to avoid pack pressure to the gas machine.	Back pressure could and did cause explosions.
Transfer the gas Pass through an incidental bath of tar for early removal of tar particles	Collect and manage spent liquids; Often sent to a *tar well* where leakage to ground water was common.	Commonly direct leakage into the subsurface, through cracks and flaws in wood or masonry wells, cisterns or tanks.
Remove the residue and recharge; usually immediately	Feedstock is spent and can yield no more; Coke is quenched	Large quantities of ammoniacal liquor generated for management
Transfer from Gas House to Condensation Area	Often in separate building, or gas passed through trenched pipe	Constant source of leaks to the subsurface
Cool the gas Drop temperature slightly by indirect or direct contact with cooling water	Direct-contact condensation water becomes valueless ammoniacal liquor in all but largest plants with specialized recovery > 1890.	Constitutes a relatively large stream of once-through contaminated process water.
Wash the gas Subject the gas to direct contact with cooler process water	Transfer as much particulate matter from gas, as possible, to the ammoniacal (wash) liquor	Constitutes a relatively large stream of once-through process water; Needs disposal.
Separate the tar Pass the gas through one of many choices of physical partitioning devices; Most depend on gravity	Tar is not welcome as it causes undue fouling of the later purification media.	*Purification media* (AKA *box waste* becomes *spent* and can be *revivified* to a degree then forms contaminated solid waste for plant disposal
Scrub the gas Remove select impurities by their sorption on/in solids or liquids	Transfer as much particulate matter from gas, as possible, to the solids placed for that purpose	Scrubber needs periodic changing of scrubbing medium; This constitutes another solid-waste stream for disposal.
Purify the gas; Remove trace insolubles originating from the feedstock.	Remove remaining impurities; mainly cyanogens and sulfides, by media sorption and oxidation.	Ultimately fouled lime wood chips/shaving/sawdust and forms of iron oxide as waste.
Store in a gas holder; The vast majority of which had stationary tanks of cooler water	Continue to drop particles from *tar fog* into cooling water for later (if ever) attention.	Periodic pump-removal of the tar and tar-water emulsion; latter usually unsalable waste.
Ammoniacal Liquor Collection System	On-plant collection drainage to capture some of the leaks/spills	Conveyed to tar well, off-site drainage or town sewer (if any).
Constant Collection of Plant Solid Waste	Accumulations of broken retorts, gas machine scurf (carbonaceous slag deposits) and spent purifier media.	Generally dumped in plant-site depressions or within short wagon haul (few blocks); Often to adjacent swamp (wetland).

Compiled by the author.

Unlike source-area equilibrium expansion, VOC/SVOC gas-plant residuals comprising contaminant migration plumes continue to move as down-gradient dispersion and advection and downward, through the aquifer(s) via gravitational attraction. DNAPLs in particular are particularly responsive to movement through geologic microfractures of aquitards as well as into and through bedrock joints, shear plants, faults bedding planes and dissolution features, and can and will then redistribute into to the next lower aquifer.

In dealing with manufactured gas production wastes, a further thought to remember is that there is no one *contaminant plume*. The body of gas-manufacturing residuals (wastes) forms in chemically variable bands within an overall plume, but within this plume we should expect to find variable masses of tar constituents.

COAL-TAR IMPACTS ON STATE TRANSPORTATION DEPARTMENTS

Transportation Departments face a unique challenge in dealing with "coal-tar" sites. This stems from the fact that most FMGPs and other historic coal-tar sites are now long-forgotten and probably 90 percent no longer are directly recognizable, even by experts. Hence, if the Department does not take careful cognizance of the generic problem, it may be that gas-manufacture residues will not be discovered until the contractor has been selected and is engaged in site preparation and grading.

Once discovered in grading the site becomes instantly recognizable to almost anyone in the area, as the human sense of odor can detect the PAHs at about one or two parts per billion, far lower than residents and concerned environmentalists need be stimulated into activist response. Furthermore, the Department must then mobilize a response team in order to take appropriate regulatory and legal action, while striving to maintain the integrity of the project and contract and to protect the latter from liabilities under alleged variable site conditions claims. In other words, once cut and bladed not even the most skillful NIFRAP (*No Further Remedial Action Planned*) magician can make the site "go away", not that it should, bearing demonstrably carcinogenic tar wastes.

State DOT Experience

The author believes that at least half of our State Departments of Transport have experienced unplanned gas-plant coal tar encounters. Illinois DOT, for example has experienced such shock in the past and now has a 1997 agreement in place with the State Geological Survey to locate, to the maximum extent possible, FMGPs of that State, as a primary planning index. The then-Missouri Department of Highways and Transportation encountered Hannibal FMGP no. 1 in its downtown cut for State Route 36, in the 1960s, New Hampshire DOT met with off-site disposal pockets of tars left from ammoniacal liquors at Concord, NH, in construction of a feeder bridge to I-93, in 1995.

CALTRANS is dealing with the long-after-construction legal impact of an 1875 FMGP adjacent to the suppressed segment of Interstate 5 at Sacramento, the right-of-way of which does not actually impinge on the original gas plant bounds. The Los Angeles Metro project is conducting direct treatment of coal tar from the original (1863) gas plant in that city, as discovered in excavation for the Red Line Subway. Highway

improvements in Washington State have brought the 1885 Tacoma Gas & Electric Company gas plant, and the original 1873 Seattle Gas Company Works into the limelight (lime-purified manufactured-gas light!) for the New Union Station development. The Tacoma site is has part of the Commencement Bay SUPERFUND National Priority List site. The 1887 FMGP of Spokane Falls Gas Light Company appeared before WADOT in the Railroad Avenue relocation in that city.

East Greenwich Gasworks (established 1894), at London, were discovered in the construction of the Jubilee Line subway extension at the intersection of the Docklands Highway along the south bank of the Thames River. Incidentally those who viewed Stanley Kubrick's 1980s *Full Metal Jacket* version of the Battle of Hue (Vietnam, 1968) will have already seen this derelict gasworks, making its last bow.

What the Transportation Advisory Agencies Have to Say

Broad hazardous waste planning and response is an old issue with State DOTs, in that the National Cooperative Highway Research Program (NCHRP) of the National Transportation Research Board (TRB) has, since 1980, published fairly extensively on the subject.

According to NCHRP Compendium 310 (1988) there were, more than 15 years ago, among the first transportation-project encounters with hazardous waste included FMGPs at a rate of 33 to 50-percent. These projects were in the States of Nebraska, Rhode Island, Vermont and Washington. And, worse yet, none of these States has a notably high population of derelict coal-tar sites.

In 1990, the FHwA, through its nine regions, sponsored a 30-hour traveling short course entitled *Hazardous Waste; Impacts on Highway Development.* (G.I. McGregor, TRR 1192, p. 94-102) with the objective to "increase awareness and provide possible solutions so state highway agencies can improve their programs" (Denbow; p. 57). Now is the time to redouble this effort to discover FMGPs in the planning stage.

EXPECTING GAS PLANTS AND OTHER COAL-TAR SITES

The author strongly recommends that each DOT initiate a one-person program of identifying FMGP and other coal-tar threats on a state-wide basis. The individual selected should be an engineering geologist or geotechnical engineer with a strong interest in the history of technology. Working materials are abundant and the author has prepared his forthcoming book (Hatheway, 1999) that that use in mind. About a two or three person-year effort should provide the necessary degree of awareness for the States with the greatest population of derelict sites. Such an effort should also produce a response plan for activation on informed discovery or linked indication that one of these sites may impact a planned project.

In terms of real practicality, the most economical pathway would be to identify the former coal-tar site and then select an alignment, where possible, that tends to avoid the source areas and their likely contaminant transport plumes. Taking, however, the findings of the author (Hatheway, 1997a and 1997b), many of the coal-tar caches will remain difficult to anticipate if they prove not to be FMGPs, but one of the numerous

alternate PAH sites, for which the available historic evidence will not be as directly accessible. The author herewith proposes (Table 4) a generic program of Statewide assessment of derelict coal-tar sites.

SYNOPSIS OF FMGP REMEDIATION

FMGPs and other coal-tar sites should give strong concern for permanence remediation by reduction or removal of the source areas, as represent the basis for the environmental concern. The process of consists of several steps, as noted in Table 5.

Table 4
General Scheme of Developing a Statewide Survey of Derelict Coal-Tar Sites

General Step	Purpose	Typical Sources
Scan the USEPA "Radian" Report.	Develop an initial appreciation.	USEPA Rept. EPA/600/7-85/004
Set up appropriate database for locating Statewide presence of FMGP and other PAH sites.	Preservation, correlation of, and access to site-specific and gas corporation and holding company policies and generalities.	Computer database software; Author makes use of *Claris File*.
Systematically scan *Brown's Directory of North American Gas Plants* (1887-1960).	Extend the initial listing; Plot gas and by-product production for all years; Select points of inflection as critical periods for reported modifications, accidents and change.	*Brown's Directory of North American and International Gas Companies* ", 1996; Composite microfilm copy is available, from 1887.
Obtain Sanborn Fire Insurance Map coverage for State; Scan each city to identify location and general nature of the site (Fig. 2).	Search to confirm exact location. Make tabular form to identify all gas-plant components and their role through years of coverage.	Utilize State Library or Historical Society, or purchase microfilm set, or order site historic coverage from EDR, Inc.
Engage senior-citizen historians to work town library & newspaper coverage on the site.	Concentrate on the years of the production-curve inflections from Brown's or other source data.	US WPA Federal Writers Project newspaper file (through 1938), at the State Historical Society.
Seek historic plant maps and layout drawing, birdseye views and historic photographs.	Interpretation of site details and locations of source areas and probable contamination plumes.	State Historical Society and museums; advertise in classifieds, for historic photographs.
Scan prominent historic national weekly engineering journals; employ university or college history students, with faculty supervision.	Concentrate on years believed to represent important changes in plant nature and layout.	*Engineering News, Engineering Record, American Gas-Light Journal. Gas Age, Progressive Age, Journal of Electricity, Power & Gas, Gas Industry*, to name a few.
Notify past and present property owners, and, in particular, utility companies.	Request access to utility corporate archives, site summaries, and corporate historical summaries.	Edison Electric Company listing of investor-owned public utilities.
Utility company historical documents.	Develop site-specific historical data; Search for utility holding company employee/consumer magazines for historic details.	State Historical Library; State Archives.
Graduate theses and dissertations in civic and public history.	Details of town history and of gas manufacture in the State.	Bibliographic searches and interlibrary loans.
State Public Commission (most formed 1905-1919) Archives.	Annual Reports, Property Evaluations, Rate-Making Cases	Typically held in State Archives; Can be very productive.
Other steps and sources.	Identified in A.W. Hatheway forthcoming technical book.	*Remediation of FMGPs and Other Coal-Tar Sites*; due in 1999.

Compiled by the author.

Figure 2: Sanborn Map of the oil-gas FMGP site of the Prescott Electric Light & Gas Company, December, 1910, when the City had a population of 11,000. Miller Creek. The site lies along the south bank of Miller Creek, just north of West Sheldon Street, and on line with the north extension of North McCormick Street (Courtesy U.S. Library of Congress, through the Chadwyck-Healy Company).

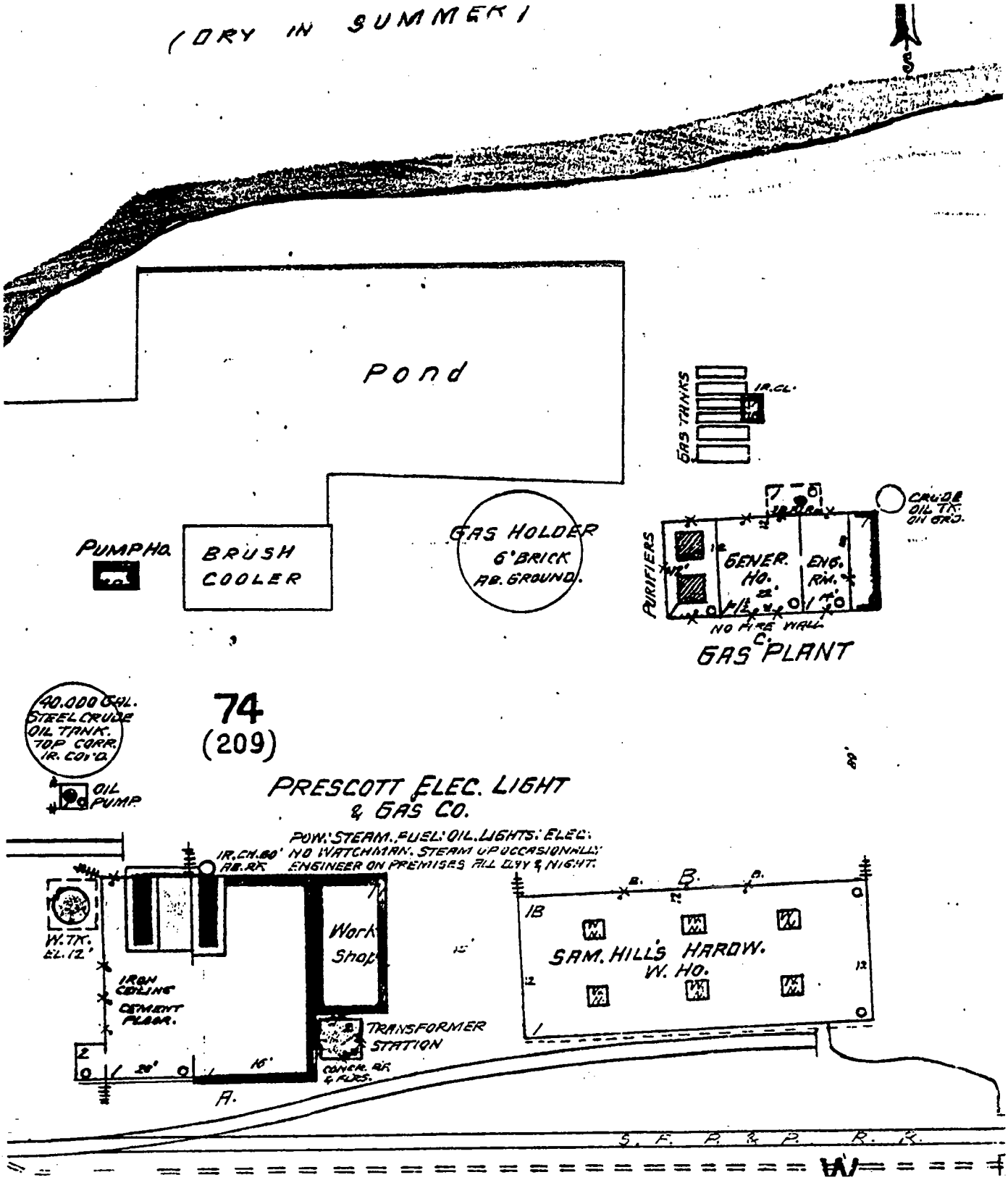


Table 5
Procedural Steps in Remediation of Abandoned Coal-Tar Sites

General Step	Purpose	Result
Characterize the site, for its gas process type, generating capacity, and probable waste stream, as well as its geologic controls over the presence and migration of its contaminants.	Conduct geologically and historically-based site exploration to locate and quantify *Source Areas* and fate and transport of its bodies of *Migrated Waste.*	Properly done, the effort locates all bodies of waste and the pathways by which such wastes have and will continue to contaminate the environment.
Sample Source Areas; At FMGPs and Other Coal-Tar Sites, these are the various components of gas manufacture, condensation, purification, and storage, plus residuals disposal areas.	Relates site wastes to point or place of origin and reduces chances that such will be over-looked and thereby continue to create environmental damage.	Systematically reduces the possibility of undetected wastes being left in the ground with potential for ongoing negative environmental impact.
Identify Contaminants of Concern (CoC)	Reduces numerous contaminants to manageable number of contaminants representing key chemical groups.	Remediation based on group representative as the means to meet threats from homologue chemical compounds or elements.
Identify Geologic Pathways of Migration	Remedial planning without such discovery and disclosure is basically flawed.	Avoid false security about removal of health and environmental threat; Achieve responsible site remediation.
Select Action Levels for Cleanup	Reduce or alter contamination to acceptable threat level.	Provides a performance-based target for achievement of remediation goals.
Identify *Candidate Alternatives* for site remediation	Recognizes that most sites can be remediated by a selection of various *Technologies* and *Remedial Alternatives*.	Basis for achievement of reasonable cost savings but not at the injudicious selection expense of improper *Technologies*.
Select the Site *Remedy*	Choice represents the best achievable approach to threat reduction or removal.	Site has best chances for not creating ongoing or worsened environmental threat; Should be so selected as not to return.
Design the *Remedy*	Serves as the basis for contract construction of remediation	Serves as regulatory evidence of responsible environmental compliance.
Construct the *Remedy*	Take the site to *Closure* .	Serves as evidence of removal of environmental threat.
Install *Performance Monitoring*	Verify that remediation has been appropriate and effective.	Provides on-going evidence of success of site remediation.
Redevelop the site.	Take advantage of often prime and nature of property location.	Places formerly dangerous derelict land on the tax roles or as serves positive sociological value.

Compiled by the author. Note that present-day environmental remediation terms of art are italicized.

SUMMARY

Coal-tar sites today constitute serious potential economic considerations, even burdens, on property owners. EPA began its campaign to identify and remediate FMGPs in 1983, as a result of lessons-learned in its campaign against the serious air emission problems of the nations coke works. Today, nearly all the coke works have been put out

of business, through regulatory ratcheting, but the national campaign to remediate FMGPs has fallen on hard-times, through incessant political lobbying of present-day owner groups who have seen to legislative initiation of serious impediments to competent and thorough site remediation requirements.

EPA seemingly now seemingly has lost its will to promote coal-tar remediation in general and has passed most of the responsibility on to State environmental agencies, many of whom do not have staff trained to deal with this specific challenge. Currently there is a wave of unsound attempts of Responsible Parties (RPs) to foster low-cost and environmentally unsound NIFRAP (*No Further Remedial Action Planned*) solutions. NIFRAP tactics generally include incomplete or otherwise flawed site and waste characterization, lack of attention to historic site details, and use of the technically unsound RBCA (Rick Based Corrective Action) application to PAH sites. RBCA which itself was developed for the totally different suite of petroleum hydrocarbons, and strong adherence to *Intrinsic Site Remediation* itself consisting of no meaningful actions, but with groundwater monitoring for five or six years. It is the stated hope that somehow natural "attenuation" conditions will reduce the public health and environmental threat of the site, with time. In reality, the monitoring may only reach a point at which regulatory personnel have lost contact with the problem and the site will continued as a deferred environmental threat.

This aside, the author has strong advice to State DOTs and to the FHwA as well. Anticipate these coal-tar sites before they are encountered by transportation route construction and avoid making contact with them, and accomplish this in the planning stage. The prudent move will be to choose an alternate route, one that lies outside the former gas-plant bounds and outside of the likely areas for off-site dumping of residuals and surface discharge of its liquid wastes, and, certainly, up-gradient for its contaminated ground water.

ACKNOWLEDGEMENTS

Since his mid-1960s graduate-school days of part-time employment with the Arizona Highway Department (District II, Tucson), the author has focused on site and waste characterization for engineered construction and remedial engineering. His first FMGP assignment was in 1989, at Burlington, Iowa, assisting the IADOT in dealing with the unforeseen discovery and mitigation engineering for the new Mississippi River suspension bridge, with its west abutment anchored in the 1856 Burlington Gas Light Company works. Since that time he has devoted unlimited energy and effort toward learning the now dead art of gas manufacture, along with developing the technology of present-day residuals management and site characterization and remedial engineering of coal-tar sites. He travels extensively, usually armed with copies of historic Sanborn Fire Insurance Maps and visits and takes observations and photographs of any and all visible remnants of FMGP activity. He has visited hundreds of FMGPs and has provided consultations at more than 75 such locations.

At UMR he acknowledges his research home at the Center for Environmental Science and Technology (CEST), where is he manager of the Office of Coal-Tar Site Remediation. His research to date has been funded out of pocket, but he regards his University of Missouri-Rolla internet page as a source of contact with individuals,

(www.umr.edu/~gee/research/hatheway/gas_coal-tar.html) agencies and companies through which he spreads the word and technology of responsible management of coal-tar sites. Readers are encourage to take advantage of this medium and the author's e.mail (hatheway@umr.edu) for gratis contact and exchange of ideas and data.

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STRUCTURAL CONTROLS OF DEVILS KITCHEN SINKHOLE; SEDONA, ARIZONA

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Abstract:

A number of sinkholes, all suspected of sharing a common origin and belonging to a special type of karst topography, occur in the Sedona, Arizona area. The largest of these, the Devils Kitchen, has experienced dramatic enlargement in historic times. According to pioneer records it is believed to have first collapsed in the "early 1880s" but recent mapping suggests that a small surface opening must have existed prior to that time. In late 1989 another sudden collapse increased the surface opening by half again to its present size of 90 by 170 feet. The sinkhole cuts through well exposed "red bed" sandstone outcrops of Permian age Supai, Hermit, and basal Schnebly Hill formations. Prominent N.50^{0}W. joints, with a subordinate set trending N.15^{0}E., cut through the entire region and influence bedrock erosion. The northwest-trending joints in the area are believed to extend to great depths in the earth's crust because they are locally occupied by mafic dikes that once fed overlying Miocene lava flows, long since eroded away. Intersecting joint sets, especially where they are more numerous, allowed groundwater to percolate down through the carbonate-poor sandstone strata to dissolve subsurface Redwall Limestone and Martin Dolomite. The top surface of these carbonate beds is estimated to lie 660 feet beneath the ground surface, or well below the present-day water table. In time the solution caves grew too large to support the overlying strata and collapse breccias began to form. The upward migration of these breccia columns have been breaking through to the modern erosion surface in relatively recent geologic time. Where observed at a bedrock site the surface expression surrounding the sinkhole displays arcuate, bell-shaped fractures that enlarge downward. This clearly implies withrawal of support from below. A breccia column of the magnitude observed at Devils Kitchen requires the collapse of a subsurface cave opening of about 300 feet in diameter. Arcuate surface fractures along the southwestern side of the sinkhole indicate that the opening will more than double in size in future years. Recognition of these anomalous arcuate surface fractures may provide advance warning for engineering studies to avoid sinkhole-prone collapse areas. Suspected sinkholes that are filled with surface alluvium and hidden from view might be detected by employing seismic survey techniques or by the simple obsevation of anomalous groundwater drainage.

Introduction:

A number of sinkholes, all suspected of sharing a common origin, occur in the Sedona, Arizona area. They share many features in common and all display evidence of very recent subsidence. The two sinkholes closest to the Sedona townsite, Devils Kitchen and Devils Dining Room, are shown in Figure 1. Two other prominent sinkholes lie to the west and southwest of town. Other suspected sites, exposed and/or covered, await further investigation. By far the best exposed and largest of the sinkholes is the Devils Kitchen that lies just beyond the northern edge of West Sedona. The writer is currently monitoring the various sinkholes in the area and is soliciting dated historic photographs from the public at large that may assist in documenting past changes in sinkhole morphology.

Historical Records:

Before the turn of the century the Sedona area was sparsely populated and few historical records have survived from that period. However, a brief description of the Devils Kitchen sinkhole collapse was presented by pioneer Albert E. Thompson in an interview for a local hiking and historical group (Sedona Westerners, 1968). He stated: "My parents were living in Sedona in the early 1880s and heard the crash when the spot caved in. Mother said the dust from the cave-in filled the air all day and the sun looked like it was shining through heavy smoke. Her brother, Jim James, was the first one to see the new hole in the ground." An examination of the site at the present time confirms the relatively fresh scars from that event. Most edges of the sinkhole are still quite angular and show little evidence of "rounding off" by recent weathering. In the southeastern corner the outcrops appear to be somewhat more rounded and weathered than the rest of the area and it is possible that a small surface opening existed prior to the "early 1880s" catastrophic collapse. Until recent time no other historical records or scientific studies of the Sedona sinkholes are known to exist. In late 1989 the northern third of the Devils Kitchen sinkhole suddenly collapsed and enlarged the surface opening to its present size of 90 by 170 feet (27 by 52 meters). It was this unexpected collapse that prompted the need for a comprehensive geologic study of the area.

Public Access and Safety Concerns:

The Devils Kitchen and Devils Dining Room sinkholes are popular destinations for jeep tours and are readily accessible to the general public. Activities in this area are managed by the U.S. Forest Service. Because of potential public danger following the 1989 Devils Kitchen sinkhole collapse they advised Red Rock Jeep Tours that a professional report on geologic conditions at the viewing site was required. At that time jeeps from the tour company regularly passed within a short distance of the abrupt southern edge of the sinkhole on a flat, seemingly solid bedrock bench. That location, even though it is undercut by as much as 35 feet, appears to be relatively safe at the surface because it is underlain by an arch-shaped rock buttress. The writer was contacted by the tour company to investigate the area. I agreed to perform the study on a volunteer basis (Lindberg, 1990). The report made from that study forms the basis of this present article.

Geologic Stratigraphy in the Sedona Area:

The prominent and colorful cliffs that form Sedona's dramatic backdrop are located along the Mogollon Rim of the Colorado Plateau. Flat-lying sedimentary strata, mainly of Permian age, crop out in bold relief and are similar to their counterparts in the upper third of the Grand Canyon, located 90 miles to the north. Figure 2 shows a stratigraphic column for the local area. The Devils Kitchen sinkhole is situated in outcrop near the base of a series of cliff-forming Paleozoic sedimentary strata that rise another 2300 feet to the top of Wilson Mountain to the northeast. Above the sinkhole site are cliffs of "red rock" sandstones of the Schnebly Hill formation which are capped by cream colored, cliff-forming Coconino Sandstone and Kaibab Limestone. These strata extend upward to an elevation of 6800 feet.

The sinkhole lies near the base of the Schnebly Hill formation close to its boundary with the underlying Permian age Hermit formation. The Hermit formation is about 250 feet thick and contains minor interbeds of shaly-siltstone within the dominantly red, well bedded sandstone. The Hermit formation rocks erode to form broad, low-relief, slope-forming benches. Most of Sedona is built on Hermit formation erosional benches. Fresh exposure to the elements causes newly blasted rockcuts of Hermit formation strata to decrepitate over a period of a several years. Exposures of this incompetent strata can be seen in sloughing roadcuts along Highway 89A just west of the Sedona Post Office and further to the west of town. At relatively shallow depths beneath town level lies the competent Supai, or Esplanade, Sandstone of Pennsylvanian-Permian age. The Supai is not exposed near West Sedona but is well exposed in a higher standing fault block to the northeast of "Uptown" Sedona near Midgley Bridge on Highway 89A. At that location it is a cliff-former. The Supai Sandstone is about 450 feet thick and is composed of massive reddish sandstone. Hermit formation and Supai Sandstone strata that underlie the Devils Kitchen sinkhole do not contain an appreciable carbonate content and, by themselves, do not contribute to the development of karst topography. The base of the Supai Sandstone is not exposed locally but elsewhere in the region it rests disconformably upon the karsted surface of the underlying Redwall Limestone.

Redwall Limestone of Mississippian age underlies the Sedona area and plays a key role in the development of subsurface solution cavities as well as being a major aquifer for the city water supply. The 180 foot thick limestone is not exposed in town but is quarried for the manufacture of cement in Clarkdale on the west side of the Verde Valley. The type section for the Redwall is within the Grand Canyon where its upper surface displays a karsted weathering surface that dates back to Late Mississippian time. Shortly after deposition the very pure marine limestone was raised above sea level and subjected to subaerial solution weathering. The karst topography and incised erosion channels at the top of the Redwall Limestone within the Grand Canyon are locally filled with Surprise Canyon formation sediments (Beus, 1990). Some portions of the extensive Mississippian Redwall Limestone cave system in the Grand Canyon area are believed to have resisted collapse and remained as open cavities long after burial by Permian Supai Sandstone and younger strata. It is not known if any of these open Mississippian age cave systems ever existed in the Redwall Limestone that underlies Sedona. It remains as a possibility that some parts of this old cave system were re-activated within the last 8-10 million years.

Martin Dolomite of Devonian age underlies the Redwall Limestone and is approximately 450 feet thick. The dolomite is kown to contain small modern caves where theses rocks are exposed on the slopes of Mingus Mountain to the west of Sedona. Beneath the Martin Dolomite is the Tapeats Sandstone of Cambrian age which rests unconformably on Precambrian basement rocks. The closest outcrops of the 1750 million year old Early Proterozoic rocks occur at Jerome, 22 miles to the southwest.

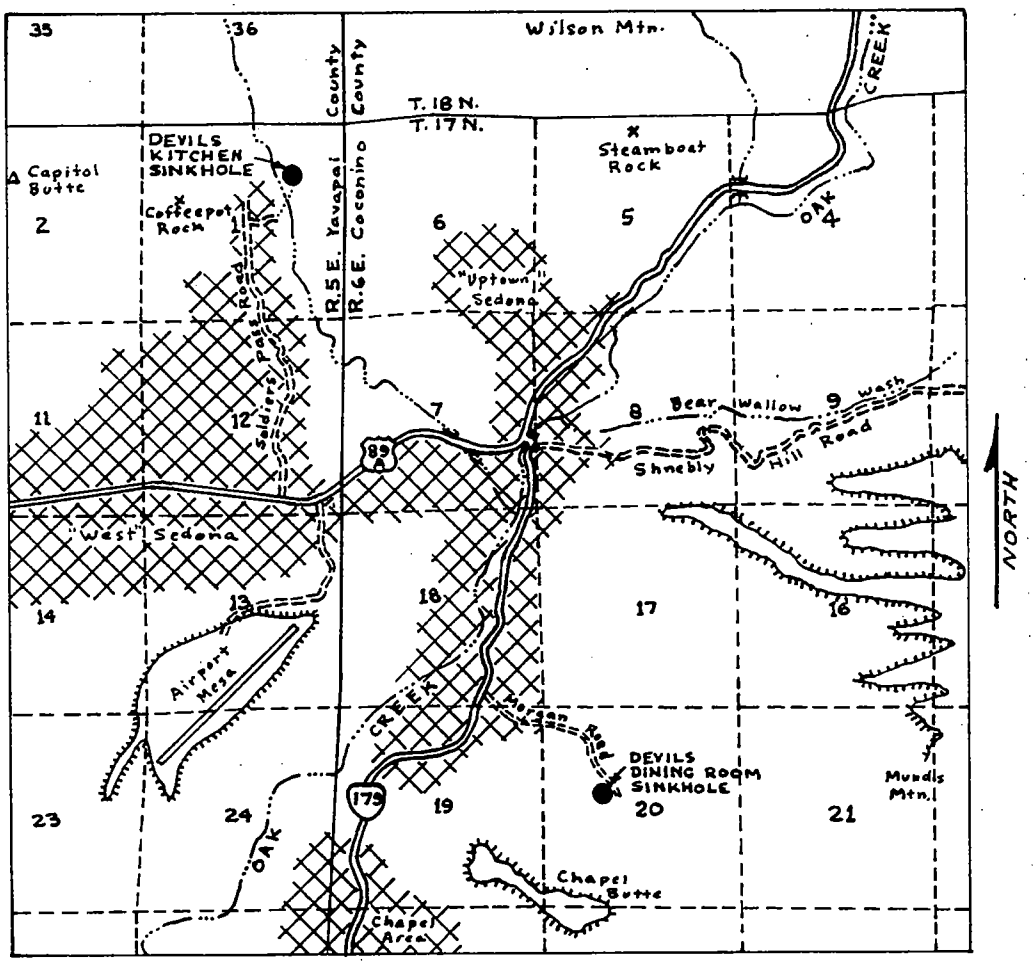
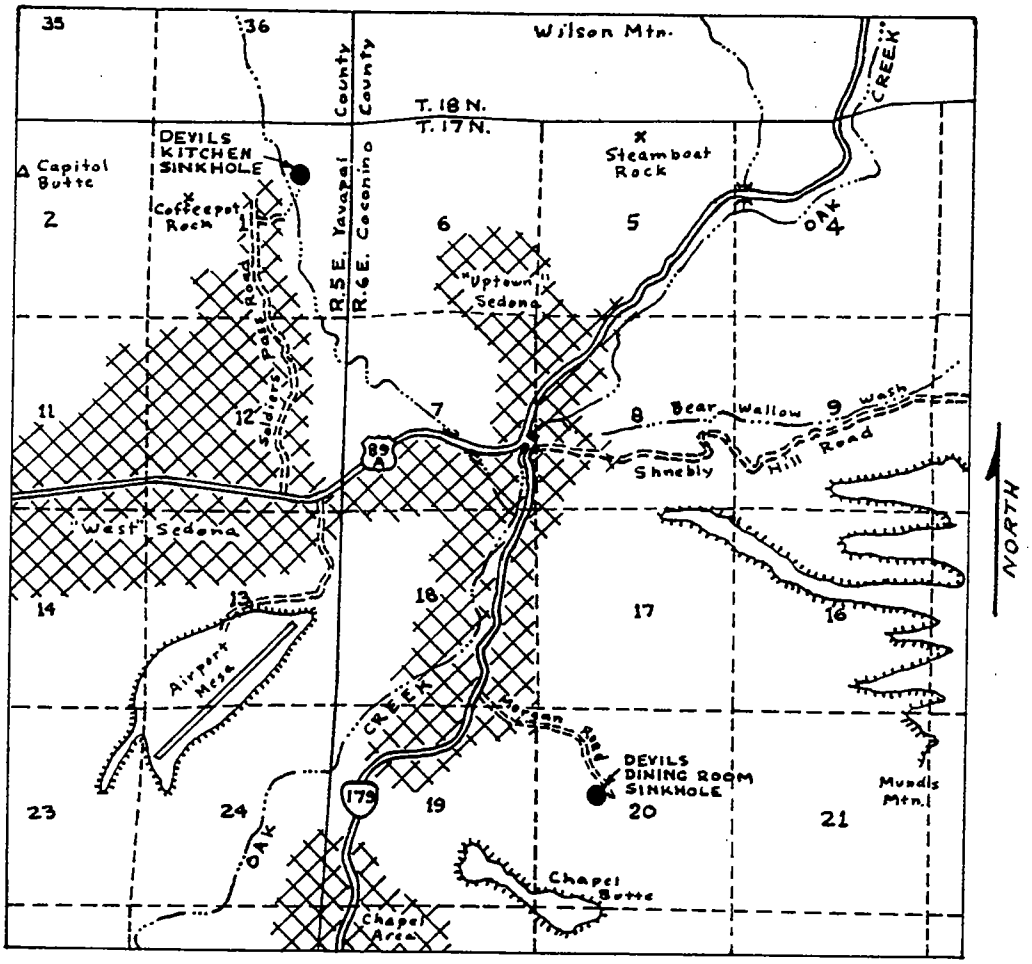


Figure 1: Location map of the Devils Kitchen and Devils Dining Room sinkholes, Sedona, Arizona.

Quaternary:	Alluvium
Tertiary:	Basalt lava flows
	---------- Unconformity ----------
Paleozoic:	Permian; Kaibab Limestone
	Permian; Coconino Sandstone
	Permian; Schnebly Hill formation; red sandstones
	(Includes thin Fort Apache Limestone member)
	Permian; Hermit formation; red sandstone and shaly sand/siltstone
	Pennsylvanian/Permian; Supai (or Esplanade) Sandstone
	---------- Unconformity and karst development ----------
	Mississippian; Redwall Limestone
	Devonian; Martin Dolomite
	---------- Disconformity ----------
	Cambrian; Tapeats Sandstone
	---------- Unconformity ----------
Precambrian:	Early Proterozoic basement; 1750 million years old at Jerome, Arizona

Figure 2: Simplified stratigraphic column for the Sedona area.



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Sedona Water Wells and Solution Caves in the Subsurface Redwall Limestone:

Sedona obtains its water supply from numerous wells that penetrate down to the Redwall Limestone aquifer. The regional water table lies midway between ground surface and the subsurface limestone. In-hole video cameras have been used to monitor some of these water wells. The resulting tapes provide an outstanding visual record of conditions below ground level (Arizona Water Company files). The video pictures show that where a drill hole passes through Supai Sandstone the sidewall caliper of the hole remains uniform and the wallrock is solid in appearance. But upon entering the underlying Redwall Limestone the drill hole passes through limestone that is often riddled with water-filled solution cavities that resembles the texture of a sponge cake. One of the wells encountered many scores of feet of water-filled cave openings with no sign of rock observable within the range of the camera's lights. This type of visual display clearly proves the existence of open, water-filled cave systems deep beneath the Sedona area. Flow rate tests have shown that relatively rapid water movement can take place between water wells that are some distance apart (personal communication, Arizona Water Company personnel, March, 1998). Presumably this water can move quite rapidly through interconnected Redwall cave systems.

Regional Faulting and Joint Patterns:

The dominant regional joints in Sedona area rocks trend in a northwest direction and their presence has a strong influence on the erosion of the landscape. These joints are readily seen from the air and they occcur in swarms of varying frequency. No significant displacement of strata can be observed in a typical joint but the narrow fractures allow for localized groundwater penetration of bedrock. When the joints are more numerous and closely spaced, say every 5-20 feet apart, they permit a greater degree of surface weathering to take place where the rocks are most fractured. When the joints are spaced farther apart the landmass is more resistant to erosion and a high standing butte results. Many of the local high standing buttes in the Sedona area are elongate in a northwest direction and have resulted from the differential weathering of northwest-trending joint sets that bound each side of the ridges. Subordinate joints, typically crossing the dominant joint set at a high angle, also cut through the region. In the vicinity of the Devils Kitchen sinkhole the dominant joint set is N.50^{0}W. and the subordinate joint set is N.15^{0}E. It is assumed that these intersecting joint sets allowed more groundwater to percolate downward at this site and permit the subsurface solution of limestone cave openings in the underlying Redwall Limestone.

Some of the northwest-trending joints contain basalt dikes which once fed overlying lava fields. One conspicuous dike passes through the southern wall of the Devils Dining Room sinkhole. It can be assumed, therefore, that the subsurface cave opening that has recently collapsed at that location must have been generated since the time the lavas were extruded in Miocene time between 8-15 million years ago. Another dike passes between the central spires of Cathedral Rock which is located about 5 miles to the southwest of Devils Kitchen. These dikes probably broke through the earth's crust along marginal fractures associated with the earliest stage of development of the Verde graben (Lindberg, 1983; 1986) . Erosion over the last 8 million years or less has removed the lava cap that once overlay the Sedona area sediments. No dikes have been found to date in the immediate vicinity of the Devils Kitchen sinkhole but the presence of sub-parallel dikes occur further north within the Dry Creek drainage.

Description of the Devils Kitchen Sinkhole:

Figure 3 shows a plan map of the Devils Kitchen sinkhole. The edges of the collapse zone are almost entirely free of soil cover. The steep inner walls can be viewed from all angles to see the dominant and subordinate joint patterns. All walls display portions of perfectly flat and very steep joint faces where rock slabs have fallen from the sinkhole walls in past collapses. The northernmost wall still shows a thin caliche layer and numerous tree roots that still cling to the vertical face following the 1989 cave-in. The older edges dating from the "1880s" collapse appear to be more rounded. The center of the sinkhole is entirely filled with broken rubble that lies 50-60 feet below the edge of the collapse area, as shown in Figures 4 and 5. The present surface opening measures 90 by 170 feet (27 by 52 meters) but the maximum undercut dimension is estimated to be about 110 by 225 feet (34 by 70 meters). All four walls display undercut edges that contain arcuate fractures that enlarge downward. The anomalous arcuate fractures cut across the pre-existing regional joint sets and can be examined at close range near the southwestern corner of the sinkhole. Recognition of the significance of these arcuate fractures is essential in predicting potential sites where collapse breccias have not yet broken through to surface.

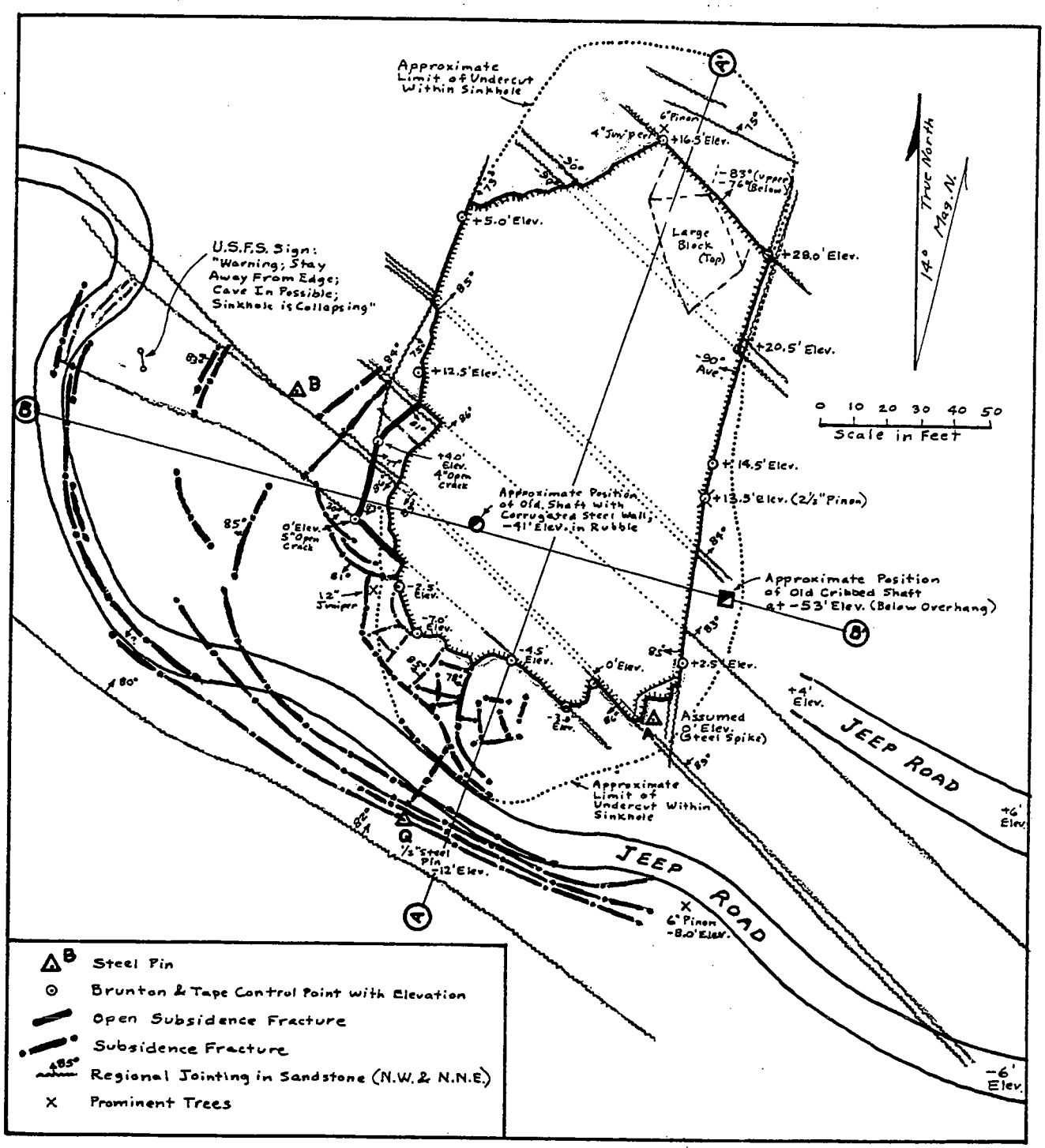

Figure 3: Plan map of the Devils Kitchen sinkhole, Sedona, Yavapai County, Arizona. The sinkhole is located in the NE 1/4, Section 1, Township 17 North, Range 5 East. Bedrock surrounding the sinkhole is composed of massive red sandstones near the contact between the Schnebly Hill formation above and the Hermit formation below. The map shows the present-day shape of the sinkhole which, according to pioneer records, collapsed in the "early 1880s". A dramatic enlargement took place in late 1989 when the northern third of the sinkhole fell in.

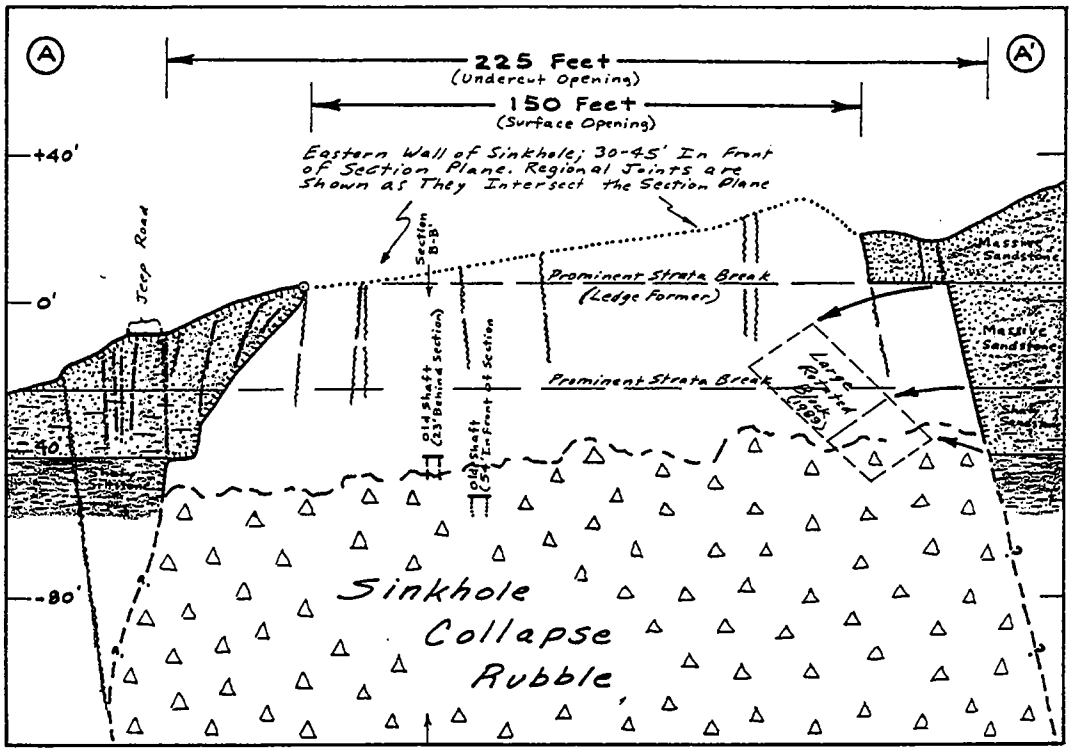

Figure 4: Longitudinal section through the axis of the Devils Kitchen sinkhole, viewing toward the west-northwest. The overhanging southern edge (left) marks the oldest portion of the sinkhole, while the northern edge (right) collapsed in late 1989. Note the downward enlarging arcuate fractures superimposed on the northwest-trending steep bedrock joints which are in the line of view.

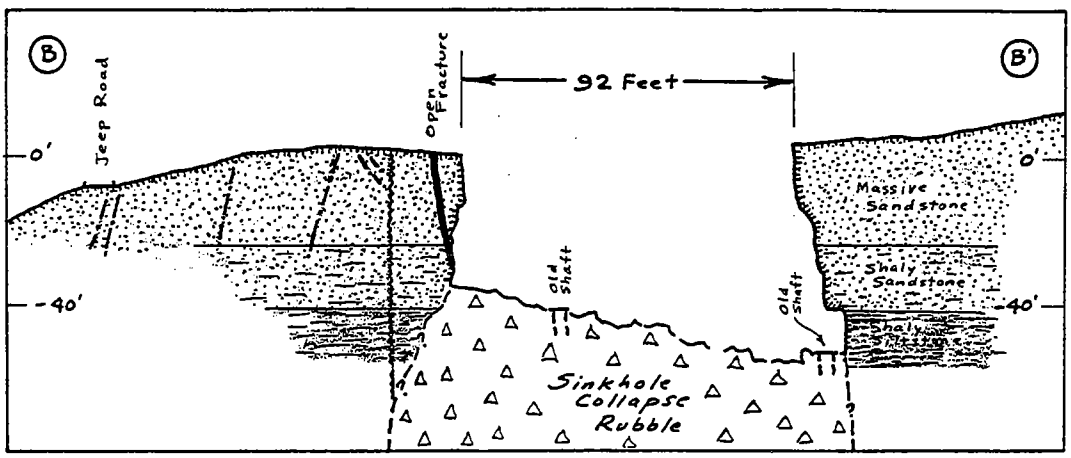

Figure 5: Cross section through the south-central portion of the Devils Kitchen sinkhole. Note the undercut eastern side (right) of the sinkhole where the softer shaly siltstone has caved in more than the more resistant capping sandstone. The western side contains a chunk of bedrock that is beginning to detatch from the wall and is soon expected to collapse.

A partially loosened block of bedrock, 10 feet wide by 40 feet long (3 by 12 meters), can be seen on the western wall of the sinkhole where it has separated from the rest of the outcrop by a space of 5-6 inches (13-15 cm). That block, and part of the severely undercut north face, are thought to be the most unstable masses that are expected to fall into the sinkhole walls in the relatively near future. Arcuate surface fractures can be found for up to 100 feet (30 meters) to the west of the present opening. These fractures dip steeply away from the sinkhole opening and clearly imply withdrawal from below. Along with the other unstable walls to the north, east, and south, it indicates that the sinkhole will ultimately enlarge to a hole about 250 feet (80 meters) in diameter. It is unknown how long it will take before the period of collapsing would eventually stabilize.

Conceptual Model for Generation of the Devils Kitchen Sinkhole:

Figure 6 presents a hypothetical cross section through the Devils Kitchen sinkhole that portrays a conceptual model for its generation. The model requires that carbonate-poor Hermit formation and Supai Sandstone caprocks are collapsing downward into a large subsurface cavity of some sort. The only logical opening of this magnitude would be a cave within the underlying carbonate strata that includes the Redwall Limestone and Martin Dolomite. The topmost surface of the carbonate strata is estimated to be 660 feet (200 meters) below the present sinkhole opening. Because broken rock occupies a greater volume than solid rock the breccia that fills the collapse column must reach from the top of the breccia fill (50 feet below the sinkhole surface) to the bottom of an original cave opening. It is estimated that a 300 foot diameter (90± meters) opening must have once existed at depth. Since this is more than the thickness of the Redwall Limestone itself it is possible that the cave may have extended downward into the underlying Martin Dolomite. If the original cave opening was largely confined to the Redwall Limestone itself it must have been much broader than 300 feet across.

Recognition of Anomalous Arcuate Fracture Patterns Associated with Sinkhole Development:

By far the most significant feature expressed by the Sedona sinkholes is the presence of anomalous arcuate fracture patterns that surround the sites. The only known method of creating fractures of this type is by collapse of surface strata into a hidden subsurface cavity. The only logical explanation for the subsurface cavity is the presence of a solution cave in the underlying limestone. It is theoretically possible that before a new collapse breccia breaks through to the surface its presence can be deduced from a detailed study of the bedrock fracture patterns. Although it has not yet been put to the test in practical terms a search for such fractures should be made prior to the commencement of any major engineering project. Sinkholes that are hidden from view and completely filled by alluvium present a different problem as discussed below. Geologists and engineers should be made aware of this special type of bedrock fracture as an early warning sign that may precede a sinkhole collapse.

Features of Other Sinkholes in the Sedona Area:

Four sinkhole sites examined to date in the Sedona area share a number of common features. All occur on bedrock sites where Permian sandstones crop out. They all show evidence of recent enlargement and downward collapse as evidenced by sharply defined rock edges, deep rubble-filled cores, unstable sidewalls, and tree roots still clinging to recently exposed joint faces. All sinkholes display prominent northwest-trending joints in their exposed walls. Every site examined so far displays some degree of arcuate fracture patterns in the bedrock surrounding the collapse area. One site only, lying well to the west of town and about a mile north of the Sedona wastewater treatment plant, shows some bleaching of iron oxide from the red sandstone strata off to one side of the sinkhole. Whether or not this is related to sinkhole development remains unresolved.

Possible Means of Detection of Alluvial Covered Sinkholes:

Alluvial covered sinkholes present a major problem of detection. Some suspected sites may shunt surface water and/or septic waste directly back to the limestone aquifer in gravel-filled sinkholes. This has already raised concerns in Sedona that has led to startup of installing a proper sewer system. It is possible that seismic survey techniques could be applied to test for hidden, gravel-filled sinkhole sites. Geophysical experience from other regions where karst topography exists should be solicited prior to commencement of major engineering projects if the possibility of hidden sinkholes is suspected.

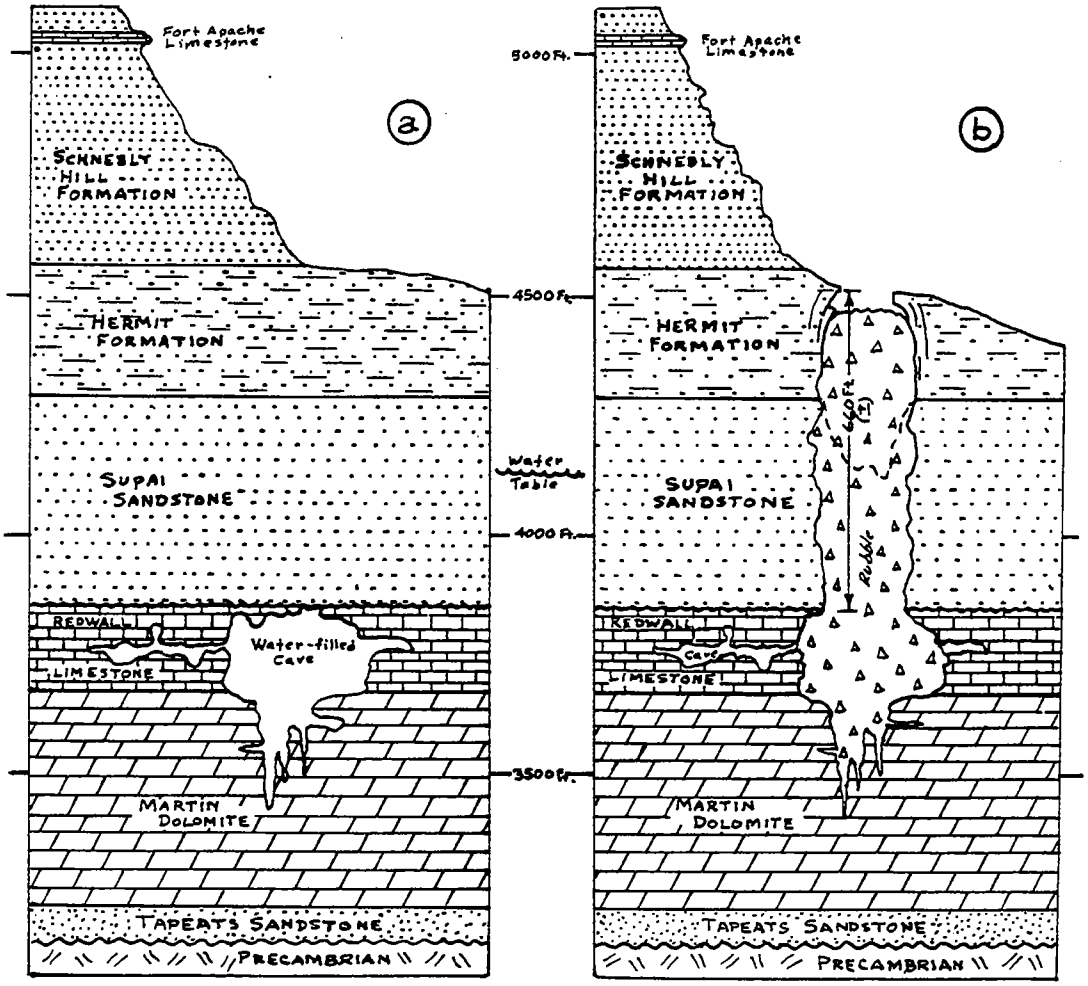

Figure 6: Hypothetical cross sections through the Devils Kitchen sinkhole that portray a conceptual model for its generation. (a) Pre-collapse reconstruction of strata and a hypothetical open solution cave in the subsurface Redwall Limestone and Martin Dolomite. (b) Post-collapse scenario whereby the overlying rocks have collapsed into the former cave opening. It is not known how long it would take for the upward migration of the collapse breccia to propogate up to the surface. In this case it has just recently breached through to the modern erosion surface.

References:

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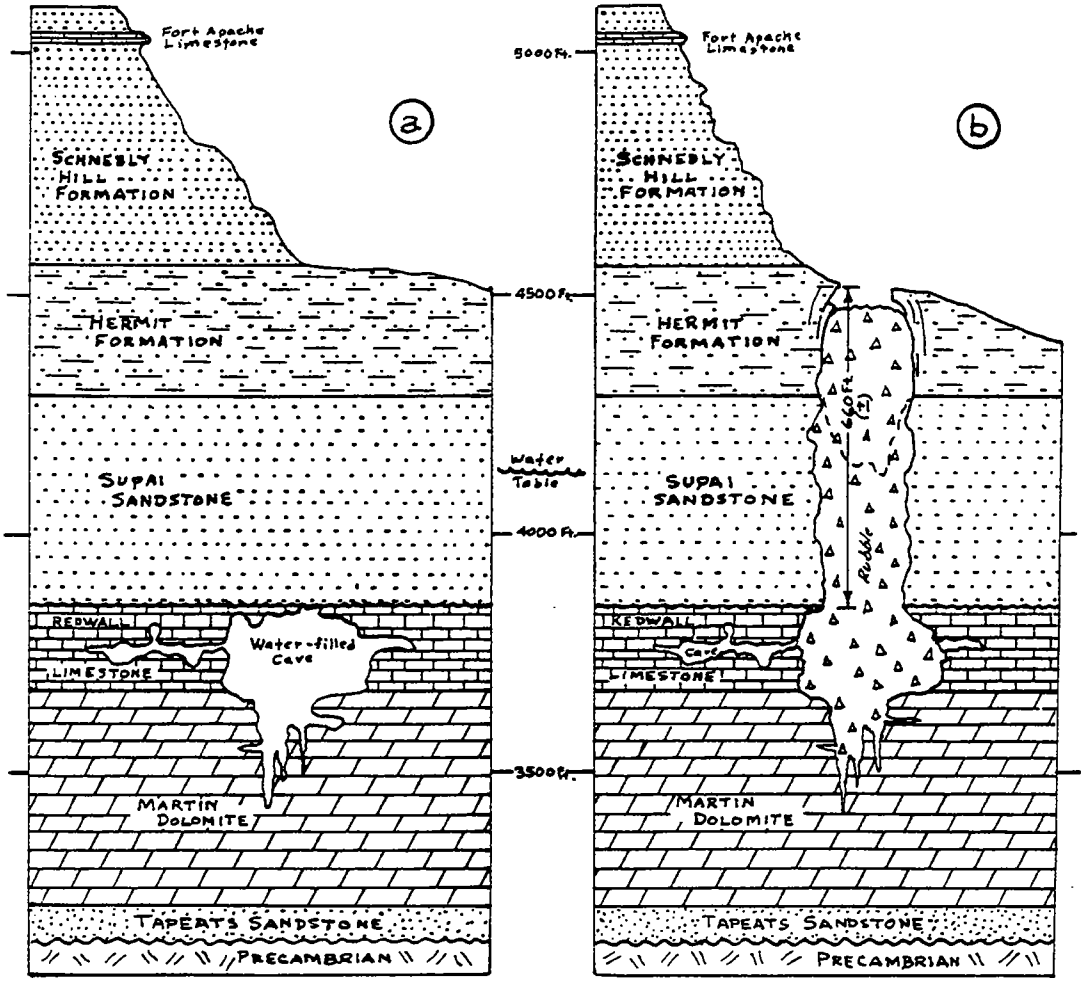

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GEOLOGIC INVESTIGATION AND CONSTRUCTION OF THE SPRING RIVER BRIDGE

BY

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ABSTRACT

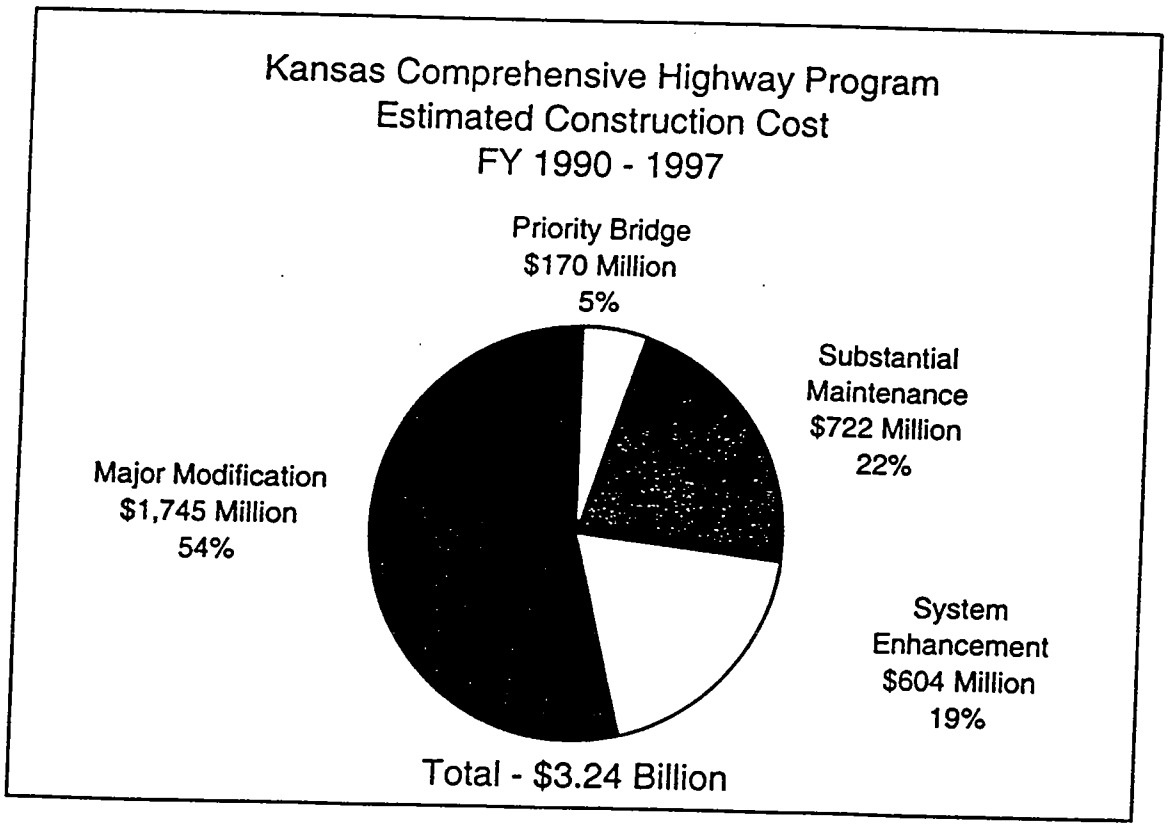
The Kansas Department of Transportation is in the final stages of a 3.24 billion dollar "Comprehensive Highway Program". Included in this massive highway system renovation is the realignment of US-69 highway across the Spring River in southeast Kansas. This small region of Kansas is typically known as the "Ozark Plateau". The Ozark topography is characterized by heavily timbered hills combined with deep cut valleys.

The geology of this region consists of interbedded limestones and cherts of the Warsaw-Keokuk Formation, which is Mississippian in geologic age. This formation is subject to numerous post-depositional alterations: solution cavities, hydrothermal mineralization, clay- filled fissures and interconnecting solution tunnels.

Diverse drilling and investigative procedures contributed to an accurate useful preliminary investigation of the high-variable bedrock. K-DOT geologists and structural engineers combined to design bridge foundations that were constructed in a timely, cost effective manner.

INTRODUCTION

In 1988 the Kansas Legislature passed the Comprehensive Highway Program. This 8 Year, 3.24 billion dollar program included, Priority Bridge, Substantial Maintenance, Major Modification, and System Enhancement projects. Funding for the program was done by three methods: 1) state funded 2) combination of Federal-aid and matching state funds 3) combination of Federal, State and local funds. The Spring river project was a portion of the Major Modification section of the program and was funded by a combination of Federal and State funds.



GEOLOGIC SETTING

The Spring River flows from South Central Missouri through the Southeastern corner of Kansas and into Grand Lake of the Cherokees in Northeastern Oklahoma. The Spring river drains over 6,000 square kilometers of land area. This land region is labeled as an Ozark Physiographic Province or Ozark Plateau. The Ozark Plateau covers most of southern Missouri and northern Arkansas, parts of northeastern Oklahoma, and the extreme southeastern corner of Kansas. The topography is characterized by steep limestone ridges and knobs separated by narrow valleys. For generations locals have called this region the "Ozark Mountains" because the tremendous topographic relief between ridges makes travel difficult; even today, most state highways in the region do little more than wind up and down the countless hills. The Ozarks are heavily forested with second-growth hardwoods--oak and hickory, primarily--and long vistas are dominated by a thick canopy of leaves that shrouds the terrain. Streams in this region are clear and cold, usually fed by springs, and streambeds are filled with pebbles of a red-brown variety of chert called jasper. Because of the Ozark's natural beauty and, until recently, its sparse population, the area has been a popular vacation destination for most of this century.

Bedrock in the Kansas portion of the Ozark Plateau ranges from lower Pennsylvanian to Upper Mississippian in age. The Pennsylvanian sediment is a dark gray conglomeritic shale representing the Krebs Formation. Primarily a clay shale, this unit contains pyrite nodules and layers of weathered chert fragments. The pyrite nodules are a secondary mineralization derived from the weathering of the overlying Pennsylvanian coal seams. The chert fragments incorporated into the shale from the local topographical highs in the underlying limestone. These Pennsylvanian rocks are resting unconformably on the cherty limestone of the Warsaw-Keokuk formation. This formation was post-depostionally exposed and eroded into a classic example of Karst topography. The shales of the Krebs formation were than deposited into the Karst valleys. Erosion has remove all but the very deepest deposits of Pennsylvanian sediment. The maximum thickness of shale observed was approximately 3.2 meters. The Warsaw-Keokuk Formation is a light gray to gray brown limestone with an abundance of gray chert. This chert can be found as individual nodules, mottled with limestone fragments, or as massive thick beds. At some locations, the bedrock just below the mantle is weathered and brittle for several tenths of a meter, at

others, very competent chert and limestone are found just beneath the alluvial material. Fractures and jointing in the limestone was prominent to depths of 20 meters or more even the most competent section of the formation. Thin shale seams are present at oblique angles to the bedding planes of the limestone. These shale seams are injections of Pennsylvanian sediment into the weathered cracks and fissures of the limestone. Other Karst features that are prevalent in this area are solution cavities. These cavities range from a few meters to over 35 meters deep. Most of the cavities near the surface are filled with clay. Nearly all of the larger voids have been partially or completely filled with limestone "pellets". The pellets are thought to have been precipitated directly from the lime-rich ground water that move through the bedrock in recent times. This pelletal limestone varies in the amount of calcite cementation, but the resulting rock is always much softer than the surrounding host rock. Knowing the diverse geologic problems we were to encounter, a variety of investigative measures were required.

GEOLOGY INVESTIGATION

Our bridge designer requested a geology investigation based on the construction of a 10 pier structure. This bridge was to be 1735 feet long and span the Spring river and approximately 1000 feet of its flood plain.

The Geotech investigation of the bridge site utilized; power augers, core drills, Standard Penetration tests and resistivity surveys.

Power Augers soundings were made at all accessible foundation locations. A float and small trailer mounted power auger were used to drill out the pier in the middle of the river. These soundings gave us depth to bedrock, extent of weathering present and type of overburden. The carbide tipped bits did not with stand the very abrasive chert.

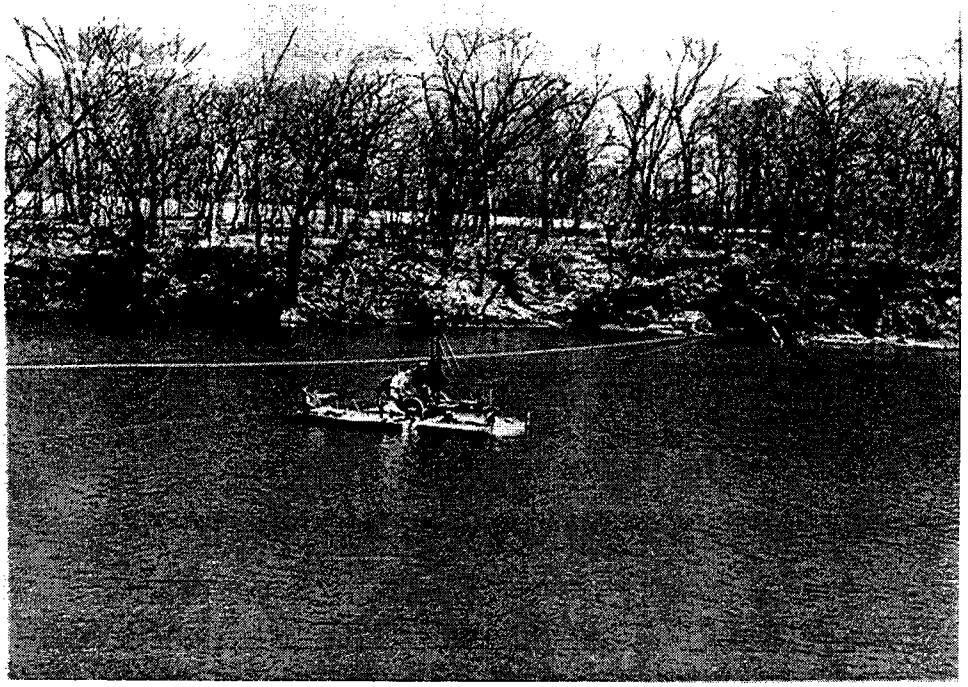
Core drill soundings were advanced at each foundation element, if possible. The larger rigs required a large amount of brush and timber to be cleared. Diamond bit core barrels were used for all borings. In coring 5 sites 14 bits were used. The weathered top of the bedrock allowed broken pieces of chert and limestone to fall into the boring hole causing severe wear on the diamond bits. In one hole an angular piece of chert jutted from the wall and nearly cut the outer corebarrel in two. Select specimens of the core were tested for unconfined compressive strengths. These tests resulted in strengths of 300 to over 900 tsf.

In areas of deep solution cavities filled with the pelletal limestone, Standard Penetration tests were run to obtain relative strengths of the

material. These tests had a variety of results ranging from 2 to 80 blows per 0.5 feet.
Several attempts to retrieve core samples of the pelletal material were unsuccessful.

The use of resistivity surveys were extremely helpful in delineating the top of bedrock and the extent of the solution cavities. Surveys were also utilized at foundation locations inaccessable to the truck mounted auger and core rigs. To have a better understanding of the resistivity information, preliminary surveys were run between drill holes. Interpretation of the results were than considered to have a much higher degree of accuracy.

CORE DRILL SITE AT PIER #2



POWER AUGER SOUNDINGS USING FLOAT AND TRAILER MOUNT DRILL

ENGINEERING RECOMMENDATIONS

The field information was compiled into 3 basic sections: 1) General plan notes 2) Pile foundations 3) Spread footing foundations.

Due to the Karst features found in the bedrock, several problems needed to be addressed., Although individual samples of the chert and limestone are highly competent, extensive fractures in the bedrock, along with the presence of numerous voids, makes lighter design loads necessary. Rock excavation on this structure will be extremely difficult due to the hard and abrasive nature of the chert. Substantially greater excavation times and equipment wear should be expected .

Pile foundations were designed for both abutments and pier #1. Pier #1 fell directly in a very large, irregular, deep, pelletal filled, solution cavity. This cavity was found to be in excess of 67 feet deep. Scouring of the pelletal material was a great concern. We therefore recommend, predrilling all pile at pier #1 to ensure that a minimum penetration is achieved. The final pile enbedment below the predrilled depth will vary and should be based on computed bearing during driving.

Abutments for the structure were also founded on pile. We recommended a minimum of 9 ksi. Due to the close proximity of bedrock at abut #1 predrilling will be required. All pile should be fitted with pile points to facilitate driving through the heavy gravel zones in the mantle.

Spread footings were designed for all other pier locations. Drilled shafts were not recommended due to the fractured nature of the rock and extreme difficulty in the drilling of such large diameter holes in this type of bedrock. Design pressures were recommended at 958 kPa or 10 tsf. At the base of each footing, the contractor was to drill one test hole per 25 square feet of footing area. The test holes should be drilled a minimum of 4 feet

deep. This is to make sure no solution cavities exist beneath the footings. All spread footings are to be placed a minimum of 8 feet into the bedrock.

CONSTRUCTION

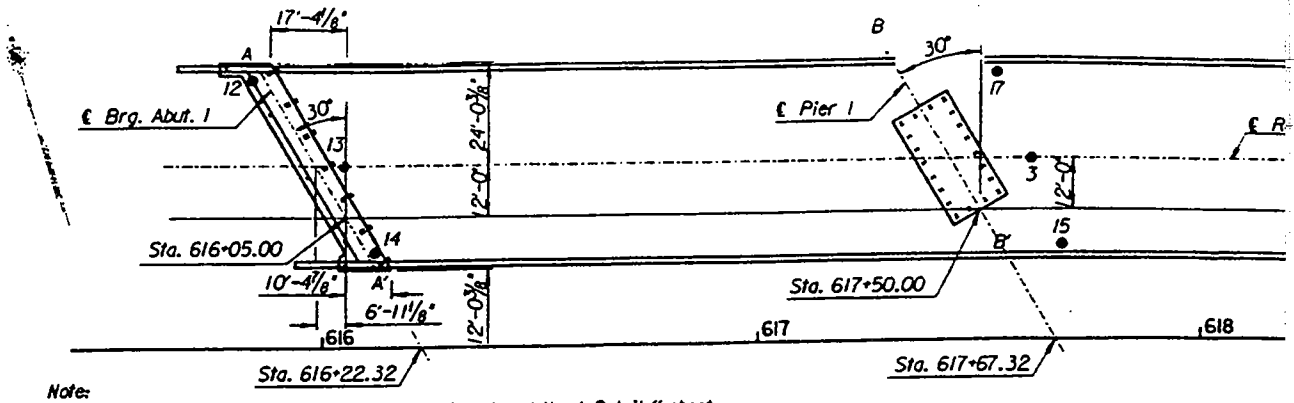
The final design for the structure was to construct a composite welded steel plate girder bridge. Piers 1 through 6 were set on a single 10 X 27 foot spread footing. The structure over these piers is supported by a single column, set at a 30 degree skew. Piers 7 through 10 were founded on 3 separate spread footings and columns. These spread footings were to be 8.5 X 7 foot set perpendicular to the roadway.

This contract has several environmental issues involved in it. 1) This area is winter habitat for Bald Eagles. No work may be done will an eagle is within 1/4 mile of the work site. 2) All brush cut is to be dozed into piles for spotted skunk habitat. However, no spotted skunks have been seen in this area. 3) The Spring River with its chert gravel bottom is prime habitat for the Neosho Madtom. (a 3 inch catfish) No work may be done in the river from May 1 to July 15. 4) Due to the number of mature trees removed for the construction of this bridge, 180 vary species of trees had to be planted and maintained for a minimum of 3 years.

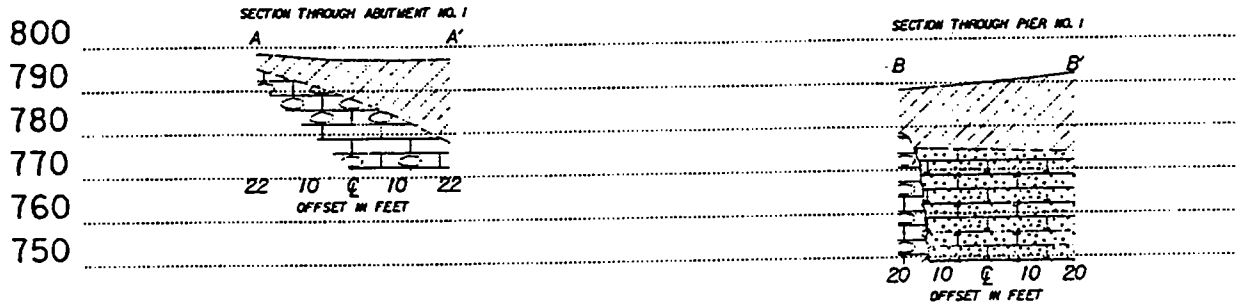
Construction of the Spring river bridge began on April 10, 1997 and is to be completed on August 28, 1998. Excavation for the spread footings went as stated in the plans, very slow and difficult. The contractor used a hydraulic hoe-ram and needed 20 or more days to excavate the first two footings. Seeing that he would not meet the calendar completion date at this rate, the contractor brought in a rock drill and predrilled all footing locations. This reduced his time to approximately 10 days. The issue of letting the contractor blast was brought up on every footing. The contractor finally understood the reasons for not blasting on the construction of the final two pier foundations. Pier #2 had been poured and the excavation was completely full of groundwater, as the contractor began excavating in Pier #1, he encountered a massive clay filled joint. This joint was directly connected to Pier #2. As the clay was removed, water flowed into Pier #1 at an approximate rate of over 200 gallons per minute. A shot set off in Pier #1 may have severely damaged the green concrete in the adjacent footing.

Pier #1 is set directly over a very irregular, deep solution cavity. Moving of the pier was not an option due to the lateral extent of the cavity. So, we had to come up with a foundation in material that was extremely variable. Soundings indicted that competent bedrock was between 5 and 67

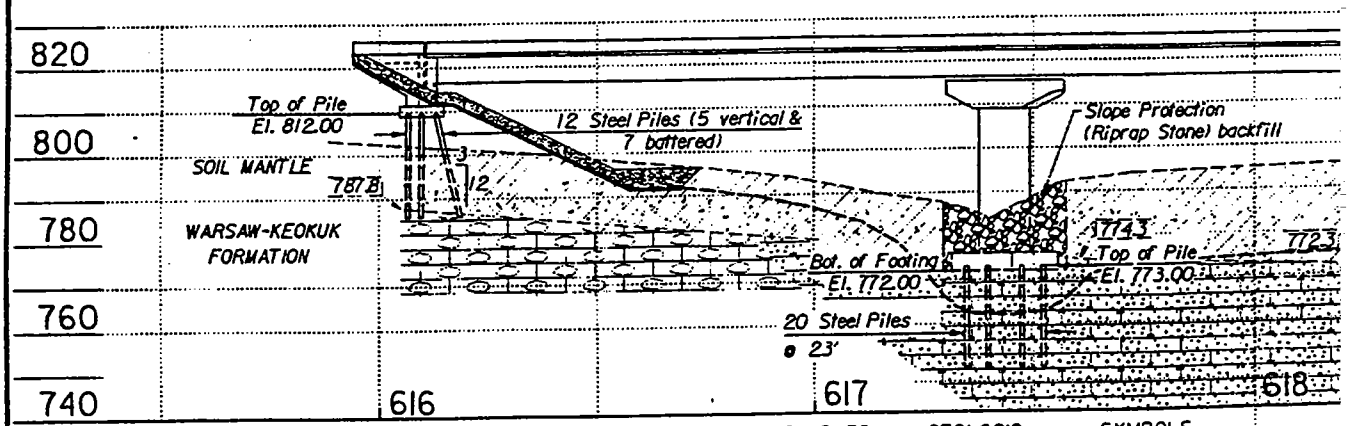
feet beneath the top of the solution cavity. We recommended predrilling 20 H-beams and than driving them to plan bearing. The biggest problem was that the contractor wanted us to predict the exact depth of bearing. Given the nature of a solution cavity, nothing more than a best quess was available. The plans called for 360 feet of pile and the end result was the driving of 711 feet, an over-run of not quite 200 percent!!



Note:
For Pile spacing at Abutment No. 1 see "Abutment No. 1 Details" sheet.
For pile spacing at Pier No. 1 see "Footing Details - Pier 1" sheet.



NOTE: Pier 1 piles shall be predrilled to Elevation 755.0.



STANDARD GEOLOGIC SYMBOLS

Clay
Silty Clay
Silt
Sand
Gravel
Boulders
Caliche
Silty Clayey Shale
Limy Shale
Black or Fissile Shale
Sandy Shale
Gypsiferous Shale
Weathered Shale
Sandstone
Shaly Sandstone
Siltstone
Gypsum
Limestone
Cherty Limestone
Shaly Limestone
Sandy Limestone
Weathered or Broken Limestone
Pelletal Limestone

SOUNDINGS
Coredrill
Power auger
Hand tools
Air hammer
Dutch cone penetrometer
Unconfined compression
Standard penetration
Water level
3/95

000.0 Elevation or from soundings
000.0 Actual so elevation
000.0 Drive s
000.0 Refusal
CASING DRIVE TEST

FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
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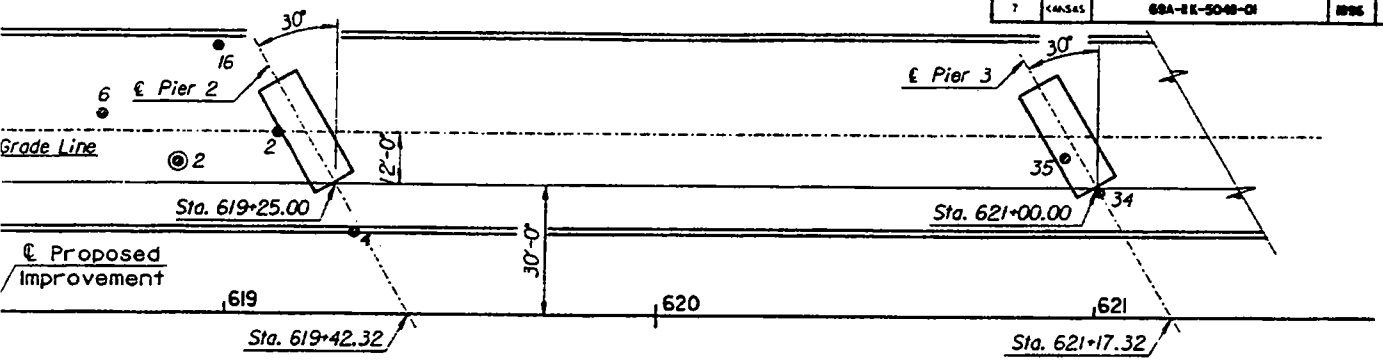
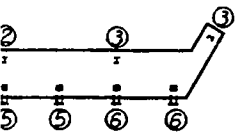
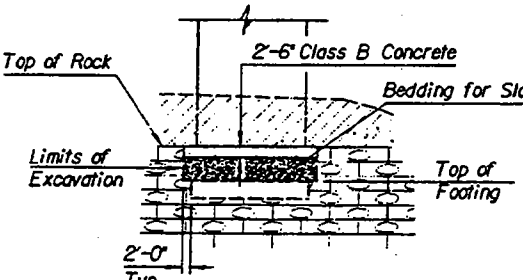




I PILE LAYOUT

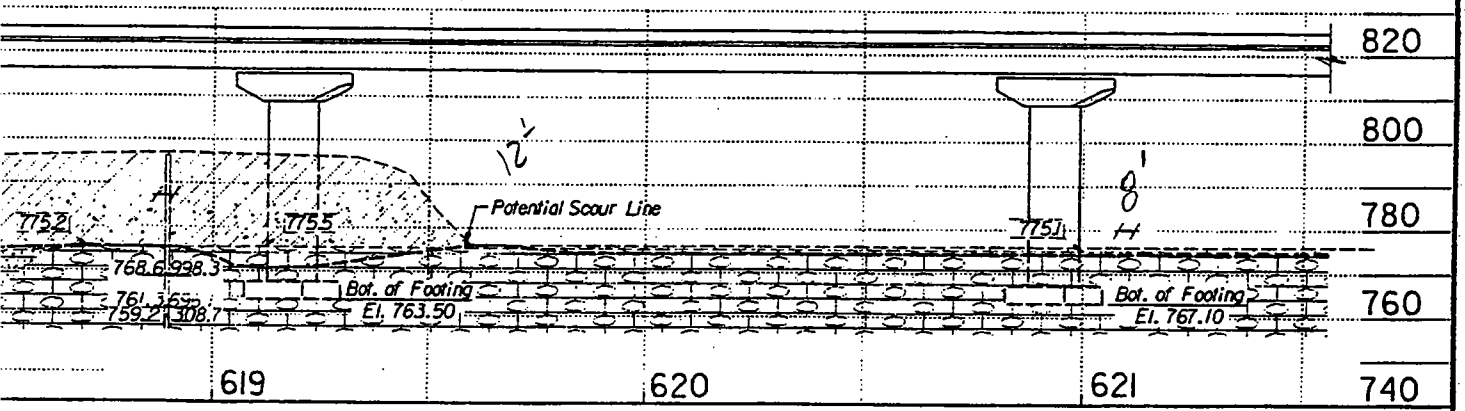
36' ④ - 37'
26' ⑤ - 27'
19' ⑥ - 20'



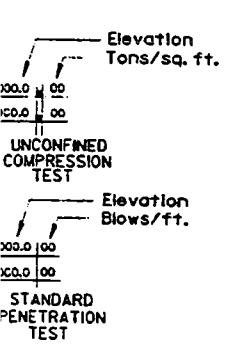
FOOTING BACKFILL DETAIL
(Piers 2 through 10)

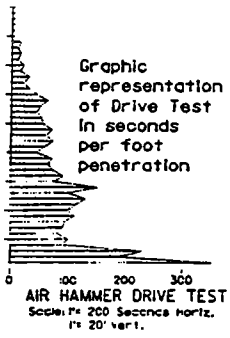
Note:
Special footing backfill shall be provided as shown for Pier No. 2 through Pier No. 8 and Pier No. 10.
Pier No. 9 shall receive all concrete backfill.
The limits of excavation shown are based on the "Bridge Excavation Standard" sheet. Quantities were calculated based on the limits shown.
Bedding for slope protection shall be Stone for Filter Course, Type I.
Any additional backfill required due to over-excavation outside the limits shown will not be paid for directly, but shall be subsidiary to "Class B Concrete" and "Bedding for Slope Protection".

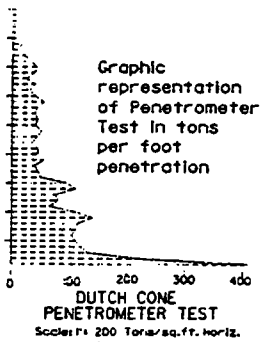
ad footings should be cut to exact dimensions
ete placed to neat lines within bedrock.



PROFILE







NOTE: The soundings shown on these plans are taken from notes in the field and represent the best information available. The logs of these soundings are in the files of the Kansas Department of Transportation and are available at their offices at Topeka, Kansas for inspection by interested and qualified bidders.

SCALE: I"= 20 Horiz. I"= 20' Vert.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

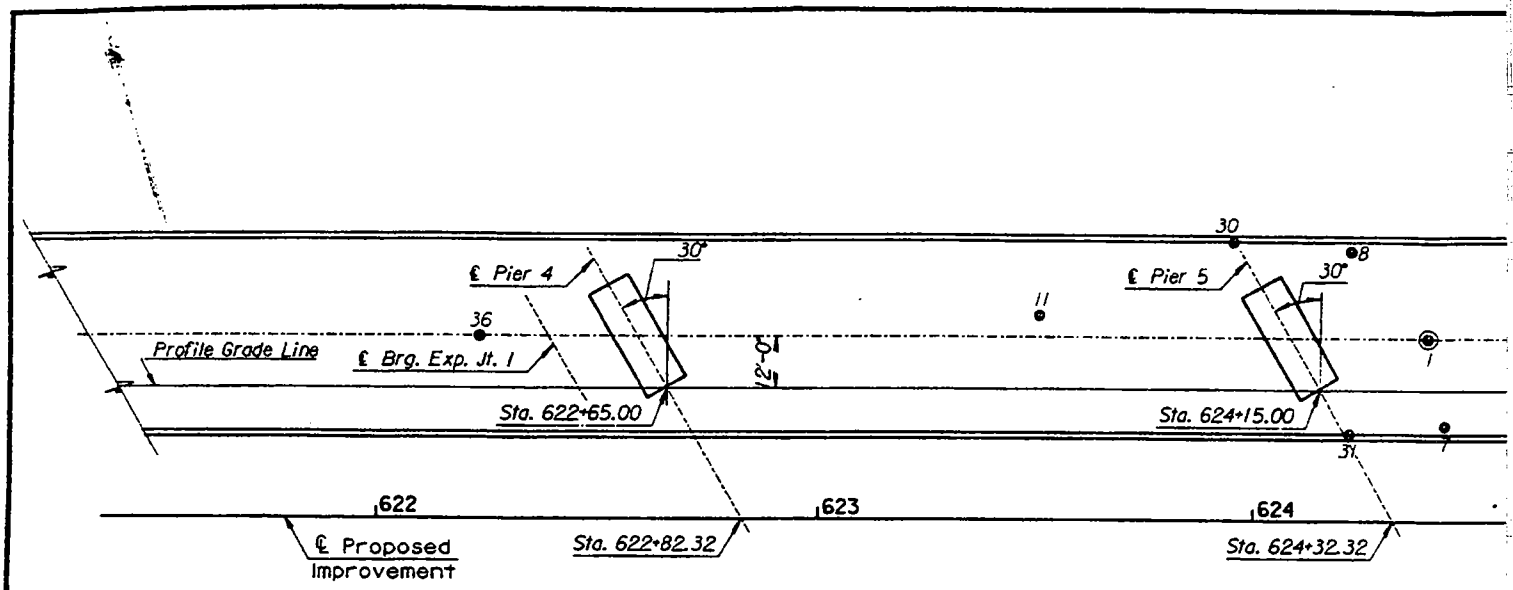
KANSAS DEPARTMENT OF TRANSPORTATION
BR. NO. 400-II-40.96 (104) STA. 624+70.00

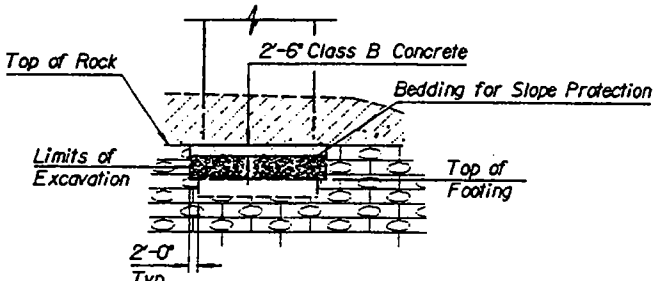
ENGINEERING GEOLOGY
US-400 OVER THE SPRING RIVER

PROJ. NO. 69A-II K-5048-01 CHEROKEE CO.

SHEET NO.	OF	SCALE		APP'D	
DESIGNED		DETAILED	SAK	QUANTITIES	TRACED
DESIGN CK.		DETAIL CK.	SMW	QUAN. CK.	TRACE CK.

130





FOOTING BACKFILL DETAIL
(Piers 2 through 10)

Note:
Special footing
for Pier No. 2 thro
Pier No. 9 shal
The limits of ex
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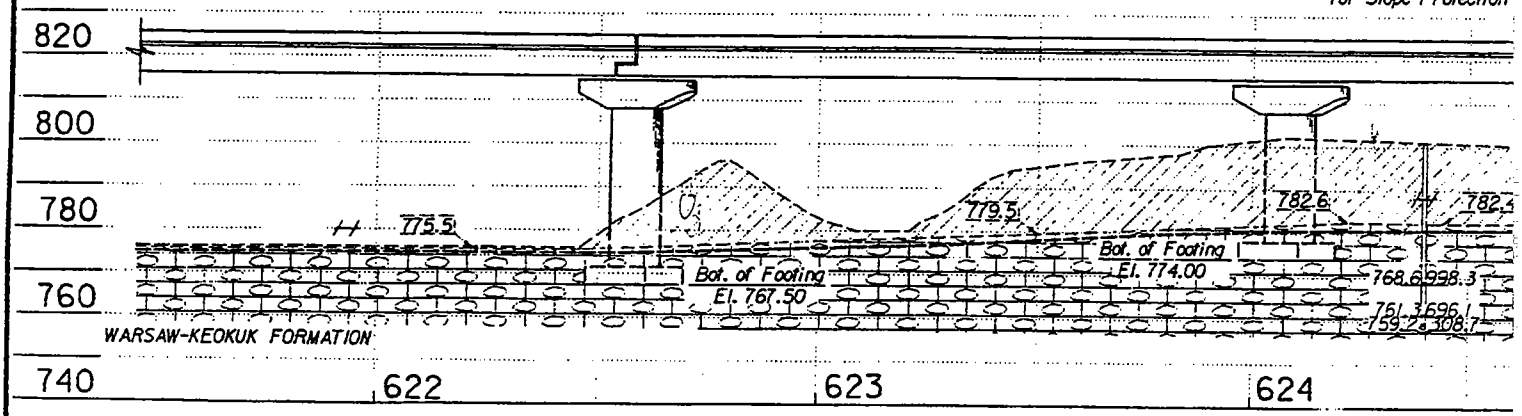
Bedding for slo
for Filter Course, Ty
Any additional b
outside the limits sh
shall be subsidiary t
for Slope Protection"



STANDARD GEOLOGIC SYMBOLS

Clay
Silty Clay
Silt
Sand
Gravel
Boulders
Caliche
Silty Clayey Shale
Limy Shale
Black or Fissile Shale
Sandy Shale
Gypsiferous Shale
Weathered Shale
Sandstone
Shaly Sandstone
Siltstone
Gypsum
Limestone
Cherty Limestone
Shaly Limestone
Sandy Limestone
Weathered or Broken Limestone
Pelletal Limestone

SOUNDINGS
Coredrill
Power auger
Hand tools
Air hammer
Dutch cone penetrometer
Unconfined compression
Standard penetration
Water level
12/94, 3/95

000.0 Elevation interpolated or from adjacent soundings
000.0 Actual sounding elevation
000.0 Drive started
000.0 Refusal

CASING DRIVE TEST

GEOLOGIC

FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
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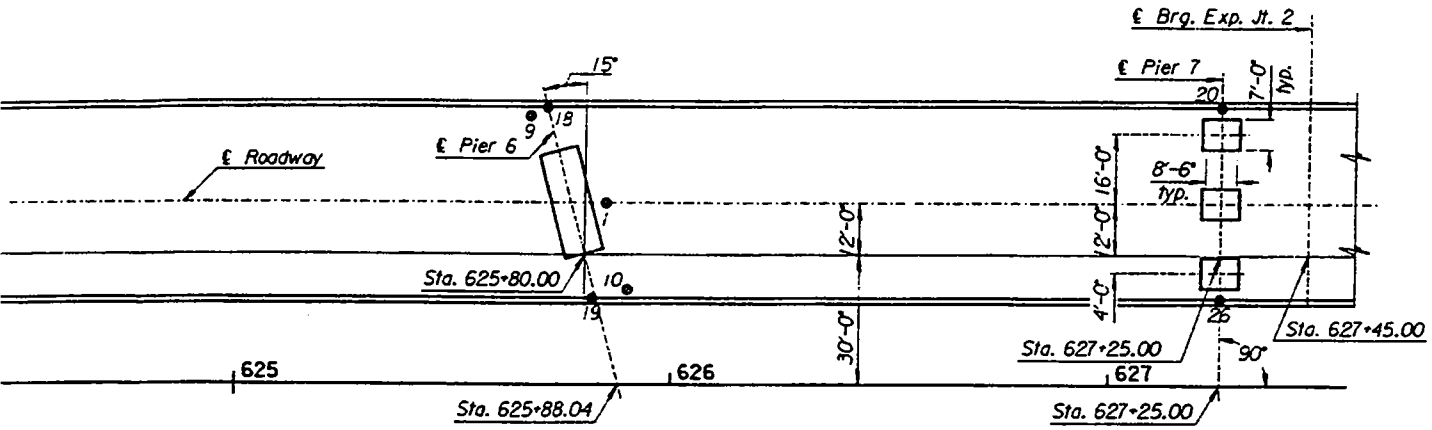
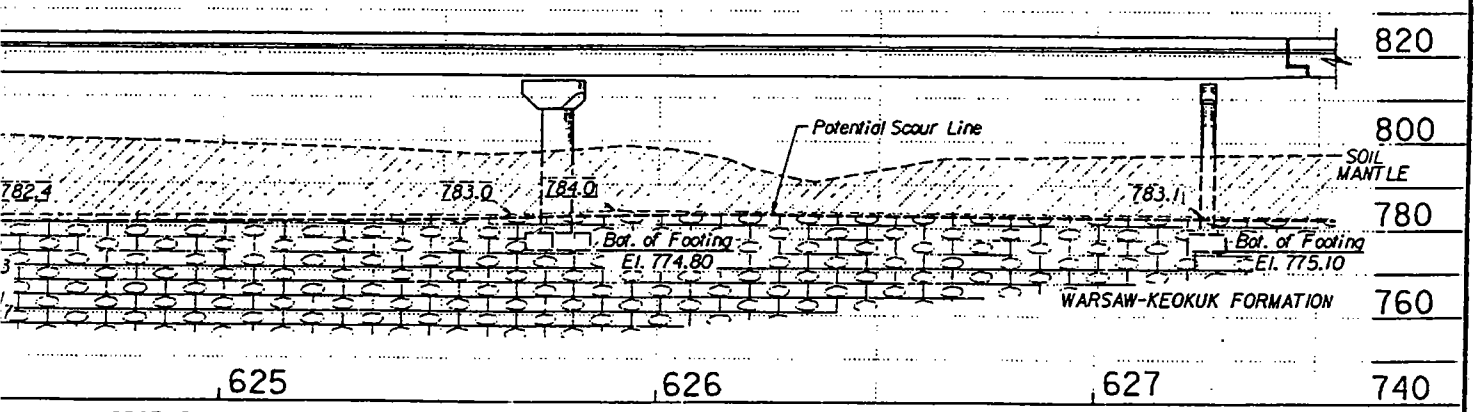
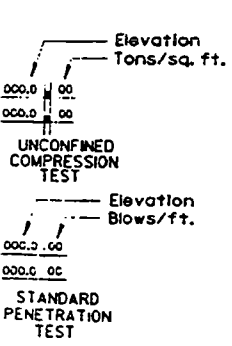


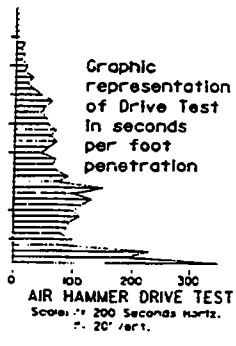
ting backfill shall be provided as shown through Pier No. 8 and Pier No. 10. shall receive all concrete backfill. of excavation shown are based on the ation Standard" sheet. Quantities based on the limits shown. r slope protection shall be Stone se, Type I. nal backfill required due to over-excavation ts shown will not be paid for directly, but ary to "Class B Concrete" and "Bedding ction".

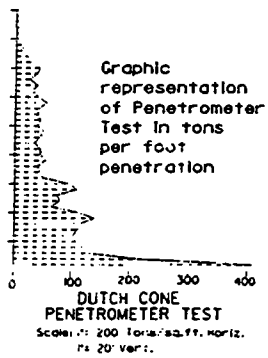
NOTE: Spread footings should be cut to exact dimensions and concrete placed to neat lines within bedrock.



PROFILE







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SCALE: I"= 20 Horiz. I"= 20' Vert.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

BR. NO. 400-E-40.96 (104) STA. 624+70.00

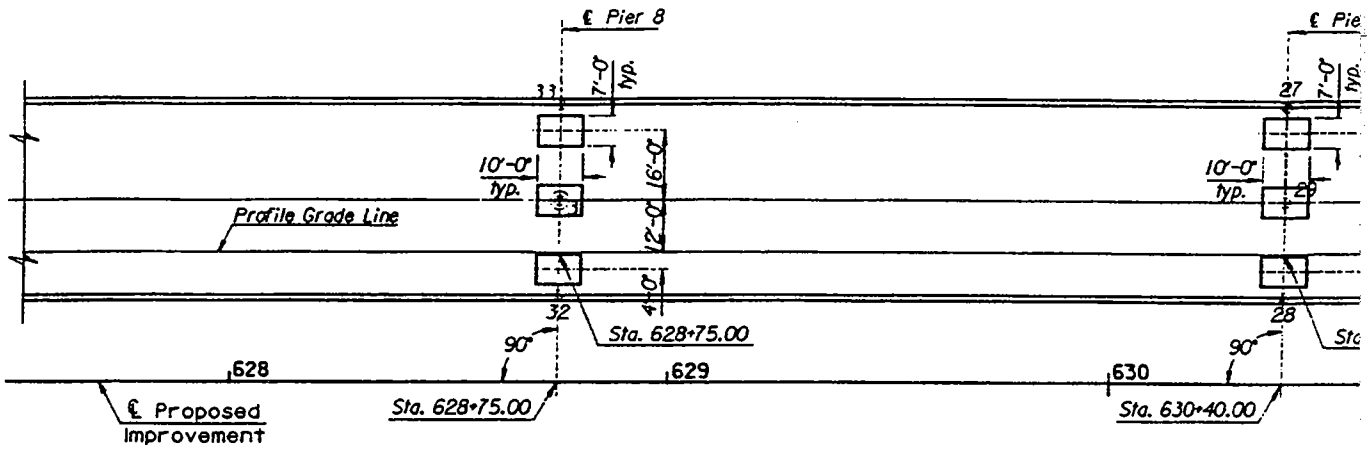
ENGINEERING GEOLOGY

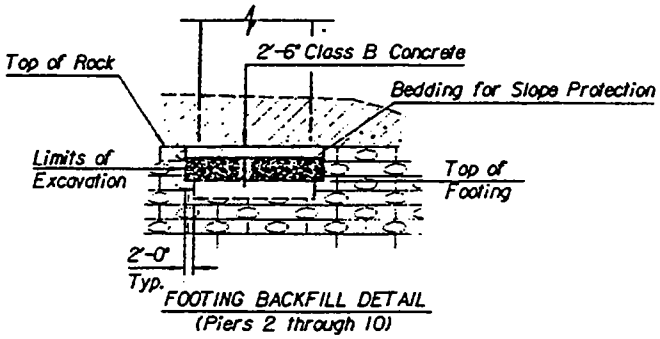
US-400 OVER THE SPRING RIVER

PROJ. NO. 69A-8K-5048-01 CHEROKEE CO.

SHEET NO.	OF	SCALE		APP'D	
DESIGNED		DETAILED	SAK	QUANTITIES	TRACED
DES. CK.		DETAIL CK.	SMB	QUAN. CK.	TRACE CK.

131





FOOTING BACKFILL DETAIL
(Piers 2 through 10)

NOTE: Spread footings should be cut to exact dimensions and concrete placed to neat lines within bedrock.

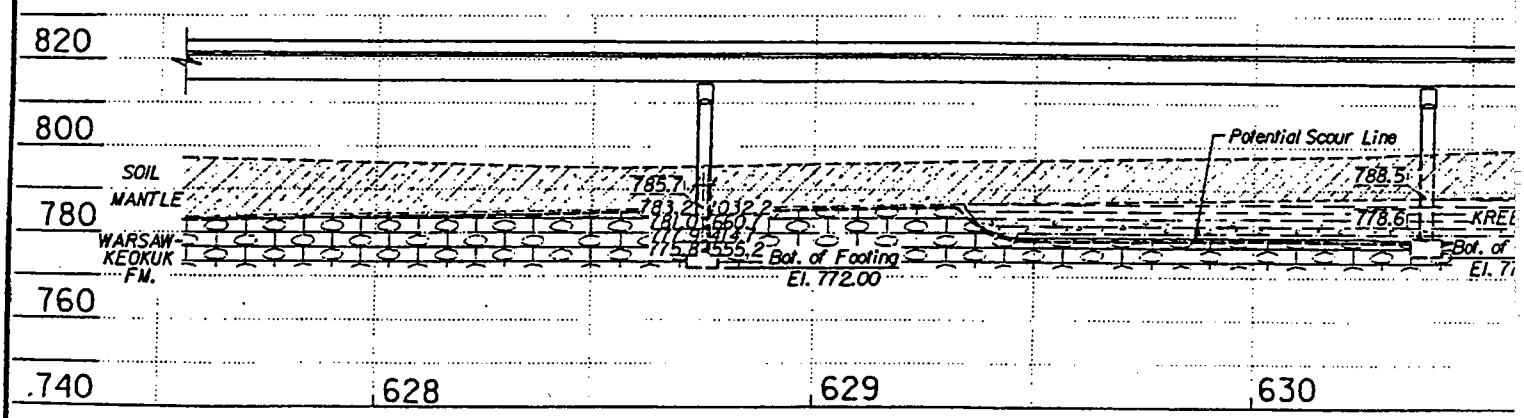
Note:

Special footing backfill shall be provided for Pier No. 2 through Pier No. 8 and Pier No. 9 shall receive all concrete backfill.

The limits of excavation shown are based on "Bridge Excavation Standard" sheet. Quantities were calculated based on the limits shown.

Bedding for slope protection shall be Special for Filter Course, Type I.

Any additional backfill required due to excavation outside the limits shown will not be paid for and shall be subsidiary to "Class B Concrete" and "Bedding for Slope Protection".



STANDARD GEOLOGIC SYMBOLS

Clay
Silty Clay
Silt
Sand
Gravel
Boulders
Caliche
Silty Clayey Shale
Limy Shale
Black or Fissile Shale
Sandy Shale
Gypsiferous Shale
Weathered Shale
Sandstone
Shaly Sandstone
Siltstone
Gypsum
Limestone
Cherty Limestone
Shaly Limestone
Sandy Limestone
Weathered or Broken Limestone
Pelletal Limestone

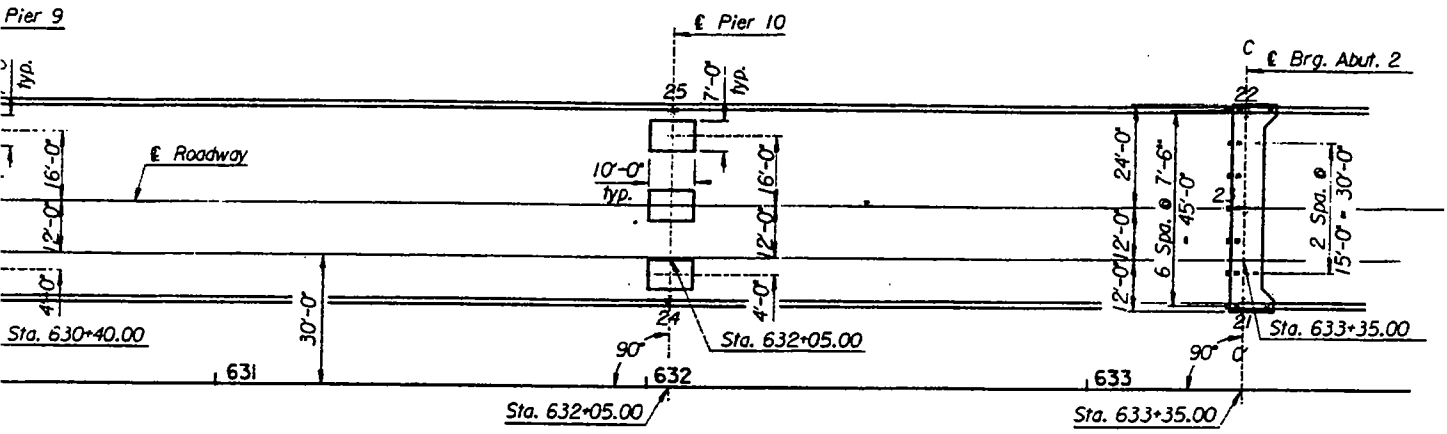
GEOLOGIC SOUNDINGS

Coredrill
Power auger
Hand tools
Air hammer
Dutch cone penetrometer
Unconfined compression
Standard penetration
Water level 3/95

000.0 Elevation interpolated or from adjacent soundings
000.0 Actual sounding elevation
000.0 Drive started
000.0 Refusal

CASING DRIVE TEST

FHWA PROJECT NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	69A-II K-5048-01	1996	132	483



Note:
For pile spacing at Abutment No. 2 see "Abutment No. 2 Details" sheet.

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Pier No. 10.
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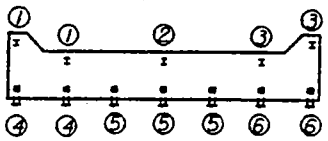
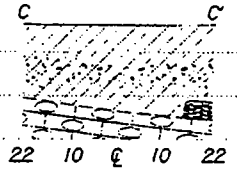
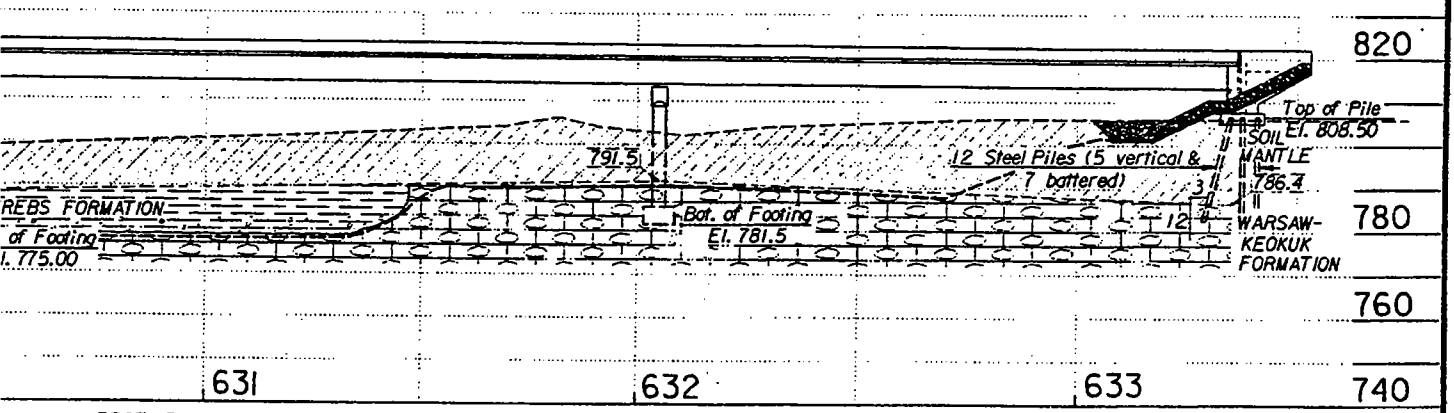


ABUTMENT NO. 2 PILE LENGTH

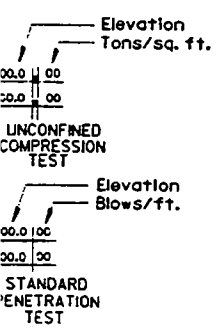
① - 21' ④ - 22'
② - 24' ⑤ - 25'
③ - 25' ⑥ - 26'

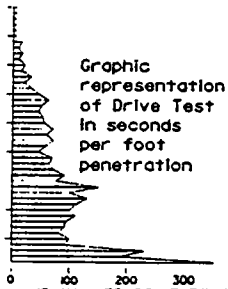
SECTION THROUGH ABUTMENT NO. 2

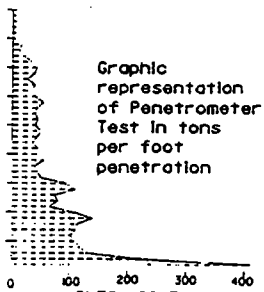




PROFILE







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SCALE: 1" = 20 Horiz. 1" = 20' Vert.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION
BR. NO. 400-II-40.96 (104) STA. 624+70.00

ENGINEERING GEOLOGY
US-400 OVER THE SPRING RIVER

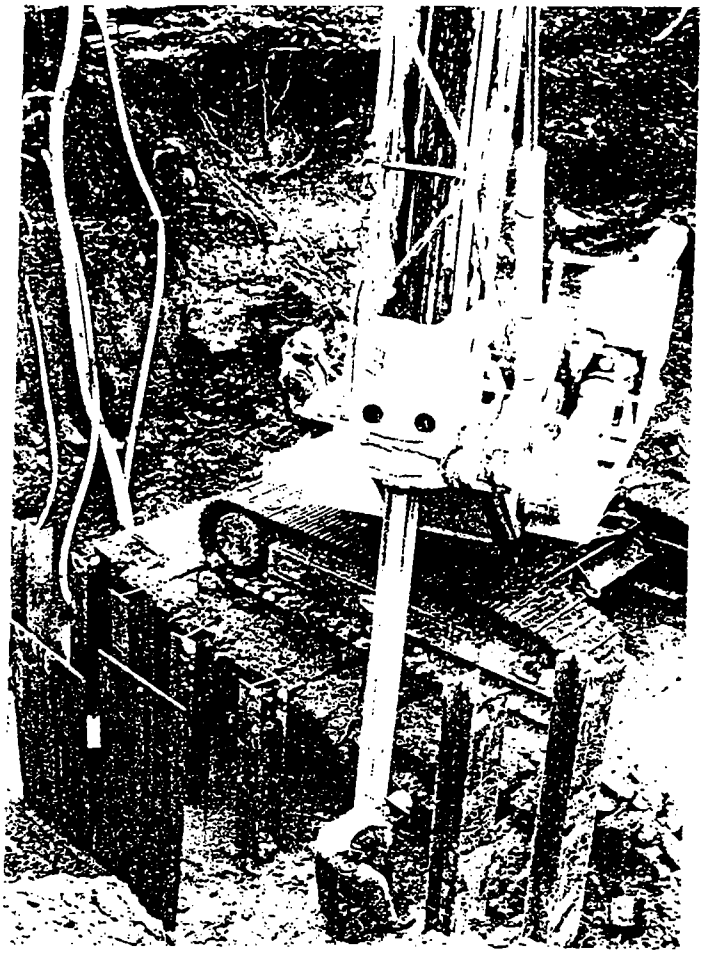
PROJ. NO. 69A-II K-5048-01 CHEROKEE CO.

SHEET NO.	0"	SCALE		APP'D	
DESIGNED		DETAILED	SAK	QUANTITIES	TRACED
DESIGN CK.		DETAIL CK.	SMW	QUAN. CK.	TRACE CK.

132

CLOSE-UP OF FOOTING WALL SHOWING INTERBEDDED NATURE OF THE BEDROCK

EXCAVATION OF PIER FOOTING USING HYDRAULIC HOE-RAM



PREDRILLING PILE AT PIER #1

LOOKING EAST FROM PIER #2

SIDE VIEW OF PIERS 2 THROUGH 6

CONCLUSIONS

As of this writing, the construction is on schedule to meet the calendar completion date of August 28, 1998. With the exception of the pier #1 pile over-run there have been very few change orders on this project.

Working in a Karst topography area, minor changes from the geotech or bridge designer can significantly alter recommendations. It is imperative that both groups work as a single unit.

Investigative measures represent the best available information. Changes will occur during construction and all involved should be prepared to address them.

Contractors need to be aware of the extreme difficulty in excavating the Warsaw-Keokuk Formation. Added time, cost and wear of equipment should incorporated into their bids.

We have began investigations on two additional bridges in this area. Techniques, problems, solutions and recommendations used on the Spring River have aided us greatly in these geotech studies.

REFERENCES

1) Zeller, Doris E., **The Stratigraphic Succession in Kansas,** Kansas Geological Survey, Bulletin 189.

2) Carlson, E. Dean, **Kansas Comprehensive Highway Program,** Kansas Department of Transportation, FY 1996

3) Croxton, Neil M., Ryan, Richard F., Henthorne, Robert W., **Bridge Foundation Geology Report, US-69 Alternate Over the Spring River, 69A-11-K-5048-01,** Kansas Department of Transportation, Bureau of Materials and Research, Geology Section.

4) Merriam, Daniel F., **The Geologic History of Kansas,** Kansas Geological Survey, Bulletin 162.

Roadway Construction Considerations Through a Young Volcanic Center at the South End of Big Sandy Valley, Near Wikieup, Arizona

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Senior Geologist
AGRA Earth & Environmental, Inc.

LORENA M. ROSBERG, G.I.T.
Staff Geologist
AGRA Earth & Environmental, Inc.

RONALD E. PRICE, P.E.
Senior Engineer
URS Greiner, Inc.

ABSTRACT

A thick sequence of volcanic rocks associated with a young volcanic center and basin fill sedimentary rocks has filled the south end of the Big Sandy Basin. The reconstruction of the section of U.S. Highway 93 which crosses through steep relief associated with the volcanic center, approximately 15 miles south of Wikieup, will be impacted by the presence of the various geologic materials. Rock cuts in this area will reach a maximum depth of about 80 feet. Thick sections of fill also will be required in this area of high relief. The sequence of volcanic units is Pliocene in age and includes basalt flows, flow-banded rhyolite domes with associated flow breccias, lahar and tuff deposits. The sedimentary units associated with the basin consist of interbedded tuffaceous conglomerates, conglomerates and sandstones.

Due to the young age of these rocks, the regional structural fabric is not expressed in the rocks, and flow breccias, lahar, tuff and basin fill deposits generally are unfractured. However, the rock mass of the flow breccia is fragmented with large remnant blocks from the flow, and the various volcanic flows are jointed in response to cooling stresses. Joint spacing in the flows and domes varies from very close to moderately close. The geologic units also vary in hardness from hard in the volcanic flows and domes to very soft in the basin fill materials.

These variable geologic conditions will impact both slope design along the alignment and use of excavated materials for embankment fills. Since discontinuities are uncommon in the bulk of the materials, new roadway slopes can be constructed at relatively steep slopes, typically from 1/2:1 to 3/4:1 (horizontal:vertical) if coupled with appropriate collection ditches to contain random rockfalls. The rockfalls are most common in the basin fill conglomerates as a result of hard particles eroding from a softer surrounding matrix. Unconfined compressive strengths of these rocks also are highly variable, ranging from about 100 psi in the softer sedmentary rocks up to 8,000 psi for the harder volcanic deposits. Materials generated from cuts generally will be usable as embankment fill for the new alignment with some special consideration of oversize materials from the more widely fractured sections of rhyolite domes, flow breccias, and basalt flows. Swell factors for the materials will be highly variable and range from about 0 to 5 percent in the sedimentary rocks and up to 15 to 30 percent in the volcanic rocks. Although considerable variations of the geotechnical properties exist in this sequence of young volcanic and sedimentary basin fill materials, they will provide good foundation conditions for both roadway and bridge construction.

INTRODUCTION

Improvements to the Kaiser Spring Wash Section of U.S. 93 will require construction through a complex assemblage of geologic materials associated with a young volcanic center. The Kaiser Spring Wash Section of U.S. 93 is located in northwest Arizona between Wickenburg and Wikieup. The project extends from Mileposts 138.0 to 132.9, which is about 15 miles south of Wikieup. The improvements will consist of a reconstruction of the existing highway from a 2-lane roadway to a 4-lane divided highway with two lanes in each direction and a median width of 84 feet. This section of U.S. 93 crosses rugged terrain with areas of steep relief associated with the young volcanic deposits. Thus, the reconstruction will require considerable excavation up to about 80 feet deep where the alignment crosses through the elevated ridges and thick sections of fill up to 95 feet high where it crosses primary drainages. Two bridges, 1500 to 1600 feet in length, also are planned in order to span the canyons in the Kaiser Spring Wash drainage area. The location of the project is shown in Figure 1. The figure includes a photograph of the existing Kaiser Spring Bridge spanning the main canyon.

Based on general rock types, the project alignment is divided into three geologic domains each with a distinct geomorphology. The most rugged and steepest relief area is the central domain where the dominant rock types are silicic volcanic domes derived from local vents. The Southern Domain is dominated by basalt flows and the topography typically is characterized by a flatter topped but tilted mesa. The northern domain is dominated by sedimentary deposits and is characterized by a series of more rounded and elongated ridges but still with an occasional steep sided drainage.

The sequence of rocks is very young and, thus, has not been subjected to extensive structural adjustment and is only locally fractured. Discontinuities typically are limited to cooling joints within the domes and flows and very isolated fractures and faults within other geologic units. Thus, slope stability of cuts through most of the geologic units will be governed by general rock strengths and potential erodability rather than the orientation of discontinuities. Similarly, excavatability of the harder volcanic rock units will be affected by the lack of fractures and blasting will be required in many areas.

Photo 1. Rugged terrain in Kaiser Spring Canyon. Sedimentary rocks of the Northern Domain in foreground and volcanic domes of the Central Domain in background.

This section of U.S. 93 is part of a designated scenic area and has many environmental aspects that needed to be considered during project development. These considerations included archaeological sites, the Desert Tortoise, the scenic designation, and natural riverine systems that could not be disturbed. In response to a combination of the rugged terrain and the intent to minimize disturbance to the site, the geotechnical investigation was completed with portable rigs with the assistance of a helicopter to place the rigs at the drillholes. Photo 1 shows the nature of the rugged terrain in the area of Kaiser Spring Canyon and Photo 2 shows the drill rig in transport between borings.

Additional construction considerations include the following:

- Usability and potential swell of the variable geologic materials locally available from road cuts for fill sections.
- Slope designs including shaping or sculpturing to minimize impacts to visual quality along the scenic road.
- Construction sequencing to optimize use of available materials while posing minimal impact to roadway users, since this roadway is a main artery between Phoenix and Las Vegas. The sequencing issues includes temporary road closures during blasting, haulage of materials from cut to fill

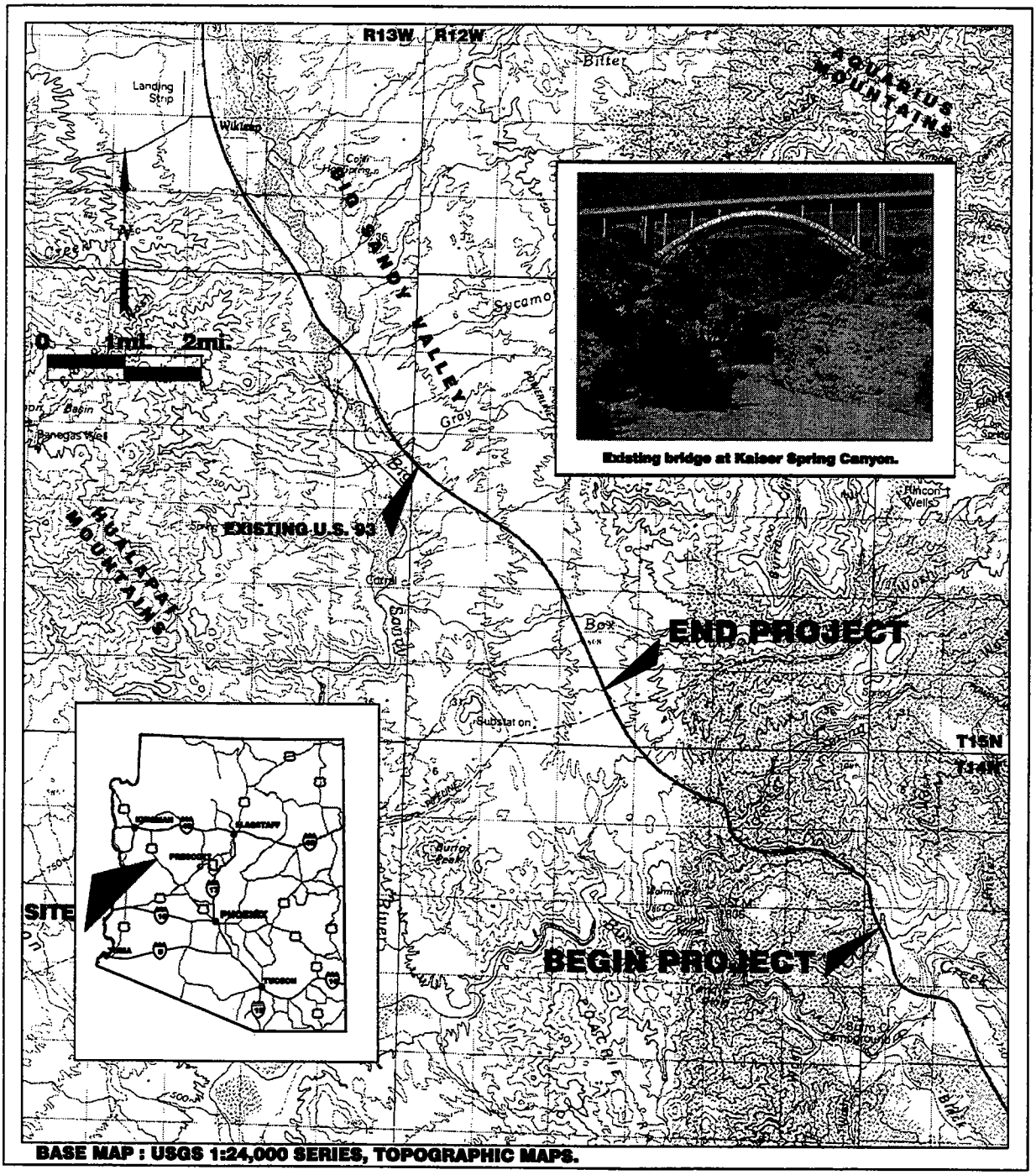

Figure 1. Location map.

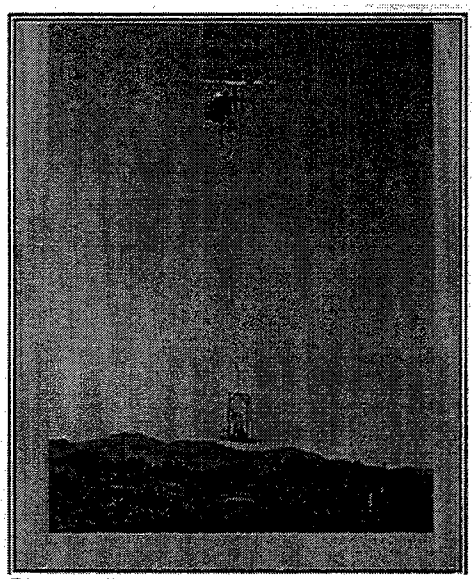

Photo 2. Drill rig in transport to boring in Northern Domain.

areas and general strategies to maintain traffic flow.

GEOLOGIC SETTING

The Kaiser Spring Wash Section of U.S. 93 lies along the east edge and at the southern end of a structural basin within the Basin and Range Physiographic Province. The geomorphology typical of this province consists of a series of parallel mountain ranges separated by intervening valleys or basins. The Hualapai Mountains to the west and the Aquarius Mountains to the east represent uplifted structural blocks. The Big Sandy Valley, along which U.S. 93 follows, represents the intervening structural subsidence feature.

The major structural framework in the region relates to the Basin and Range physiography with a dominance of approximate northwest to southeast trending faults. A second major set of north to south trending faults intersects the primary northwest to southeast faults at the southern end of the basin. Eruption of volcanic materials occurred at the intersection of the two sets of structural features and resulted in the formation of a volcanic center (Moyer, 1982). Continued structural adjustments along these features resulted in localized compression and folding, and development of anticlines.

The geologic units associated with this geomorphology and exposed in the general area include Precambrian rocks in the mountain ranges and Tertiary to Quaternary basin fill deposits in the valley. The Precambrian rocks predominantly consist of granite, gneiss and schists within the Hualapai and Aquarius Mountains; and the Tertiary basin fill consists of a sequence of volcanic and sedimentary rocks. Tertiary to Quaternary alluvial and colluvial deposits locally overlie the Precambrian and Tertiary basin fill sequence along the flanks of the mountain ranges and within the Big Sandy Valley. The basin fill rock sequence deposited in the southern end of the Big Sandy structural trough, and just west of the Aquarius mountain block, includes a dominance of volcanic rocks derived primarily from a local cluster of vents associated with local Basin and Range structural features (Moyer, 1982). The geologic units exposed to the north of this volcanic center within the basin include a sedimentary sequence that progressively becomes finer grained toward the north. Near the volcanic center, the sedimentary rocks are composed of fluvial conglomerates. The sedimentary rocks become finer grained to the north, grading to sandstones, and ultimately grading into a lake bed sequence of siltstones and claystones near Wikieup. The lake bed sequence indicates that the Big Sandy Valley was at one time internally drained.

The Tertiary volcanic sequence at the southern end of the basin consists of a voluminous deposit primarily of rhyolite and rhyodacite domes and volcanic breccias with associated pyroclastic deposits of tuff and fewer basalt flows. This sequence of volcanic rocks completely filled the basin in this area and subsequent erosion of this volcanic pile resulted in steep mountainous terrain similar to the adjacent mountain blocks. To the north, where the sedimentary rocks dominate in the basin, the geomorphology of the basin is still evident, with the mountain blocks elevated from the valley floor.

SITE GEOLOGY

The Kaiser Spring Wash Section of U.S. 93 consists of three distinct geomorphological terranes. From south to north, the highway alignment intersects a tilted interbedded sequence of Tertiary sediments and tuff overlying a basalt flow; two bulbous volcanic domes composed of rhyolite and rhyodacite flows associated volcanic breccias and tuff deposits; and

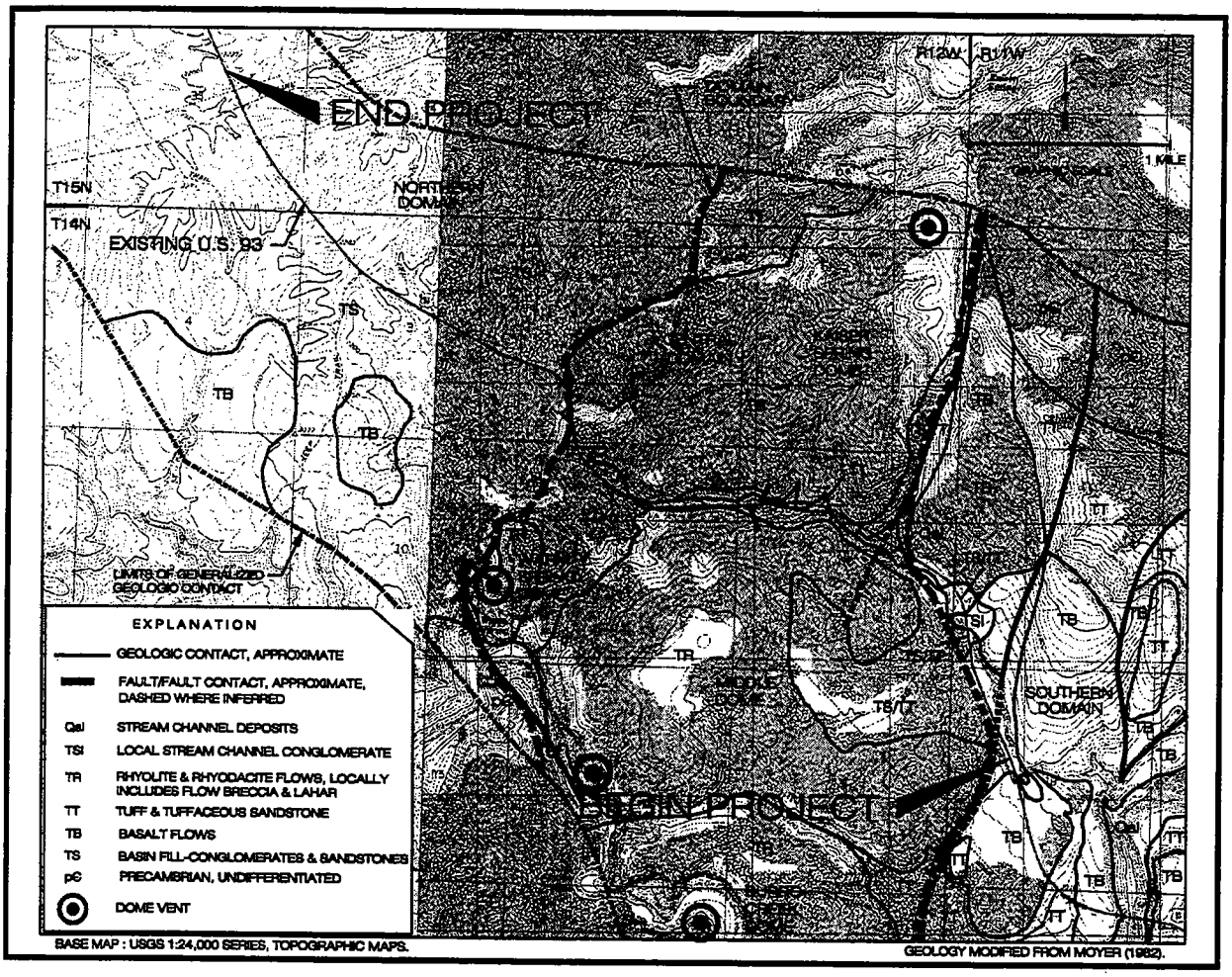

Figure 2. Generalized geology of the project area including the three geomorphological terranes of the Southern, Central and Northern Domains.

rolling hills predominantly of conglomerates with interbedded sandstones and occasional tuffaceous conglomerates. These terranes have been generalized into three geological domains and designated Southern, Central and Northern Domains. The areal distribution of the geologic units and geological domains are presented in Figure 2.

The geologic units encountered along the alignment in the Southern Domain consist of a locally tilted sedimentary sequence, from youngest to oldest, of stream channel conglomerate and interbedded sandstone, tuff and conglomerate overlying a basalt flow. The surface of the tilted sequence is relatively flat and dissected by relatively shallow to moderate drainages. On the north side of the highway alignment, the sequence gently dips to the south at an angle of 10 to 12 degrees. The dip flattens to near horizontal towards the south of the alignment. The tilted sequence and basalt flow form the limb of a structural fold or anticline that dips to the south. The axis of the anticline trends generally northwest-

southeast, paralleling the regional fabric, and is located north of the project site and outside the mapped area of Figure 2. The anticline is terminated on the east and west by north-south trending faults. The eastern fault is located outside the southern limit of the project area. The western fault crosses the project alignment and defines the boundary of the Southern and Central Domains. The location of the western fault is based on aerial photograph interpretations and projections from exposures to the north. It represents a significant structural feature within the volcanic center, although it is not exposed along the alignment.

The stream channel conglomerate is a localized deposit rich in basalt clasts that caps only the southernmost ridge of the Southern Domain. This stream channel conglomerate represents the youngest bedrock material within the project area and is derived from erosion of the directly adjacent basalt materials likely following the formation of the anticline.

The Central Domain extends primarily from the north-south trending fault that defines the western edge of the basalt to Kaiser Spring Canyon. The bulbous ground surface of the domes, which are the predominant geomorphological expression of this domain, is more irregular and contains steeper relief where dissected by drainages, than the basalt flow of the Southern Domain. The two domes are composed of slightly differing lithology and represent two relatively large and thick silicic volcanic features derived from different vents. As shown on Figure 2, the vent of the rhyolite dome is located north of the roadway and the vent of the rhyodacite dome is located south of the alignment. Thus, the rhyodacite dome is exposed on the west side of U.S. 93 and the rhyolite dome is located on the east side of the roadway. The rhyolite dome is denominated by Moyer (1982) as the Kaiser Spring Dome, and the rhyodacite dome as the Middle Dome. Other rhyolitic vents and domes that are part of the Kaiser Spring volcanic center as mapped by the writers and Moyer can also be observed in Figure 2, however, these are not intersected by the highway alignment.

The rhyodacite flow with its associated volcanic breccias on the west side of U.S. 93 is the oldest of the two domes since the rhyodacite flow dips beneath the rhyolite dome exposed in the higher cliffs to the east of the roadway, although some sections may have been deposited contemporaneously. Tilted or dipping tuff beds locally overlie the Middle Dome and underlie the Kaiser Spring Dome on their eastern edge.

The rhyolitic domes are locally faulted. A fault zone is exposed along an existing roadway cut within the rhyodacite. The fault zone is characterized by a 35- to 40-ft wide zone containing localized fault gouge and brecciated rock. Considerable vugs, black manganese staining and low density gouge material (silty sand) were encountered in the core from borings completed in this area. The fault strikes N30W and is steeply dipping.

Brecciation of the highly viscous flows during continued transport of a combination of partially molten and partially solid material, but prior to complete solidification of the lava, resulted in the formation of localized sections of volcanic breccias within the flows. The volcanic breccia locally includes interfingered rhyodacite flow breccias and lahars. However, the lahar appears to have been deposited locally on the edge of the flow breccia unit upon an influx of water. The flows and volcanic breccias are not differentiated on Figure 2 since their geotechnical properties are similar, and because the individual brecciated and lahar sections are not readily discernable at a large scale.

Localized sections of sedimentary rocks, predominantly composed of interbedded conglomerate and sandstone, also occur within the Central Domain. The exposures occur in the hills south of Kaiser Spring Canyon, since the youngest portion of the sedimentary sequence was deposited on the volcanic rocks and some erosional remnants remain as a capping unit. In this area of the project, the sedimentary deposits occur as isolated pockets, wedges or capping ridges.

The geologic units exposed in the Northern Domain consist predominantly of sedimentary rocks dominated by conglomerate with some interbedded sandstone and localized tuffaceous conglomerate. The exposures typically form more gentle slopes and rolling topography than the volcanic rocks to the south. The boundary between the Central and Northern Domains is somewhat more irregular than the boundary between the Southern and Central Domains since the sedimentary rocks overlie the volcanic rocks along a highly irregular erosional surface. The boundary selected herein is set at the

northernmost exposure of the volcanic rocks in the area of Kaiser Spring Canyon.

The sedimentary rock sequence is the youngest bedrock material exposed at the site, with exception of the local stream channel conglomerate at the south end of the project. However, portions of the sedimentary rock sequence likely were deposited contemporaneously with the volcanic rocks. The sedimentary rocks are the predominant rock type exposed north of Kaiser Spring Canyon. Due to their young age, the sedimentary rocks are generally unfractured, or only locally fractured with rare faults. The faults typically are high angle and intersect the proposed alignments at nearly perpendicular angles.

GEOLOGIC UNITS

Basalt Flow - The exposures of basalt and core retrieved from borings in the Southern Domain have an aphanitic texture with considerable fine to medium grained phenocrysts. The exposures and core vary from slightly weathered to locally highly weathered. The basalt commonly is moderately to highly vesicular in layers, dark gray to black and typically is hard to very hard. As a result of local cooling joints within the hot flow, the basalt predominantly is closely fractured (0.2 to 1.0 foot) to very closely fractured (less than 0.2 foot) with some moderately close zones (1.0 to 3.0 feet). The orientation of the fractures are highly variable and curved varying in dip from 30 degrees to vertical. Fractures within the shallow sections of the flow typically are filled with a fine-grained silty sand with gravel-size basalt fragments indicating that fissuring or opening of fractures occurred in the past. Deeper fractured surfaces are medium rough to rough and occasionally lined with calcium carbonate. Calcite amygdules and partially calcite-lined vugs locally occur in the basalt. Photo 3 shows the typical texture and fractures of the basalt.

Refraction seismic compression wave velocities measured within the basalt ranged from 9,600 fps to 12,900 fps at near surface depths and 16,600 fps below a depth of about 65 ft below ground surface.

Photo 3. Typical textures and fractures of the basalt

Tuff -Tuff is exposed in the Southern and Central Domains of the project area. In the Southern Domain, the tuff is interbedded with a sequence of tilted sedimentary rocks overlying a basalt flow. The orientation of the bedding of the tuff within the tilted sequence ranges from N83E to N55W and dips from 11 to 12 degrees to the south and southwest, becoming flatter to the south. The tuff exposed in the Central Domain occurs between the rhyodacite and rhyolite domes. The orientation of the tuff deposit in the Central Domain ranges from N60W to N80E and dips from approximately 30 to 35 degrees towards the northeast and northwest. The dip of the tuff appears to correspond to its angle of deposition on the pre-existing variable ground surface of the rhyodacite dome, although local tilting could have occurred.

In general, the tuff consists of white to light gray lapilli tuff composed of coarse grained sand and fine grained gravel in a silty sand matrix. Typical texture of the tuff in a cut in the Central Domain is shown on Photo 4. The tuff grades to a tuffaceous sandstone and is medium to thickly bedded and stratified. The tuff is very soft to soft, generally unfractured and slightly weathered to unweathered.

Refraction seismic compression wave velocities measured in tuffaceous rock types ranged from about 5,300 to 6,600 mps.

Photo 4. Typical texture of the tuff in a cut of the Central Domain.

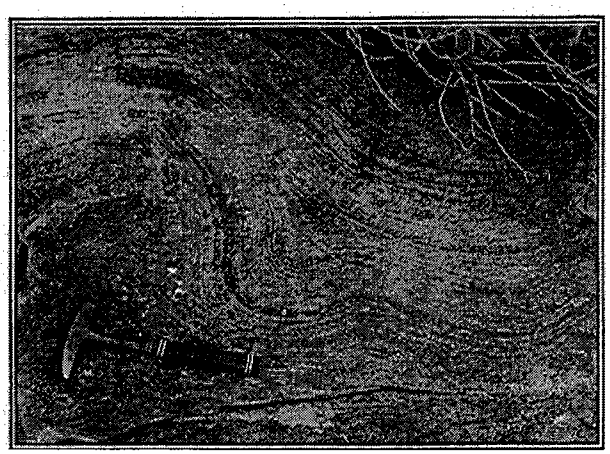

Photo 6. Flow banding features within the rhyodacite flow.

Rhyodacite/Rhyolite Dome Flows & Volcanic Breccias - The texture of the rhyolite/rhyodacite flows exposed in the Central Domain is aphanitic with considerable fine-grained phenocrysts of feldspar and biotite in a fine-grained crystalline to glassy groundmass. Exposures vary from unweathered to slightly weathered, and the core is mostly slightly weathered with some moderately weathered zones. The exposures are hard to moderately hard with some moderately soft to soft zones, and light gray to gray. The rhyolite/rhyodacite flows are closely to moderately closely fractured (see Photo 5). The fractures represent local cooling joints within the hot dome. The predominant fracture set appears to parallel the flow banding. Other fracture sets present in the exposures appear to terminate primarily on the flow banding set and/or the fractures paralleling the flow banding. However, flow banding features within the dome are highly irregular and folded and do not have horizontal continuity, as shown in Photo 6.

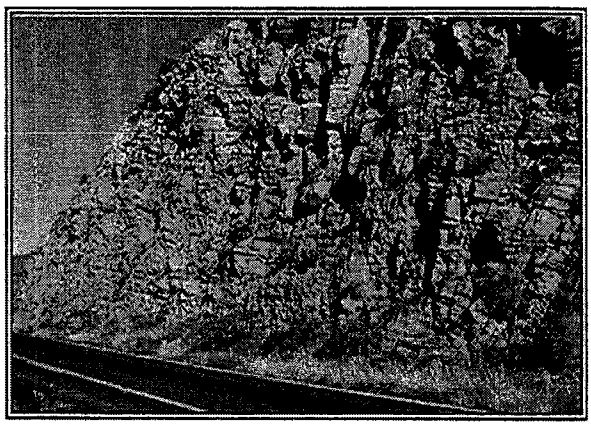

Photo 5. Fractured rock mass in existing cut of the rhyodacite flow exposed in the Central Domain.

The edges of the rhyodacite dome structure within the project area are commonly interfingered with a volcanic breccia consisting of flow breccia and less frequently lahar. The contact between the rhyodacite and the volcanic breccia is highly irregular, transitional and generally steeply dipping. The flow breccia consists of angular to subangular gravel through boulder-size remnant blocks of the flow as much as 15 to 20 ft in diameter in a fine-grained rhyolitic groundmass. Typical texture of a fine-grained section of the flow breccia is shown in Photo 7. The blocks within the breccia are hard to very hard and predominantly unweathered. The blocks are gray to dark gray, commonly flow banded and contain some fractures remnant of the flow. The groundmass is moderately soft to moderately hard, and slightly to moderately weathered. The groundmass generally is unfractured and pinkish brown to brown in color.

Photo 7. Typical texture of flow breccia in Central Domain with large remnant blocks up to 3 to 4 feet across.

The lahar consists of gravel to boulder-sized clasts in a silty sand and locally glassy matrix. The lahar matrix is weakly to moderately welded, hard with moderately soft zones, unfractured and light gray to occasionally brownish gray. The lahar is weakly stratified. The lahar features are commonly wedge shaped deposits and appear to overlie the flow or flow breccias. Photo 8 shows the texture of the lahar within Kaiser Spring Canyon.

Photo 8. Texture of lahar within Kaiser Spring Canyon.

Compression wave velocities measured in rhyolite flow or volcanic breccia rock ranged from 5,900 to 15,600 fps with the lower velocities occurring in areas where flow breccias are predominant.

Conglomerate - The conglomerate exposed extensively in the Northern Domain and locally in the other two domains is composed of gravel to cobble-sized clasts in a predominantly fine to medium grained sandy matrix with localized boulders. Locally, the conglomerate is composed of gap graded to well graded gravels in a well graded sand and silt matrix. The fine-grained matrix of the conglomerate is slightly to moderately calcareous and varies from very soft to rarely moderately hard (see Photo 9). In general, the conglomerate is the hardest adjacent to the bridge site and is the softest on the north end of the site where core recovery was more difficult.

The clasts in the conglomerate are subrounded to subangular and mostly are hard, except for the pumice fragments in the tuffaceous conglomerate beds which are soft. Where present, the boulder-sized clasts typically are 1.0 to 1.5 ft in maximum dimension, with rare boulders up to 4.0 ft. The conglomerate is slightly weathered to unweathered and light brown.

Sandstone to Silty Sandstone- The sandstone is composed of predominantly fine to medium grained sand with some silt and occasional to some fine grained gravel. The sand and gravel is well graded in lenses. Locally, the sandstone is slightly to moderately calcareous and grades into a silty sandstone. The sandstone is medium to thickly bedded, very soft to moderately soft and brown to light brown. Interbedded layers of sandstone also are shown on Photo 9.

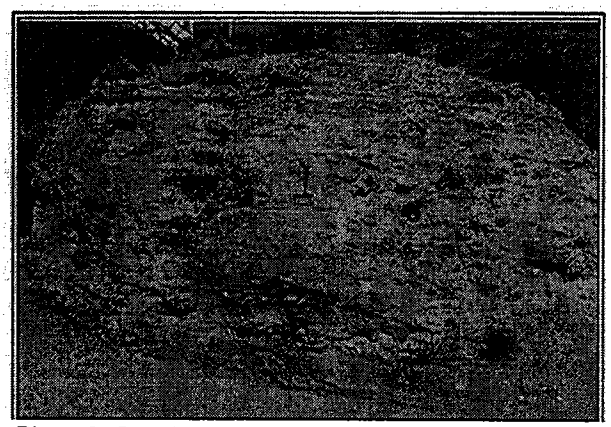

Photo 9. Conglomerate interbedded with sandstone lenses.

Tuffaceous Conglomerate - The tuffaceous conglomerate is predominantly composed of subrounded to rounded, well graded gravel-sized pumice clasts in a silty sand matrix. The tuffaceous conglomerate is soft to moderately soft and brown to light brown. In general the tuffaceous conglomerate beds vary in thickness from approximately 1/4-in to 3.5 ft and were encountered locally immediately north of the Kaiser Spring Canyon.

Refraction seismic compression wave velocities measured in the sedimentary rocks varied from 3,600 to 9,900 fps at very shallow depths.

Local Stream Channel Conglomerate - The localized stream channel of basalt clast-rich conglomerate is exposed as a capping unit on the southernmost ridge crossing the project alignment in the Southern Domain (see Photo 10). This conglomerate is generally softer and younger than other conglomerates encountered at the site. The contact of the stream channel conglomerate with the underlying sandstone dips steadily to the south but is irregular since smaller-scale channels have been eroded into the sandstone. These channels subsequently filled with gravels. The conglomerate

is composed of well graded, subangular to subrounded gravels and cobbles in a very soft to soft, locally calcareous silt and predominantly fine grained sand matrix. The conglomerate is brown with dark gray clasts and locally contains finer-grained lenses that grade into a sandstone.

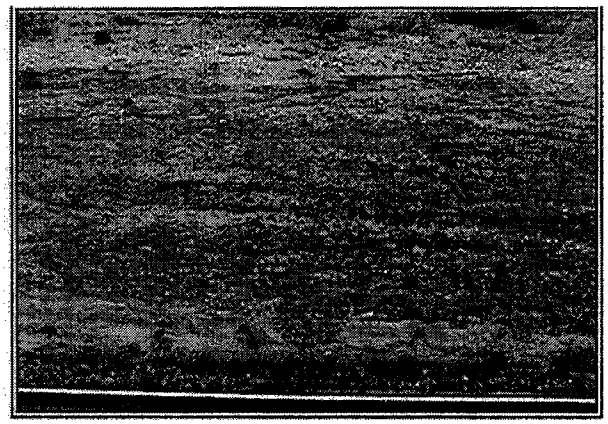

Photo 10. Local stream channel conglomerate is the youngest bedrock material of the alignment.

Refraction seismic compression wave velocities measured over the stream channel conglomerate varied from 4,300 to 7,300 fps at very shallow depths.

SLOPE DESIGNS

Construction of the proposed realignment of U.S. 93 Kaiser Spring Wash Section will require considerable excavation within bedrock materials along sections of the entire alignment. The cuts will vary from about 10 feet to just over 80 feet in depth. The objective of the slope designs is to provide a safe slope for roadway users while not impacting visual quality and optimizing cut slope angles to meet project cut and fill balances with planned construction sequencing.

The parameters for establishing cut slope design angles are general rock strengths, fracture frequency, condition and orientations and historic slope performance of existing slopes in similar geologic materials. Due to the young age of the volcanic center, most of the rock units are unfractured or only locally fractured or faulted. Thus, the stability of slope will depend upon general rock strength, hardness and potential for erodability rather than on potential failures along discontinuities. Rock strengths based on unconfined compressive strength tests vary from 100 psi in the softer sedimentary rocks to 8,000 psi in the harder volcanics.

The two rock units that have an enclosed fracture system include the rhyolite dome flows and the basalt flows. The basalt will be exposed in cuts along a very short section in the Southern Domain. The basalt flows typically have a predominant vertical and near horizontal joint orientation with some other joints that are highly curved and warped. Thus, a detailed slope stability analysis of jointing of the basalt was not performed since the horizontal and vertical sets will not contribute to significant slope failures and the other curved set would be difficult to predict. In this case the slope angle will be based on existing slope performance.

The rhyolite dome flows similarly have a joint system that is related to cooling of the rock mass. The dominant joint sets similarly are near vertical and near horizontal along flow banding. The Central Domain also includes numerous cut areas that will encounter a fractured rhyodacite flow that frequently grades into a flow breccia. In this area, general discontinuities could influence slope performance. Thus, an analysis of discontinuity data was performed for this rock type.

The rhyodacite are strongly flow banded as a result of the origin of the volcanic structure. The flow banding results from high viscosity lavas that form these domes when flowing out of a vent and radially spread from the source. The flow banding is an alignment of minerals that typically develops parallel to the surface of the flow but is commonly folded and contorted due to the high viscosity nature of the lava, as shown previously on Photo 6. Fracturing commonly develops along the flow banding as the flow cools. However, due to the folded flow banding, the fractures vary considerable over relatively short distances.

In addition to the flow band fracture set, other discontinuities form as a result of cooling within the flow. These sets typically cross the flow banding and develop normal to the surface upon which the flow was deposited. These fractures are not very continuous, typically are slightly curved, and locally terminate on or are offset by the flow banded set. All fracture surfaces are moderately rough to rough.

Orientations and dips of both sets of fractures are irregular and dependent on the local flow banded fabric. A fault zone containing a fault breccia also is exposed in the central portion of the main cut slope and was encountered in a nearby boring. The fault crosses the alignment at nearly perpendicular angles and thus will not negatively impact slope performance.

Discontinuity data was collected in the vicinity of the large cut in the central section of the Central Domain to evaluate the potential for failures in the slope along discontinuities. The orientations of 70 discontinuity planes, which included 20 flow banded sets and 50 non-flow banded sets, were measured and plotted on a equal area Schmidt stereoed using RockWare Utilities v.4 software. The distribution of the poles of the discontinuity planes for the flow band set is presented on Figure 3. The plot shows a wide scatter of data and no well defined clusters which normally would represent specific planes that would be anticipated to occur throughout the rock mass. Conversely, the data show that the orientations vary considerably within the rock mass. This likely is a product of development of curved cooling joints similar to those in the basalt flows, and distortion of fractures during brecciation and contemporaneous cooling and continued transport of the mass. However, the plot of poles does show a general trend of numerous near vertical joints.

Considering rock mass conditions including strengths and general lack of fracturing of the geologic units along the alignment, slopes will be constructed at angles varying from 1/2:1 (horizontal:vertical) to 1:1. Most of the unfractured rock and the basalt will be constructed at a 1/2:1 slope. The fractured dome flows will be constructed at a 3/4:1 slope, since undulations in the flow band joint set locally dips toward the slope faces and could contribute to failures in steeper slopes. The 1:1 slopes will be constructed only in a few locations where the softer conglomerate and sandstone exist at the northern end of the project. An inventory of existing slopes along the current roadway was completed during the field investigation for this project. The current slopes vary from 1:1 to near vertical (see Photo 11). Performance of the existing slopes generally has been good with the main problems associated with random rockfalls onto the roadway, since shoulders are very narrow. The new slopes angles generally will be consistent but less

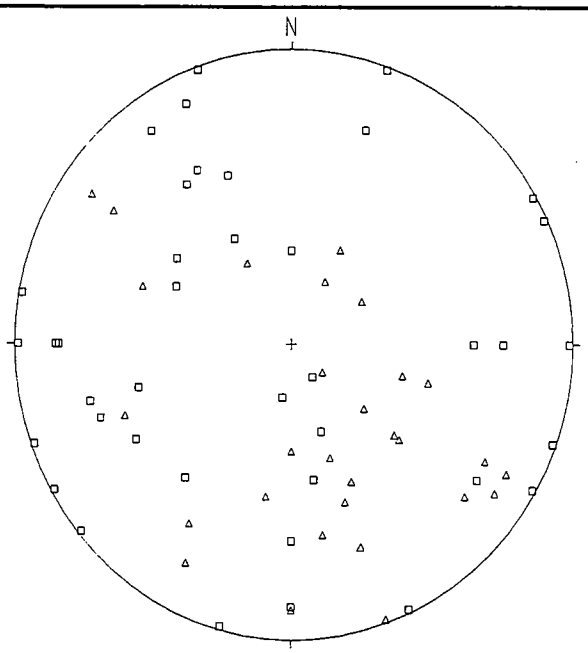

Figure 3. Plot of ples of discontinuity planes for rhyodacite flow.

steep than with the existing slopes. The new slopes also will be designed with an approximate 20 foot toe ditch to contain rockfalls, since erosion of softer matrix surrounding the harder clast will continue to occur in the new slopes.

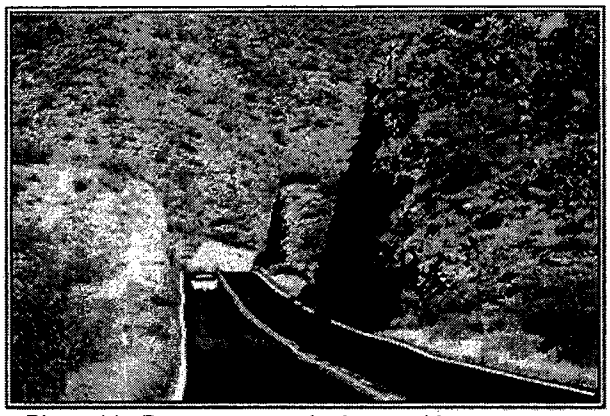

Photo 11. Deep, near vertical cuts with narrow shoulders along the existing roadway.

Visual quality impacts also will be considered in slope designs. The current very steep slopes have been referred to as "meat cleever cuts" by the URS Greiner, Inc. team during project development due the vertical nature of the slopes. Slope shaping including rounding and localized sculpturing will be performed on the final cuts in order to more appropriately have them blend in with the surrounding landscapes. Additional concerns of slope construction include method of excavation. Due to the unfractured nature and hardness variations and as indicated by seismic refraction velocities, considerable areas will likely need to be blasted to facilitate efficient excavation of the slopes. Blasting plans and associated traffic control plan will be required to minimize impacts to road users including limiting road closures to less than 15 minutes (see Photo 12).

CONSTRUCTION MATERIALS

Construction of the Kaiser Spring Wash Section of U.S. 93 will require 1,850,000 cubic yards (1.4 million cubic meters) of roadway excavation. Since the project also will require considerable cut slope excavations, it is desirable to use the material generated from adjacent cuts in construction of the fills. The construction sequencing will be important to promote efficient construction and minimize impacts to road users.

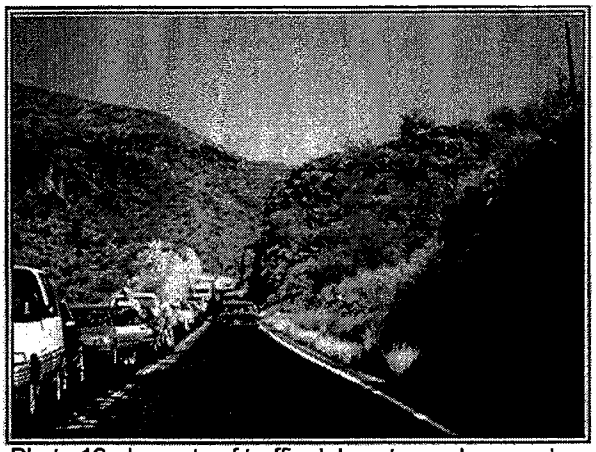

Photo 12. Impacts of traffic delays to road users along the existing roadway.

It appears that nearly all of the materials that will be excavated from roadway cuts from within the project limits will be suitable for use as embankment fill. The majority of potential embankment materials consist of conglomerate and sandstone and various volcanic rocks including rhyodacite flows, flow breccias and localized tuff. Localized sections of basalt also will be encountered in the Southern Domain. The materials should provide excellent subgrade support, provided the materials are broken down sufficiently. It is anticipated that nearly all the conglomerate and sandstone is soft and will break down to a non-plastic to low plasticity coarse grained granular material. Some isolated sections of harder conglomerate in the northern end of the Central Domain may be more difficult to break down due to cementation in lenses. The closely fractured rhyodacite flow and flow breccia also should break down to a non-plastic silty sand, gravel and cobble size material. However, some zones of more widely spaced fracturing of the flow and some large isolated boulder size blocks within the flow breccia will produce some larger fragments which may require additional treatments. The section of basalt similarly may produce some oversize materials due to localized wider spaced fractures.

Estimated earthwork factors for the various geologic materials to be encountered on site are presented in Table 1. Since the source of materials is consolidated deposits, there will be a net swell during placement in fills. The apparent large swell factor associated with the basalt is anticipated only in a small portion of the project. Actual earthwork factors in the harder volcanic materials will be influenced by general low densities in some units,

Table 1
Estimated Earthwork Factors

Material(s)	Geologic Domain	Representative In-situ Rock Mass Density kg./cu.m	Estimated Embankment Density @ 97% of Standard Proctor Density	Estimated Earthwork Factor
Sandstone/ Conglomerate	Southern	1,750	1,700	5% Swell
Basalt	Southern	2,400	1,840	30% Swell
Volcanic Breccia & Rhyodacite	Central	1,900	1,670	15% Swell
Conglomerate, local Sandstone	Central	1,920	1,670	15% Swell
Sandstone/ Conglomerate	Northern	1,650	1,650	Even

the methods of excavation, the particle sizes to which the materials are broken down, and the degree of blending which occurs with other finer grained fill materials. It is assumed that the sedimentary materials will break down to some degree during the excavation and placement process, and that the laboratory densities obtained for the sedimentary materials reflect that process.

CONCLUSIONS

Construction of the Kaiser Spring Wash Section of U.S. 93 will be impacted by the complex assemblage of geologic materials associated with the young volcanic center in the south end of the structural Big Sandy Basin. The geologic materials include a central core of thick rhyolite and rhyodacite dome flows and associated breccias, an interbedded sequence of tuff, conglomerate and basalt at the southern end and an interbedded sequence of sandstone and conglomerate at the northern end of the project.

Bedrock materials generally are young and unfractured and therefore will support steep slopes varying from 1/2:1 to 3/4:1 and locally 1:1 in the softer rocks at the northern end of the alignment. The design includes, at a minimum, a 20-foot wide bench to collect random rockfalls that will be generated from these slopes as a result of harder clasts/fragments embedded in a softer more erodable matrix.

It is anticipated that excavations by both conventional earth moving equipment and blasting will be required due to variable strengths and hardness of the rock masses. Careful construction sequencing of excavation in cuts and transportation of material to fill areas will be required to minimize impacts to road users. Road closures on this main roadway artery between Phoenix and Las Vegas will be limited to 15-minute intervals. In addition, slope rounding and sculpting will be performed to blend cut areas with surrounding landscapes to reduce visual impact to the road users. Blast hole traces and equipment marks also will be removed from the slope faces.

The highly variable geologic materials exposed along the alignment generally will be suitable for use as fill. However, the variable textures and fracture pattern in a few units will result in generation of materials of variable gradations. Thus, placement methods of fills may require periodic adjustments to obtain acceptable embankment fills.

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Caves discovered during the building of expressways in the Slovene karst region

by

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Abstract

Karstologists are involved in the construction of the expressways in the karst region. Through preliminary studies, we select the right-of-way together with the planners, and through regular karstological monitoring we help the builders span karst features and at the same time attempt to save as many as possible. During the construction of the newest section of the expressway, more than two hundred caves were opened in the thirty kilometers of the right-of-way. Different types of caves reflect the development of the through-flow and run-off aquifer due to the lowering of the underground water level and the karst surface.

Introduction

Selected karst phenomena discovered during the construction of the expressway in the classical karst region have already been presented (Šebela and Mihevc 1995, Slabe 1996, 1997a, 1997b), as well as the work that the staff of the Karst Research Institute ZRC SAZU carried out in the planning and construction of individual sections of the expressways (Kogovšek, Slabe, Šebela 1997). To these accounts, I am adding new ones since the construction of the expressways in our karst region will continue for several more years. In particular, I wish to present ways to open types of caves characteristic of the aquifer and ways to preserve the most important ones during construction.

In planning and selecting a right-of-way, we consider the geological, geomorphological, speleological, and hydrogeological characteristics and the entirety of the karst areas on which the expressway is under construction. We avoid the more important karst features such as sinkholes, large depressions, caves, and karst walls, and by making the expressways impermeable we try to prevent the influence of sewage from the roadway on the underground water. From experience, we can discern caves without ceilings on the karst surface before work begins on the land. For this, the characteristic forms in the karst surface, alluvium, and calcareous sinter deposited in the caves are of help.

During the construction of expressways when excavations lay open karst surfaces, cutting deeply into it and hollowing it with channels, numerous karst cavities are opened that reflect the development of the aquifer. We have researched all the caves, studied the alluvium and calcareous sinter in them, and tried to save the most important. The caves are an important part of our natural heritage.

Karstological monitoring is carried out under the auspices of the Institute for the Protection of Natural and Cultural Heritage and financing is provided by the Expressway Corporation of the

Republic of Slovenia. The presentation of the research was also made possible by the Ministry of Science and Technology of the Republic of Slovenia.

The cavernosity of the aquifer

A significant proportion of the expressways in the Slovene karst region will run through Kras, the region which gave its name to this unique landscape of carbonate stone (Kranjc et al. 1997). This is a larger, currently through-flow and run off aquifer through which flows water from the flysch hinterland of the Vipava Valley and the Brkini mountain range and from the Soča Valley which is covered by the gravel alluvium of the river. Some 65% of the water emerging from springs on the coast of the Bay of Trieste originates on the permeable karst surface. The aquifer was originally surrounded and covered by flysch. The carbonate stone of the karst aquifer was therefore enclosed and the underground water restrained, preserving surface streams. The aquifer was first hollowed out by well-fed groups of caves through which streams flowed. We can follow the slow karstification of the aquifer with gradual lowering of the underground water level linked primarily to the vertical neotectonic fissuring of the karst areas and marginal conditions. The irregularity of development is reflected by traces in the caves. Periodic minor oscillations in the level of the underground water are mostly the consequence of changing climatic conditions. In some places, flysch patches remained on the surface for longer periods of time. From these, water flowed through swallowhole caves to the karst underground. On the surface, the epikarst part of the aquifer developed with a denser network of fissures from which the water accumulated in shafts. Due to the lowering of the aquifer, old caves and potholes began to open. In uncovering the karst surface, we could only find alluvium that had been deposited in the caves. Many were already without ceilings and the alluvium therefore is on the surface. However, we did not uncover alluvium that would suggest the flow of water over karst surfaces that had lowered considerably. The precipitation water percolates dispersedly through the karst surface, lowers it, and partly transforms old caves. The level of underground water is 200 meters and more below the surface.

I divide the caves that are parts of the network hollowing the upper part of the aquifer into two basic groups. The first group includes the old, dry caves through which streams flowed when the level of the water in the aquifer was higher and the second includes the potholes. Old caves were through-flow and swallowhole-like. We can divide them into empty caves in which calcareous sinter as well as some alluvium can appear that are found at various depths below the surface and caves that are entirely filled with alluvium. The type of alluvium is a reflection of their development. The majority of the alluvium in them is granulated, indicating that in the last periods of their distinct development these caves were flooded many times and for long periods. The flood waters were obviously the consequence of climatic changes, since previously the caves were dry for a long time and calcareous sinter had been deposited in them. The level of the underground water rose and reached part of the caves but then dropped again relatively quickly. In some caves, gravel has remained, brought in by faster streams. In some places on the walls of caves that during a certain period of the development of the aquifer were on the level of the underground water, there were small scallops (Slabe 1995). Many such caves are already without ceilings and were preserved by fine granular alluvium that densely coated the circumference of the cave and prevented permeating water from reshaping the caves more distinctly. In some caves, there is also sharp-edged rubble resulting from the breaking up of the stone surface during cold periods in the development of the aquifer. Due to the lowering of the karst surface and the opening of the ceilings of the old caves, they are filed with this rubble. Old empty caves that emerged when the level of the

water in the aquifer was much higher merged into the fissured karst surface. Trenches and dolines formed from caves without ceilings filled with alluvium.

The water percolating from the karst surface flows through the fissures and potholes toward the underground water.

The opening of caves during construction of the expressways and their preservation

The caves opened during the construction of the expressways therefore reflect the development of the aquifer. In the thirty kilometers of the twenty-five meter wide expressway sections that were constructed in Kras in recent years, more than two hundred caves or accessible karst cavities were opened. Of course, there were many more fissures and pipes with smaller diameters. Two thirds of the newly opened caves were old caves, either empty or filled with alluvium and gravel; the remainder were potholes. One third of the old caves were filled with alluvium and were already without ceilings.

The caves measured from one to eight meters in diameter; the deepest pothole (Fig. 1) reached 110 meters, while the majority of potholes extended to thirty meters deep. The cavernosity of the aquifer is relatively uniform and dispersed.

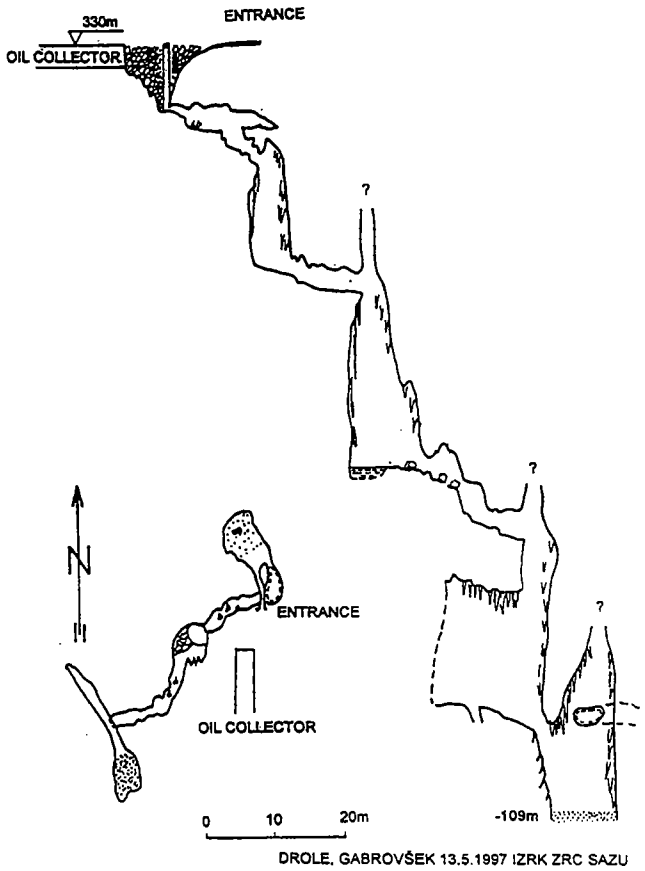

Fig. 1: Pothole with artifical entrance

Caves without ceilings (Fig. 2) were opened during the grading of the karst surface, many of which we predicted even before construction started, along with old caves that had the thinnest ceilings and potholes. In the process of blasting, many smaller caves were certainly destroyed, which was demonstrated by the calcareous sinter in the rubble. During the excavation of road cuts, caves were opened as ceilings fractured by the blasting collapsed and when the work cut their galleries. Entrances to these caves therefore appeared in the walls of the cuts as well. The ceilings that collapsed were five meters thick and more. The caves in the walls of cuts were both empty and filled with alluvium. In tunnels, caves opened across the entire circumference. Because of the blasting, the stone circumference of the caves was often fractured up to fifteen meters from the point of the blast, as well as the calcareous sinter. The nearest rock was fractured into pieces, while farther away, there were only cracks in the stone, with dripstone formations preserved on the circumference (Kogovšek, Slabe, Šebela 1997). It was only possible to study these caves with extreme caution. In investigating caves with poor ventilation, special attention must be paid to gasses that can remain trapped after the blasting. After one of our team suffered unpleasant nausea in one such cave, we decided to check the air first in all of caves of this type. Caves also opened later in the construction process during the compacting of gravel on the roadway with a vibration roller. The resulting breaks were one to five meters in diameter, and ten of them occurred in a thirty-kilometer stretch of the expressway. Ceilings above old caves and potholes collapsed, whose chimneys were hidden under the right-of-way. Later right-of-ways were therefore inspected using geo-radar.

Fig. 2: Cave without ceiling

Potholes were most frequent in the floors and sides of larger, multithroat (Šušteršič 1985) funnel-like dolines. The majority of the potholes were without a visible natural entrance. Numerous narrow and impassable fissure-like mouths appeared when the soil was removed. The more spacious potholes were usually discovered and researched during the digging of larger cuts and a tunnel and are ten to thirty meters below the surface. This is the consequence of the uniting of dispersed water percolating through the permeable surface; the more spacious

potholes—including those with natural entrances since the surface had previously dropped considerably—are the collectors of this water.

To guarantee the firmness of the roadbed, many caves were blasted and filled. The fine granular alluvium was removed from the caves without ceilings, which were then filled with rocks and rubble. The caves under the right-of-way whose ceilings collapsed due to the blasting of the rock above them or whose circumference was fractured had to be blasted and filled. Smaller caves below the right-of-way whose ceilings were thin and in danger of collapsing were also filled and covered with concrete lids. Thus, the sections of the expressway under which there might have been inaccessible continuations of the uncovered caves were protected.

Many caves were preserved. Entrances to potholes revealed during the removal of alluvium and soil from dolines were covered with concrete lids. The dolines were later filled with rocks and rubble (Slabe 1997, 188) and the potholes are now under the expressway. Concrete lids were also put on narrower potholes under the right-of-way. Caves opened in the walls of road cuts that were either empty or filled with alluvium have been protected behind rock walls. The circumferences of the empty caves were fractured due to the blasting and the caves are therefore not suitable for visiting, while the alluvium could be washed away by rainwater. We entirely preserved a characteristic old cave (Fig. 3) opened during excavations for the

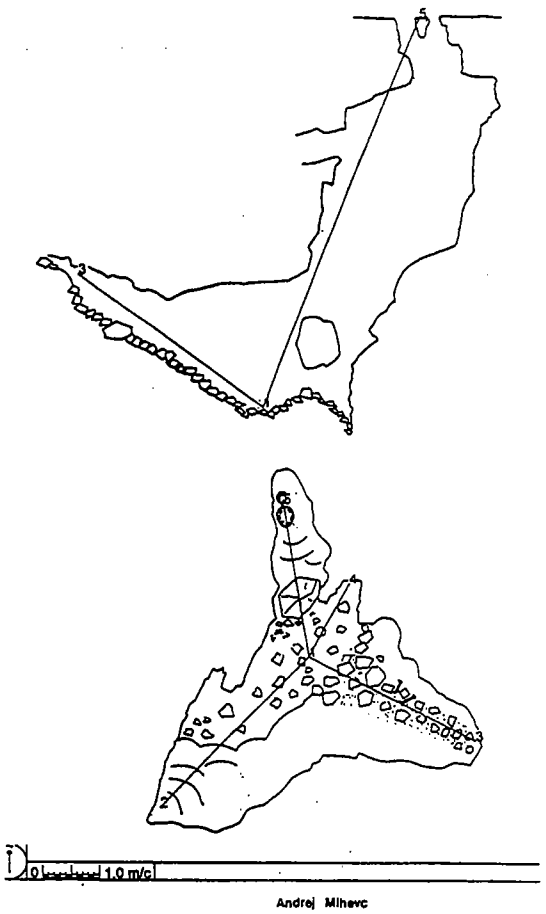

Fig. 3.: Old cave with artificial entrance

foundations of an overpass. It is hidden under the large foundation of the overpass, and it is possible to crawl into it through a concrete pipe closed by a lid ending at the green verge of the road. Thus, the deepest uncovered pothole (Fig. 1), which opened at the bottom of a doline at the edge of the expressway, will remain accessible. The construction of an oil collector is planned in the doline. An old cave exposed by a cut for the expressway before the Italian border, whose floor is covered by fine granular alluvium and numerous stalagmites, remains open and on view for travellers.

Conclusion

A solid knowledge of karst aquifers and their cavernosity makes it possible to foresee the kinds of caves that might be opened during construction projects. Where the caves are and what size they are a special problem, especially in the type of karst region described which developed in the diverse and tectonically reshaped and fractured carbonate rock.

Through karstological monitoring of the construction of the expressways in Kras where the right-of-way uncovered and cut into the karst surface several times, we established that old caves and potholes are found as often in Cretaceous as in Paleocene limestone and Lower-Cretaceous bituminous dolomites. Old caves in dolomite are usually smaller. Most of the potholes were discovered in Cretaceous limestone, the deepest in dolomite, and relatively few were uncovered in individual sections of Paleocene limestone. However, the latter is also pierced by numerous small pipes that can be seen in the cuts along the expressway. Only a few developed into larger caves. Stratification of the rock and its fracturing is more important for the occurrence and formation of caves than the properties of the rock, in particular. the properties of specific bedding planes (Knez 1996), cracks, and the network of fissures so that along them cavities develop under various conditions in the development of the aquifer.

In some places along specific bedding planes and open fault zones where known caves have developed, it is possible to anticipate the increased likelihood of opening caves. In other sections, the inspection of the right-of-way with geo-radar helps.

We encounter unique features when planning expressways over the shallow and partly alluvium-covered karst region of southern Slovenia, over the breccia karst below the high limestone plateau, and over the cavernous turbidite carbonate rock found in flysch.

In the view of the great cavernosity of the aquifer and the links between underground waterways that we established with numerous tracings, it is necessary to constantly draw attention to the danger of polluting the underground waters during the construction or use of expressways; new expressways should therefore be made impermeable. It is possible to find traces of pollution along major traffic routes in the waters of nearby caves. For the moment, the traces of mineral oils that we found in standing waters (Knez, Kranjc, Otoničar, Slabe, Svetličič 1944) are small, but it is necessary to prevent the occurrence of even these quantities.

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GEOTECHNICAL INVESTIGATION IN KARST: BENT ROCK VERSUS FLAT ROCK

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ABSTRACT

Not all karst is the same. There is the flat-lying, relatively young "limerock" of Florida and the Caribbean; the faulted and folded Paleozoic-aged limestones and dolomites of the Ridge and Valley Province of the eastern United States; the gently-dipping, Ordovician to Devonian-aged carbonates of the central United States; the many caves of Arizona; and even a few areas underlain by Proterozoic carbonates. Any field investigation must recognize the differences between these carbonates, the prospective mode of failure of concern to any proposed highway-related construction, and the variable nature of any karst area.

This paper addresses the differences in possible failure modes that develop in these different rocks in relation to potential highway structures, roadways, bridges, and stormwater direction/detention/ retention facilities. Any investigation is, of necessity, multi-phased, particularly if the geotechnical work is used as input for route selection. A review of the aerial photography and available geologic information, as well as a reconnaissance for ground-truthing the aerial photography and previous regional mapping, are all generally part of the first stage of an investigation in karst areas. Subsequent phases consist of developing a geologic model of the site, then testing and refining it through field investigation. Direct investigation tools include test pits and test borings using specific techniques, with perhaps the use of indirect (geophysical) tools to correlate (very carefully) between known data points.

In karst terrane, the subsurface target is always treacherous. The equipment and techniques discussed are not unique, but are not usually applied in the manner described in this paper.

INTRODUCTION

Karst terrane is international in its extent, but is often studied as a singular unit. Very generally, karst can be developed in three types of terrane, although perhaps only areas similar to the folded rocks of the Aegean Sea province actually named Karst should be considered truly karstic in nature. In the United States, karst topography is considered, very generally, to form in areas underlain by three types of rocks; A) geologically recent (Cenozoic) sediments, B) older (Mesozoic and Paleozoic) limestones and dolomites, and C) occasional metamorphosed carbonates (marble).

Much of the literature in the United States is related to the flat-lying rocks of the mid-continent (Figure 1) such as the Mammoth Cave area of Kentucky. Not all carbonates are flat-lying, nor composed of hard, massively bedded rocks. Florida carbonates, for example, may be flat-lying, but are generally soft, sandy "limerock". Appalachian Valley carbonates are hard, crystalline rocks, but are folded and fractured with seemingly always tilted bedding.

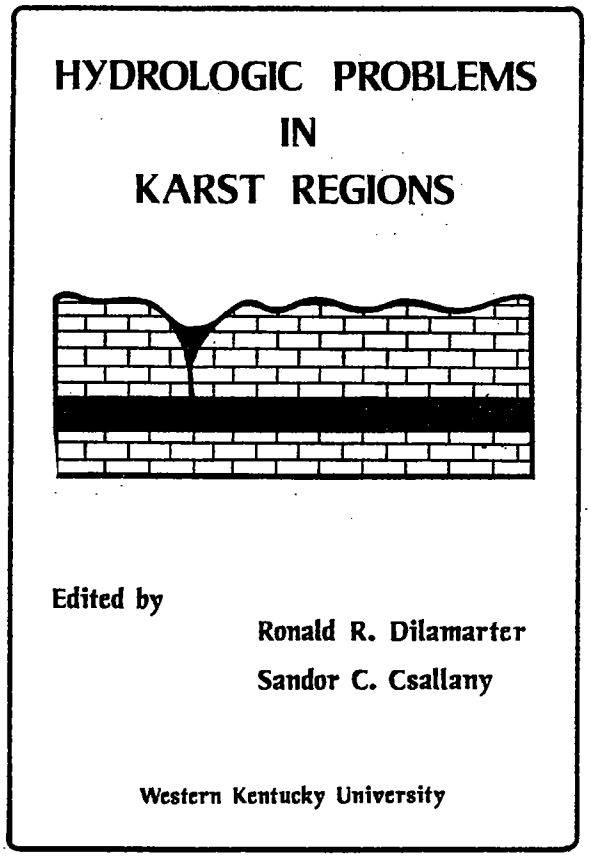

FIGURE 1 - Text Book Karst

Arizona has a wealth of examples. The limestones of the hills south of Tombstone are folded and faulted, similar to the Valley and Ridge Province rocks of the Mid-Atlantic states. However, the Grand Canyon limestones are relatively undisturbed and more similar to the cratonic carbonates of the mid-continent.

Thus, the first step in attempting to design a true geotechnical investigation of a route through, or a structure atop, carbonate terrane is to have an appropriate geologic understanding of the nature of the karst in the locale of concern.

Without attempting an extensive or complete characterization of potential failure modes, it may be useful to list some of the typical failure modes of each type of karst previously noted. Depending upon the focus of the investigation, "failure" can be related to either (or both) structural integrity and groundwater contamination.

A) Soft, recent karst often generates "cover collapse" sinkholes (e.g., Chen and Beck, 1989) where loose sandy materials fall or erode into generally near-vertical solution features within the relatively flat-lying "limerock" below. In Florida for example, the permeability of the overburden is often high and both the primary and secondary porosity of the soft limerock can be large.

B) Structural failure within the flat-lying, but hard Paleozoic rocks of the central United States generally occurs through "cave collapse" (e.g., Fischer, et al, 1996). The roof of a more linear solution feature (cave), formed along bedding or structure, may collapse as a result of continued solutioning or increased loading. Virtually undiluted contaminants can travel for miles through open channels in these flat-lying rocks (e.g., Hannah, et al, 1989).

Conversely, a ravelling-type of failure generally occurs as a result of the erosion of overburden materials into solution features along inclined bedding or shear zones in, for example, the folded and faulted rocks of the Valley and Ridge Physiographic Province of the eastern United States (e.g., Fischer, et al, 1996). Contaminants may not travel as far in open channels, but wells are often relatively closely spaced in some high population areas and well head protection considerations are important (e.g., Canace, et al, 1996).

C) Solutioned cave features are often found in hard metamorphics of, for example, the western United States. Failure usually requires the overloading of a cavity roof rather than persistent dissolution.

Beck, 1991, in characterizing sinkhole occurrences (Figure 2), again used flat-lying rock. Revising this yields Figure 3 for folded rocks (Fischer, et al, 1996).

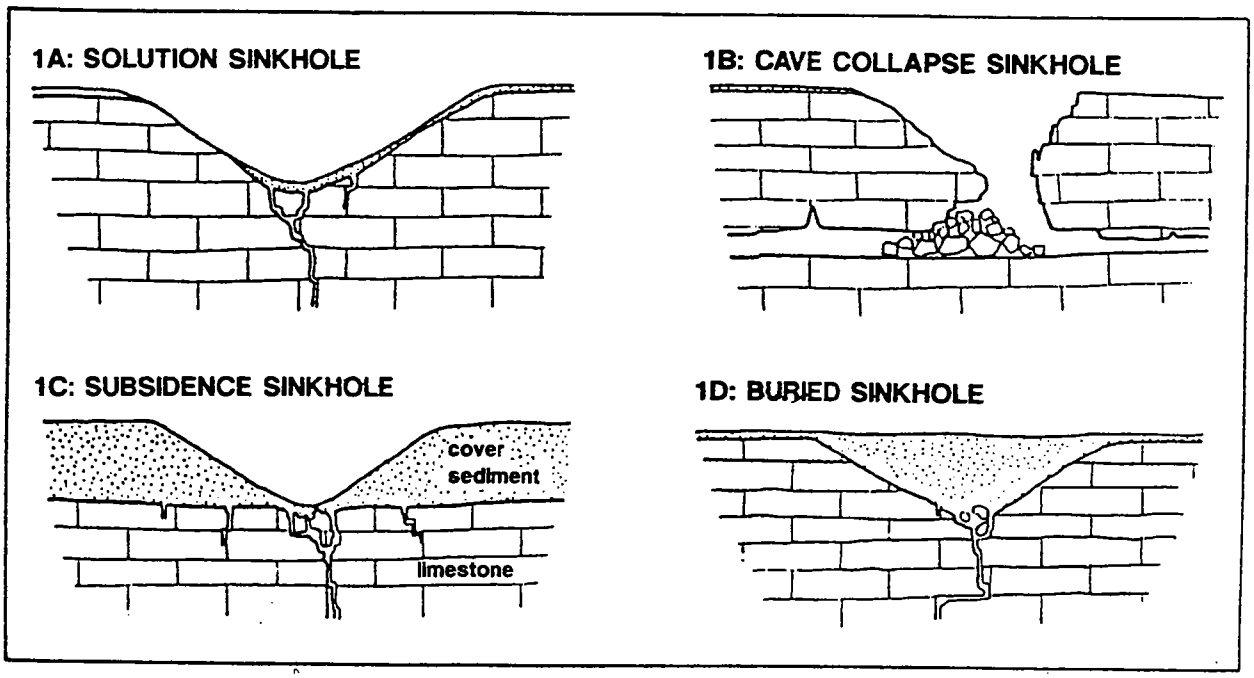

FIGURE 2 - Types of Sinkholes (Beck, 1991)

Before attempting to design an investigation, one should have some understanding of the nature of the prospective mode of failure of the subsurface, as well as the more conventional concerns of loading (both structural and contaminant) and grading (cuts and fills) likely for a site or along a route. Often, the "bent rock" and limerock type of failure can be considered a soil condition constraint.

In the more flat-lying, hard, crystalline rocks, a structural failure likely results from the collapse of a cavity roof, more of a rock mechanics failure. In addition, one must also recognize that variations in the nature of the karst will likely be more severe in a "bent rock" locale as a result of the numerous rock formations and/or beds that express themselves as the rock surface. Ground water contamination concerns result from the lack of chemical absorption or the trapping of particulate in undisturbed overburden materials, as well as rapid transport through open rock conduits common to all karstic terrane.

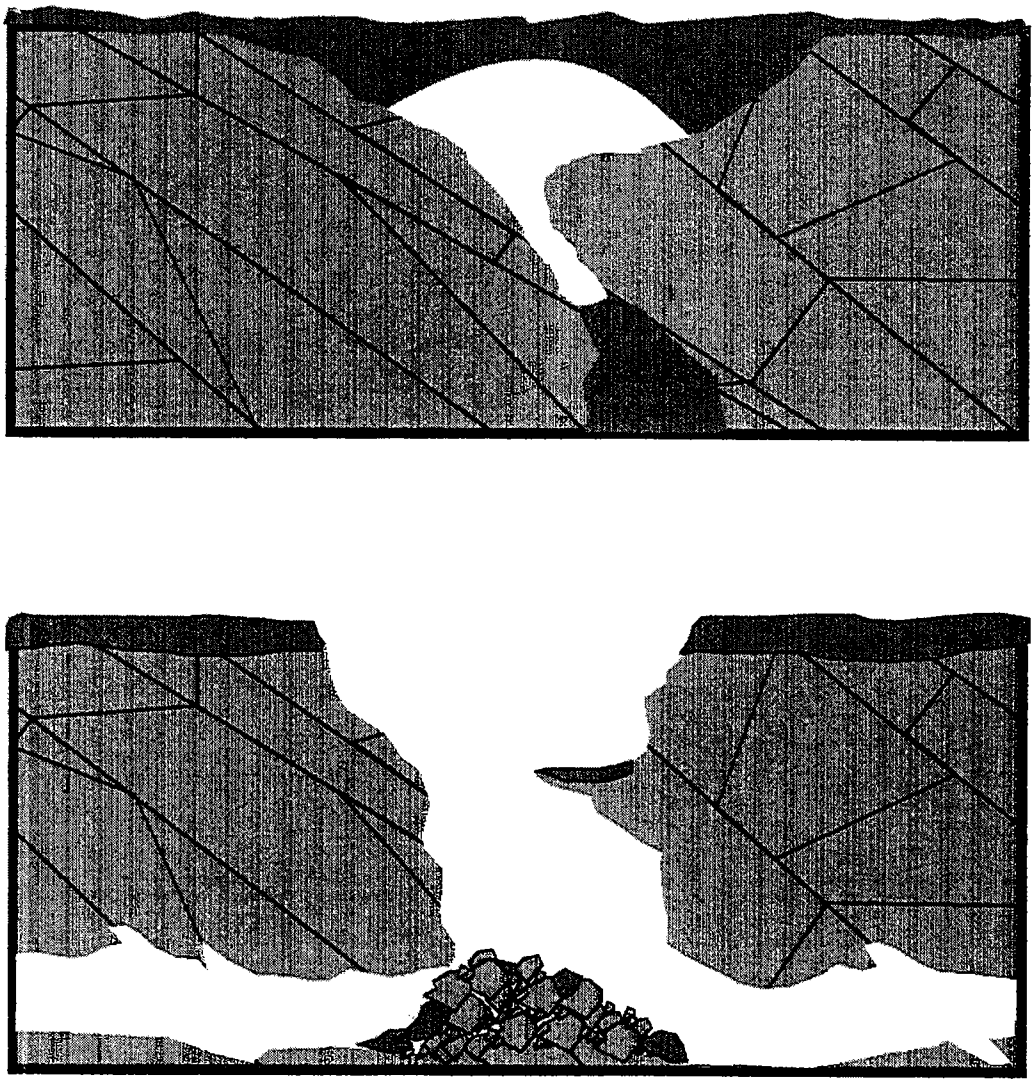

FIGURE 3 - Types of Sinkholes (Fischer, et al, 1996)

As a consequence of the varying nature of the karst subsurface and the nature of the resulting hazard, conventional soil mechanics investigation procedures are not usually effective. In addition, the basic geophysical tools such as reflection and refraction surveys, resistivity, and ground penetrating radar are often of marginal use. The existence of highly irregular and variably weathered carbonate surfaces must be considered and the resolution required to "see" soil voids as well as air, water and/or soft soil-filled cavities are seldom considered in geophysical investigations. In addition, residual soils with high clay contents and varying moisture levels will limit radar penetration depths.

Conventional permeability tests are often too specific for general site understanding. Sometimes aquifer parameters can be developed from large scale well testing with numerous monitoring points, although conventional well modeling formulae are inapplicable to an irregular, solutioned aquifer condition. Solutioned carbonates are not isotropic, anisotropic, or slab-fissured/fractured rock. A finite element difference solution is appropriate, but at what level does the cost of measuring physical properties outweigh the usefulness of the model.

The authors have performed investigations from differing perspectives, both research and commercially driven. However, the need to understand the subsurface is equally important whether one maps an area for basic geologic purposes or for eventual construction. Once the geotechnical, geological or geohydrological engineer has a sound geologic data base, he should proceed with the formulation of planning and design recommendations for the planned facilities. Route or facility layout in karst terrane as well as pavement or facility design can only be predicated upon a reasonable (and unfortunately never complete) understanding of the subsurface.

In our experience, almost any subsurface investigation is driven by economics, however, it is necessary to understand that one cannot define all possible subsurface variations at most karst sites. Assuming that no geotechnically oriented investigation will be funded at a level that allows a complete picture of the karst subsurface conditions, studies are generally performed in phases and with a strong emphasis on obtaining "geologic" data rather than trying to define the actual physical parameters of the soils or rock along a route or below a structural load until the final design stage. A multi-phased approach has been developed with the understanding that data will even be obtained during the construction phase and incorporated into final design. The field procedures were developed as a result of the combined experience of a number of geological, hydrogeological, geotechnical, drilling, and construction personnel. The use of this multi-phased approach to investigations allows a development design team to become more familiar with the site subsurface conditions as the program progresses from phase to phase.

PHASE I - ROUTE OR SITE AREA INVESTIGATION

In any karst investigation, the greatest return on investment is possible during the initial (Phase I) stage of an investigation. The first step is to review any data available from state and federal (public domain) sources in the form of research reports and geologic maps which may indicate the formations underlying the route/site and the extent of folding and faulting which have affected the formations. Aerial photography, taken over a period of at least a decade, should be obtained as karst features may develop or change over time. Annual cycles of precipitation and drought can highlight different features at different times. Features visible in early photographs may be obscured later by man's activities. Persistent lineaments and circular shapes are particularly suspicious when they are observed on a number of photographs taken over extended time periods. Photographs taken after periods of heavy precipitation can reveal interesting patterns of subsurface and near-surface water movement which can aid in developing a preliminary geologic model of the area in question. "Home-flown" oblique aerial photographs taken at low sun angles and after precipitation events can be particularly effective in delineating areas requiring further on-site study.

Aerial photography, and sometimes satellite imagery, in combination with the available geologic information are used as the basis for a geologic reconnaissance. Areas that farmers avoid during cultivation often represent rock pinnacles or persistent sinkholes. Forested areas in otherwise fully farmed lands generally indicate areas of shallow rock or possibly intense sinkhole activity. Changes in vegetation sometimes indicate incipient sinkhole formation. Obviously, photo-lineaments, depressions, possible sinkholes, disappearing streams, springs, wetlands etc. must be investigated on-site. One of the observations usually made in a geologic reconnaissance is that ground surface elevation variations across an area underlain by carbonates is generally greater than they appear from a distance or as indicated on topographic mapping. These variations usually represent stratigraphic differences or geologic structure that has been emphasized by differential weathering.

A geologic reconnaissance using the readily available data is a necessary step in any investigation in karstic areas. A geotechnically or geohydrologically oriented study provides the opportunity to confirm the data developed to this point and the necessary ground work for the next phase of investigation.

The amount of useful information that can be developed by an experienced engineering geologist/geological engineer from this preliminary phase is surprisingly large. The findings usually allow a number of decisions to be made for the subsequent phases of geotechnical investigation as well as the facility planning and preliminary design. From a planning or construction standpoint, the simplest approach may be to remove the site or a portion of the site from further construction considerations. This conclusion is often valid. More often, however, the decision is made to build upon the area of concern. This decision should be made with an understanding of potential remediation costs. All too often, the decision is made without an appreciation of the potential increase in design and construction costs that occur in karst terrane.

PHASE II - GEOTECHNICAL INVESTIGATION

The results of the preliminary investigation and the needs of the proposed construction will likely lead to additional studies for any important project. A Phase II investigation is mostly used to confirm and expand Phase I findings through direct subsurface investigation which generally includes a number of test pits and borings at key locations. From a geohydrological standpoint, establishing ground water levels and monitoring water losses in test borings are generally all that is done for this phase.

This phase of the investigation should be used to further explore geologically suspect areas identified in Phase I as well as critical facility areas and storm water detention/retention areas. The emphasis of a Phase II investigation is to provide both hard data and allow, at least conceptually, for structures, roadways, utilities and other facilities to be located and designed in a manner appropriate for the subsurface conditions likely to be encountered throughout the area of concern (not just at the locations drilled or excavated). However, so as not to escalate the cost of this phase, much extrapolation is necessary to expand the limited information gathered to this point into a loose but coherent geologic model of the entire route or facility site. One must also recognize that the variations that can occur over short distances will usually preclude the development of an all encompassing geologic model.

Test pits can be performed with conventional equipment and are obviously quite informative, particularly when a comparatively large area of rock is revealed in the pit bottom. The undulating or even disappearing rock surface is a surprise to many investigators. Often, 10 foot long test pits have encountered shallow rock at one end and none at the other. Soil characteristics are also diagnostic. Is there a simple pattern of weathering in the pit side; or is there evidence of leaching or ground water movement; or an old filled sinkhole; or soil voids; is the bedding dip identifiable in residual soils? Backfill in test pits should always be compacted to densities essentially the same as the in situ materials. Very often, soil log test pits become new sinkholes because poorly placed backfill allowed surface water to erode soils into an existing underlying cavity or seam.

Test borings should be drilled using rotary wash drilling procedures without the addition of drilling "muds" to allow water loss depths and quantities to be monitored. Drill water is often lost just above the top of the rock indicating ground water flow had previously carried the generally clayey residual soils into a nearby (but not necessarily directly below) cavity within the rock. In the authors' experience, these water losses sometimes occur well above the rock (several meters) indicating a subsurface "throat" (often a precursor to soil void formation), a soil void (a sinkhole waiting to happen) or the gradual raveling of soils into underlying rock cavities. The soils encountered atop the rock in these areas of drilling water loss are usually soft and sticky, or even non-existent. Sampling is usually accomplished with a conventional split-spoon soil sampler, although the softer materials are sometimes difficult to retain even with the use of an appropriate sampler trap.

Encountering rock is always an adventure. Is it sound, broken, weathered (to what degree), and what is the nature of the rock surface? Numerous core barrels, roller bits and drill rods remain in the ground as testimony to the difficulty of attempting to advance a hole in creviced, solutioned or pinnacled limestone. If "rock" is encountered, is it a pinnacle, boulder, or sound bed underlain by a series of saprolitic beds?

Rock core should be extracted with a double- or triple-tube, split core barrel. This equipment takes no longer to advance in the borehole and perhaps only an extra few minutes to extract the core. The information gleaned from a core run lying in the split barrel, not "hammered" out of a conventional core barrel, is invaluable. Numerous fractures, some open, some soil-filled, are evidenced. Often, cavity fill materials are recovered. Three to six meter runs with a 1.5 meter core barrel are not uncommon. Making a 1.5 meter core run through competent material is rare in many karst terranes. Obviously, a driller with extensive karst drilling experience is a must.

Many indirect procedures have been advanced for the detection of subsurface cavities and rock contours which have not yet become surface features (depressions, outcrops or dolines). A host of geophysical techniques have been suggested as suitable including seismic reflection and refraction, electrical resistivity and conductivity, self potential, ground penetrating radar, and gravity surveys. If one is aware of the nature of the subsurface target and the resolution available with most commercial equipment using conventional procedures, it is difficult to visualize how these geophysical prospecting procedures can be considered any more than marginally useful in karstic areas. They function best in an auxiliary role to the performance of direct measurements of subsurface parameters. Pinnacled

rock, large floating boulders, the size and nature of soil voids, clayey soils, and the lack of a coherent ground water table compromise the use of geophysical surveys in most karst engineering applications.

It is valuable to recognize, however, that at times such indirect methods can be effectively combined with direct methods of investigation such as test borings and test pits. However, geophysical methods cannot be substituted for carefully drilled test borings, qualified full-time inspection, experienced and professional drillers, and large diameter (Nx-sized or larger) double- or triple-tube split barrel coring. There is invaluable information to be gained through examining water-softened soils immediately above the rock surface, noting drilling water losses at various depths, observing clay-filled seams within the rock, transported soils, stained joints, and/or weathered cavity or joint sides in a core run laid out in the core barrel, or watching the drill rods fall through a 3 meter void and into soft cavity fills.

Pneumatic probes can be quite valuable when used in conjunction with test borings because of an air-track's mobility, speed of drilling and relative economy, at least at shallow depths (less than about 12 meters). Although no core is extracted, rock depths and qualitative competency can be estimated. With an experienced operator, it is possible to see or feel major changes in the stiffness of the overburden soils as well as measuring penetration rate differences in variable quality rock. Noting the depths that air loss is experienced also aids in understanding the subsurface. Observing air returns from a previously drilled probe hole after noting its disappearance from the hole in progress can be quite informative.

Aside from the soil and rock data available from test borings and pneumatic probes, grouting (or attempting to grout) the borehole is informative. A conventional cement, water, bentonite grout can be used and the quantities of grout placed downhole should be measured. However, after several cubic meters of grout have been introduced into a 0.1 cubic meter borehole, it is often necessary to employ unconventional means to plug the hole so as to prevent water infiltration and possible future sinkhole occurrence.

PHASE III - FOUNDATION OR GEOHYDROLOGICAL INVESTIGATION

Phase III takes the conclusions drawn from all of the work to this point, considers the proposed route or facility layout, and performs more detailed subsurface investigations (as necessary) to further expand the information in critical areas. The investigation can be performed in a single field program or just prior to planned, phased construction. In either case, all subsequent work should always build upon the previous work. For example, if construction is planned for an area identified as likely having shallow rock, relatively closely spaced test pits may be appropriate to help in estimating whether rock will be encountered and whether it is "rippable" or will require some other means of removal (e.g., blasting or "hydrohammer"). Signs of past of incipient sinkhole formation may be observed from the ground surface in the test pit wall. If highly sensitive or heavily loaded facilities are included, then drilling below the proposed locations or even at each column location may be warranted.

Geohydrological concerns are not as easily addressed. How do you characterize an almost infinitely variable medium. Aquifer tests may provide generalized an-

swers for large areas when numerous observation wells are used and when great precision is not necessary. When one wishes to find out "where does it go and at what concentrations?", dye tracing is a must. (e.g., Hannah, et al, 1989). This paper will not further discuss dye tracing in carbonate terrane. A specialist is a necessary adjunct should the need be there.

Sometimes the Phase III investigations can include remedial grouting such as suggested in Fischer and Fischer, 1995. An exploration/grouting program is generally quite cost-effective at this stage (prior to construction), rather than after failure has occurred. Although remediation can be accomplished in a number of ways, downhole tremie grouting and shallow excavation with sinkhole (rock) throat closure are generally the most economical.

Dynamic compaction (or perhaps more appropriately - dynamic destruction) can also be quite effective in remediating larger areas or along a route (or portion thereof). Dynamic destruction entails dropping large weights from great heights to collapse any near-surface cavities and/or voids, and to densify the founding soils.

SUMMARY AND CONCLUSION

Karst terrane is wide-spread, but not necessarily well understood from an engineering design and construction standpoint. Misunderstanding karst concerns can be partially attributed to the lack of appreciation of failure types in light of the geologic conditions and proposed projects uses. Thus, to adequately perform an appropriate geotechnical investigation and provide satisfactory answers to karst-related design and construction problems, a multi-faceted study is usually necessary.

One must understand the nature of the subsurface in relation to design and construction concerns. The great variations in subsurface conditions over short horizontal distances in karst terrane make it economically impractical to completely characterize its' physical nature. An investigation will likely not be able to completely define a route or site. Both direct and indirect procedures for investigation are useful, but only certain direct procedures are effective in identifying the actual subsurface conditions, and only at the test pit or test boring location. Each boring samples only <u>one</u> spot. Hence, extrapolation of these data are more difficult at a karst site than in most geologic environments. Be careful.

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ASSESSMENT, PREDICTION AND REMEDIATION OF KARST CONDITIONS ON I-70, FREDERICK, MARYLAND

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ABSTRACT

Locating and characterizing areas which are susceptible to subsidence and sinkholes is a formidable task. It requires considerable geologic insight based upon experience as well as a unique strategy for field measurements. The objective is to create a conceptual model of conditions and then make measurements and observations to confirm or change the conceptual model while converging on the true conditions as rapidly and cost effectively as possible.

This strategy was recently applied to a section of I-70 in Frederick County, Maryland where a significant number of sinkholes have occurred. A karst investigation was carried out to assess I-70 and nearby areas. Work included: a review of available data, EM conductivity and gravity measurements, a review of aerial photography, extensive observations of sinkholes in the area, as well as observations in a nearby quarry and discussions with the quarry geologist. This paper describes the problem and its impact to traffic safety, the strategy of the technical approach for karst characterization, the resulting data, and development of a conceptual model of subsurface conditions, and the prediction of areas where subsidence and collapse are most likely to occur. Follow-up studies and remediation are also underway.

THE PROBLEM

Carbonate rocks are susceptible to dissolution and are characterized by sinkholes, caves, and underground conduit drainage. The effects may manifest themselves by slow subsidence or may occur suddenly and catastrophically when unconsolidated sediments are washed into cavities within the bedrock. Often, changes in hydrologic conditions (i.e. pumping of groundwater, drainage modifications, increased groundwater recharge) trigger the transport of sediments, ultimately leading to sinkholes.

A significant number of sinkholes have opened up along I-70 and nearby roadways south of the city of Frederick, Maryland. This portion of I-70 runs east-to-west and is centered just north of the Genstar Quarry (Figure 1). The risk associated with subsidence and sinkholes along highways are higher than many other locations due to

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the relatively steady steam of traffic flow. Extensive emergency grouting has been undertaken at certain locations along I-70 in an effort to remediate the problem. A karst investigation was carried out to assess subsurface conditions along a total of 7,050 feet of highway. Figure 1 shows the extent of the area investigated. The focus of the work were to characterize the overall karst setting and to locate any areas in which potential collapse may be concentrated.

Boyer (1997) presents a number of regional indications of karst, including geomorphic features based upon topographic and soils maps. An initial site specific characterization of top of rock and karst conditions had been developed by Fahs (1996) and the Maryland State Highway Administration (Boyer, 1997).

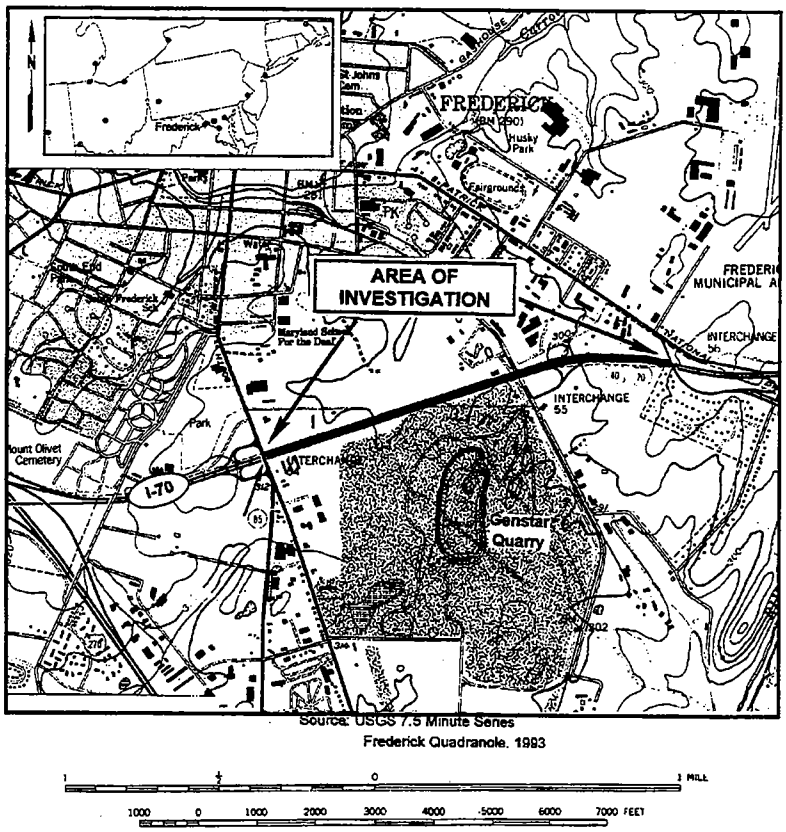


Figure 1. Site location map

THE APPROACH

The approach to assessing karst conditions consisted of a review of existing data, on-site observations, aerial photo interpretation and a comprehensive investigation using surface geophysical measurements. Table 1 summarizes the wide range of observations and measurements made at this site along with the property measured or observed and their purpose. Table 1 also illustrate the approximate depth of these

data. Selection of geophysical measurements were based upon decades of successful experience in characterization of karst conditions (Yuhr, et al., 1993).

> *Our focus here is upon gathering appropriate data, accurate data and adequate data which can be interpreted and integrated into a reasonably accurate conceptual model of site conditions.*

Regional geologic structure and karst features were identified from existing data and aerial photo interpretation. Site specific conditions were characterized by existing site specific reports, and observations as well as surface geophysical measurements. Field observations are a key aid in the interpretation of all data and guide the conceptual model.

> *The many hours on-site provides the opportunity to make observations, which are often critical to understanding site conditions and cannot be obtained in any other way.*

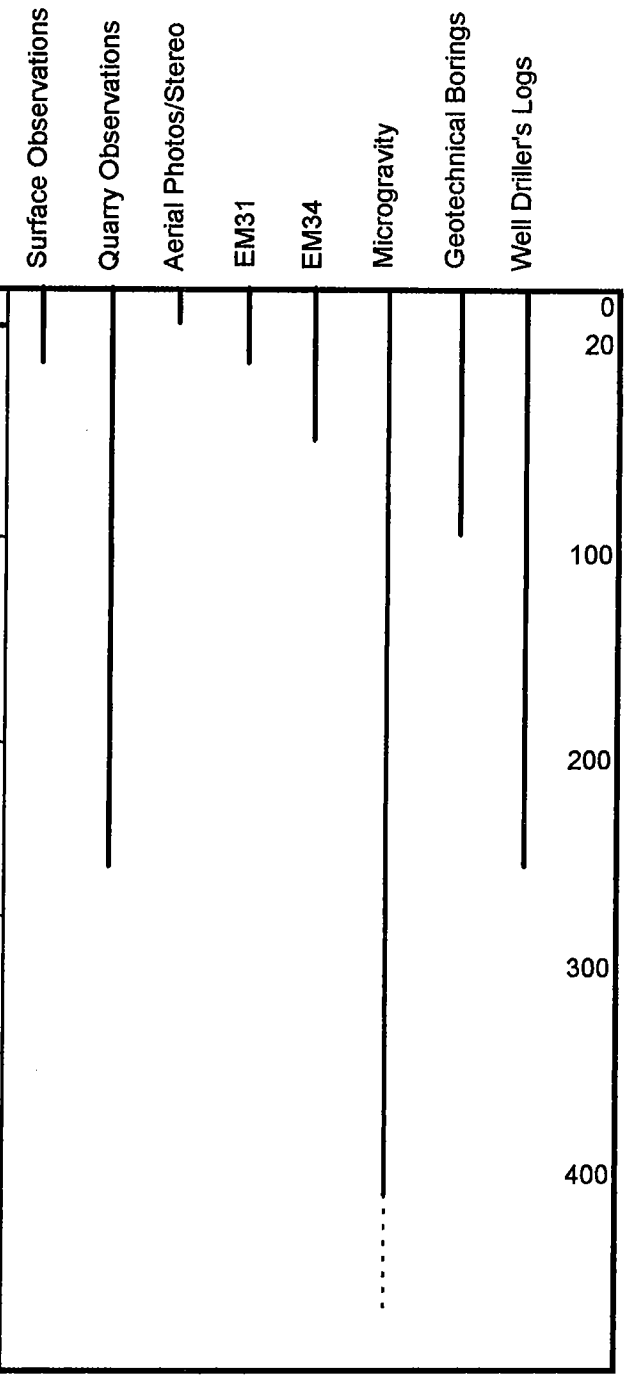
The EM31 and EM34 conductivity data were used to determine the relative thickness of soil cover (i.e., cutter pinnacle profiles), while the microgravity data were used to assess deeper karst conditions (cavities) within the rock. Since the EM measurements can be affected by metal guard rails and culverts, the use of multiple lines provides the necessary redundancy in these measurements to discriminate cultural noise from changing geologic conditions.

> *An important approach to resolving complex conditions is to use a variety of different measurements to characterize subsurface conditions (Benson and LaFountain, 1984). The support from different sets of data obtained in different ways provides a reliable, defensible means of testing the hypothesis upon which the conceptual model is based.*

A detailed site map of key features was developed on an overlay of an aerial photo to place all of the data into spatial perspective. Then interpretation and integration of data was carried out to develop concepts of subsurface conditions. Finally, a conceptual model of subsurface conditions supported by data.

Table 1 - List of Data Used

Measurement	Property Measured	Purpose
Review of Available Data - State Geologic Survey - Site Specific Reports - Personal Communication - Grouting Records	Geologic structure, caves Driller's well logs to 235 ft. Geotechnical borings to 50 ft.	Provides regional setting data for site specific investigation
Observations - Detailed along 7,050 ft. of roadway, ±1,000 ft. each side - Reconnaissance beyond 1,000 ft. - Within Genstar Quarry	Drainage and recharge area indications of active soil piping and collapse Surface manifestation of underlying karst.	Develop a spatial pattern and statistics for existing conditions
Aerial Photo Interpretation and Stereo Pairs - various scale and times	Surface manifestations of underlying structure and karst.	Provides surficial indications of soil joints and dissol-ution-enlarged fractures, drainage patterns, sinkholes, etc.
Continuous EM31 Profiles - nine parallel profile lines (37,770 linear feet)	Electrical conductivity of the subsurface	Determine relative thickness of clay soil cover
Continuous EM34 Profiles - five parallel profile lines (20,970 linear feet)	Electrical conductivity of the subsurface	Determine relative thickness of clay soil cover
Microgravity Station Measurements - along 7,050 foot profile line with measurements every 15 ft. (total of 471 stations)	Density contrast within the subsurface system	Assess dissolution-enlarged joints, cavities and cutter/ pinnacles in limestone



THE RESULTS

Regional Geologic setting

The study area lies roughly perpendicular to the central axis of a NNE striking syncline, and includes the Grove Formation bounded on the east and west by the Lime Kiln Member of the Frederick Formation (Reinhardt, 1974). The Grove Formation dips up to approximately 45 degrees on the western edge of the syncline and is vertical to overturned on the eastern edge of the syncline. Figure 2 shows the location of the study area within these formations, and a generalized cross-section through the syncline.

The Lime Kiln Member (590 feet thick) is the upper portion of the Upper Cambrian age Frederick Formation. It is characterized by highly burrowed, very thin to thick limestone beds. The Grove Formation is composed of thick-bedded nearly pure limestone with massive beds of fine-grained dolomite in the lower part and highly quartzose limestone at the base. The maximum total thickness of the Grove Formation in the study area is about 650 feet. The Grove Formation limestone is mined at the Genstar Quarry south of the study area, which is presently at a depth of about 300 feet.

A sandy layer is present at the base of the Grove Formation and outcrops to the west and east within the surveyed area of interest. This sandy layer results in a silty/sandy residual soil which can easily be transported into deep cavities. One larger sinkhole just off of I-70 occurred in this sandy material.

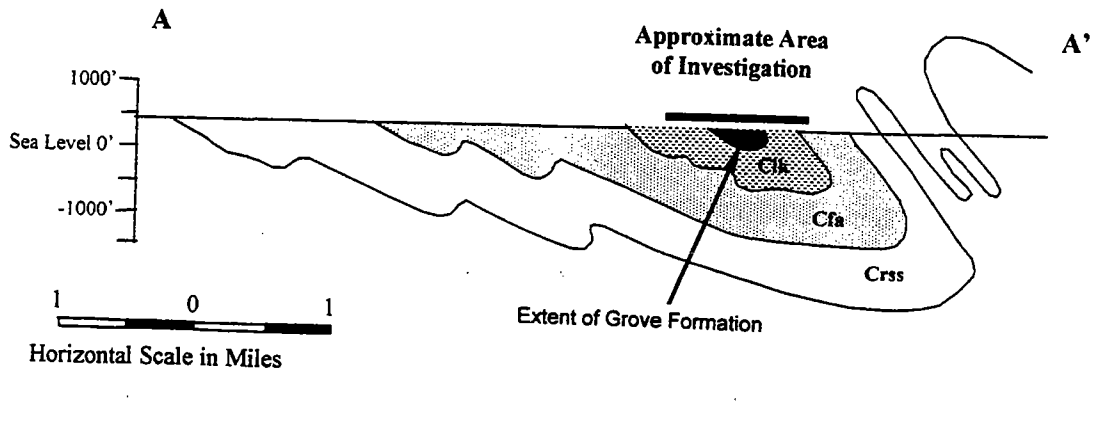


Figure 2. Geologic cross-section (Reinhardt, et al., 1974)

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Regional Indications of Karst

Numerous sinkholes and closed surface depressions were identified in the aerial photo analysis of the area surrounding I-70. Field checks of these sinkholes revealed that many of these sinkholes are currently active. During the period of this investigation, one sinkhole swallowed an estimated 10,000,000 gallons of water overnight after opening up beneath a flooded low-lying area.

A majority of the identified sinkholes occur within the Grove Formation limestone and are aligned with the strike of the formation. The sinkholes are generally less than 20 feet deep and typically result from piping of soil and small rock into open bedrock cavity zones. Individual active sinkholes around the site are generally small (one to ten cubic yards of unconsolidated material). A few active areas of subsidence encompasses an area of about 1,000 sq feet and include smaller localized sinkholes. A few larger (about 10,000 sq feet) sinkhole areas are active, while others are greater than 10 years old.

Regional literature indicated a total of 10 caves have been located in Frederick County (Franz and Slifer, 1971), and four springs are within a radius of 3 miles of the site (Meyer and Beall, 1958). These facts by themselves indicate a lack of large karst features.

A regional water level map (Nutter, 1973) indicates a broad area of deeper water levels (~50 feet) running along the eastern contact of the Grove Formation, and water yields from wells may be used as a general indicator of the permeability of the bedrock in which they are completed (Duigon and Dine, 1987).

Local Conditions

Based upon observations in the quarry, a system of cutter and pinnacles with residual clay soils and weathered bedrock are seen around much of the perimeter of the Genstar Quarry. Geotechnical boring data indicate the top of rock is highly irregular and ranges from 0 to 50 feet below grade and that cavities are commonly found within the rock (RK&K, 1996).

Long term water level measurements from the three geotechnical borings nearest the site show the water table at 15 to 28 feet below grade (Fahs, 1996), while a piezometer located just 550 feet to the southeast indicates a water level between 160 and 180 feet deep. The Genstar Quarry, approximately 300 ft deep, is dewatered by pumping up to 2×10^6 gpd (Herbert, 1996) which lowers the surrounding water levels to the northeast and southwest along geologic strike.

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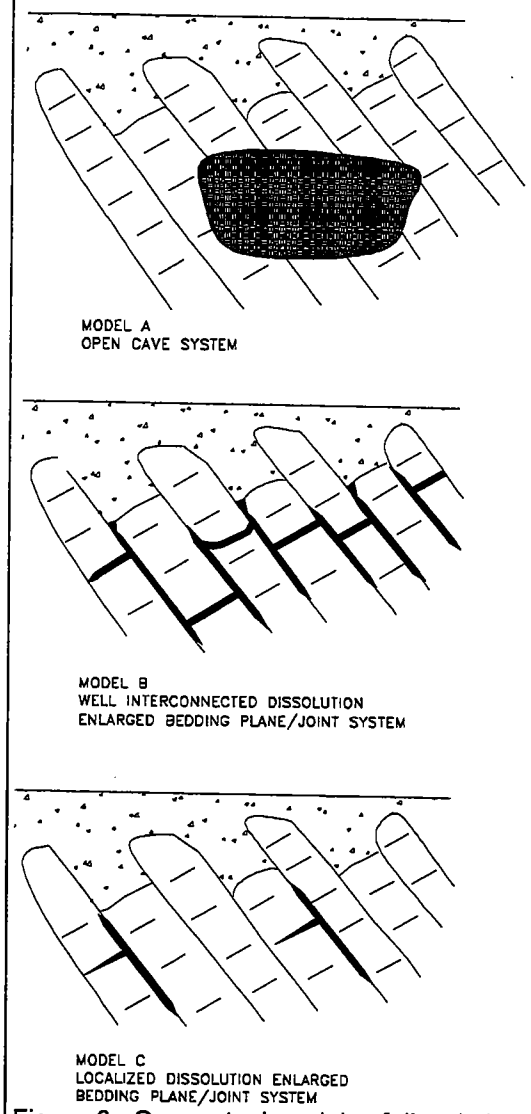

Figure 3. Conceptual models of dissolution zones within the bedrock

Based on the regional and local data along with observations, an initial conceptual model of subsurface conditions was developed. The models consist of residual clay soils overlying a zone of weathered bedrock having a cutter and pinnacle profile. In the initial conceptual models, the bedrock contain cavities of unknown character and size (Figure 3).

Geophysical Measurements

The gravity profile along I-70 (Figure 4) shows a very large obvious anomaly (lower gravity value) of about 800 μGals. The anomaly has a lateral width of about 3,800 feet and its edges correlate with the extent of the Grove Limestone.

Density Contrast between Grove and Lime Kiln Bedrock

The large gravity anomaly is simply too large to be due to cavities within the limestone alone since it would require an inordinate amount of void space. It is interpreted to be partially due to a small density contrast ($\Delta\sigma$ = 0.07 g/cc) between the Grove Formation and Lime

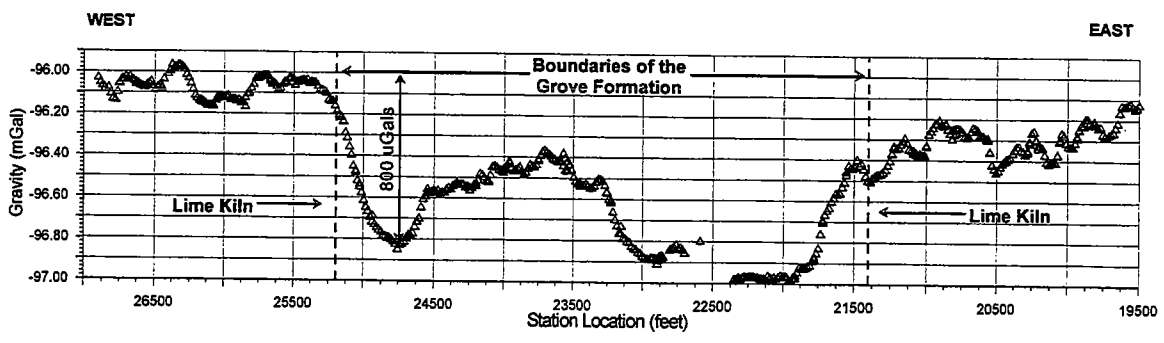

Figure 4. Microgravity profile data along I-70

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Kiln Member limestones. A model of this small density contrast accounts for approximately 400 μGals of the gravity anomaly.

Cutter and Pinnacle Locations

Narrow anomalies were observed in the gravity data and interpreted as being due to the cutter and pinnacle nature of the weathered bedrock. These anomalies are 100 to 500 feet wide and 30 to 200 microGals in magnitude. The gravity anomalies show a clear correlation with the EM31 data (Figure 5) which confirms the presence of cutters. The gravity values decrease with an increase in soil cover (due to the density contrast between soil and rock), while the EM31 values increase with an increase in soil cover (due to thicker electrically conductive clayey soils).

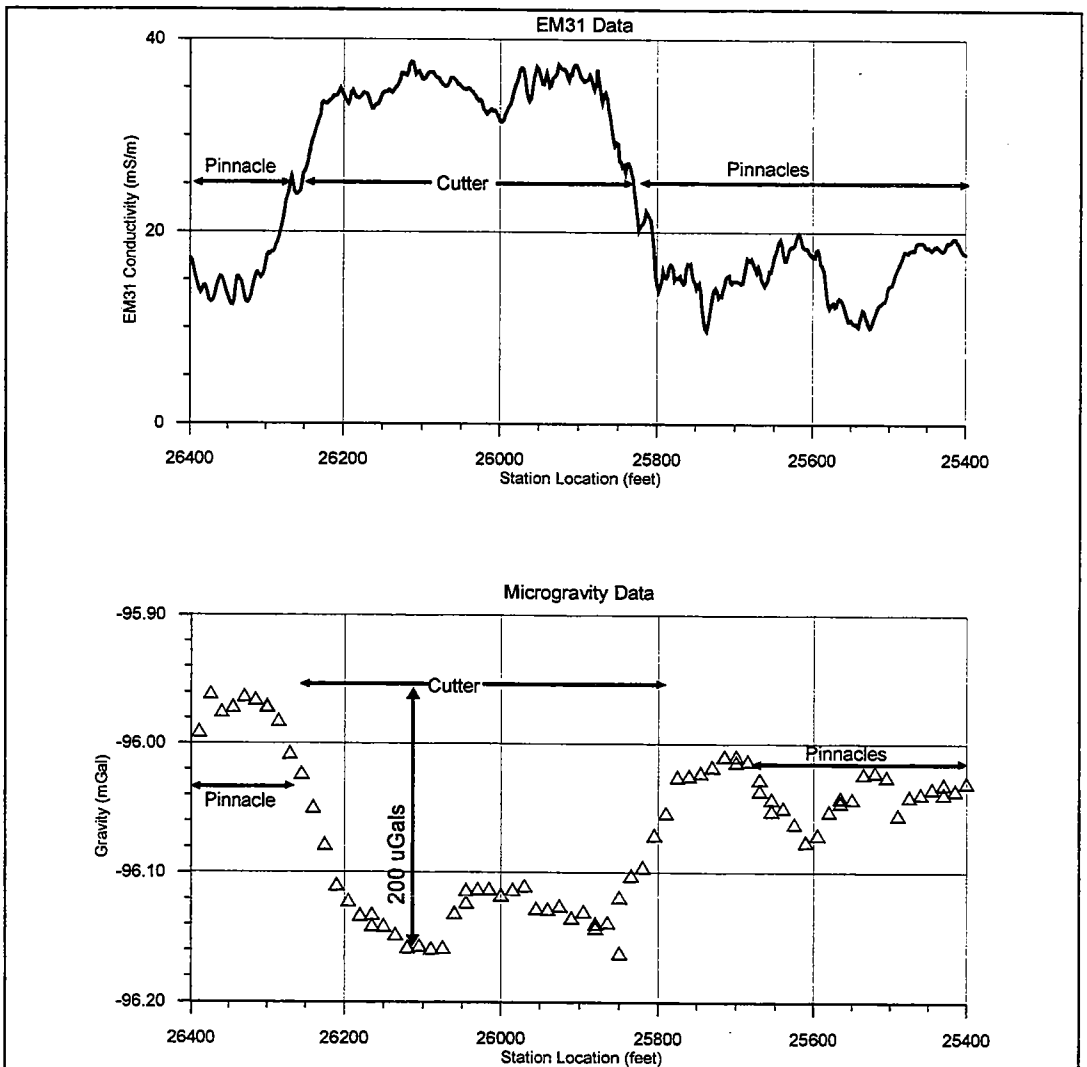

Figure 5. EM31 and gravity profiles locate and characterize cutters and pinnacles

Gravity models indicate a maximum thickness of the cutters of 60 feet, which is in good agreement with boring data. The cutter widths range from less than 20 feet to 500 feet,

and are typically separated by steep-sided pinnacles. Observed rock outcrops along the highway were used to constrain the models.

Other Factors

Other subsurface features also have an effect on the observed gravity data; the quartzose zone at the base of the Grove Formation, fill materials added for highway construction and fill materials for ramps to bridges also affect the gravity to some degree. The missing mass from the quarry has a small negligible effect across the entire gravity profile (<40 μGals). These features along with the bedrock density contrasts are modeled using constraints derived from outcrops, borings and topographic maps. Figure 6 shows the modeled gravity data after taking all of these factors into account. The remaining portions of the gravity anomaly not accounted for in the modeling are assumed to be due to other features such as karst.

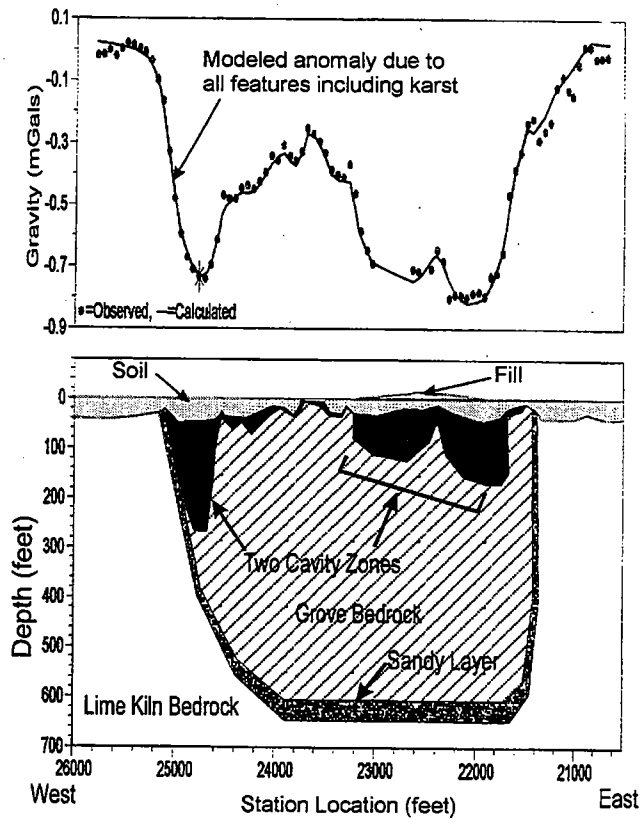

Figure 6. Gravity model of two sinkholes prone areas along I-70

Karst

After removing the various non-karst aspects from the gravity data, two areas of substantial gravity anomalies (mass deficit) remain (Figure 6). The western and eastern boundaries of these remaining anomalies are bounded by the western and eastern contacts of the Grove Formation.

The western gravity anomaly is approximately 500 feet wide, while the eastern gravity anomaly is approximately 1500 feet wide. These anomalies are interpreted as being due to dissolution-enlarged cavity zones within the bedrock. Note that the eastern gravity anomaly is located in an area of extensive recent and previous karst activity on I-70 and north and south of I-70 (Figure 7).

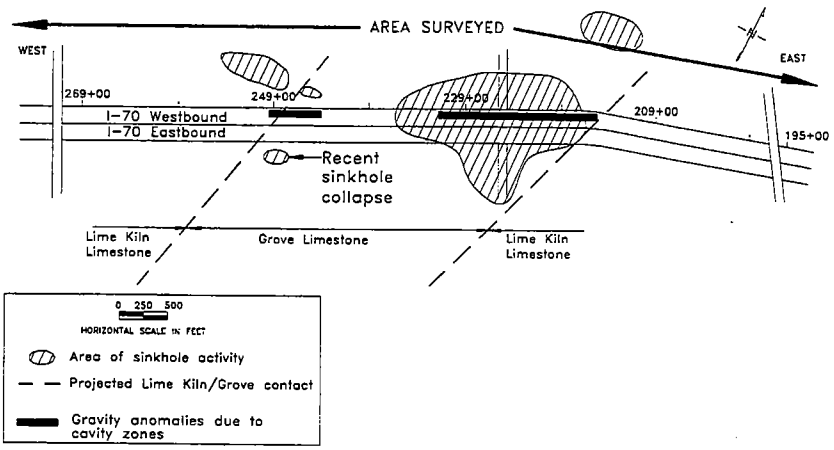

Figure 7. Locations of sinkhole prone areas based on gravity anomalies along I-70 with location of recent sinkhole collapse

These remaining gravity anomalies were then modeled to assess their character (Figure 6). Forward modeling is used to estimate maximum depth and void space. For modeling purposes, the cavity zones are assumed to consist of 70% unweathered bedrock and have a secondary porosity (dissolution-enlarged cavity zones) of 30%. The secondary porosity is modeled with 50% clay and weathered rock, and 50% water. Using these assumptions, the gravity modeling suggests that the cavity zones extend to a maximum depth of 260 feet at the western anomaly and 170 feet at the eastern anomaly. This model thickness is reasonable based upon a weathered, more permeable zone defined by well drillers in the area (Meyer and Beall, 1958).

Assuming a width of 200 feet which may affect the roadway, the western cavity zone has a water-filled volume of 90,000 cubic yards, while the eastern cavity zone has a water-filled volume of 175,000 cubic yards. The modeled gravity now fits the observed values with a small level of error.

> *Note: The gravity model shown in Figure 6 is non-unique, and cavity zones may, in fact, consist of a different volume, depths or material fill and void space that produce the same anomaly. A modeling analysis was performed that shows a range of realistic void spaces for the cavity zones. To be valid, gravity models must have good supporting data and be guided by experienced interpretation.*

THE CONCEPTUAL MODEL

Separating Data from Opinions

In any complex site characterization, all data is used with caution until it is supported by collaborating data. Once two or more pieces of data agree, we begin to have greater confidence both of them. We create hypothesis (opinions) about subsurface conditions and then go about supporting them or abandoning them as data become available.

A final conceptual model of subsurface conditions has been developed and is illustrated in Figure 8. The conceptual model shows the integrated results of all observations and measurements conducted in the karst investigation. The conceptual model for the site consists of:

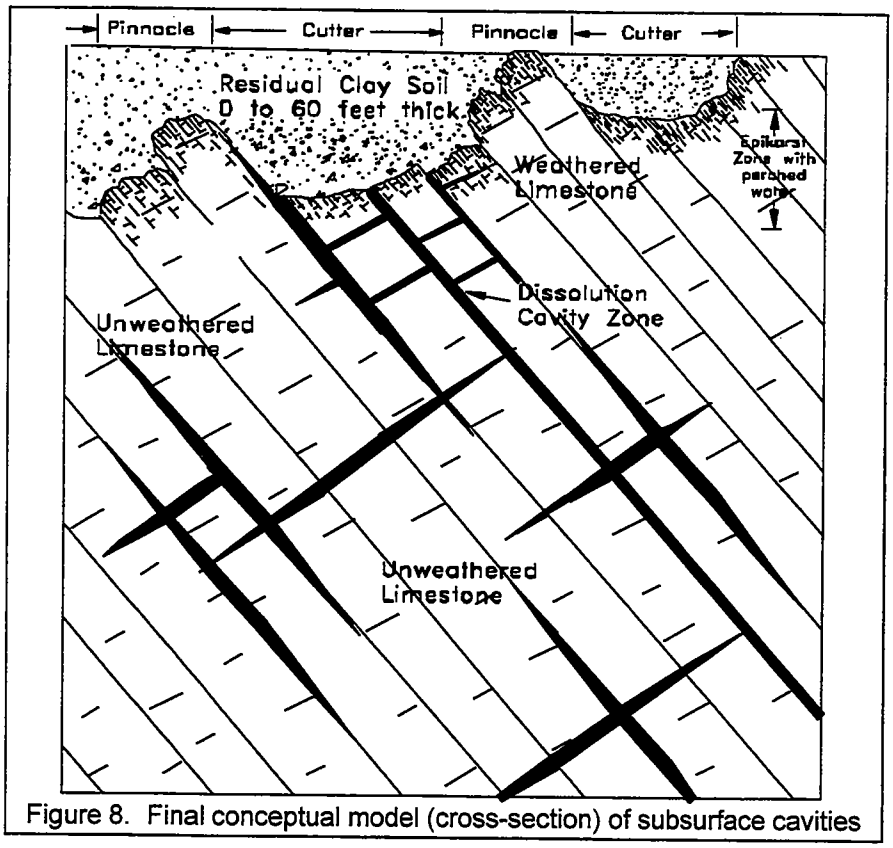

Figure 8. Final conceptual model (cross-section) of subsurface cavities

- residual clay soils and weathered bedrock (0 to 60 feet thick);
- a cutter and pinnacle top of rock profile;
- a zone of perched water at the soil bedrock interface (epikarst zone); and
- competent, unweathered bedrock with cavity zones (dissolution-enlarged joints and bedding planes) extending to a depth of about 260 feet. Individual void size may range over several orders of magnitude.

The cause of the subsidence and collapse observed at the surface is due to infiltration of surface waters which move unconsolidated material into well developed dissolution-enlarged joints and bedding planes within the bedrock (Figure 3b). At the same time, rapid movement of unconsolidated materials (including small chunks of rock) may be eroding and enlarging the cavity zone.

A valid conceptual model must be developed by integrating appropriate data, accurate data and adequate data. Statistical methods or sophisticated

computer imaging cannot substitute for invalid or missing data. All of the pieces of the puzzle must fit together and key hypothesis must be confirmed by multiple observations or measurements. As always, the results must be checked by an experienced professional(s) to assure the model is consistent with reasonable geologic and hydrologic conditions.

THE PREDICTION

There are three components which promote subsidence and collapse to take place at this site: a cutter zone, the presence of a deeper underlying interconnecting cavity zone within the rock, and infiltration of surface water via the cutter to aid the movement of unconsolidated materials into the cavity zone. The most likely location for subsidence and collapse is where cutters (water recharge areas correlate with the karst related gravity anomalies (Figure 5). The greatest probability of underlying bedrock cavities occurs at the locations of cavity zones identified by gravity anomalies shown in Figure 6.

A few months after completion of the karst investigation, a major sinkhole collapse (100 feet in diameter and 60 feet deep, Figure 7) occurred 100 yards south of the western gravity anomaly which was identified as being highly susceptible to sinkholes. Other sinkholes have subsequently occurred north and south of I-70 along the southwest to northeast axis of the western gravity anomaly (Figure 7). More recently, sinkhole activity has now been occurring over broader area (Herbert, 1998).

The location and characterization of cavity zones and incipient sinkholes is relatively straight-forward, the prediction of their time of collapse is not. We can identify certain conditions which act as precursors and are the initiating cause of subsidence and collapse (Benson and LaFountain, 1984), but unless a site is instrumented or temporally monitored, we cannot reliably predict their time of collapse.

Follow Up Studies and Remediation

An initial study of groundwater levels is being carried out by Genstar Quarry using existing wells and piezometers around the quarry (Herbert, 1998). The Maryland State Highway Administration (SHA) has installed and is monitoring piezometers along the highway. The cone of influence of the dewatering of the quarry and the interaction of groundwater with the karst setting will be better defined by these measurements.

Seven dye trace studies have been initiated and baseline sampling continue on a monthly basis (Herbert, 1998).

Subsequent borings along the highway confirmed the location of the gravity anomalies and deeper cutters (Figures 6 and 7).

Both SHA and the city of Frederick continue grouting programs to remediate conditions.

SUMMARY

This study defined areas along I-70 that have a high potential for future collapse. The analysis of existing data, field observations, and geophysical measurements enabled these areas to be defined with a high degree of accuracy and confidence.

For decades, site characterization of karst has been thought to be beyond our technological capabilities. In fact, during the 1960 and 1970's, karst and sinkhole activity were commonly treated as an "act of God", impossible to characterize and predict. Many practitioners continue to believe that one cannot provide an accurate characterization of karst; we disagree adamantly with that position. While most site characterization of karst are relatively incomplete and inaccurate (commonly 10 to 30%), if we focus upon obtaining accurate, adequate and appropriate data and integrate it into a conceptual model, experienced karst professionals can achieve levels of completeness and accuracy of 75% or more.

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Ground Penetrating Radar: An Effective Tool for Locating Dry Caves Along a Portion of State Route 69 Near Mayer, Yavapai County, Arizona.

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Abstract

Widening and realignment of State Route 69 traverses an area of Tertiary-age travertine bedrock near Mayer, Arizona (Figure 1). During the design phase of the project, subsurface exploration encountered small voids within the right-of-way. A moderate-size cave structure in the area was mapped by local speleologists. Arizona Department of Transportation (ADOT) was concerned that cave structures of unknown size might be found within a few feet of the new roadway subgrade. As a result, highway construction specifications contained special provisions requiring geophysical surveys to identify cave structures that could adversely affect the roadway and expose the traveling public to possible subgrade failure hazards. ADOT's concern was realized during construction when a D-9 Caterpillar tractor broke through a cave roof and dropped about six feet into the void.

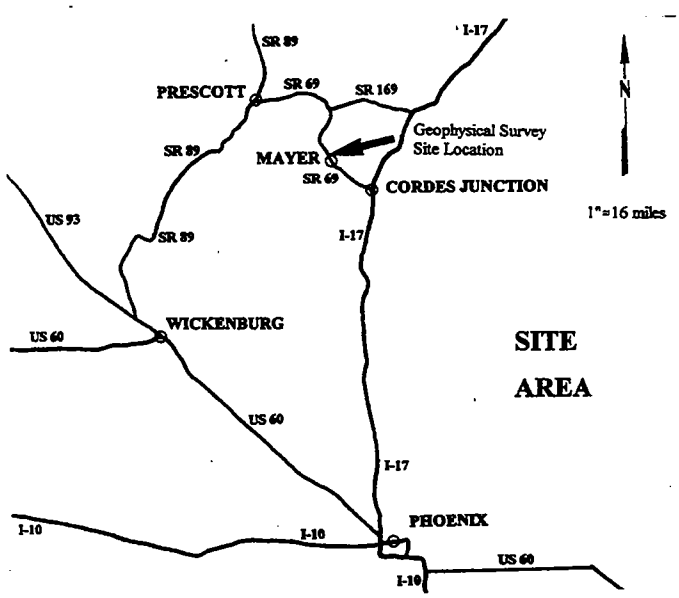


Figure 1 Project area location map

Geological Consultants Inc. of Phoenix, Arizona was contracted by C. S. McCrossan, the general contractor, to conduct the geophysical surveys of the cave-affected alignment. The investigation area included both the north and south bound lanes, shoulders, and median along an 800-foot section of the alignment. Ground penetrating radar (GPR) was the geophysical technique of choice because it has the highest spatial resolving capability of any geophysical method, and the data can be quickly collected and interpreted. More than 6,800 GPR measurement were collected using a Sensors & Software pulseEKKO model 100 system with a 100-MHz antenna.

One-hundred-thirty cave-type anomalies were identified from the raw GPR data along the alignment section. More than 460 anomalies were successfully interpreted from the computer processed GPR data within an interval that extended from the roadway subgrade to at least 40 feet below subgrade elevation. Air-track test drilling of suspect anomalies was used to confirm the anomalies and to refine the cave structure dimensions ranging from 2 feet in diameter to more than 40 feet in diameter. The success ratio of confirmed voids to anomalies was about 70 percent.

Recommendations were provided to ADOT, and the contractor, to remediate the cave-affected highway section. The selected remediation method was to eliminate major voids within 20 feet of the roadway subgrade using a process of (1) removing (or collapsing) the cave roof by blasting and backfilling the opening with structural fill or (2) grouting the voids full. The collapse technique was employed where large (greater than 10 feet in diameter) cave were found. Grout was use to fill smaller voids. Although deeper cave structures were present, only the upper 20 feet was remediated to effectively bridge the deeper voids while providing long-term stability of State Route 69. Survey monuments were established for monitoring roadway performance and potential subgrade settlement.

The application of an indirect geophysical method, such as ground penetration radar surveys, and a thorough understanding of the local geological conditions, provides for a rapid, economical, focused subsurface exploration program. This type of program can be used successfully to find caves (or other underground structures) with a highway right-of-way. The data can then be used to develop cost effective mitigation measures to remediate potential hazards during project construction.

Background

Location

The cave-affected section of State Route 69 is located in the vicinity of Milepost 271 about one mile north of Mayer, Arizona, or about 75 mile north of Phoenix, Arizona. The study area is situated in the central part of the Arizona Transition Zone, a structurally distinct northwest-southeast trending zone that separates the Colorado Plateau Province to the north and the Basin and Range Physiographic Province to the south. The Transition Zone contains many of the topographic attributes of the bounding provinces: deeply incised canyons, high peaks, tablelands and mesas, broad valleys, and low mountains (Pierce, 1985).

Highway Corridor

Several years prior to the start of design studies to widen SR 69 in the project area, ADOT was aware of unusual runoff conditions in the drainage ditch adjacent to the south bound lane within a cut section of the existing highway. Following intense rainfall events, runoff flowed toward the center of the cut section and drained into a small opening at the toe of the cut. Subsequent research by the ADOT Geotechnical Section revealed that local cavers (or speleologists) knew of an accessible cave in the area. Supported by ADOT, two Prescott area speleologists mapped a small cave system, referred to as the "Mayer Mystery Cave" in 1993. Results of the mapping showed the cave extended to depths from the entrance ranging from 20 to 40 feet below the surface with connected rooms and corridors totaling a few hundred feet in length. The entire cave was within the existing highway right-of-way.

In the early 1990's, design studies were initiated to complete a 4-lane divided highway to replace the existing 3-lane segment of State Route 69. Site-specific alignment investigations were initiated along the Big Bug Creek section near Mayer, Arizona in 1993. The preliminary geotechnical investigation for the SR 69 realignment was completed in mid-1993. Small voids and loss of drilling fluid is documented in the investigation drill hole logs. ADOT attempted to characterize suspect cave features in the new alignment area in 1995 using GPR, but penetration and resolution was extremely limited by the thickness of clayey overburden soils above proposed subgrade elevation. ADOT deferred further investigations until construction of the cave-affected alignment section had been excavated to remove overburden to near grade.

During excavation of the south bound lanes and shoulder in 1996, a D-9 Caterpillar tractor was using is single-shank ripper to breakout strongly cemented breccia and travertine limestone bedrock that had been previously fragmented by blasting. Without warning, the D-9 broke through the ground surface and dropped about 6 feet into an open void (Figure 2).

During construction, Geological Consultants Inc. was contracted to conduct ground penetrating radar (GPR) geophysical survey along the new alignment section of SR 69, north of Big Bug Bridge, near Mayer, Arizona. Specifically, the surveys were located along the southbound lanes between stations 3698+00 and 3706+00 and along the northbound lanes between stations 3697+50 to 3706+00. The median and shoulder areas were also surveyed. The geophysical data and findings were used to identify voids or caves in close proximity (within 30 feet) to the roadway grade. Large voids identified within this zone exhibit the potential to collapse under traffic-induced loads.

Highway Geology

Overview

The project area is located in the central part of the Arizona Transition Zone containing deeply incised canyons, high peaks, tablelands and mesas, broad valleys, and low mountains (Pierce, 1985). These features have been formed as

a result of several episodes of crustal deformation thus creating complex geologic structures including fault bounded basins and uplifted mountain massifs. The rocks of the Transition Zone consist of Pre-Cambrian age crystalline granitic and metamorphic rocks, Paleozoic through Cenozoic age sedimentary units, and substantial accumulations of Cenozoic volcanic rocks (Geological Consultants, 1992).

Following a series of complex mid-Tertiary episodes of volcanism and deformation, the basins of central Arizona (including the Mayer, Arizona area) were loci for the accumulation of extensive deposits of clastic and basaltic rocks. Thick deposits of lacustrine limestone and spring-related travertine carbonates (limestones) were deposited during extended periods of interior drainage (Thompson, 1983).

Cave-Affected Alignment Geology

The geophysical survey was conducted in an area locally referred to as the "onyx pit". The 'onyx' is a Tertiary age banded travertine that outcrops north of Mayer in the vicinity of milepost 270, along State Route 69. The travertine is interbedded with marl, calcareous sandstone, and breccia. Significant voids are found throughout the travertine unit. The travertine is locally underlain by a very hard, well cemented breccia Prior to excavation of the roadway area, drill hole logs (ADOT, 1994 and AGRA, 1993) reported that the travertine was overlain by the residual soil and poorly to moderately cemented breccia.

Figure 2 Cave area collapsed by D-9 dozer.

Travertine: The travertine is distinctively banded with alternating layers of white to cream color, and red brown to brown and tan colored layers. Within the red color-banded zones, a lime green calcite is also found. The red colored beds are believed to be caused by oxidized hematite taken into calcite crystal structure during their formation. The white and tan bands of the travertine range in thickness from less than one-half inch to several inches. The unit is composed of the spring-deposited mineral calcite with grain size ranging from fine to coarse crystalline. The travertine ranges from soft to hard. The travertine contains numerous voids, ranging in size from vugs that are less than an inch across, to caves greater than ten feet in diameter.

Where exposed, the travertine banding is horizontal to gently folded, and locally dips forming relatively small anticlines, synclines, and kink folds. The travertine is massive, poorly jointed, and moderately thick. The unit is known to be at least 50 feet thick based on test drilling data, but may be as much as 100 feet thick. The unit outcrops locally in Mayer, covering less than about ¼-square mile. Although the unit is not located on any of the available geologic maps due to the scale of the maps, it is believed to be a member of the Tertiary-aged Hickey Formation, which is also mapped in the vicinity of the study area (Anderson and Blacet, 1972 a&b).

Breccia: The unit is a light gray to dark gray, soft and massive matrix supported breccia. The breccia is poorly to moderately cemented with calcite. It is composed primarily of subangular to angular clasts of metamorphic rock fragments cemented in a matrix of angular sand. The rock fragments range from ½-inch to 12 inches in diameter. The breccia is generally not well exposed in outcrops within and adjacent to the study ar Composed of gravelly to bouldery clasts of igneous and metamorphic rock in a sandy matrix. The alluvial fane deposits were observed to be only weakly to moderately caliche cemented at the surface. Strongly caliche cemented materials could be encountered at depth, particularly near the bedrock/alluvium boundary. ea. The breccia was encountered above and below the travertine in exploratory drill holes along portions of the alignment (AGRA, 1993). The breccia was also exposed in debris created by portions of the excavation where blasting was done. No void features appeared to have been developed in the breccia.

Karst Features Identified Along Roadway Alignment

Throughout the project and adjacent areas that are underlain by travertine, only one surface expression of a karst feature was known to exist prior to the highway realignment construction. This surface expression of a karst feature was a small cave entrance, located along the shoulder of the original southbound lanes of SR 69 at about station 3702+00. The cave opening was large enough for three spelunkers to enter the cave and to survey its passageways. The approximate location of the "Mayer Mystery Cave" is depicted on Figure 1.

Exploration drilling conducted by as part of the highway alignment design investigation encountered reportedly small voids. ADOT also conducted drill hole exploration during October 1994. The purpose of the ADOT program was an attempt to determine the extent of voids in the vicinity of the "Mayer Mystery Cave". Logs of both auger and core borings reported voids with open intervals ranging from at least one foot to eight feet. The top of the voids range from about 3 to 40 feet below existing grade.

Field Investigation

Geophysical surveys, along the SR 69 realignment section, were conducted in two phases. Phase I concentrated on the southbound lanes segments, including the southbound shoulder area and a portion of the median. Phase II concentrated on the northbound lanes segments including the remainder of the median and the northbound lanes shoulder. The field work for Phase I and II, was performed from October 29, 1996 to November 1, 1996 and from June 26, 1997 through July 3, 1997, respectively.

Geological/Geophysical Model

During a GPR survey, repetitive, short duration (a few nanoseconds) electromagnetic pulses are radiated into the earth from a broad-bandwidth antenna placed on the ground surface. Pulses are partially reflected back to the antenna by dielectric discontinuities in the subsurface. Dielectric contrasts can be caused by changes in moisture content, stratigraphy, man-made objects, or voids within a limestone.

Strata changes within the limestones may produce semi-continuous radar reflections that change over tens to hundreds of feet on the data

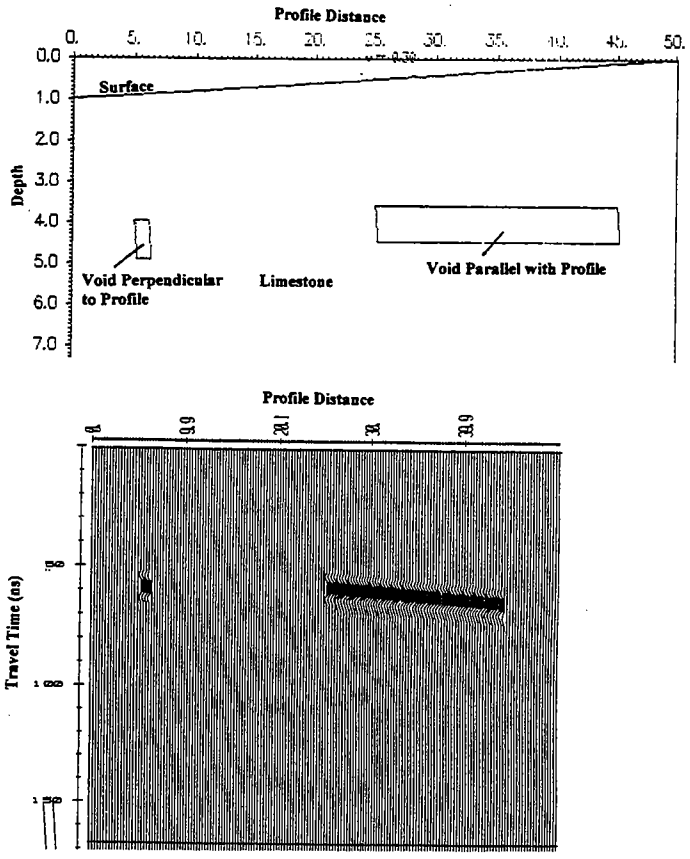


Figure 3 Theoretical radar data of subsurface void.

caused by a void will change according to the void's geometry and the orientation of the data profile over the void. Figure 3 shows the theoretical anomaly associated with a narrow, long axis void (Sensors and Software, 1995). The left of the plot shows the characteristic hyperbolic shape anomaly over a void with its long dimension perpendicular to the data profile and is also valid for a semi-spherical anomaly with no long axis. The right side of the plot shows a long anomaly with a large amplitude top and bottom representing the void/limestone contact when the profile is subparallel with the void's long axis.

The GPR method is the most appropriate surface geophysical method to use to locate voids at this site. The method has the highest subsurface object resolution of any method because very high frequency (narrower pulse width) signals are used to probe the subsurface. The major limitation of radar is its ability to penetrate conductive soils, such as saturated alluvium and clays. These conditions do not exist at the site. The data were collected either directly on the dry limestone or on a thin layer of gravelly fill on top of the dry limestone.

GPR Survey Design

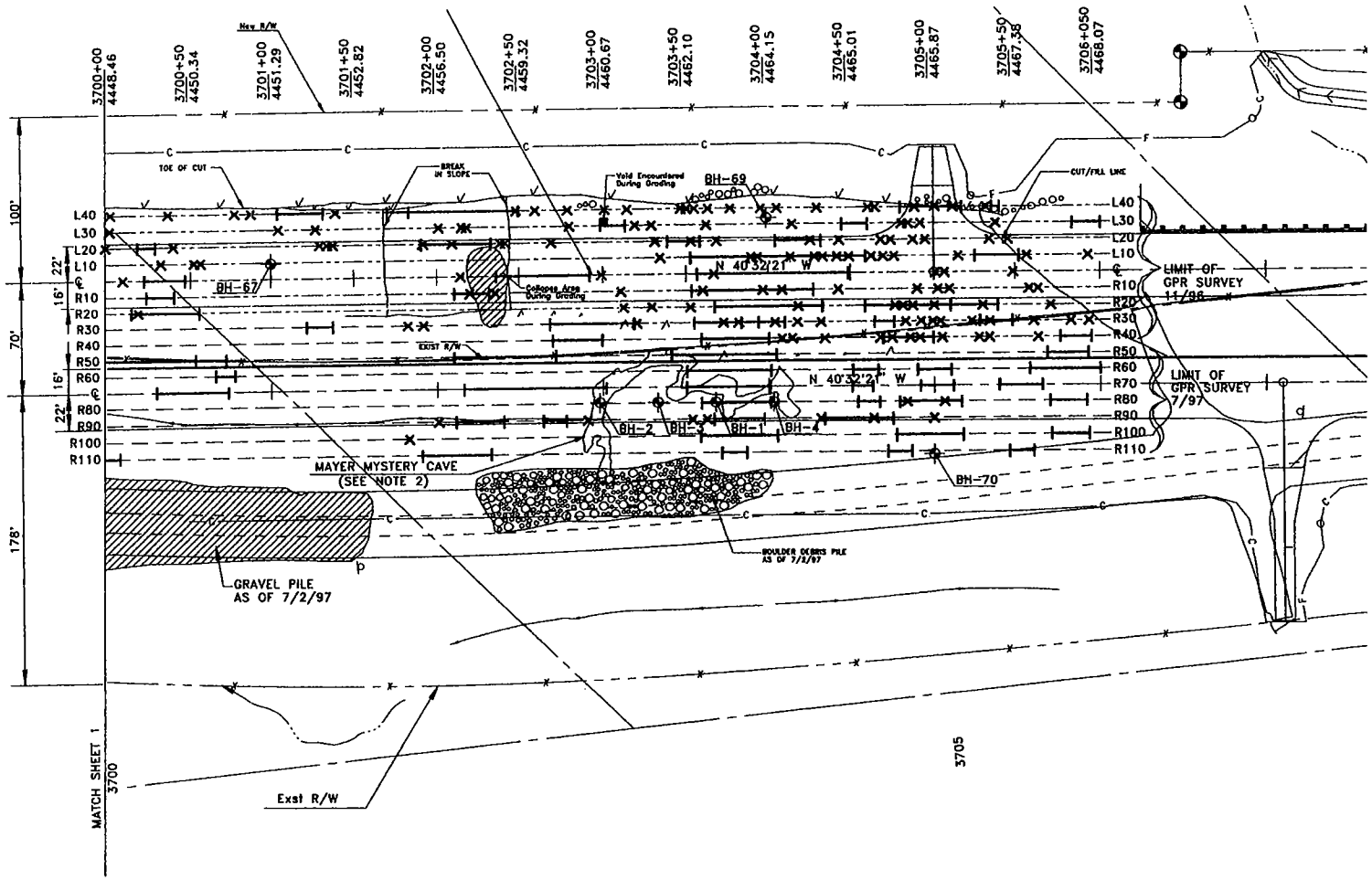
The radar survey was designed to detect voids within the upper 30 feet that could collapse and expose the traveling public to potential hazards along the roadway. The smallest void of interest was estimated to be a cube two feet on each side. Collapse of smaller dimension voids would probably not seriously effect the roadway. The GPR measurement parameters were chosen to detect voids with a vertical dimension larger than two feet (Figure 4).

Figure 4 GPR antenna/receiver collecting data along profile line

The measurement profile geometry was also designed to detect voids with spacial dimensions larger than two feet. A grid of parallel profiles were constructed along the roadway axis from Station 3698+00 to 3706+00 (Figure 5). The profiles extended from the south shoulder of the southbound lanes about 150 feet to the northern shoulder of the northbound lanes. Adjacent profiles were 10 feet apart. Sixteen parallel transects were continuously surveyed producing a total of almost 14,000 lineal feet of GPR profiles. Data were collected with the 100 MHZ antenna that has a beam width at depth about four feet on each side of the profile. Therefore, this antenna should be able to detect a void even if it has a four feet perpendicular offset from the profile. Each profile probed a data swath eight feet wide (four feet on each side of the centerline) resulting in a 2-foot wide zone between two adjacent profiles that was covered by the surveys. Data were collected at two foot intervals along each profile which should be adequate to detect a two foot cube.

GPR Survey Results

The processed sections for Profiles L40, L30, L20, L10, southbound centerline, and R 10 through R110 were analyzed individually. Large-amplitude anomalies appearing in the lower part (longer travel times) of the section were identified as possibly being air-wave reflections from nearby surface features. When possible these correlations were made and those anomalies ignored as possibly representing voids. Candidate anomalies were highlighted along the bottom of the sections (Figure 6) On the southbound lanes, 209 possible void anomalies were interpreted and 257 anomalies on the northbound lanes.



SUMMARY NOTES:

1. Drawing Modifed From Plan Sheet STA 3700+00 to 3705+00; Big Bug Bridge #1 to #4; SR 69; Sheet 14 of 15; Prepared by Sverdrup Corporation 9/94.
2. Mayer Mystery Cave, Taken From Map Location Prepared By Bill Rodgers, Andy Yednak & Bill Lowell-Britt, 12/21/93.

EXPLANATION:

- Blast Hole Location (Data Provided by C.S. McCrossan Inc. 11/96)
- GPR Survey Line (Southbound Lane)
- GPR Survey Line (North Bound Lane Survey)
- Bore Hole Location (BH-1 to BH-6 Data Provided by A.D.O.T.)
- Bore Hole Location (BH-67, BH-69 & BH-70 Data Provided by AGRA Earth & Environmental Group)
- GPR Linear Anomaly
- GPR Anomaly

Figure 5 GPR Profile line location plan showing approximate site of "Mayer Mystery Cave" and GPR anomalies.

Finally, all sections were compared and the candidate anomalies screened to determine the ones most likely to represent voids. Interpreted voids' location, depth and orientation are depicted on the anomaly plan map (Figure 5)and cross sections (Figure 7). Many individual anomalies identified on adjacent profiles were grouped together to form large dimension anomalies or zones containing many small dimension anomalies that could represent interconnected voids

Void Verification and Remediation

Southbound Lanes

Anomaly Verification Program: The anomaly verification program involved the test drilling of selected areas based on the results of the GPR survey conducted along the future southbound lanes of the SR 69 realignment. The GPR survey located 51 target areas where subsurface voids were interpreted to be present. The target areas were drilled using an air track drill (Figures 8). If a suspect anomaly was confirmed as a void additional test holes were drilled around the initial drill hole on five foot centers to determine the void limit. All of the test holes (for the southbound lane) were drilled to a depth of 46' below grade. A total of 191 test holes were drilled.

During the drilling operation, voids were encountered at most of the identified anomaly locations. When voids were intersected, the drilling response was dramatic with the loss of air circulation and drop of the drill rod. The anomaly site was drilled first and, if the voids' presence was confirmed, additional bores were drill in a pattern away from the initial bore to 'chase' the void in an attempt to determine it's lateral extent. The depth to the top of the voids ranged from as shallow as four (4) feet to as deep as 40 feet below existing grade. The open intervals (top to bottom) of the voids encountered by the drilling are estimated to ranged from one-foot to at least 26 feet with lateral dimensions estimated to range from less than 5 feet to as great as 60 feet. The areas that appeared to exhibit the most critical void features are depicted on Figure 9 which also depict the GPR survey lines, GPR anomalies identified from the process geophysical data, drill holes that encountered voids, and drill holes that did not encounter voids.

Void Remediation Program: The void remediation program was conducted during January and February, 1997 by the contractor to prepare a stable subgrade. The program involved the southbound lanes of SR 69 between station 3698+00 and station 3706+00, including portions of the median and shoulder. As directed by ADOT, void remediation was limited to the depth of about 20 feet below subgrade elevations. Although voids were encountered below 20 feet, it was ADOT's opinion that remediation of the upper 20 feet would effectively bridge the lower voids. The void remediation program activities were monitored and documented by Geological Consultants Inc.

This phase of work included the drilling out (defining) of void areas which were initially characterized during the anomaly verification test drilling program (Figure 10). Once a void area was defined, it was then selected for either blasting or grouting at the direction of ADOT. Three void areas were shot during this period of work to remove the void roof. Areas opened by blasting were further mitigated by removing all of the blast debris and backfilling the open void with a controlled fill. A geogrid type filter fabric was installed prior to backfilling to prevent piping of fine grain sediments into deeper voids. As directed by ADOT, some of the smaller void areas were not shot. Void areas not shot were filled to the fullest possible level using a sand-cement grout.

Northbound Lanes

Anomaly Verification Program: Because of the high degree of success with the cave anomaly/verification program experienced in the south bound lanes phase of the project, the anomaly verification test drilling program was not conducted along the northbound lanes. The locations of the GPR Survey anomalies were used as the basis to proceed directly with the void remediation program.

Void Remediation Program: The void remediation program was conducted by the contractor during July and August, 1997. The drilling program to delineate void areas was conducted between station 3697+50 and station 3706+00, including portions of the median and shoulder. Interpretation of the GPR data identified 173 anomalies within the north bound lanes suspected to be voids from the surface to a depth of about 40 feet. Anomalies within 20 feet of the surface were drilled, and the location and depth to voids encountered, as well as the size of the void, were

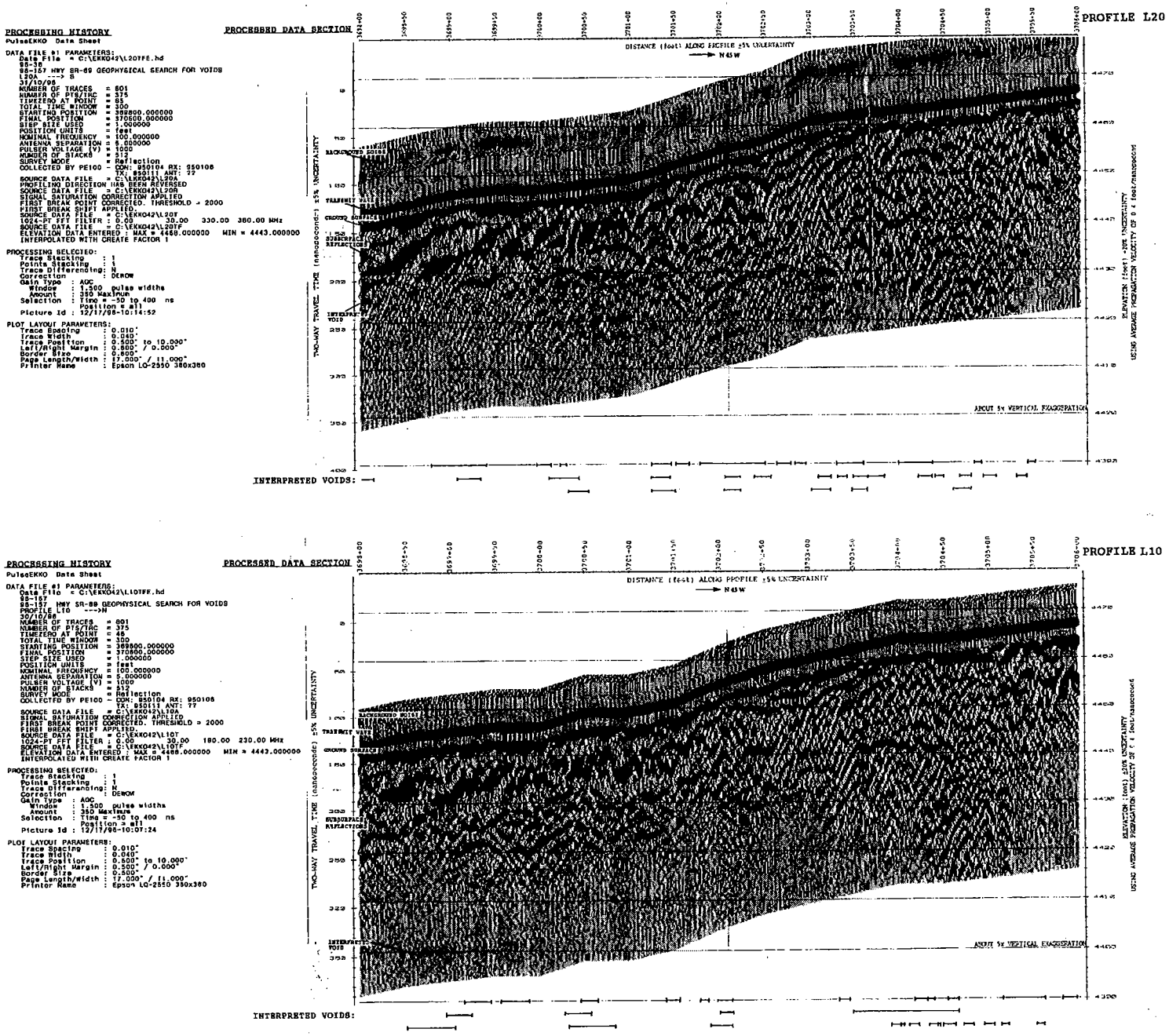


Figure 6 Typical GPR data profiles collected along cave-affected alignment section

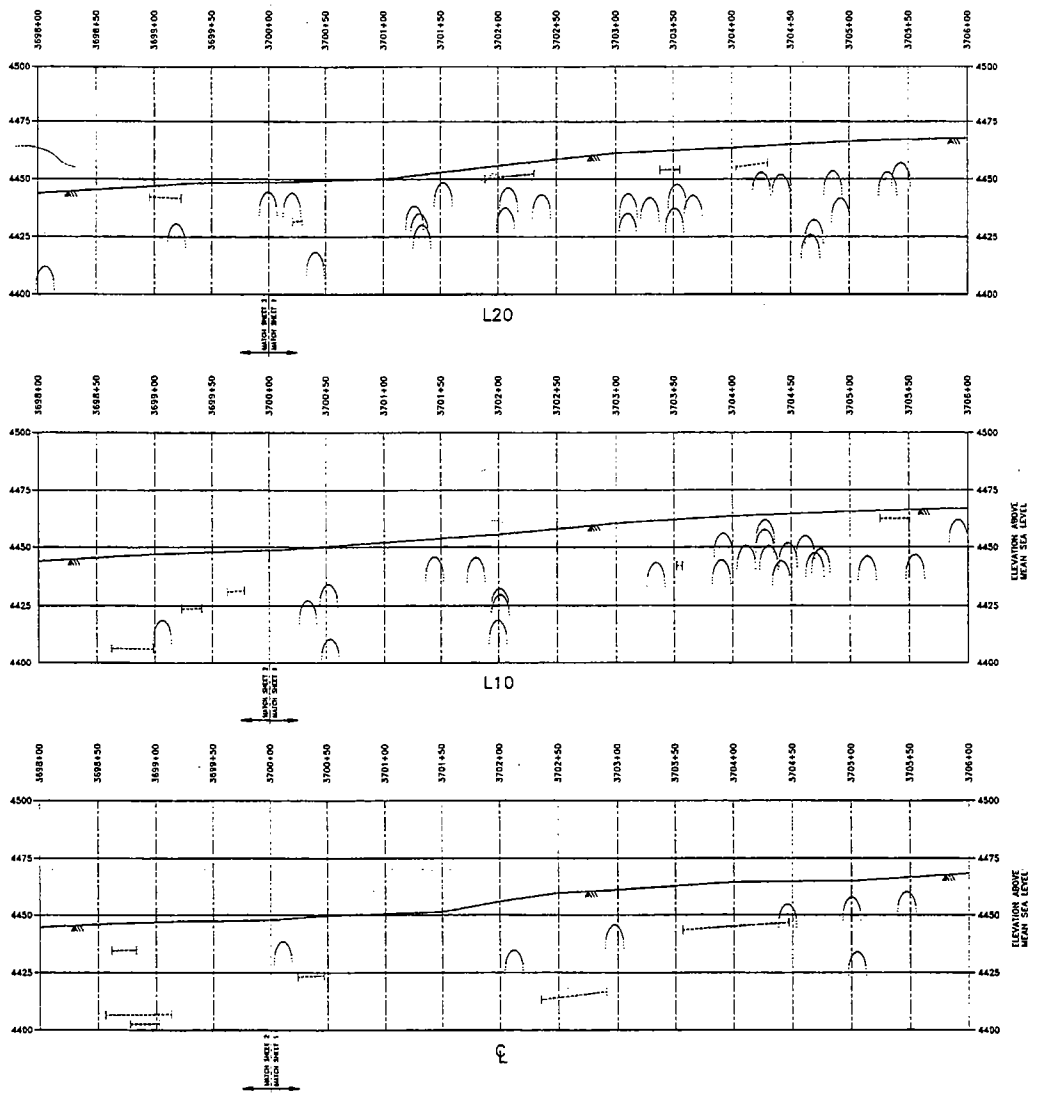


Figure 7 Profiles along cave-affected section of alignment depicting interpreted GPR anomalies. Hyperbola indicates individual anomaly; bracketed, dashed line indicates linear anomaly.

documented in the field by Geological Consultants Inc. This drilling process continued until void areas were defined and surrounded by drill holes in which no voids were encountered. Once a void area had been defined, a remediation method of either blasting and backfill or grouting of the void was implemented.

Twenty-four of the GPR anomalies were confirmed as voids during drilling within the upper 20 feet of the section. Voids encountered during the drilling of the geophysical anomalies in most cases were interconnected with adjacent voids. Nine large void areas were defined by a combination of drilling GPR anomaly sites and chasing the voids to their limit. Three of the caves were remediated by blasting and backfilling with the remainder closed by grouting.

SUMMARY

GPR surveys were conducted along a portion of the SR 69 highway realignment between stations 3698+00 to 3706+00 where known karst features had been previously documented and which were known to be in very close proximity to proposed roadway grades. The survey included both the northbound and southbound lanes, their respective shoulders and the median between the lanes.

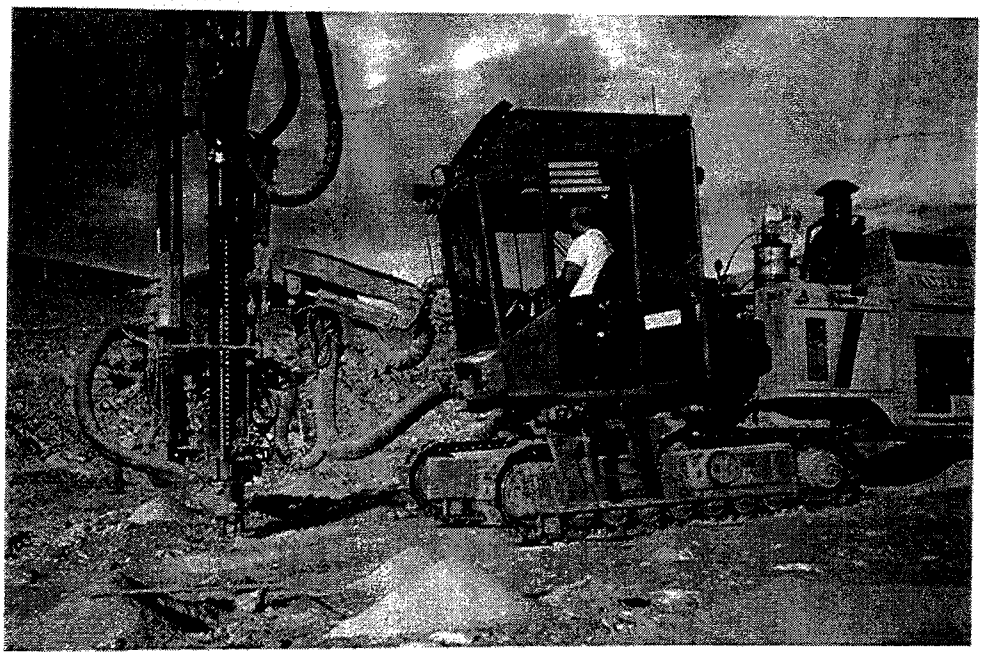


Figure 8 Air track drill used for anomaly verification along south bound lanes segment.

The GPR survey detected subsurface anomalies that were interpreted as voids and many were confirmed by subsequent drilling. This survey proved a rapid method to statistically characterize the travertine (onyx) formation in which the voids were found and focus remediation to locations where it was deemed needed. The GPR survey collected data almost continuously along 16 profiles spaced 10 feet apart and oriented parallel to the roadway centerline. Measurement parameters were designed to resolve voids larger than a cube, two feet on a side, within the upper 30 feet. Data was extensively processed to produce sections emphasizing anomalous areas. A theoretical model was constructed for different void shapes and shows the corresponding anomalies shapes. These models were used as a guide to identify anomalies within the data sections that could represent voids.

Numerous irregularly-shaped voids of small (± 2 feet in diameter) to large (greater than 20 feet in diameter) were interpreted in a zone that extends from 3 to 40 feet below roadway grades. Remediation efforts were concentrated within the upper 20 feet of the section in an effort to form a reasonably stable platform to "bridge" the known lower voids. Remediation in the upper 20 feet of the roadway section included the opening of large near surface voids by blasting followed by their subsequent backfill with an engineered fill. Smaller void areas were filled with a standard grout mix.

Three monitoring points have been established adjacent to the void-affected, remediated roadway. These points can be used by ADOT to monitor the performance of the highway and its response to void remediation.

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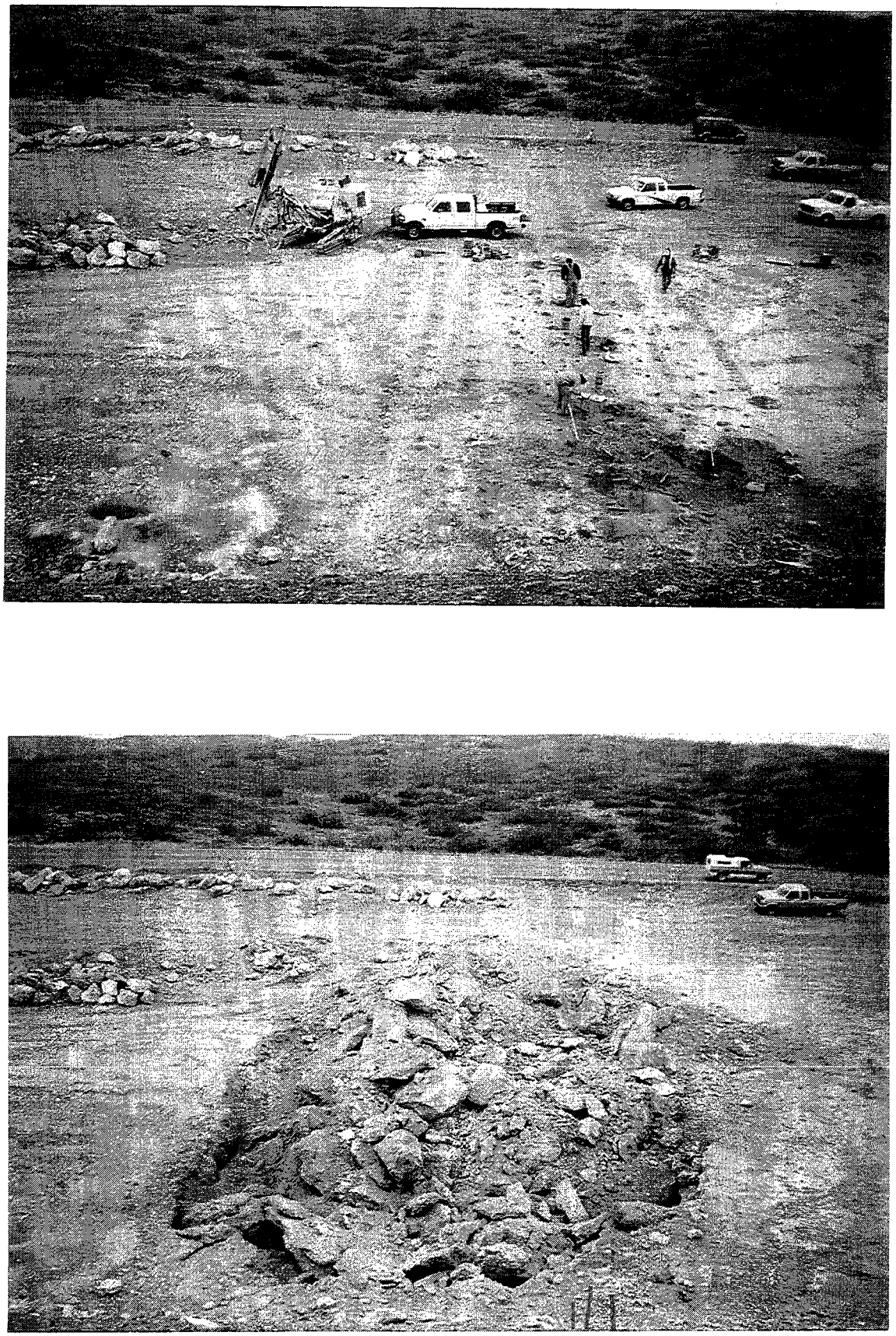

Figure 10.1 (Top) Shot hole pattern for shot designed to collapse roof of near-surface cave structure; (Bottom) Cave roof collapsed by blasting.

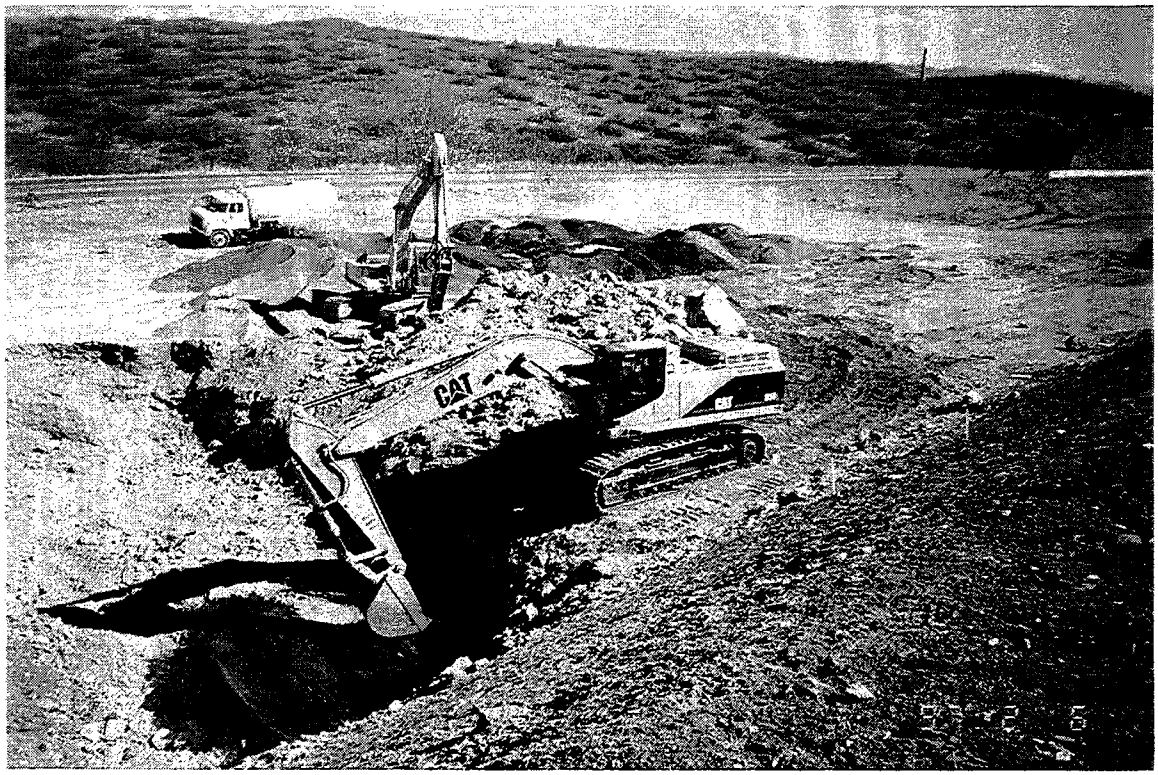

Figure 10.2 (c) Final remediation of collapsed cave structure with select backfill.

Design Considerations for Bridge Foundations in Karst Terrain

By Bashar S. Qubain,[1] Eric J. Seksinsky,[2] and Jianchao Li[3]

ABSTRACT: A complicated karst setting along a major highway widening project in southeastern Pennsylvania presented considerable foundation design challenges for three new bridges. The project, situated in Montgomery County on the boundary between Triassic and Cambrian age rocks, includes three carbonate formations. Significant voids in the upper carbonate rock and other subsurface complexities were revealed by a field exploration program consisting of many deep rock cores together with detailed geologic mapping. Three distinct categories of karst features were identified: (1) deep, sloping rock with voids and boulders; (2) severely sloping rock without voids; and (3) cavernous rock. The foundation considerations for each category together with the adopted designs are discussed herein.

INTRODUCTION

A widening project for the Pennsylvania Turnpike (I-276) in Montgomery County presented many design challenges related to karst terrain. This paper illustrates these challenges and provides a detailed description of how they were addressed in the design stage. Three new bridges and a total of 7,700 ft of roadway are involved. Two more lanes, wider shoulders, and a median (about 50 ft total) are being added to the existing four lanes between Flint Hill Road and Ridge Pike. From west to east the bridges are: six spans, 1224 ft; two spans, 150 ft; and simple span, 80 ft. The specific roadway section containing the three bridges is about 2,600-ft long. The widening is completely to the south in the area of the three bridges.

The six-span bridge is designed as an entirely new, independent eastbound structure in between an existing eleven-span Turnpike bridge and a seven-span Conrail bridge to the south. Its deck will be about 70 ft above the Schuylkill River. Site conditions are tight: the southern edge of the new bridge will be less than 15 ft from the Conrail bridge.

Farther east, an existing, 625-ft long, deteriorated, ten-span viaduct over the Diamond Run and Conshohocken Road will be replaced with two shorter bridges at the ends and a 40-ft high fill embankment in between. At the western end of the embankment there will be a two-span bridge while the eastern end will have a simple-span bridge. A similar existing Conrail embankment and a double-bridge arrangement parallel the highway immediately to the south.

GENERAL SITE GEOLOGY

The Norristown-Valley Forge locality has a unique transitional geologic setting. Not only does the area lie near the southeast margin of Pennsylvania's piedmont plateau (Cuff and Young 1989), but it also straddles the southern border of the Triassic basin where an eroded feather edge of Triassic age clastic strata unconformably overlie three formations of Cambrian carbonate and metamorphic rocks.

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The Cambrian rocks have been altered, fractured, and deformed by several mountain building and rifting events in their geologic history. The upper carbonate extreme of the same rocks has also been subjected to as many as four different periods of large-scale erosion since their formation plus intensified local downcutting along waterways like the Schuylkill River and Diamond Run (Wise 1996). The result is a complicated karst setting characterized by significant rock voids, boulders, sloping rock, and numerous faults.

The uppermost Cambrian rock belongs to the Ledger dolomite formation. This rock type was in general the most problematic, especially at the western and eastern ends of the project where significant rock voids presented foundation stability concerns for the Schuylkill River bridge and the simple-span bridge. Those two locations were found to have a buff, pure, coarsely crystalline, thickly bedded form of the Ledger which is highly susceptible to solution activity and cavern formation. In contrast, the Ledger strata toward the middle of the project, i.e., the location of the two-span bridge, are generally argillaceous and therefore, much less soluble.

A second, older Cambrian carbonate formation, the Vintage dolomite, underlies the Ledger dolomite everywhere across the extent of the project. Perhaps due to its minor thickness in this area (40 ft or less), published geologic maps (Berg and Dodge 1981) do not even identify the Vintage in Montgomery County, but seem to lump it together with the Ledger. For this project, however, it has been advantageous to distinguish it from the Ledger due to its more favorable engineering properties. The Vintage dolomite is highly phyllitic and intensely laminated. Strongly altered by metamorphism, this rock has extremely closely spaced, contorted phyllite beds which give the rock a unique banded or gneissic appearance (Cleaves 1952). Its color is light and dark gray.

The Vintage dolomite at the site makes a gradual transition with depth into the lowest formation in the stratigraphic sequence—the Harpers phyllite. The Harpers rock has a banded fabric similar to the Vintage, but is light gray and dark greenish gray in color and lacks carbonate content. In terms of foundation stability, the Vintage and Harpers formations at this site fall into the same category: neither is subject to rock voids, but boulders and sloping rock surfaces are common.

The last formation of importance is the Triassic Stockton. Clastic beds of this formation partially cover the upper Cambrian rock (i.e., the Ledger dolomite pure form) at the eastern end of the widening project. Although highly faulted and fractured, the Stockton rock dips mildly to the north and has a relatively flat surface. The basal Triassic strata which blanket the pure form of the Ledger at the simple-span bridge location consist of red and light gray limestone conglomerate. This rock has a low compressive strength, but is not susceptible to dissolution due to its mudstone matrix.

FIELD EXPLORATION & LABORATORY TESTING

Detailed field exploration was instrumental in developing the subsurface picture at each of the three bridges. This in turn helped optimize their foundation designs and minimize uncertainties inherent to foundation construction in carbonate rock formations. The program included 121 test borings, detailed geologic mapping, and laboratory testing on soil and rock core specimens. Information from 59 previous borings (Sprague and Henwood 1952) for the existing Schuylkill River bridge and Diamond Run viaduct was also checked and correlated to the latest drilling data.

It is important to point out that not all of the test borings were performed in a single phase. Rather, when complications such as voids or severely sloping rock were found at certain substructure

locations, additional borings were added to develop a more thorough understanding and refine foundation designs. In problem areas, the borings were also extended deep (100 to 120 ft) to check for geologic contacts where karst features and weaknesses were found at this site. Several borings did actually penetrate two separate rock formations, which is unusual for foundation explorations.

Detailed geologic mapping was employed to correlate the borehole data with surrounding geology and also reliably establish rock bedding and joint orientations, which are critical to foundation design. Railroad cuts, natural outcrops, and cut exposures at nearby quarries provided valuable information.

Finally, close examination and testing were carried out in the laboratory on selected soil and rock specimens. Testing on 34 soil samples comprised classification, unconfined compression, triaxial compression, and one-dimensional consolidation. Rock testing included unconfined compression, microscopic examination of thin sections, and unit weight measurements. A total of 45 rock breaks were conducted to represent the intact unconfined compressive strength of fresh to moderately weathered rock cores. Unit weight measurements were also taken on many rock fragments which were too small to break in unconfined compression. This provided a relative indication of the unconfined compressive strength for a given rock type and its state of weathering. To account for fracturing and field variations, the rock mass compressive strength was taken as 15% of the average laboratory strengths of the intact specimens (PennDOT 1993).

SUBSURFACE IDEALIZATION

Upon idealizing the subsurface conditions for the project, three distinct categories of karst features emerged: (1) deep, sloping rock with voids and boulders; (2) severely sloping rock without voids; and (3) cavernous rock.

Figure 1 illustrates the first case. It occurs at the west abutment of the Schuylkill River bridge where fifteen closely spaced, deep test borings were taken. In that area the overburden is 70 ft deep, contains numerous elongated boulders, and has a 15-ft zone of weak fill at a depth of 30 ft. Poor quality Ledger dolomite bedrock is present above the Vintage as a system of thin, fingerlike, partially dissolved remnants of the original formation,

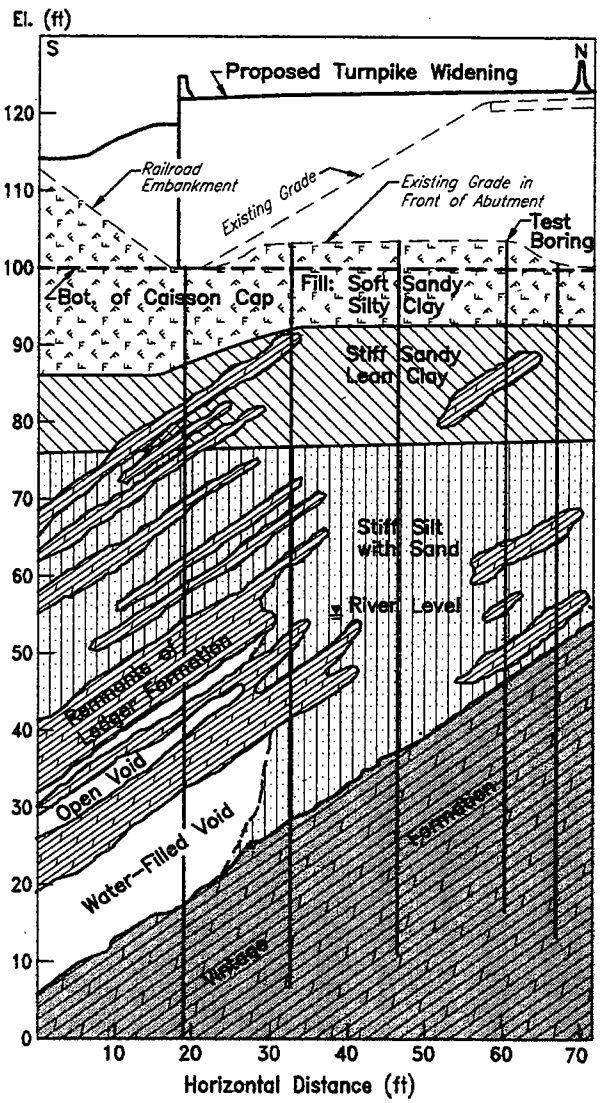

Figure 1. Deep, sloping rock with voids and boulders at the west abutment of the Schuylkill River bridge.

which was probably as much as 2,000 ft thick (Geyer and Wilshusen 1982). Continuous voids—open and clay-filled—were found between the fingers. In spite of the voids in the rock, the field exploration did not reveal any sinkholes or weaknesses in the natural soils on top of the rock.

As shown in Figure 1, the fingers and voids of the Ledger do not extend underneath the existing west abutment. Rather, they form cantilever extensions projecting at an angle of 35° from the massive Ledger which lies to the south and abruptly end at the edge of the existing roadway. Due to the presence of the railroad embankment, the southern edge of the rock fingers was not determined. The voids between the fingers average 6 ft in thickness, and have a maximum thickness of 15 ft. The clay filling in some of the voids, along with the presence of colorful mineral veins in the rock characteristic of Mesozoic hydrothermal activity, are evidence that the voids could be 200 million years old (Wise 1996)! The Vintage formation was free of voids. The rock mass has a compressive strength of 60 tsf.

A different karst feature is portrayed by Figure 2, which is a geotechnical section across the west abutment of the two-span bridge. Although this bridge is only 150 ft long, it is supported on three different rock formations due to the presence of a north-south normal fault which raised up older rock units on the west side. Argillaceous Ledger dolomite supports the new pier and east abutment, while Vintage dolomite and Harpers phyllite support the new west abutment. In all, 50 test borings were used to investigate this bridge site. As can be seen in Figure 2, the two-span bridge area has no rock voids but the top of rock slopes severely (as much as 50 °). The average rock mass compressive strength of the three different formations under this bridge is 40 tsf.

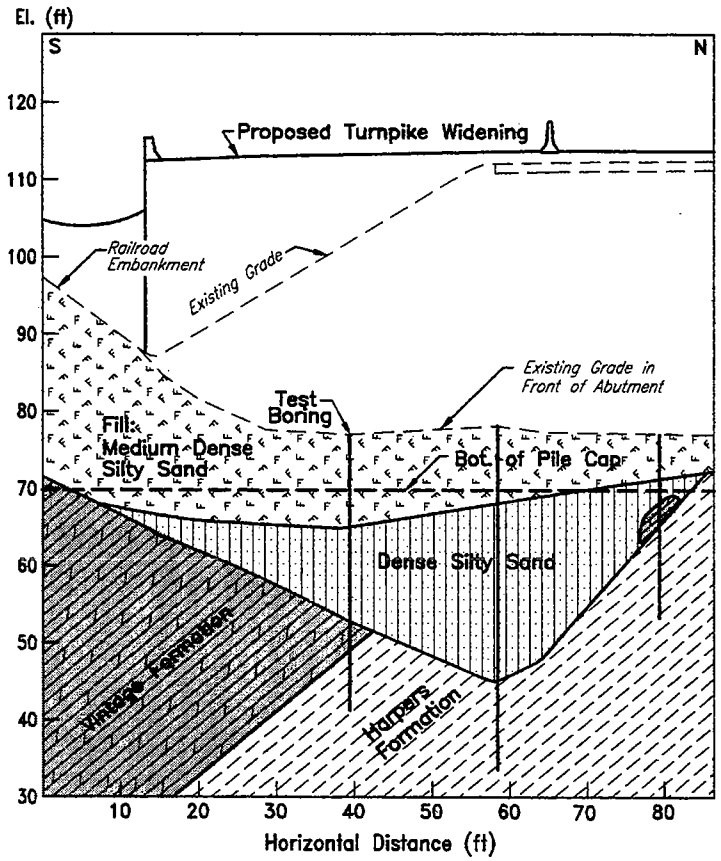

Figure 2. Severely sloping rock without voids at the west abutment of the two-span bridge.

The third major karst category occurs in the vicinity of the east abutment of the simple-span bridge, as depicted in Figure 3. A total of seven borings were used to investigate this area. The field exploration revealed a 30-ft cavern in the pure, soluble dolomite beds of the Ledger just below a thin layer of Triassic limestone conglomerate. The soil overburden above the conglomerate is 10 ft thick.

Like the Schuylkill River bridge area, the rock void in the Ledger runs east-west along the bedding, dips moderately toward the south, and terminates at the existing edge of roadway. Its southern extent is not known. In a geologic sense the cavern was judged to be comparatively recent (200,000 years old), as suggested by a lack of clay and hydrothermal infills (Wise 1996). Adding to the concerns at this site, the conglomerate is a weak rock with a rock mass compressive strength of 20 tsf. Also, the recovered core percentages during drilling were typically about 60% due to disintegration and washing out of the mudstone matrix within highly fractured zones during coring.

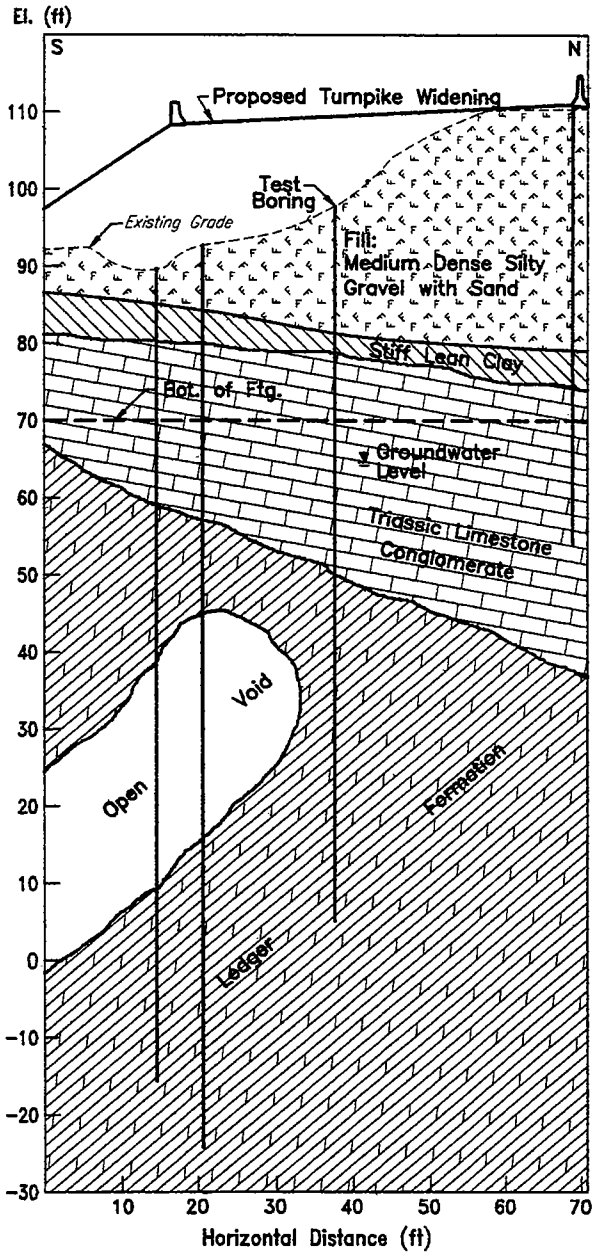

Figure 3. Cavernous rock at the east abutment of the simple-span bridge.

DESIGN CONSIDERATIONS

Deep, Sloping Rock with Voids and Boulders

To avoid the deep, sloping rock with voids and boulders at the Schuylkill River bridge location, the feasibility of supporting the new bridge on soil was first examined. (It is worth noting that one of the piers of the existing bridge is supported by a spread footing on soil and is functioning well.) Preliminary calculations using critical loading combinations yielded a maximum bearing pressure of about 3 tsf for a 15-ft wide footing. Although this value is not too high, the existing soil conditions were marginal in terms of settlement at the west abutment location. Thus, several soil improvement techniques to minimize potential settlements were evaluated. Compaction grouting turned out to be an appropriate ground modification technique although there were uncertainties associated with its cost. Deep dynamic compaction was ruled out due to tight site conditions and the danger of damaging the existing substructures. Also, the on-site soils are too clayey for vibro-compaction to be effective. Considering the cost uncertainties associated with grouting, spread footings on soil were not pursued.

The design effort was then directed toward deep foundations. Driven piles were ruled out due to the following reasons even though as-built drawings (1952) showed the existing west abutment to be supported on concrete-filled, steel Monotube piles. First, upon close review of the old drawings, it was discovered that the piles were not installed as designed; the original design called for them to be end-bearing on rock, but they are actually floating in clay or bearing on boulders 10 to 20 ft above the rock. Second, there is a 2-in. wide separation between the existing abutment and wing wall, indicating differential settlement. Third and most important, the new bridge location has the added complication of rock fingers and voids which are not present under the existing structure (see Figure 1).

Unlike driven piles, drilled piles or caissons would make sound foundation systems for the conditions at hand. Their common disadvantage is that they are expensive, especially when dealing with boulders, poor rock and voids. Of these two systems, caissons were preferred because of their larger diameter which allows a visual inspection of the bearing level. The effectiveness of caissons for highway projects in sinkhole prone areas is demonstrated by Brown (1990).

In an effort to avoid the difficulties of advancing caissons through the rock fingers and voids, compaction grouting was considered to fill the voids so that the caissons could be supported directly in the Ledger dolomite rather than the underlying Vintage. This would reduce their length by as much as 30 ft. The disadvantage to this approach is the difficulty in estimating the required grout volume in light of the steep dip of the rock and the unknown southern boundary of the voids. Lacking a reliable cost estimate, this approach was eliminated.

Finally, it was decided to extend the caisson shafts below the problem zones in the Ledger and found the sockets entirely within the underlying Vintage formation. This was the adopted design. Although it would result in longer, more expensive caissons, constructing the sockets in the Vintage rock did have one advantage—higher capacities could be used than if the sockets were drilled into grouted Ledger. The allowable unit socket tip resistance and side friction in the Vintage rock were calculated as 5 tsf and 17 tsf, respectively, compared to 5 tsf and 10 tsf in the grouted Ledger. The final allowable caisson capacity was 150 tons for a nominal 5-ft socket length. The design was based on a 3-ft shaft diameter and 2.5-ft socket diameter. The predicted maximum settlement of the caisson groups was 0.3 in. The lateral capacities of the caissons, determined using computer program *COM624P* (FHWA 1993), ranged from 15 to 30 tons and the uplift capacities were between 50 and 75 tons.

In establishing the caisson socket tip elevations, only the recovered lengths of the Vintage cores were counted, since poor recovery is usually caused by highly weathered or severely fractured (less than 1-in. size) material which does not reflect the rock mass characterization for the Vintage formation. Accordingly, the sockets were lengthened beyond the nominal values to account for such weaknesses. Actual socket lengths varied from 5 to 11 ft, as shown in Table 1. Where voids in the Ledger are expected, a permanent casing was specified to extend all the way to the top of the Vintage dolomite. This was important to prevent stressing the rock fingers above the voids and confine the shaft concrete.

The above design criteria were readily applied to the caissons where test borings were performed. Elsewhere, linear interpolation was performed using top of rock (Vintage) contours generated from the borehole data and superimposed onto plans of numbered caissons for each substructure. Figure 4 depicts this information for the west abutment. For caisson bidding purposes, five separate bid quantities were tabulated for each substructure, as shown in Table 1. These are: (1) caisson length in soil; (2) boulder/finger thickness; (2) rock void size; (4) socket length; and (5) permanent casing length.

Table 1. Design Information and Bid Quantities for Selected Caissons at the West Abutment of the Schuylkill River Bridge

Design Information									Bid Quantities				
Caisson No.	Ground Surface El. (ft)	Caisson Cap Base El. (ft)	Boulders/ Fingers El. (ft)	Groundwater El. (ft)	Top of Rock El. (ft)	Top of Socket El. (ft)	Rock Void El. (ft)	Socket Tip El. (ft)	Caisson in Soil (ft)	Boulders/ Fingers(ft)	Rock Void (ft)	Socket Length (ft)	Permanent Casing Length (ft)
9	121.4	100	-	60	48.9	48.9	-	41.6	36	15	0	7	0
14	105.1	100	87.0-20.0	61	54.4	29.4	45.0-25.0	23.4	56	15	5	6	71
17	101.7	100	65.0-51.8	63	47.8	47.8	-	36.6	36	16	0	11	0
24	99.9	100	83.9-20.4	59	54.4	20.4	46.0-25.0	15.4	54	26	11	5	80

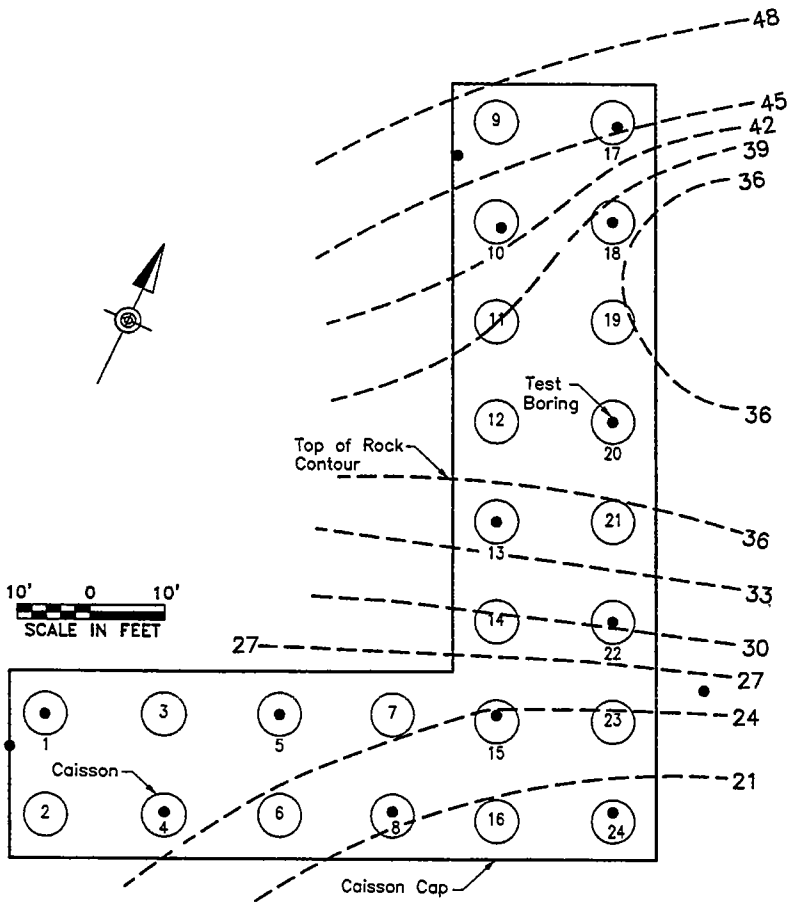

Figure 4. Caisson layout and top of rock contours for the west abutment of the Schuylkill River bridge.

Table 2 presents the general requirements, relative cost, and limitations of each foundation alternative considered for the Schuylkill River bridge.

Table 2. Aspects of Various Foundation Systems for Deep, Sloping Rock with Voids and Boulders.

Design Alternative (1)	General Requirements (2)	Relative Cost (3)	Limitation (4)
Spread Footings on Soil	Compaction grouting of soil to improve strength	Low	Uncertain compaction grouting cost
Short Caissons with Sockets in Ledger dolomite	Grouting of rock voids	Medium	Uncertain rock grouting cost
Long Caissons with Sockets in Vintage dolomite	Drilling through boulders, fingers, and voids; casing into deep rock	High	Requires a highly qualified caisson contractor

Severely Sloping Rock Without Voids

A sloping rock surface is the predominant feature at the two span bridge site. Since the bedrock is not as deep as the Schuylkill River bridge site, spread footings on rock were initially considered. However, the need for benching and stepping of the footings on the steep rock surface together with the associated shoring complications indicated that this would not be a cost effective and practical foundation system. Spread footings on improved soils, although a feasible approach, was also not pursued due to grouting cost uncertainties. Consequently, the following two foundations were carefully evaluated.

Caissons with rock sockets are technically sound and significantly less complicated than those previously discussed for the Schuylkill River bridge. The problems associated with boulders, rock fingers and voids are not present here. Thus, high vertical and lateral capacities can be easily developed from the rock sockets. For a socket with a nominal length of 7.5 ft, the vertical allowable capacity was calculated to be 190 tons and the uplift capacity 85 tons. The estimated settlement was less than 0.2 in. Lateral capacities of the caissons controlled the design of each substructure foundation at this bridge and were analyzed using *COM624P* computer program (FHWA 1993). The resulting allowable lateral capacities were in the range of 20 to 35 tons.

The caisson socket tip elevations were determined in a similar manner as described for the Schuylkill River bridge, interpolating between borings as necessary. Design tables similar to Table 1 were prepared for each substructure unit.

Due to the lack of voids and minor amount of boulders, driven piles were considered for the two-span bridge. Nevertheless, the pile foundation design involved two major obstacles: the stability of driven piles on severely sloping rock, and the associated low lateral capacities of short piles—less than 5 ft at certain locations. The first obstacle was overcome by predrilling certain pile locations 2 ft into the rock to prevent slippage of the driven piles. Steel H piles (*HP 12x74*) are then installed in the predrilled holes and AASHTO No. 10 sand was specified to fill the annular space between the predrilled holes (18 in. diameter) and the H piles. The second obstacle was resolved by utilizing battered piles to completely resist all of the lateral loads. A 1 horizontal : 3 vertical batter was adopted for design. Also, piles less than 5 ft long were predrilled to ensure their stability. A design capacity of 80 tons per pile was calculated.

The following approach was utilized in determining which piles had to be predrilled: Vertical piles on sloping rock surface steeper than 2 horizontal : 1 vertical or 26° are generally unstable because this is the limit after which tip reinforcement becomes ineffective in providing adequate seating into a sloping rock surface. When the direction of battered piles is favorable to the slope of the rock, the 26° limit is

increased by the batter angle. Conversely, when the direction of the battering is unfavorable, the 26° limit is reduced by the batter angle.

Figure 5 shows the pile layout with top of rock contours at the west abutment. The shaded areas designate piles that are required to be predrilled. In the north shaded area the battering is unfavorable, whereas most piles along the southern edge are favorably battered—thus no predrilling is required there. The two piles in the southwestern corner required predrilling due to boulders.

Both caissons and piles are technically feasible for the two-span bridge and driven piles are usually more economical than caissons. However, karst terrain imposed special restrictions on the piles. About 50% of the total piles require predrilling and embedment 2 ft into the rock. As a result, the usual cost advantage of driven piles over caissons was not apparent. Consequently, both design alternatives were included in the contract documents so that the ultimate choice is decided by the actual bids.

Cavernous Rock

The simple-span bridge site has the condition of cavernous pure Ledger dolomite. As previously described, the 30-ft opening in the rock runs across the entire length of the wing wall but does not extend under the existing east abutment. The Ledger in this area is covered by a blanket of Triassic limestone conglomerate which is free of voids.

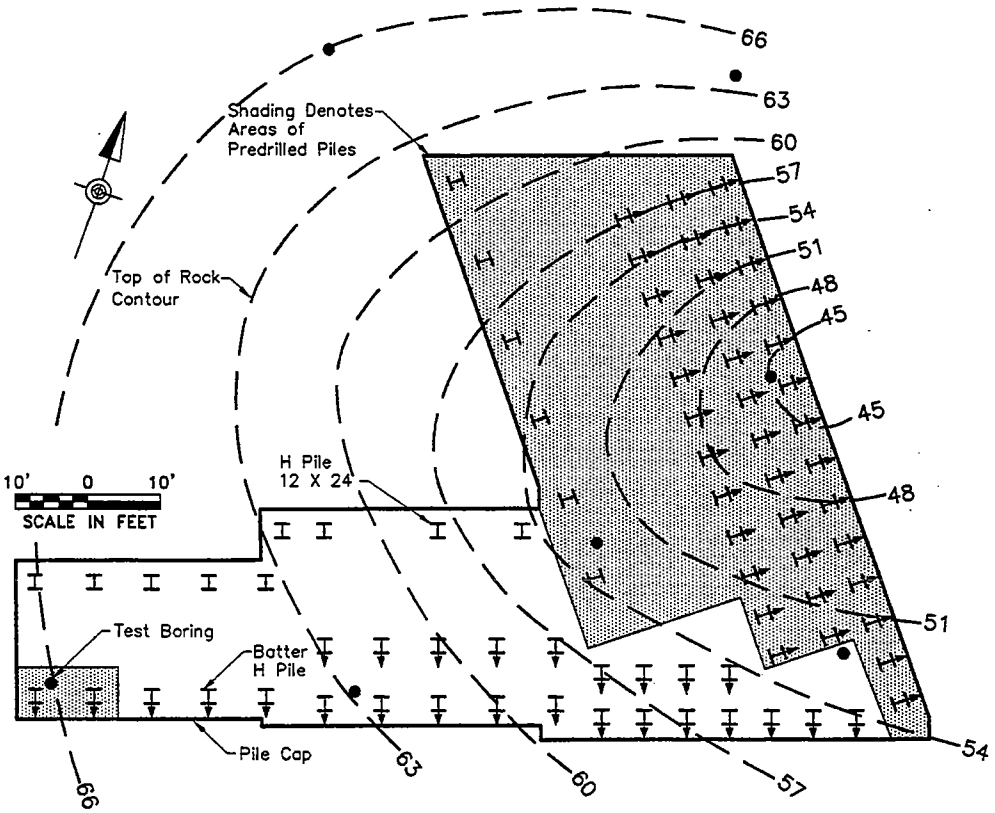

Figure 5. Pile layout and top of rock contours for the west abutment of the two-span bridge.

The presence of the conglomerate moderates the severity of the huge solution opening by creating a buffer between the cavernous Ledger and the bridge. For this reason, and because the conglomerate was level, a spread footing on rock was considered to be the best foundation option for the new structure. The footing of the new abutment was designed to line up with the existing one at El. 70 ft, as shown in Figure 3. At this elevation there would be about 10 ft of conglomerate on top of the cavernous dolomite. Given the weak strength of the conglomerate and the unknown southern boundary of the cave, a grouting program was developed to fill the rock void. For footing design, an allowable bearing pressure of 5 tsf was used, and the predicted maximum settlement was 0.5 in.

The grouting program included an allowance for performing additional exploratory drilling at the start of construction to further define the cave's boundaries, since the intent of grouting was to fill the void immediately below the footing, not the entire feature. Given this objective, compaction grouting was the selected technique due to its low slump (less than 1 in.), which would prevent excessive loss of grout. The program involved primary, secondary, and tertiary holes laid out in a 5 to 20-ft grid. Each of the three injection categories had a specified grout cutoff volume.

CONCLUSIONS

This paper demonstrates the importance of detailed field explorations toward a comprehensive understanding of the subsurface conditions in difficult karst settings. The various subsurface features, especially the unusual rock fingers and inclined voids at the western end of the Schuylkill River bridge, could not have been understood without geologic mapping and diligent drawing of sections.

Spread footings on soil should be considered for karst settings that have a stable overburden or one that can be improved. This system avoids the adversities associated with boulders, sloping rock, and voids.

When carefully considered, driven piles coupled with selective predrilling can be an effective foundation system for structures underlain by severely sloping rock. However, detailed subsurface information and very specific requirements regarding predrilling and pile battering are needed.

Although expensive, caissons will always provide a sound foundation system even in the most difficult carbonate formations. Most subsurface problems in the soil or rock can be effectively handled by caissons. Nevertheless, their design and construction in adverse carbonate formations require detailed geotechnical drawings and reliable estimation of quantities. When included with the bid documents, these items clarify the subsurface conditions likely to be encountered and minimize misunderstandings, which are the leading cause for problems, change orders, and cost overruns in caisson construction.

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Prospecting for Local Material Sources

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Abstract

It has been common practice for many agencies to prospect and test material pits to locate economical sources of aggregate for nearby construction projects. By making the information obtained available to all bidders, agencies expect to reduce bidding costs and risk for bidders; thereby receiving lower bids. However, providing information on availability of local materials entails risks for the agency due to the variability and uncertainties that are inherent in many subsurface explorations.

Subsurface exploration can involve both intact ledge materials such as basalt, granite and limestone and loose redeposited materials of sand, gravel and boulders. Each deposit will likely require its own type of exploration. Options include continuous auger drilling, open excavation with backhoe or bulldozer, or continuous coring. Seismic exploration may be beneficial, particularly for ledge deposits. The extent, frequency and method of exploration may not be evident until some preliminary exploration has been performed.

Adequate numbers of samples should be tested to develop the information required by the bidders and for preparation of the project specification. Information must be sufficient to define properties to allow bidders to estimate production methods and costs and/or project designer to incorporate achievable requirements into the specifications.

Introduction

The construction of a new roadway or the rehabilitation of an existing roadway either by realignment or overlay generally requires a large quantity of natural materials. These materials, be they unclassified borrow, aggregate base, aggregate for portland cement concrete or asphalt concrete, represent a major portion of the total construction/maintenance costs. Particularly for roadways in remote areas, the transportation costs of the materials can exceed the initial material costs. Many transportation agencies have undertaken programs to locate economical sources of materials for nearby construction projects. These programs were prominent during the early development of the interstate highway system. Many agencies developed

extensive inventories of potential material sources and even designated which sources could be used for specific projects. The agencies expected, by making this information available to all bidders, to reduce bidding costs and reduce risk to the bidders; thereby receiving lower bids.

Today, it is a rare occurrence for an agency to designate a particular material source for use on a project. This position resulted from numerous claims or litigious actions brought against the agencies by contractors citing changed conditions. Allegations range from claims of insufficient quantities and inadequate qualities to inability to mine with conventional methods. Consequently, as the cost and time allocated to defend the claims grew, the agencies relegated the responsibility for material supply to the contractors.

The need for prospecting material sources still remains, whether performed by agencies or contractors. The methodology presented herein can serve as a guide for addressing the issues associated with the development of new material sources. In all cases, four basic factors must be considered: quality of material, quantity of material, economics of production and delivery, and impact on the environment.

Purpose of Prospecting

Although prospecting local pit materials and providing information to bidders entails risks, the potential return in lower construction costs and higher quality material are incentives for performance of this work. The goal of any engineering endeavor should be to produce a product that meets the project requirements at a cost within the project budget. The objects of material pit prospecting are:

- Gather information on the quality of local pits materials.
- Gain knowledge of the quantities of materials available.
- Compare production, transportation and construction costs for alternative material sources.
- Collect information sufficient to satisfy designer and bidder's needs.

For pit prospecting information to be of value, designers and bidders need to be able to depend upon it. If information is not reliable, those parties utilizing it will still need to assume high risk factors and little has been accomplished. Consequently, the "hold harmless" clauses and qualifications of data often given, render the efforts and information near useless. Statements relative to normal variation within deposits should be given and heeded; however, this can be done while still having confidence in the basic data provided by prospecting efforts. It also is reasonable that if information provided to bidders while developing bids is found to be inaccurate during prosecution of the subsequent contract, the contractor is entitled to fair compensation or specification revisions due to these changed conditions.

If dependable data from local pit prospecting are to be provided, prospecting and subsequent testing and analyses must be carefully and accurately performed. The extent and type of exploration and testing will vary from source to source. Accepted standards need to be followed and combined with the application of practical judgment. In all cases, the manner in which the work is performed must be clearly defined for designer and bidder's information. The risks involved are too great for incomplete or haphazard work.

Preliminary Research

Few areas exist in the world for which there is not some information accumulated which is of potential value in the location of material sources. The information may have been accumulated for purposes quite extraneous to material source development. The prospector needs preliminary subsurface information to intelligently and economically plan explorations. The subsurface should be well understood in general before it is studied in detail. A good regional concept of the general origin and development of the materials at or near the surface is a valuable asset in:

- formulating a typical profile of the subsurface,
- estimating the depth of bedrock,
- planning a physical exploration program, and
- preventing misinterpretation of individual boring logs.

Preliminary information can be gained by reviewing printed publications from four basic sources:

- engineering information,
- pedologic information,
- geologic information, and
- aerial photography information.

Engineering Information

Engineering soil maps exist in forms from very small scale to very large. Unfortunately, these maps are unavailable for most areas and limited to rather shallow depths.

Topographic maps are generally available for most areas of the United States and can be obtained from the U.S. Geological Society and major sports or map stores. Drainage maps and flood plain maps have been developed mostly on a county basis and are available from local agencies or universities.

Pedologic Information

The science of pedology is concerned with the classification of soils, based upon their origin and development and their consequent physical and chemical characteristics. The pedologist studies the effect of five basic factors in classifying soil profiles:

- age,
- parent material,
- climate,
- slope or topographic position, and
- vegetative cover.

Not all of these factors are of primary concern, however, age and parent material are highly pertinent.

These maps have been developed by the Soil Conservation Service and the coverage is for a county size unit and reported in a Soil Survey, e.g., "Soil Survey, Safford Area, Arizona, 1970."

Geological Information

There exists a wealth of information on geological issues, most of which have been compiled for scientific or economic purposes unrelated to transportation routes. Maps are most useful and some of the common types are:

- areal or bedrock,
- surfical, and
- structural or subsurface.

There is a profusion of geologic literature. An effective geologic information search should incorporate a query of the state geologic survey for the area of interest. Local universities or colleges also can be a source of information.

Training and experience are, of course, required to derive optimum benefit from geologic maps and reports. The engineer with some geologic knowledge and the geologist with some engineering knowledge can, by cooperative effort, bring considerable power to bear upon the problems of material source location.

Aerial Photographs

Photographic coverage can be used to interpret or infer the nature and distribution of soils and rocks. The "appearance" of a soil or rock material on the photograph defines the pattern which is studied by its various subordinate elements:

- topography,
- landforms,
- photographic tones,
- drainage pattern,
- gully shape and gradients,
- vegetation cover, and
- culture or land use by man.

The study of these elements and their relation to various soil and rock types often allows development of a rough geologic map of an area. Aerial photographs are available from state and municipal agencies and from local aerial survey companies.

Material Sources

Roadway materials are obtained from two basic sources, pits and quarries. Pits are used to obtain unclassified borrow, aggregate base and concrete aggregate after processing. Quarries usually are used to generate high quality aggregate, ballast, rip rap or dimension stone.

Sand and Gravel Sources

The geological terrains wherein sand and gravel may be found are numerous:

- *Alluvial Fans* are deposits found in the valleys at the base of mountains and contain particles ranging from boulders to fine-grained materials. Usually the coarse-grained material is found near the head of the fan, upstream end, and the finer materials are near the outer edges, downstream end and sides.

- *Terrace Deposits* commonly occur along the sides of streams or rivers. The materials are generally well graded and rounded. Many of the gravels may be severely weathered and soft or the materials may contain chemicals which render them useless as portland cement concrete aggregates.

- *Flood Plain Materials* occur within the boundaries of a stream valley. Fine-grained materials are predominant near meandering streams whereas coarser materials are prevalent adjacent to braided streams. The materials from these sources usually require considerable processing and washing to remove the fine-grained material for aggregate production.

- *Glacial Outwash* is material deposited from streams flowing from a glacier. These deposits tend to be stratified and contain all particle sizes from clay to boulders.

- *Sand Dunes* are very common in arid and semi-arid regions. The sand is generally fine to medium in size and poorly graded.

- *Residual Deposits* occasionally can be a source for sand. Typical sources are granite, sandstone and quartzite. These deposits are usually very limited in extent and depth.

- *Talus Deposits* may be used as gravel or rip rap if grading permits. Crushing is commonly required to obtain a sufficient proportion of small sizes.

- *Volcanic Cinders* can be processed to serve as aggregate base materials. These materials are generally quite porous, resulting in some undesirable characteristics such as degradation under freeze-thaw conditions and high absorption of asphalt cement.

Quarries

Quarries may be either open or underground. The open quarries may be shelf (ledge) quarries where the rock is extracted from a hillside ledge or pit quarries where rock is extracted from a certain depth in the ground. The rock type can be igneous, metamorphic or sedimentary; however, the most prevalent types used for crushed rock, rip rap or concrete aggregate are igneous and sedimentary. Particular attention should be given to the geological features of the locality as the strike and dip of the strata and the jointing of the rock may dictate the rock processing methods. The depth to the rock and the presence of groundwater also impact the processing method and costs. Similar geologic features are important in the development of underground quarries.

Environmental Concerns

Even though the preliminary paper research indicates a potential material source may exist, other concerns need to be addressed prior to performing any field explorations. The development of a pit or quarry should be preceded by the establishment of a general plan for minimizing the short and long term effects on the environment. The environmental goals should be to avoid any detrimental effects; to minimize detrimental effects if they can not be avoided; and to enhance the environment whenever possible.

Do nothing is an obvious solution, but it does not resolve the need for construction materials. The development of a pit or quarry should address the following issues:

- vegetation, destruction and restoration;
- hydrology, surface and groundwater;
- erosion, water and wind;
- pollution, water and atmospheric;
- noise, effect on humans and wildlife;
- wildlife, habitat and reproduction;
- aesthetics, operational and permanent;
- land reclamation;
- culture, religious, historical and archeological.

Preliminary inquiries on these issues should be made with the local permitting agencies. It is better to face the issues initially to formulate economic resolutions rather than expend significant costs in exploration and testing on a source that can not be permitted.

Field Exploration

Once the environmental and cultural resource issues have been resolved or there appears to be an economical resolution to any issue of concern, a field exploration program can be designed. The program should be performed in two stages, a preliminary and final program. The preliminary phase should consist of widely spaced borings or backhoe pits to assess the areal extent of the possible material deposit and to obtain sufficient quantities of materials to perform preliminary quality tests. It is at this point where knowledge of the site geology is a great asset particularly when there is considerable topographic relief across the site. The location and depth of the borings should be selected so that the depths of the borings overlap, thus allowing a complete profile of the materials across the site. If the potential site is to be a quarry operation, the strike and dip of the strata and the jointing of the rock are of major importance. Sites where the strata have a steep dip may require different mining operations than one where the strata are relatively horizontal. Samples of the rock taken from outcrops have been used for quality tests; however, these samples may not be representative of the fresh material. Inferences of the strata thicknesses from

the outcrop profile may be misleading. Continuous coring of the formation provides the best information regarding stratum thickness and quality of the material. The percent recovery and the rock quality designation (RQD) of the recovered core provide information on the quality of the rock and provide incite to required mining operations. Geophysical exploration procedures can be used to great advantage in defining strata thicknesses between boring locations.

The preliminary exploration program for evaluating a natural sand and gravel deposit will usually require more borings or test pits than for a quarry due to greater variability of materials across the site. Backhoe pits provide information on the ease or difficulty of excavation in addition to a greater exposure of the material profile; however, pits have a limited depth restriction. Continuous flight augers are routinely used for exploration. Care needs to be exercised to avoid intermixing of materials from different strata, thus, getting a false picture of the profile.

Preliminary tests should be performed on recovered samples to assess whether the materials will meet the standard specifications for the intended use or whether the specifications can be modified to allow use of the materials. Evaluation of the logs of the borings or test pits can be used to estimate the quantity of available materials in the source and the amount of unuseable overburden that will need to be removed. A more extensive exploration program should be designed after the suitability and supply of the potential materials has been initially determined. The design of the final program is dependent upon the initial results and the level needed to establish confidence and reliability to the end user.

Testing

There is a wide array of tests that can be performed in the laboratory to define properties of local pit materials. The tests selected need to be those that characterizes the material for its intended use. Frequently used tests include:

- Gradation
- Atterberg Limits
- Specific Gravity and Absorption
- Los Angeles Abrasion
- Sodium or Magnesium Sulfate Soundness
- Fractured Faces
- Maximum Density/Optimum Moisture
- Sand Equivalent
- Resistance Value
- California Bearing Ratio
- Soluble Salts and Sulfates
- Flat and Elongated Particles

There are other more specific tests such as potential reactivity, primarily for aggregates to be used in portland cement concrete, that will be performed when required. It is desirable to perform standardized tests so that parties using the information will be familiar with the results.

Loose aggregate deposits will normally be tested for gradation and Atterberg limits. In some cases such as for borrow or select material, a product that can be used without further processing will be sought. Maximum density/optimum moisture, resistance value or California bearing ratio are common tests to characterize these products.

Where higher quality material is sought, the aggregate samples will be processed in the laboratory by crushing and/or screening to simulate actual production. Further testing of the processed materials might include Los Angeles abrasion, fractured faces, sand equivalent, soundness and several other tests that measure particular physical properties. Material that will require crushing may be evaluated for compressive strength, which provides information on the degree of crushing difficulty and the effort required.

Ledge materials will require analysis following sizing by crushing and screening in the laboratory. Once prepared, testing similar to that used for loose deposits following processing is performed. In some instances, it may be advisable to perform trial mixtures, such as for use in asphalt concrete or portland cement concrete, to evaluate the product properties.

Test results are used not only to inform bidders of the characteristics of local material deposits but are used by the designer to draft specifications that will fit the material specified. Thus, an adequate number of tests need to be performed by conventional test procedures to instill within the user confidence that the data are an accurate representation of the material source.

Conclusion

Although there are risks involved in prospecting local pits for use in developing specifications and information for bidders, the returns can be significant. If local pit prospecting is elected, the work must be thorough and accurate so that information provided is reliable. If the information provided is unreliable and of no value to the designer and bidders, it is better to perform no prospecting and require bidders to develop their own material source information.

Sequential Deterioration of Concrete, Congress Street Bridge, Boston, Massachusetts

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ABSTRACT

We performed extensive petrographic and other examinations of 65-year-old concrete piers of the Congress Street Bridge, Boston, Massachusetts. The purpose was to assay the condition of the concrete and to evaluate the feasibility of rehabilitation to add an additional 50 years to the expected service life. For this investigation we collected and tested thirteen vertical and seven horizontal cores.

There was a sequence and a zonation to the concrete deterioration. First, the concrete deteriorated because of ASR (alkali silica reactivity), which occurs throughout the concrete. This fractured the concrete, increasing its permeability to rainwater, seawater, and brackish groundwater. A variety of coarse and fine aggregate rock types deteriorated from ASR in this natural sand and gravel concrete. These are meta-graywacke, granite, greenstone, meta-siltstone, quartzite, meta-rhyolite, slate, and claystone. The sequence of ASR deterioration is first the development of silica-gel saturated rims, followed by either peripheral cracking or splitting of the particles. There is previously unreported ASR deterioration of weathered potassium feldspar.

The most severe deterioration followed, and was from dissolution of localized zones of cement paste due to sulfate attack from brackish groundwater or seawater. There was also some sulfate attack from sulfate released from slag, apparently used as fill below or alongside the piers. Sulfate attack was heaviest at depth, below the waterline. Each of the concrete piers exhibits deterioration from alkali silica reactivity (ASR) and sulfate attack. Samples from piers 2, 3, and 7 have moderate ASR and light sulfate attack. Piers 1 and 9 have light-moderate ASR and heavy sulfate attack. Piers 4, 5, 6, and 8 have moderate ASR and heavy sulfate attack (especially piers 4 and 5). Deterioration also occurred from rainwater leaching of the exposed, upper portions of the concrete.

The concrete physically ranges from dense and competent to rubble. Coherent concrete has compressive strengths from 1,100 psi to 8,760 psi. Rubble pockets, often in the tidal zone, are about 2-3 feet in diameter, and are most likely able to withstand 300-500 psi stress.

If the northern pier structures of piers 7 and 8 are repaired in the tidal zone area, the remaining useful life of the foundation system of the Congress Street Bridge appears to be excess of 50 years.

INTRODUCTION

The Congress Street Bridge was constructed circa 1932; at the time of this investigation, it was 65 years old. The east end is in South Boston and the west end is in Boston proper. It is a structural steel girder and bascule type bridge used for vehicular and pedestrian traffic over Fort Point Channel, a salt-water tidal estuary. The bridge is approximately 520 feet long from

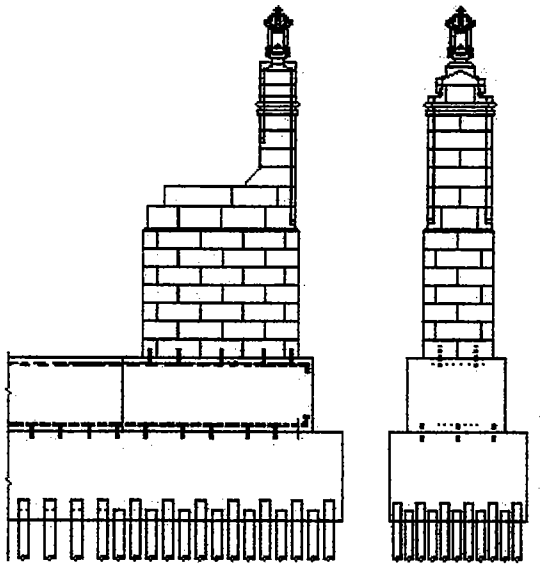

Figure 1. Typical pier construction.

abutment to abutment and approximately 60 feet wide. The bridge is supported between granite abutments by nine concrete pier foundations, each supported on wooden piles. The piers are constructed of concrete with granite block veneer in the tidal zone (Figure 1). The pier foundations are numbered 1 to 9, west to east.

There are no visible signs of settlement of the superstructure or foundations. The concrete foundations have, however, deteriorated to varying degrees at several locations. Recently the prestressed concrete pile foundations of a building at 303 Congress Street failed structurally. This event was highly publicized and the bridge owners were concerned that the bridge foundations might have a similar problem. However, the Congress Street Bridge concrete foundations are not constructed with cement and aggregates from the same material source which led to the ultimate failure of the building's foundation.

Original construction drawings dated July 1929 indicate concrete mixture proportions for the piers as being 1:1½:3 for concrete laid under water and 1:2:4 for the pier concrete. These proportions would have provided 2,000 psi at 28 days. The cement-richer 1:1½:3 mixture was constituted to allow for cement wash-away for the underwater work. Thus both mixtures most likely would have provided 2,000 psi at 28 days. For working stress design, this is an allowable stress of 800 psi. Analysis by the client indicates stress levels in the foundations do not exceed 200 psi.

In a report entitled "Congress Street Bridge Foundation Assessment and Underwater Inspection, Boston, Massachusetts," Nucci Vine Associates, Inc. state that, based on underwater exploration, the bridge's timber piles appear to be in good condition without any significant deterioration. The Nucci Vine report also states: "The condition of soil and debris surrounding the pier further indicates that the piles have apparently not been exposed to the marine environment." More importantly, the body of the report indicates in five test pits at the base of the concrete piers that the submerged concrete is soft and spalled with an estimated 3" to 6" loss of exterior concrete.

PROCEDURES

In order to perform petrographic analyses, a 2.1" diameter vertical core was taken from each of the nine piers. Stone Products Consultants then examined these cores in accordance with ASTM C 856 (Petrographic Examination of Hardened Concrete). We prepared thin (30μ thick) sections (2 x 3 in., 27x46 mm) from select portions of the concrete cores. Thin-sections were taken of rimmed aggregate particles, particles containing filled cracks, pores containing white deposits, bluish concrete, and of white porous concrete.

Nominal 5" diameter horizontal cores were extracted to provide samples for ASTM C 39-94 (Compressive Strength of Cylindrical Concrete Specimens). The cores were prepared by saw cutting, capping, and presoaking in salt water 40 hours prior to testing.

RESULTS

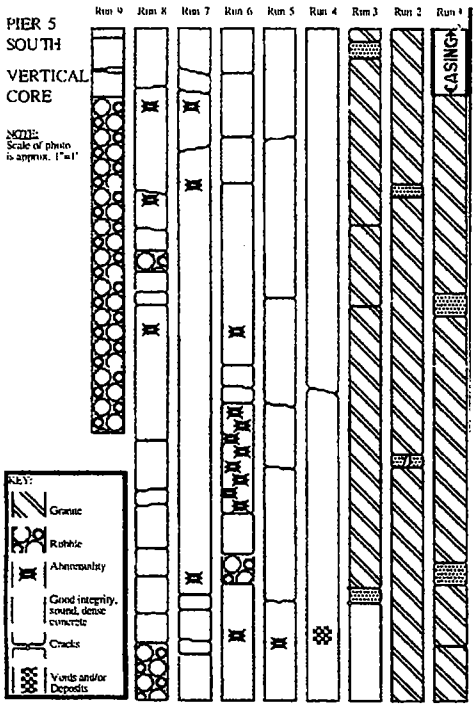

Figure 2. Typical core log.

Petrographic Examination

The appearance of the cores is best shown by Figure 2, a log of a typical core. The concrete cores exhibit various conditions, including dense sound concrete, cracked concrete, white deposits within cracks, completely deteriorated concrete rubble, etc. Each of the piers exhibits deterioration from Alkali-Silica Reactivity (ASR) and sulfate attack. Below are descriptions of these modes of deterioration with reference to Piers 1, 2, 4, 5, 7, and 8, which typify the sequence and zonation of the concrete deterioration.

In general, light to moderate ASR occurred before the sulfate attack, and throughout the concrete piers. Silica gel, the reaction product indicative of ASR, is seen in thin-section. Reactive particles are claystone, granite, slate, meta-siltstone, meta-rhyolite/tuff, greenstone, quartzite, sandstone (Figure 3), meta-graywacke, and even weathered potassium feldspar (Pier 5, Figure 4). Silica gel is often detectable to the naked eye in the form of white, or clarified rims around coarse or fine aggregate particles, or more obviously as white deposits which fill cracks in aggregate particles. Often, the sequence of ASR deterioration proceeds from clarifying rims to either peripheral cracking or splitting of the particles. In a section of Pier 4, a roughly foot-long crack occurs, extending through several particles, such as the meta-rhyolite particle in Figure 5. Thin-section analysis also reveals an associated crack in the center of the core to be filled with silica gel from ASR deterioration of a nearby meta-graywacke particle (Figure 3). A crack at the margin of a

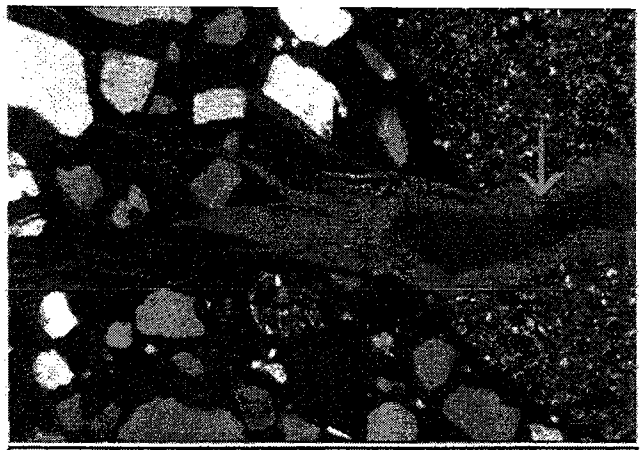

Figure 3. Photomicrograph of silica gel (@ arrow) in crack from ASR deterioration of sandstone particle. Field with = 4.8 mm.

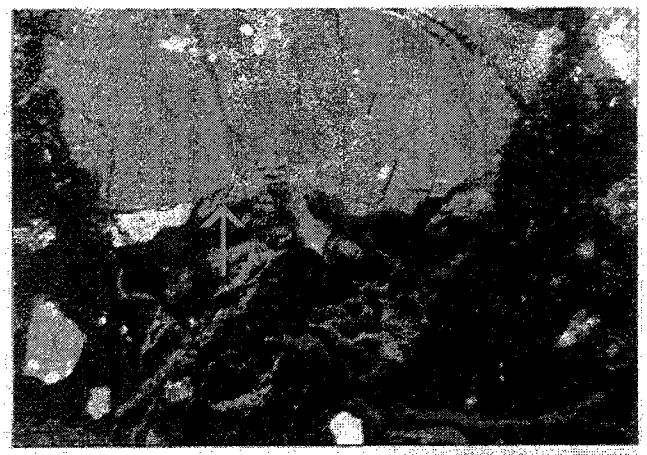

Figure 4. Photomicrograph of silica gel (@ arrow) from ASR deterioration of weathered feldspar particle. Field width = 4.8 mm.

7" Milford granite aggregate particle (Figure 6) occurred because of ASR-associated expansion of the concrete around the particle, and not by ASR deterioration of the particle. Silica gel often filled concrete pores; Figure 7 illustrates a silica-gel geopetal feature.

Following ASR fracturing of the concrete and the consequent increased permeability, additional deterioration resulted. Above the waterline, the deterioration was from rainwater leaching of the exposed, upper portions of the concrete. The concrete's saturation with water is evident from heavy deposits of Portlandite [$Ca(OH)_2$], a normal cement hydration product, in entrapped air voids and water voids under aggregate particles. More severe deterioration occurred below the waterline. This was from dissolution of localized zones of cement past due to sulfate attack from brackish groundwater or seawater. Sulfate attack was relatively light in Piers 1 and 2, but heavy in Piers 4, 5, 7, and 8. Cracks from ASR and air voids provided pathways for later sulfate attack, resulting in dissolution of cement paste and precipitation of Ettringite ($\{Ca_6[Al(OH_6)]_2 \cdot 24H_2O\}(SO_4)_3 \cdot 2H_2O$) with silica gel from ASR in cracks and pores. In Pier 8, sulfate attack was severe enough to also form Thaumasite ($\{Ca_6[Si(OH_6)_2 \cdot 24H_2O\}(SO_4)_2(CO_3)_2$), a rare secondary mineral in concrete. The concrete paste is bluish-gray to bluish-black in some portions of the concrete. This staining is ferrous iron, probably from slag used as fill near the pier. The weak, porous, and sometimes rubbly nature of the concrete in these stained portions is indicative of possible sulfate attack from sulfate released from the nearby slag.

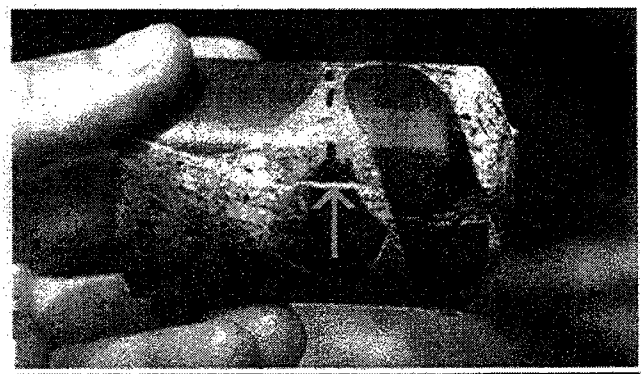

Figure 5 . Closeup photo of crack filled with silica gel (from ASR, @ arrow) extending through meta-rhyolite particle in concrete. Core diameter = 2.0".

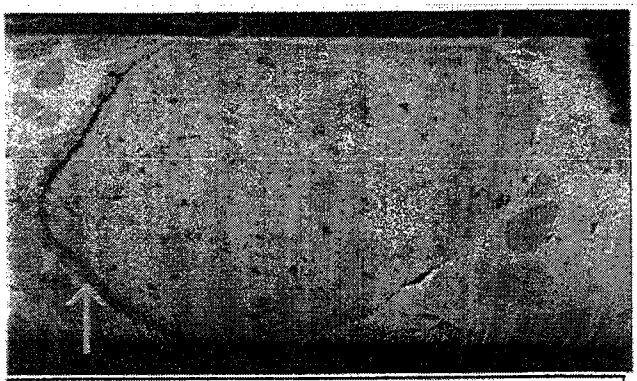

Figure 6. Macro photo of crack (@ arrow) around 7" Milford granite particle. Core diameter = 5.0".

The sequence of events, Portlandite formation, ASR, sulfate attack, is illustrated well in Figure 8. Portlandite formed first during the initial hydration of the concrete cement. In open voids it formed well-crystallized hexagonal crystals. ASR eventually occurred. The resulting silica gel formed deposits in air voids in the concrete, in the peripheral zone of the

aggregate particles, in the cement paste around the aggregate particle, and in micro cracks in the aggregate. In the portion of the concrete below the water line, sulfate-bearing water entered the concrete and initiated sulfate deterioration. The sulfate attack leached the cement paste and dissolved the Portlandite. Some dissolved material re-precipitated as secondary deposits of Ettringite, often within silica gel from ASR. Eventually, only Ettringite was deposited. This is

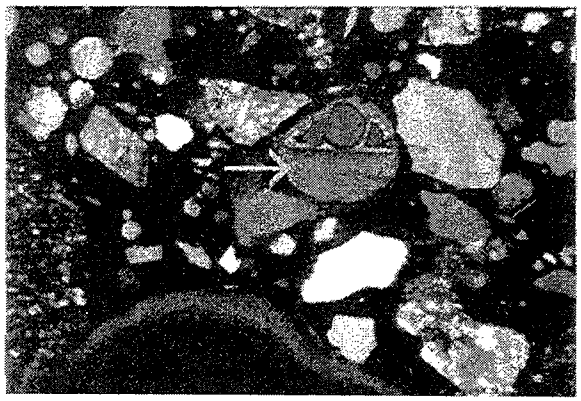

Figure 7. Photomicrograph of silica gel in a geopetal structure (@ arrow). Field width = 4.8 mm.

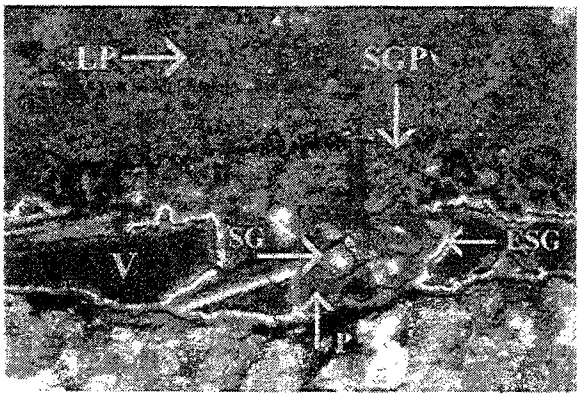

Figure 8. Photomicrograph of (@ arrows) Portlandite crystal molds (P), Voids (V), silica gel (SG) from quartzite, Ettr./silica gel (ESG) from ASR, silica-gel saturated cement paste (SGP), and leached cement paste (LP) from sulfate attack. Field width = 1.4 mm.

presumably because the ASR ceased as the sulfate attack caused the pH of the concrete to decrease because of the dissolution of Portlandite.

Compressive Strength Testing

The results of the compressive strength testing are summarized in Table 1. The actual compressive strengths range from 1,100 psi (test 9V-N#10) to 8,760 psi (test 6V-S#6) for coherent concrete. Although this concrete most likely achieved a 28-day compressive strength of approximately 2,000 psi, it is not unusual for the compressive strength to reach levels of over 8,000 psi at the current age of 65 years.

Rubble pockets in the concrete piers, often in the tidal zone, are about 2-3 feet in diameter. Because these pockets of severely deteriorated concrete are completely or partially enclosed by relatively sound, coherent concrete, they have a confined bearing capacity, most likely in the range of 300 to 500 psi.

Table 1. Results of Compressive Strength Testing of Concrete Cores, Congress Street Bridge, Boston, Massachusetts.

Pier I.D.	# of Tests	Compressive Strength (psi)			
		High	Low	Average	99% CL*
1V-N	10	6,660	5,660	6,120	5,700
2V-N	6	5,760	3,940	5,110	4,200
2V-S	5	6,910	5,910	6,440	5,900
3H-S	3	4,050	2,170	3,200	1,800
3V-N	6	4,630	3,030	3,770	2,800
4H-S	3	3,750	2,280	2,840	1,700
4V-N	4	2,970	1,360	2,230	1,300
4V-S	9	3,390	1,600	2,070	1,200
5V-S	19	5,330	3,670	4,400	3,800
6H-S	1	—	—	2,710	1,500
6V-N	13	5,030	2,960	4,310	3,600
6V-S	6	8,760	5,160	6,380	4,500
7V-N	5	5,700	3,770	4,660	3,400
8V-N	5	2,740	1,130	1,800	1,000
8V-S	6	5,440	3,440	4,500	3,400
9V-N	10	3,800	1,100	2,660	1,500

* The 99% CL tabulation is a statistical calculation based on the lowest (most conservative) value from one of two formulae as a function of the standard deviation of the results. One of the formulae is taken from ¶5.3.2.1 of ACI 318 and the other is from the lower value of a standard statistical confidence interval.

CONCLUSIONS

The amount of deterioration seen in the concrete represents less than 10% of the concrete sampled. Some small amount of the deterioration is due to original construction such as inclusions of wood, cement laitence, slag and voids. The majority of the deterioration is due to ASR, which increased the permeability of the concrete, allowing intrusion of seawater and consequent sulfate attack of the cement paste. Most of the sulfate attack occurred in the north pier structures of piers 7 and 8 behind the granite facing in the tidal zone. Otherwise, localized deterioration, particularly seen in the vertical cores from piers 4 and 5, consists of confined rubble pockets about 2 to 3 ft. high. In these pockets, the concrete is most likely able to withstand stress levels of 300 to 500 psi. With calculated stress levels of actual loads showing less than 200 psi, the rubble pockets should not compromise the foundation during normal gravity loading conditions. The compressive strength levels of the non-deteriorated concrete range from 1,100 psi to over 8,000 psi. If the northern pier structures of piers 7 and 8 are repaired in the tidal zone area, the remaining useful life of the foundation system of the Congress Street Bridge appears to be in excess of 50 years.

HOW TO AVOID EXCESSIVE EXPANSION OF CONCRETE DUE TO ALKALI-AGGREGATE REACTION[1]

by

Bryant Mather[2]

ABSTRACT

Excessive expansion of concrete due to alkali-aggregate reaction (AAR) will not occur if any one of the following circumstances exists: (a) The aggregate is insufficiently reactive, (b) The pH of the pore fluid is not too high, (c) The amount of reaction product formed is not sufficiently large or not sufficiently expansive so that its expansion can cause damage, (d) There is not enough available water to cause the reaction to progress so as to develop the expansive product and to be available for imbibition by the product so as to cause it to swell and disrupt the concrete.

Put the other way around, excessive expansion can occur only if there is enough potentially expansive AAR reaction product and water so that, as the product takes up the water and swells, the concrete expands excessively. Hence, what is needed to avoid excessive expansion is to be able to predict the probable reactivity of the available aggregates, the probable mechanisms by which the pH of the pore fluid might get well above the normal 12.6 or so, and, when justified, select and implement appropriate precautions against the undesirable consequences of these eventualities.

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[1] Prepared for presentation at the 49th Highway Geology Symposium, Prescott, AZ, September 10-14, 1998, and publication in the Proceedings.

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INTRODUCTION

In 1979, ACI published a paper I wrote (Mather, B., 1979) called "Concrete Need Not Deteriorate," which included the essence of remarks I have made at quite a number of different places over the past 40 or more years. It says, in effect, that concrete will provide the sort of performance desired, provided only that the specifications are correct and that they are followed. We know how to identify the relevant levels of properties needed of concrete to resist any reasonably anticipated set of environmental conditions. We know what sorts of quality control procedures the contractor should employ and what sorts of quality assurance activities the owner should employ to see to it that the standards of good practice in executing the work are followed.

Later I wrote a paper on how to make concrete that will be immune to the effects of freezing and thawing (Mather, B., 1990). Still later (Mather, B., 1993) I discussed at the Transportation Research Board what one needs to do to avoid premature deterioration and to obtain the desired performance of concrete in transportation. Damage to concrete by freezing and thawing and damage caused by excessive expansion due to alkali-aggregate reaction were the two principal issues relating to concrete durability that were selected for detailed research in the recent Strategic Highway Research Program. It seems appropriate now to try to do some technology transfer about preventing damage to concrete resulting from alkali-aggregate reaction.

Tom Stanton, of the California Department of Transportation, discovered alkali-silica reaction (Stanton, 1940). I went to work for the Concrete Laboratory of the Corps of Engineers in August 1941. Word of his discovery had already reached that laboratory, and one of the first bits of work I did was to look at his "siliceous magnesian limestone," which was described as the offending ingredient of the concrete that had deteriorated. At that point, the Corps of Engineers was quite concerned because, to us, magnesian limestone meant dolomite, and dolomite as crushed stone was very widely used as aggregate in concrete in Corps of Engineers structures. Our position had been that while most limestones sold as aggregate were satisfactory, substantially all dolomites were. It turned out that the "siliceous magnesian limestone" that caused the trouble in California is a very unusual rock that contains a lot of opal. As far as I know, no other dolomite in the world contains enough opal to cause trouble due to AAR.

ASSESSING AGGREGATE REACTIVITY

Normally, the first issue confronting a person contemplating the use of concrete with regard to alkali-aggregate reaction is: "Are there some reactive aggregates out there among those that might get used in the work?" Hence, one usually thinks first of the procedures available for answering the question: "Is the material in this aggregate source potentially deleteriously reactive with alkalies?"

ASTM C 33-93, the "Standard Specification for Concrete Aggregates," requires in Section 7.3 that: "Fine aggregate for use in concrete that will be subject to wetting, extended exposure to humid atmosphere, or contact with moist ground shall not contain any materials that are deleteriously reactive with the alkalies in the cement in an amount sufficient to cause excessive expansion of mortar or concrete, except that if such materials are present in injurious amounts, the fine aggregate may be used with a cement containing less than 0.60% alkalies calculated as sodium oxide equivalent (Na_2O + 0.658 K_2O) or with the addition of a material that has been shown to prevent harmful expansion due to the alkali-aggregate reaction. (See Appendix XI.)" and in Section 11.2: "Coarse aggregate for use in concrete that will be subject to wetting, extended exposure to humid atmosphere, or contact with moist ground shall not contain any materials that are deleteriously reactive with the alkalies in the cement in an amount sufficient to cause excessive expansion of mortar or concrete except that if such materials are present in injurious amounts, the coarse aggregate may be used with a cement containing less than 0.60% alkalies calculated as sodium oxide equivalent (Na_2O + 0.658 K_2O) or with the addition of a material that has been shown to prevent harmful expansion due to the alkali-aggregate reaction. (See Appendix XI.)"

The appendix to C 33 discusses the use of C 295, petrographic examination; C 289, the quick chemical test; C 227, the mortar-bar test; C 342, the Conrow test; and C 586, the rock cylinder test. In each case some advice is given for interpreting the results in terms of the potential for excessive expansion due to alkali-aggregate reaction. Action to revise this appendix to cite more recently developed test methods is currently pending in ASTM Committee C09 on Concrete and Concrete Aggregates.

The U.S. Army Corps of Engineers' Manual (1994) includes guidance to field offices in preparing specifications for concrete. It is assumed that a survey of potential aggregate sources will be made and data on samples from them will be obtained. The project specifications will contain a

list of the sources that are regarded as containing adequate amounts of material of satisfactory quality to provide aggregate for the project. The evaluation of reactivity will include examination and tests by C 227, C 289, and C 586. The manual states:

"If aggregates containing reactive constituents are to be used, the problem then becomes one of identifying the reactive constituents and determining the conditions under which the available aggregates may be used. Where the reactive constituents can be positively determined to occur in such small proportions as to be innocuous, an aggregate, if otherwise of suitable quality, may be used without special precautions. Where the reactive constituents occur in such proportions that they are potentially deleteriously reactive, it will be necessary to use low-alkali cement or an effective pozzolan, or both, with the aggregate. If the requirement of low-alkali cement would impose serious difficulties of cement procurement or excessive increase in cost, consideration should be given to the use of portland blast-furnace slag cement (a blend of portland cement and slag), portland-pozzolan cement, cement with GGBF slag, or cement with a pozzolan, or both, that will prevent excessive reaction even when high alkali-cement is used. When consideration is given to the use of any of these materials in lieu of low-alkali cement, mortar-bar tests should be conducted to verify that the potentially deleterious expansion will be reduced to meet the criteria in Appendix D or E. If petrographic examination determines the presence of potentially reactive materials in excess of the limits given in Appendix D, mortar-bar tests, ASTM C 227 shall be performed. If the petrographic examination determines the presence of strained quartz in excess of the limit given in Appendix D, the high-temperature mortar-bar test shall be performed."

I regard the treatment in Appendix D and Appendix E as the most comprehensive available. Since the Corps of Engineers' Manual is not copyrighted, it has been reprinted for general information by the American Society for Civil Engineers.

PRECAUTIONS TO TAKE TO AVOID EXCESSIVE EXPANSION DUE TO ALKALI-AGGREGATE REACTION

In some cases, no specific precautions are needed to preclude excessive expansion due to alkali-aggregate reaction. These cases include:

(1) The aggregates have been shown not to be potentially deleteriously reactive; i.e., innocuous with respect to AAR.

(2) The cementitious medium to be used has been shown to be of sufficiently low-alkali content and the possibility of alkali from any other source is sufficiently remote to be disregarded; hence, it can be confidently expected that the pH of the pore fluid will not get too high.

(3) The cementitious medium to be used includes a sufficient amount of satisfactory ground slag or pozzolan so excessive expansion will not occur, with this composition having been selected for reasons of temperature control or economy or both, unrelated to alkali-aggregate reaction.

(4) The concrete to be produced will all be in an environment in which it will rapidly get dry (internal relative humidity well below 80%) and remain so during its service life.

In some cases where the aggregate is classed as potentially deleteriously reactive, the level of such reactivity is sufficiently low so that the only precaution that is needed is to specify that the cement be "low-alkali." In others, one should preselect one or more aggregate sources that have a good service record when used in similar work with the local cement or cements. In still others, there should be a study of other constituents that might be included in the concrete such as ground granulated blast-furnace slag (ggbfs) or a pozzolan. Natural pozzolans (volcanic ash, diatomaceous earth, metakaolin, calcined shale, etc.), fly ash, and silica fume have all been shown to be able to prevent excessive expansion of potentially deleteriously reactive cement-aggregate combinations.

PROCESS AND MECHANISM

While we talk about "alkali"-aggregate reaction, in every case of damage to concrete in service I know of caused by AAR, what the presence of alkali (sodium ion or potassium ion or both in solution) did was to cause the pH of the pore fluid to rise to levels where it became aggressive to the thermodynamically metastable silica or the peculiar reactive carbonate rock in the aggregate. It isn't the alkali that does the reacting; it is the hydroxyl ions. Vivian (1951) demonstrated that: "The hydroxyl ion must be considered to be the reacting ion." He based this conclusion on the observation that tetramethyl ammonium hydroxide produced a similar reaction product to that formed from sodium hydroxide. He showed a mortar bar made with low-alkali cement to which sufficient tetramethyl ammonium hydroxide had been added to provide 0.85% hydroxyl ion by mass of the cement, that expanded 0.85% at 10 months. A control with low-alkali cement did not expand.

The alkalies that end up in concrete, from the cement or elsewhere, are almost never NaOH or KOH unless we are doing research in the laboratory; they are often potassium sulfate from the cement, or sodium chloride from sea water, etc. Solutions of such salts are pH neutral, that is about pH 7, not above the normal 12.6 or so of calcium hydroxide. The reason alkalies get a chance to mobilize hydroxide ions, raise the pH, and allow the pore fluid to become aggressive to the reactive constituents of aggregate when such are present, is that the chloride or sulfate ions react and precipitate out of the solution, usually to form calcium sulfoaluminate or calcium chloroaluminate or gypsum.

REMEDIATION

Suppose someday you get the news that your structure (or one of your structures) that you'd been counting on for another half century or more of service has been declared to contain AAR reaction product, or worse, found to have typical pattern of cracking and, upon investigation, found to have both AAR reaction product and remaining unreacted reactive aggregate constituents. What are the chances that deterioration will progress to an unacceptable level? This is not an easy question to answer--as one philosopher put it, "Making predictions isn't easy, especially about the future." What we did years ago was to store cores with length-change measuring inserts in the fog room (Luke, 1964). What was found was that a core from a dry area showed twice as much expansion as any of the others, all of which came from areas that were wet or damp. This suggested that there was a limited amount of expansion to be expected, much of which had already taken place in the wet or damp portions of the structure.

Tuscaloosa Lock in Alabama, the Corps of Engineers' best-studied ASR-affected structure (Mather, B., 1952; McDonald and Campbell, 1977) is a classic example of how a structure that when first examined was declared to be "so badly cracked that the natural impression would be that it is about to collapse" (Carlson, 1948) and 30 years later (McDonald and Campbell, 1977) be found to be "of generally good quality."

Another structure, also in Alabama, from which cores were studied in detail at our laboratory is Martin Dam (K. Mather, 1981). In that paper it was concluded that "the condition of the concrete...can be considered acceptable" after 50 years of service. Alkali-silica reaction evidence was found in all the concrete cores examined. The reactive constituent was metamorphosed quartz and quartzite that had undulatory extinction. The reaction product was typical of ASR. The concrete had less reaction product than seen in

other structures still in good condition after 10 or 24 years of service. One photomacrograph of Martin Dam concrete was so illustrative of ASR phenomena that it was used as the frontispiece in the *Proceedings* of the 1978 Fourth International Conference on Effects of Alkalies in Cement and Concrete (Diamond, 1978).

A somewhat similar story is that of Stewart Mountain Dam in Arizona where major expansions were observed, then they ceased, and 30 years later the concrete was in as good as or better condition than it had been previously.

ACI Committee 221 on Aggregates is preparing an extensive report on AAR in which remediation will be addressed. It will be interesting to see what it proposes.

CONCLUSIONS

Procedures are available to reach a highly probably accurate answer to the question: "Is the aggregate from this source reactive?" In some cases, it is not necessary to take any particular precautions to avoid excessive expansion due to AAR even if the answer is "yes." These cases include: (a) when there is an available competitive nonreactive aggregate; (b) when the cement is low-alkali and that is all that is needed; (c) when the intent is to use ground slag or a pozzolan for other reasons. However, if specific precautions are needed, they can be taken. Pozzolans or ground slag are widely available.

It is better to take the steps to avoid excessive expansion before the structure is built, but even if this is not successfully accomplished and severe cracking due to internal expansion caused by AAR occurs, one doesn't necessarily need to "run for the hills." Tuscaloosa Lock, Stewart Mountain Dam, and others with severe internal expansion have gone on for 30 or more years giving adequate service.

Meanwhile, all of us who are interested in AAR should keep up with the state of the art. ASTM has published a new edition of STP 169 with excellent papers by Stark (1994) and Ozol (1994). As previously noted, ACI 221 on Aggregates has a review in the works. The 10th International Conference was held at The Regent Hotel in Melbourne, Australia, in August 1996.

POSTSCRIPT

The reviewers of this manuscript suggested that I should also address the question of why is it that, since the principles of ASR have been known for half a century,

examples of distress due to ASR are still being found in structures much less than 50 years old and how current actual practice agrees with the concepts I outlined to avoid excessive expansion due to AAR.

There are many situations in which it was assumed none of the aggregate sources in a region were potentially deleteriously alkali reactive since no distress had ever been observed that was proven to have been caused by AAR. In one such case, there was an abrupt change upward from about 0.4% Na_2O_e to about 1.2% Na_2O_e in the alkali content of the available commercial cement. This resulted in severe distress from the use of crushed stone containing substantial amounts of opal, the presence of which was well known to geologists in the area but not to any concrete people who were prepared to realize its significance.

There have been strong efforts to dissuade users from specifying low-alkali cement "when it is not needed," since, in many locations, it is difficult or impossible for some or all the cement producers to make it. The methods for establishing the fact of reactivity or nonreactivity of materials from aggregate sources without a service record has often been somewhat capricious and the procedures for evaluating the effectiveness and needed dosage of a mitigating material--slag, pozzolan, lithium salts--are likewise less precise and straightforward than would be preferred.

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Runaway Truck Ramp Testing Program

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Abstract:

Truck arrester beds, or escape ramps, located within Arizona were used for prototype testing. Tractor semi-trailer combinations with GVW's of approximately 35,000 lbs and entrance velocities of 45 and 64 MPH were driven into these ramps. The testing was used to help evaluate the effects of ramp material, ramp preparation, and maintenance on ramp performance. Instrumentation utilizing radar an a data acquisition computer was developed to record truck velocities from entrance until run out in the ramps. The instrumentation was designed to be used during testing and for remote sensing of entries at the completion of the research. Average deceleration values of from 0.25 to 0.44g were recorded for smooth ramp entries and from 0.24 to 0.3g for tracked or following entries. The performance of the existing ramps was found to be satisfactory and the importance of maintenance following an entry demonstrated.

During 1989 and into 1991 an ASU research team conducted an escape ramp research program for the Arizona Department of Transportation. The research was conducted in four of the existing escape ramps that had been built, Figure 1. This research effort proceeded in two major areas. The first was the development of instrumentation that would allow vehicles entering escape ramps to have their motion recorded. The second area was the use of controlled truck entries to evaluate escape ramp performance.

The development of the instrumentation system proceeded in two directions initially. The use of existing inductive loop technology to measure initial entry events and vehicle velocities was the first instrumentation technique utilized. With extensive support of ADOT personnel an inductive loop array was installed at I17 Southbound escape ramp. The array was to measure initial entrance velocities. Once an entry had occurred team personnel would visit the site and manually measure the run out distance. This array unfortunately was hampered by a serious of problems that prevented successful monitoring of emergency entries.

The study team enlisted the help of the only heavy duty towing company in the vicinity of the two I17 escape ramps. Jimbo's Towing was getting frequent calls to remove trucks using the escape ramps. Jimbo's staff agreed to ask the truck driver how fast he

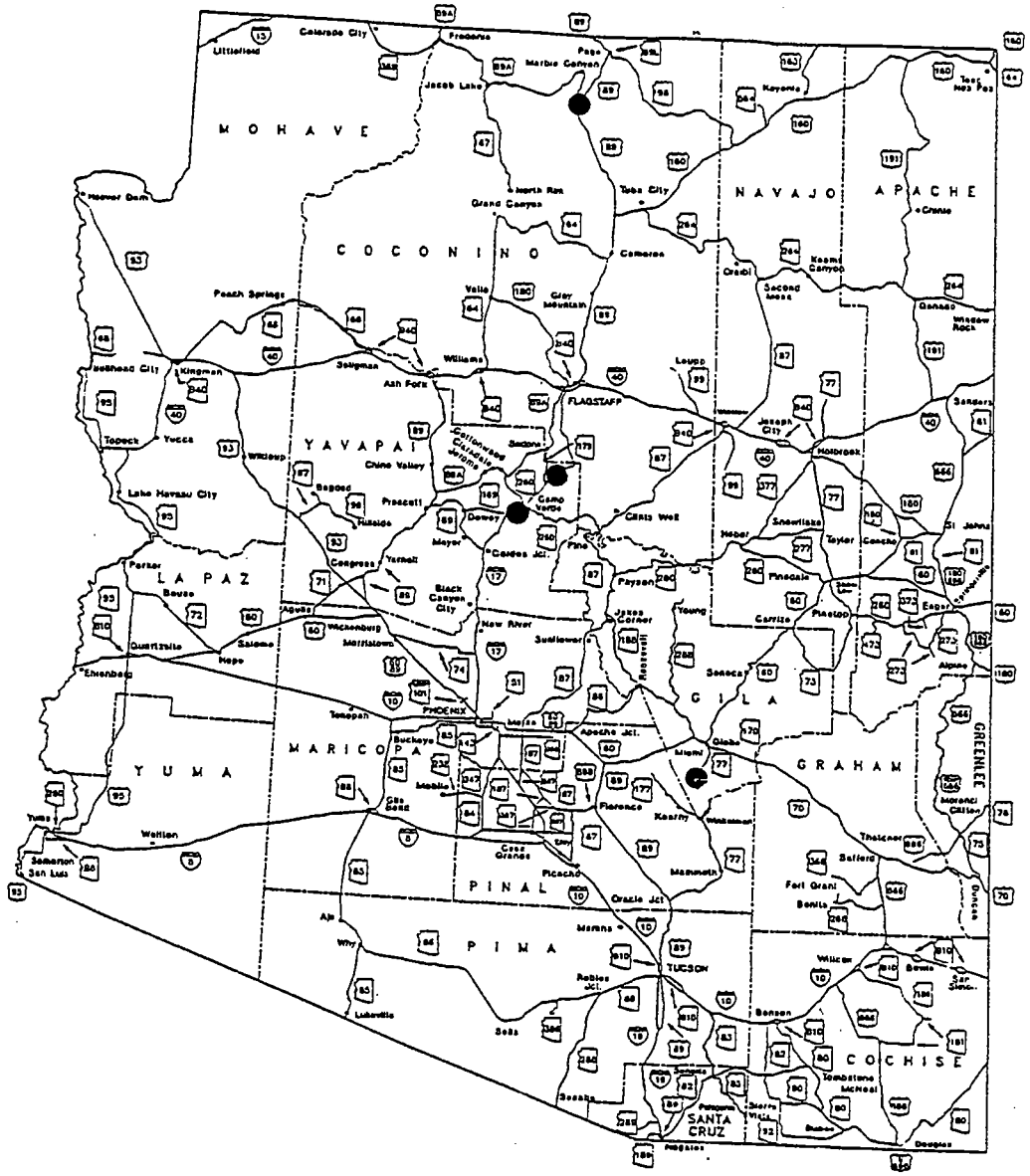


Figure 1, Escape ramp test locations

was going when he entered the escape ramp, what his gross weight was, and what the driver thought caused the need to use the escape ramp. Jimbo's staff would then call the research team who would send a member to the ramp and measure the length of escape ramp used by the truck.

The intent was to correlate driver's velocity estimates with inductive loop measurements. Persistent difficulties with the inductive loop's reliability, however, thwarted this correlation. During the observation period the relationship between driver's estimated speed and length of vehicle travel in the bed are provided in Figure 2.

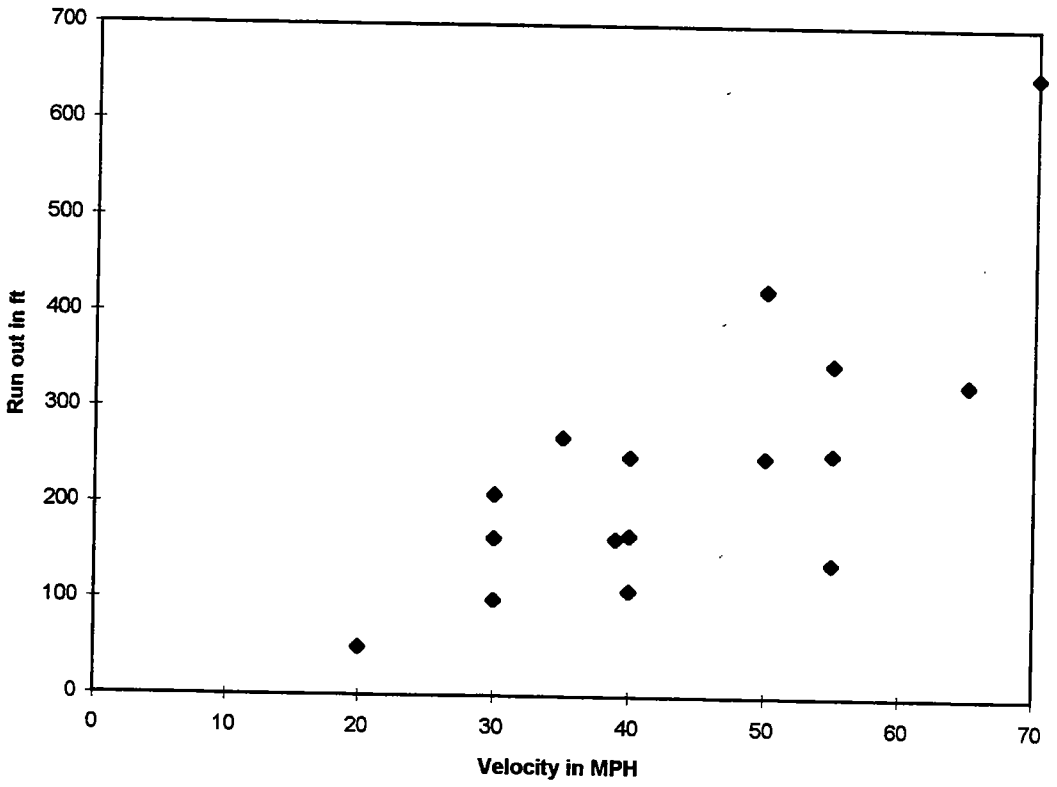


Figure 2. Driver's estimated speed vs. run out, average weight 77000 lbs with a range from 80000 to 73000 lbs

The difficulties with keeping the inductive loops operational coupled with the need to develop the relationship between entry velocity and distance traveled led to the second instrumentation program. The research team decided to develop a radar velocity measuring system that would provide continuous velocity versus distance data before and after the truck entered the escape ramp. An off the shelf radar gun was modified to provide velocities down to 1 MPH and send that data to a Hewlett Packard computer which constructed an accurate time line with the data arriving from the radar gun. The complete system was mounted in a heavy duty aluminum box for security and powered by solar cell. Two pressure switches buried at the start of the ramp were used to start data acquisition. The first switch was used to turn the computer and radar gun from standby to active modes. When the truck crossed the second switch the computer began to record the velocity data being sent to it. Once the radar gun reported loss of velocity the computer and radar gun were reset to standby mode. Plate 1 shows the cabinet mounted system without the solar panel. Figure 3 depicts the radar system as installed.

The development of this radar system was facilitated by the electronics developed by the team including Ryan Ali and the programming of Barry Meyers. The Chrysler Proving Ground personnel were also very helpful in enabling prototype testing to be conducted there. Proving ground personnel provided instrumented vehicles which enabled comparisons between the radar system and on board vehicle instrumentation.

Once the ability to track truck performance in the escape ramps was available the team began the truck in ramp portion of the research. There were several varieties of aggregates used in the beds selected for testing. Figure 4 provided the grain size distributions for these escape ramps. The angle of repose was measured for each escape ramp and the average slope of each ramp was established, Table 1. The aggregate was similar in each of the escape ramps with the exception of the US89 ramp. The aggregate used at US89 was appreciably finer. The combination of differing escape ramp geometry's and differences in the aggregates produced constraints on the experimental design.

Table 1, Escape ramp aggregate angle of repose, in degrees and ramp slope, in %

	Average values		
Escape ramp	Angle of repose	Specific gravity	Ramp slope
I17 North bound	34.1	2.64	-2.17to +2.0
I17 South bound	34.3	2.59	-5.47 to -2.0
AZ 77	33.5	2.6	-7.77 to +2.0
US 89	31.6	2.55	-3.44 to -2.62

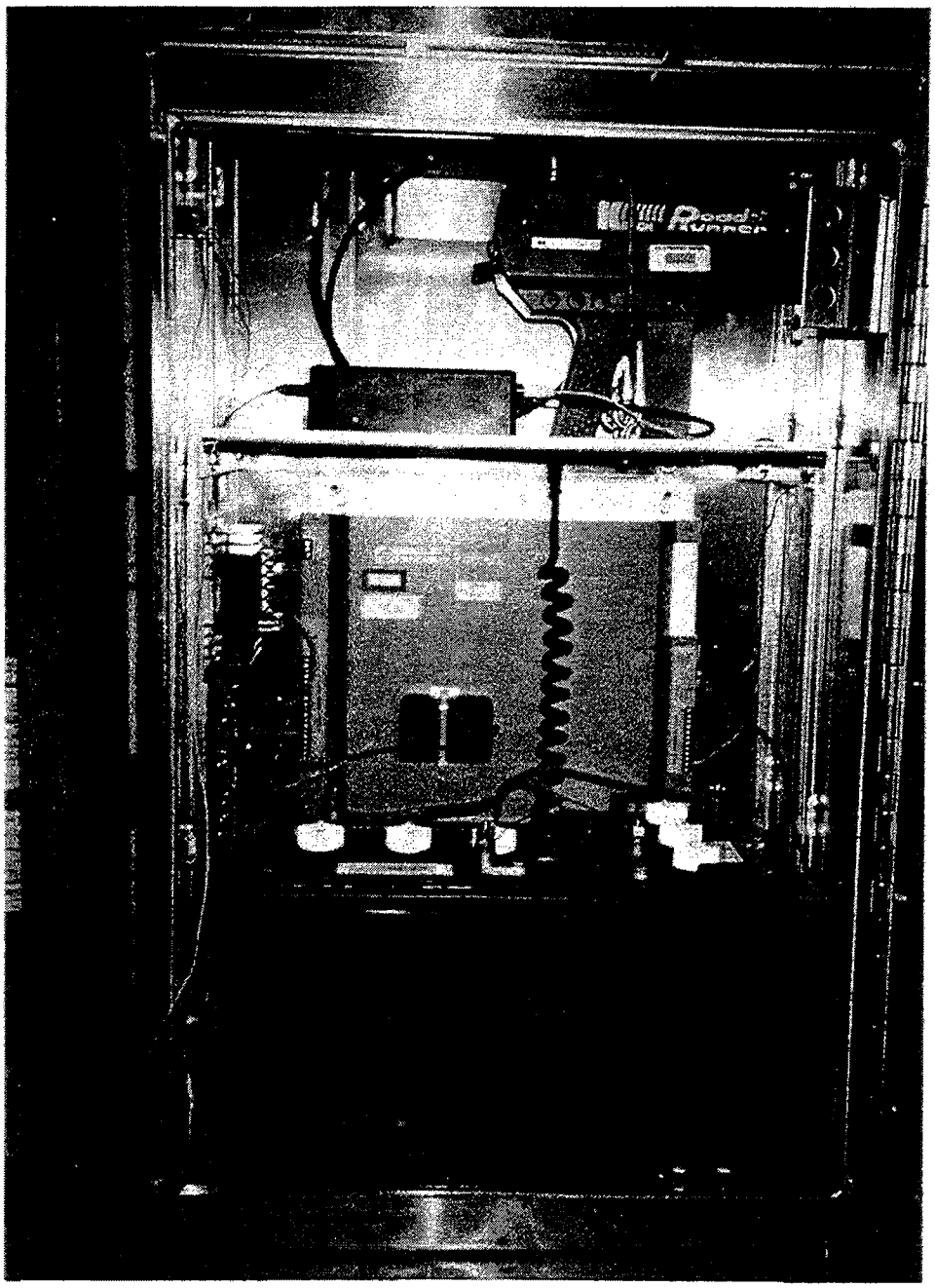
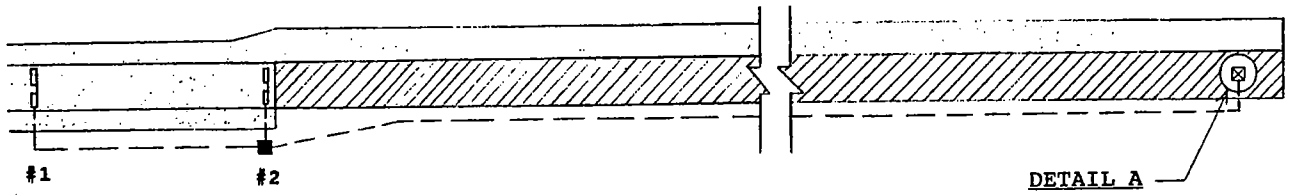
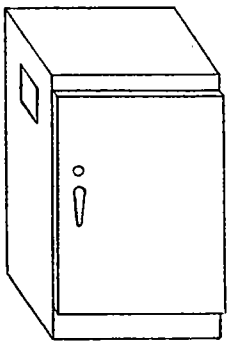


Plate 1, Radar velocity measuring system.



Direction of
Runaway Vehicle



KEY

Arrestor Bed Gravel

Junction Box

Buried Wiring

Two 10 foot Long Roadway Tape Switches
Wired Together in Series (#1 : Enables the Radar,
#2 : Initiates Data Collection)

Aluminum Equipment Enclosure, housing:
Radar, HP48060, F/V Signal Conditioner,
Pulse Extender, and 12V DC power supply.

Figure 3, Escape ramp installation, after Meyers, 1990

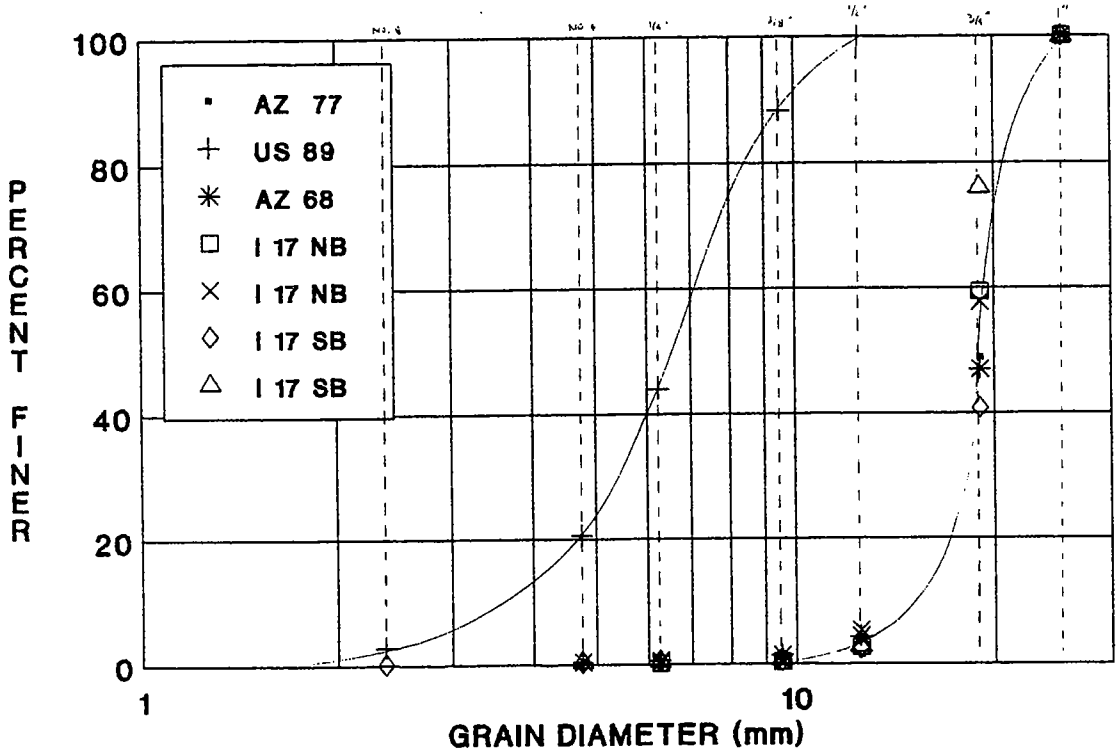


Figure 4, Escape ramp grainsize distributions

A typical plot of vehicle velocity versus time is provided on Figure 5.

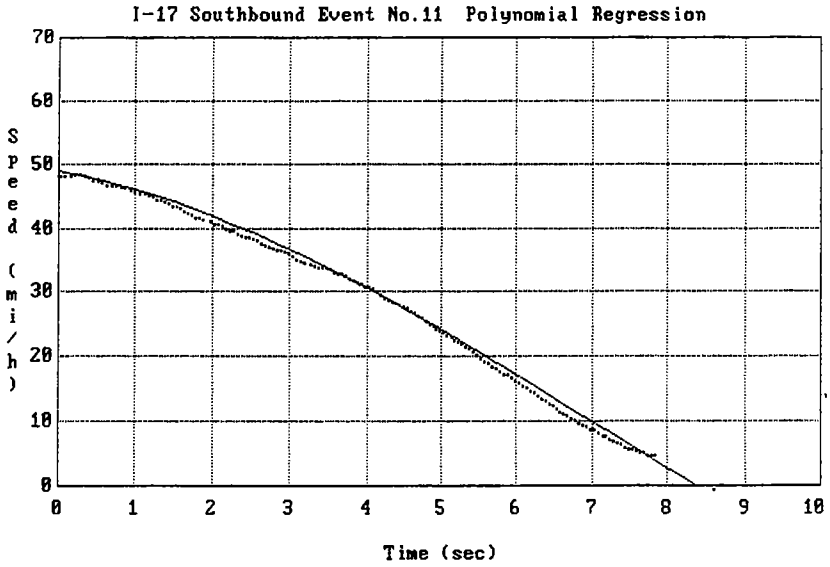


Figure 5, Typical time versus velocity relationship for vehicle entering escape ramp, after Zapata-Ospina, 1996

The testing program was designed to evaluate the method of bed preparation and the influence of prior truck entries into the escape ramp. The influence of existing tracks which could affect the rolling resistance of an entering vehicle as well as it's controllability was of special interest. There would be the possibility that another truck entry would be made before maintenance operations could re-level the aggregate. The density of the aggregate was also of interest in terms of how the aggregate was leveled. A question was would the vehicle entry length vary if the aggregate were "raked" prior to entry compared to leveled by grader or dozer blade? These different methods of aggregate preparation prior to testing were used. Plate 2 shows one of the aggregate smoothing techniques used to prepare the escape ramps for entries.

A rubber tired loader was also used to prepare the aggregate but it's mobility was poor. The maintenance personnel operating the equipment were very proficient in producing uniform escape ramp aggregate prior to each entry. Surveys of the escape ramp surface closest to the point of entry determined, however, that there was

Plate 2, Dozer with rake on blade

a loss of aggregate. The reason for the loss became apparent as the trucks entering the ramps were observed. The high velocity entries were throwing aggregate out of the bed and transferring it down ramp with the truck. The maintenance personnel were not only forced to deal with re-leveling the aggregate they had to attempt to maintain constant aggregate thickness at the entry points.

The research team thought it would be an easy matter to find a trucking company willing to rent an operated tractor semi-trailer for the testing. This turned out not to be the case even when it was explained that any damage would be paid for. There was a concern about the safety of their driver's as well as catastrophic vehicle damage. The team heard several accounts of injury and damage from the companies contacted that they said occurred when trucks entered escape ramps. None of these stories, however, turned out to be first hand accounts.

Mid-State trucking agreed to provide a vehicle and driver with the understanding that one of the professors would ride with the driver at the start of the testing. The driver who only identified himself as "Murphy" turned out to the type who would try anything, as long as he had company. Arizona Highway Patrol Officers were hired to provide entry safety. It was a concern that a car might try and pass the truck just prior to the entry or follow the truck into the ramp. Another job of the DPS officers was to observe Murphy's brake lights to make sure that at the last moment he did not hit the brakes in panic. On only

one entry did the driver use his brakes and that was well into a very rough run when he thought the truck was going to bounce out of the bed. In the end Murphy did all that was asked and he did it well. It may have been imagination but some said his swagger improved as the testing continued.

Unfortunately the large number of entries damaged the paint, broke the windshield and other glass, and damaged hoses and cables under the truck. Approximately $3,000 in damages were sustained to bring the vehicle back into pre-test condition. When the second phase of the testing was to start the company refused to provide a truck. The driver of Jimbo's towing came forward and offered to drive if a truck could be found. A truck rental company agreed to rent, after being showed photos of actual entries. With windshield and other glass protected this truck was used for all subsequent testing. When the second phase of the testing was completed less than $900 was spent on repairs.

Previous research, Wambold, et al., 1986, had provided evidence that vehicle entries were not sensitive to gross vehicle weight. The differences in momentum being transferred to the aggregate being compensated for by increased tire penetration during the entry. The obvious benefit of not having to deal with load securement was realized by using unloaded flat bed tractor trailer units. The gross vehicle weights of the units used was approximately 35,000 lbs.

Two entry velocities were used. 45 and 65 MPH were used at each escape ramp. The Arizona speed limit prevented entry speeds greater than 65 MPH. The driver's perspective of the escape ramp entry at I17 289 North bound is presented on plate 3.

The escape ramp does not appear to be overly wide even to a passenger. The image of jets landing on a carrier deck flashed through the mind of the team member in the cab on the first entry. There is no question that at night under the stress level created by failing brakes that using the escape ramp is both an act of courage and desperation.

When one looks at plate 3 it is possible to interpret the slope of the bed as positive, when in fact it is negative. There was a near entry at the I17 North bound escape ramp prior to the start of testing that provides insight into driver concerns. The truck was a fully loaded tractor semi-trailer hauling fresh produce. The truck had lost effective braking and the driver turned into the ramp entrance. He told DPS officers that he thought the truck could be slowed on the "uphill" access ramp that runs the length of the escape ramp. This driver proceed to run the full length of the ramp before he realized he was not slowing. He drove his

truck over the end of the escape ramp and down a steep slope destroying the truck and load but escaping with his life.

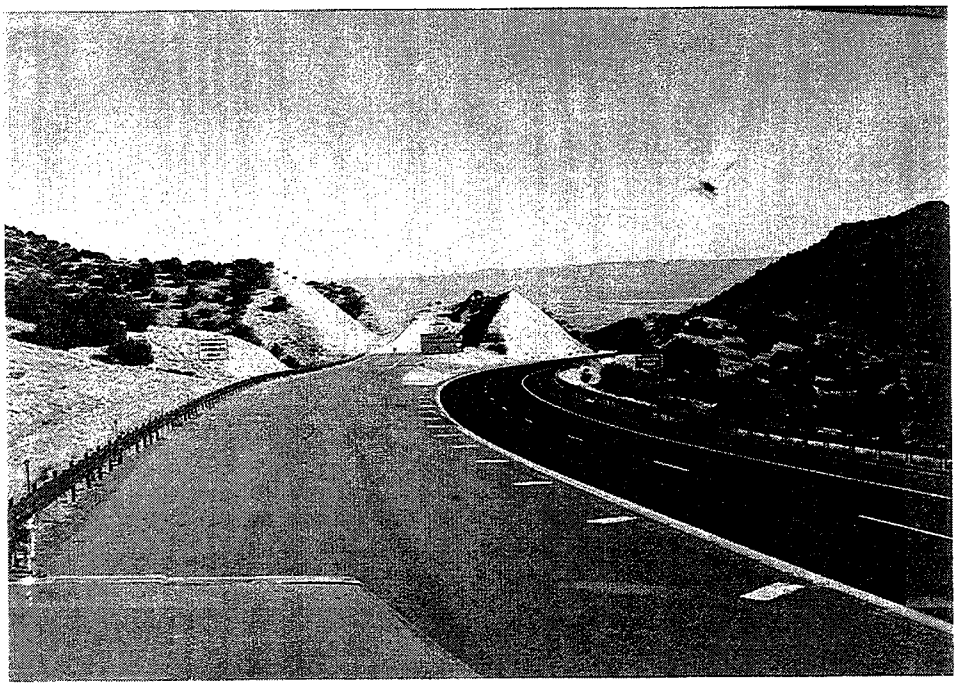


Plate 3, Drivers perspective of escape ramp at I17 South bound

In discussions with drivers who entered the escape ramps under emergency situations and with the tow truck operators, it became clear that the word on the road at the time of the testing, was escape ramps are the last resort. Drivers told of hearing stories about bad accidents in the ramps and of $500 tow out fees and then having to deal with the state troopers.

On one occasion one of the team members was replacing a defective battery at the I17 South bound instrumentation when he observed a truck making an unscheduled entry into the bed. This team member is a DPS Reserve Officer carrying Commercial Motor Vehicle inspection certification. He estimated the entry speed to be 75 MPH and saw no sign of smoking brakes. The vehicle entered the escape ramp slightly left of center and ran straight for 300 feet. At 300 feet the truck began a turn to the right and ran for the edge of the aggregate where it appeared it would roll. At the edge of the three foot high drop-off the truck swerved back into the ramp where it came to rest diagonal to the long axis of the ramp with the front bumper 15 feet from the edge of the

aggregate. The rear bumper was less than two feet from the right edge of the ramp when the truck stopped. The vehicle had effectively blocked the escape ramp. The total run out distance measured was 678 feet. An examination of the aggregate side slope showed evidence of a bearing capacity failure when the truck was running toward the right edge. The driver was very lucky he started his turn to the left when he did.

The team member produced his DPS identification and asked for the driver and vehicle papers. The driver's status was current though he had an unauthorized passenger, a woman who was sleeping at the time the brakes were lost and was terrified throughout the ordeal. The vehicles total weight was 70,140 lbs with the load being frozen fish. During the interview of the driver some interesting comments were made that shed further light on driver mentality prior to an escape ramp experience.

It was his first time down I17 and he was coming from New England. He said as soon as he noticed his brakes were smoking he tried to stop but could not. He said " I knew I had four more miles of down grade and was only going to go faster so I went into the bed". He estimated his entry speed as 65 MPH and thought he was still applying brakes when he went into the bed.

The driver was not wearing his seat belt. He knew that was a Motor Carrier violation and still admitted it. He was scared enough, prior to seeing the escape ramp signs, that he and his passenger were considering jumping from a high speed truck. He said the first part of the entry was smooth. He said that then the "gravel got too deep and it pulled to the right". It was suspected that he had been "bounced" away from the wheel because he was not wearing his seat belt.

There was only broken running light damage to the truck and Jimbo's towing was called and immediately removed the truck because of the blockage. The driver and his passenger were both shaken by their experience. If the escape ramp had not been there who can say what might have happened around the curve that lies below the ramp.

Plate 4 shows the I17 South bound escape ramp with a test entry of 65 MPH underway. The amount of dust that is generated during an entry is apparent from this plate. Plate 4 also shows the "bow" wave that forms in front of the wheels and the aggregate thrown as momentum is transferred from truck to rock. The team discovered that the entries, though dramatic and loud, were safely made. If, however, the truck is following the tracks of previous entries the run out will longer and rougher than if the ramp was smooth.

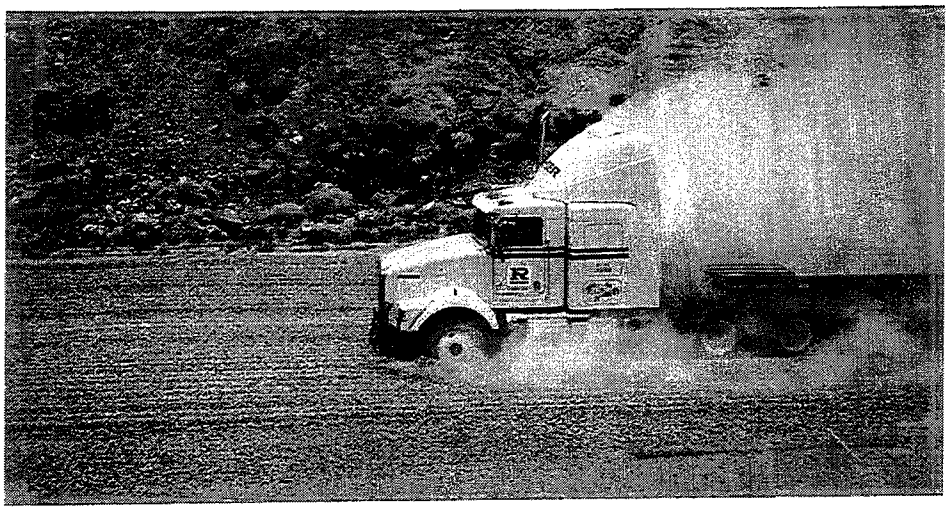


Plate 4, 65 MPH entry at I17 South bound, smooth bed conditions

The vehicles were slowed in the escape ramps by the interaction of the tires and frame with the aggregate. The influence of gravity and air resistance were also present. The pictures of the truck in the escape ramp aggregate provide some indication of the bow wave created by the tires and the transfer of energy to the aggregate. The dust associated with the entries also affects the way in which energy is transferred to the aggregate, As the escape ramps contain more dust their effectiveness will decrease. The reduction in effectiveness of the aggregate will be exacerbated in the winter when the dust will assist in the cementation of aggregate when moist and temperatures drop below freezing. Typical deceleration and for each ramp and I17 North bound run out are presented in Tables 2 and 3 respectively.

Table 2, Average vehicle deceleration in % for all ramps and test vehicles

	Ramp preperation technique and aggregate smooth or tracked from previous entries											
	Dozer				Rake				Loader			
Ramp	'45 MPH		'65 MPH		'45 MPH		'65 MPH		'45 MPH		'65 MPH	
	Smooth	Tracked	Smooth	Tracked	Smooth	Tracked	Smooth	Tracked	Smooth	Tracked	Smooth	Tracked
I17 NB	0.38	0.3	0.38	0.4	0.3	0.25	0.3	0.27	0.32	0.27	0.3	0.28
I17 SB	0.25	0.24	0.33	0.31	0.26	0.24	0.31	0.29	0.26	0.26	0.32	0.29
US 89	0.29	0.24	0.31	0.29	0.3	0.26	0.31	0.29	0.29	0.25	0.3	0.28
AZ 77	0.39		0.44									

Table 3, Truck run out I17 NB smoothed by dozer & tracked, in ft

	Entry velocity			
	Smooth		Tracked	
	45 MPH	65 MPH	45 MPH	65 MPH
	201	405	263	491
	204	362	253	386
	224	430	260	485
		411	242	516
	198	395	266	464
		378	278	491
		425		414
Average	207	401	260	464

Conclusions:
The instrumentation system developed worked essentially as designed. The difference between measured truck run out and the instrumentation system computed distances varied an average of 0.9%

The aggregate placed in the Arizona escape ramps effectively decelerates trucks. Damage to vehicles entering these escape ramps is usually slight at speeds as high as 75 MPH.

At entry speeds of 65 MPH no entry used more than 33% of the escape ramp. Tracked entries, where the truck runs in existing ruts, are consistently longer than the smooth escape ramp beds.

The escape ramp aggregate must be maintained carefully to minimize vehicle run out and controllability.

An extensive education program should be utilized to show commercial vehicle drivers that the Arizona escape ramps will safely slow their vehicles.

Acknowledgments:
The research conducted in Arizona's escape ramps was funded by the Arizona Department of Transportation. The employees of ADOT, engineers, geologists, and maintenance personnel were essential to the conducting of this research. The enthusiasm of the field personnel involved with the maintenance of these ramps was fantastic. This research could not have been conducted without their suggestions and their hard work. They clearly were working to improve highway safety which they saw as their job.

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Evaluation of Crushed Stone Used for High Traffic, Bituminous Pavement, Wearing-Courses in Indiana

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ABSTRACT

Wearing courses for bituminous pavements typically consist of the upper 2-inch portion of the highway, which comes in direct contact with vehicular tires. Crushed stone aggregates in this surface course provide, along with other properties, the frictional resistance required by a braking vehicle. As exposed aggregate pieces tend to become smoother under traffic, it is necessary that the rock particles have resistance to this polishing action. For state highways in Indiana, heavy traffic roads must have wearing courses containing 50% blast furnace slag and 50% dolomite. For state roads with moderate traffic loads, 100% dolomite aggregate is required. Limestones and crushed gravels are not acceptable aggregates.

Cases of deterioration of aggregates in wearing courses have been documented. Examples are: 1) attack by deicing salts, yielded pits where aggregates were removed, and 2) breaking loose of pieces containing both aggregate and bitumen. Evaluation of aggregate quality was used to evaluate this problem.

Testing procedures in the evaluation included: Los Angeles abrasion loss, sulfate soundness loss, freeze thaw loss, freeze thaw loss in a brine solution, chemical analysis, insoluble residue content, x-ray diffraction and petrographic examination. Three case histories are examined which illustrate the use of these procedures for evaluating aggregates in bituminous wearing courses.

Introduction

Wearing courses for bituminous pavements typically consist of the upper two-inch portion of the highway. Coarse aggregates provide the frictional resistance required for braking and to be effective must resist polishing from vehicular traffic. State roads in Indiana carrying heavy traffic loads are required to contain 50% blast furnace slag and 50% dolomite. For those with moderate traffic loads, 100% dolomite aggregate is required. Limestones and crushed gravels are not acceptable aggregates.

Research was conducted at Purdue University to evaluate dolomite aggregates from approved aggregate sources used in the past for highway construction. The purpose of the work was to evaluate the performance of these aggregates and to provide data for possible inclusion of

other rock types as acceptable materials. Several cases of deterioration of acceptable aggregates in pavement surfaces were also evaluated.

Research Procedure

In a previous study of Indiana aggregates for bituminous wearing courses, eighteen dolomite sources were evaluated (Bruner, Choi and West, 1995). This work was incorporated as the first phase of the study. In the second phase of the research, brine freeze thaw testing was added to the data base. Analysis of the combined data set provided the conclusions reached in the study reported here.

Detailed petrographic analysis was performed on the eighteen dolomite sources. Megascopic and microscopic (rock thin sections) analysis was accomplished. The Indiana #11 gradation is specified as the coarse aggregate for bituminous wearing courses for state highways in Indiana. This gradation consists of 100% passing the 1/2", 75-95% passing 3/8", 10-30% passing the #4, and 0-10% passing the #8.

Microscopic evaluation of textures of the dolomite aggregates was obtained in thin section using a petrographic microscope. Grain size, grain shape, groundmass content and void content were determined. This information is provided in Table 1.

Chemical composition of the aggregates was obtained by a combination of chemical analysis, x-ray diffraction and insoluble residue content. INDOT (Indiana Dept. of Transportation) specifications require that dolomite used in the surface courses contain 10.3% or greater amounts of elemental magnesium. Using an EDTA titration method in accordance with ASTM procedure C602, Specification for Agricultural Liming Materials, values were obtained for most of the eighteen samples. The relative percentage of dolomite can be calculated using the values for percent elemental magnesium.

Table 1. Petrographic Analysis

Sample I.D.	Grain Size (mm)	Median (mm)	Grain Stand Dev	Matrix (%)	Ground Mass (%)	Grains (%)	Voids (%)
1	0.13	0.085	9.9	21.5	28.4	59.9	11.7
2	0.079	0.061	3.6	2.8	30.5	59.1	10.4
3	0.059	0.042	3	21.6	42.8	49.3	7.9
4	0.194	0.053	12	26.4	41.9	48.4	9.7
5	0.069	0.052	3.8	30.9	49	45.8	5.2
7	0.074	0.075	3.7	12.3	27.4	64.4	8.2
8	0.087	0.065	4.2	5.5	14.8	71.8	13.4
9	0.066	0.063	4.9	13.2	27.2	63.5	9.3
10	0.07	0.049	3.9	2.8	28.1	61.7	10.2
11	0.058	0.041	3.5	7.9	27.8	64.7	7.5
12	0.082	0.051	4.1	7.5	30.5	64.9	4.6
13	0.057	0.043	4.9	47.5	64.8	31.2	4
15	0.116	0.085	7	24.8	36.9	58.5	4.6
17	0.119	0.114	8.7	9.4	25	68	7
18	0.063	0.058	3.6	47	60.7	35.8	3.5
19	0.059	0.046	7.4	7.4	12.6	77.6	9.8
20	0.085	0.068	4.2	13.2	34.7	55	10.3

Mg Fraction in Dolomite = (Molecular Weight of Mg)/Molecular Weight of $CaMg(CO_3)_2$ =

24.31g/184.41g = 0.1318

Therefore % Dolomite = $\frac{\%\ Elemental\ Mg}{0.1318}$

A simple calculation shows that for the 10.3% Mg value, % Dolomite = $\frac{10.3}{0.1318} = 78.1$

X-Ray Diffraction

The two minerals comprising the greatest percentage of carbonate aggregates are dolomite and calcite. Using traditional petrographic techniques it is difficult to distinguish between them because of their similar properties in polarized light. Determination of the composition of cement versus matrix is not always clear-cut as it involves a decision by the petrographer. X-ray diffraction was selected to quantify the relative proportion of the two carbonate minerals in an attempt to avoid these difficulties.

Acid-Insoluble Residue Test

The acid-insoluble test was performed by dissolving approximately 500 grams of the aggregate graded finer than 9.5 mm (3/8 inch) size in 6N HCl. Three tests were performed on each source and the average determined as the percent of insoluble material. The insoluble residue analysis was accomplished using ASTM procedure D 3042, Standard Test method for Insoluble Acid Insoluble Residue in Carbonate Aggregates.

Materials obtained in the acid-insoluble residue test were sieved to obtain two portions, $\geq$74 μm (No.200 sieve) and material <74 μm. This boundary, 74 μm, distinguishes the coarse size from fines for soil-like particles.

Calcite Content

Similar optical properties of calcite and dolomite make it difficult to distinguish between them using a petrographic microscope. However, for the high purity carbonates in this research, calcite content can be estimated based on a difference calculation. Essentially all the carbonate in these rocks is either dolomite or calcite. Carbonates are also soluble in HCl. Therefore if the dolomite and insoluble residue contents are known the calcite content can be estimated by the following equation:

% Calcite = 100% - % Dolomite - % Insoluble residue.

Summary of Compositional Results

Table 2 is a summary of results for the compositional analysis. The elemental magnesium values (column 3) were obtained by INDOT using the EDTA titration method ASTM C602. The x-ray diffraction results for dolomite content (column 2) were obtained at Purdue University. The dolomite contents in column 5 are calculated using the elemental magnesium values in column 3 based on the equation provided previously.

$$\% \text{ Dolomite} = \frac{\%Elemental\ Mg}{0.1318}$$

Table 2. Compositional Analysis

Sample I.D.	X-ray Diff (%) Dol	Elem Mg (%)	Sol-Dol (%) Cal	From Mg (%) Dol	Insol Res <200, (%)	Insol Res <200, (%)
1	98.3	12.4	4.2	94.1	1.23	0.48
2	93.23	12.9	1.4	97.9	0.02	0.69
3	96.87	11.3	11.1	85.7	2.07	1.06
4	82.72	11.3	12.2	85.7	0.11	2
5	90.18	11.6	2.2	88	3.55	6.28
7	98.66	12.9	0.8	97.9	0.21	1.13
8	99.78	13.1	0.4	99.4	0.03	0.2
9	84.53	10.7	11.4	81.2	4.4	3.02
10	95.84	12.6	0.2	95.6	1.81	2.35
11	93.69	12.2	1.1	92.6	2.71	3.59
12	92.35	11.9	2.1	90.3	3.39	4.26
13	76.4	10.1	17.8	76.6	0.78	4.78
15	99.5	12.8	2.4	97.1	0.27	0.22
17	99.65	12.9	1.8	97.9	0.03	0.32
18	97.85				0.94	1.21
19	87.65				4.45	2.21
20	94.1				3.14	2.76

Results for dolomite content in columns 2 and 5 are similar but not identical. Commonly x-ray diffraction values are somewhat higher than those calculated from elemental Mg content. However, sources 4 and 13 are exceptions.

Column 4 indicates the soluble portion of the rock minus the dolomite content in column 5, thus yielding the calcite content. To illustrate, for sample 1, the insoluble residue percentage equals 1.23 plus 0.48 or 1.71%. 100 - 94.1 - 1.7 = 4.2% calcite content. Values for calcite content range from 0.2 to 17.8%, with only one greater than 12.2%. These aggregates obviously represent carbonates with low calcite contents and a high percentage of dolomite.

The total insoluble residue values are obtained by adding column 6 to column 7. Total insoluble residue values range from 0.23 to 9.93% with most values below 7%. This indicates that these aggregates have high carbonate contents containing only minor amounts of impurities.

Laboratory Testing

Results of standard laboratory tests on the eighteen dolomite sources are provided in Table 3. Included are absorption, specific gravity, sulfate soundness loss, Los Angeles Abrasion loss and brine freeze-thaw loss. Note that two samples have elevated values relative to INDOT specifications for Class A stone. Sample 13 has a high sulfate soundness loss (>12%) and sample 4 has a high absorption value (>5%). Brine freeze-thaw loss is provided in the last column. It is discussed in the paragraphs below.

Freeze-Thaw Testing, Brine Solution

This test was performed in accordance with ITM 209-94, "Soundness of Aggregates by Freezing and Thawing in a Brine Solution" and AASHTO T103, "Soundness of Aggregates by Freezing and Thawing". For the coarse aggregate, 1000g from 12.5mm (1/2 in.) to 9.5mm (3/8 in.) and 300g from 9.5mm to 4.75mm (No.4) was used.

Each fraction was completely immersed as an individual sample in a 3% NaCl solution. Total immersion is described in ITM 209-94 as Procedure A, AASHTO, T103. Alternating episodes of freezing and thawing were repeated for 25 cycles in a freezing and thawing chamber in the INDOT laboratory, Division of Materials and Tests. The required freezing period lasted for at least 15 minutes below -26.5°C (-15.5°F) and the thawing period between 21.5°C (70.5°F) and 24.5°C (75.5°F). After completion of the final cycle, each sample was washed and oven dried for 24 hours. Following drying the larger fraction was sieved over the 8.0mm (5/16 in.) sieve and the smaller fraction was sieved over the 4.0mm (No.5) sieve to determine weight loss.

Table 3. Laboratory Test Results

Sample I.D.	Absorp. (%)	Specif G	Sulf Soun % Loss	L.A. Abra % Loss	F-T, Brine % Loss
1	0.79	2.732	0.36	25.53	3.5
2	2.39	2.599		28.53	1.3
3	2.64	2.605	9.57	27.05	4.0
4	6.25	2.39	9.86	29.36	1.5
5	2.2	2.616	7.3	29.19	11.4
7	3.05	2.583	4.79	32.43	8.3
8	2.38	2.594	0.76	31.73	0.6
9	1.21	2.671	5.59	22.47	2.7
10	3.76	2.489	3.29	36.98	2.8
11	2.02	2.632	5.99	24.75	4.7
12	2.74	2.588	2.24	29.5	5.1
13	4	2.48	13.18	30.28	11.9
15	1.03	2.669	2.56	29.76	2.5
17	1	2.718	0.31	25.69	0.6
18	2.48	2.619	3.62	27.25	3.0
19	1.59	2.648	6.45	30.75	13.1
20	2.95	2.59	5.33	28.54	12.9

Results and Discussion

A comparison of petrographic data and laboratory results for the eighteen samples is shown in Table 4. Correlation coefficients were calculated for these data but for simplicity, have not been included here.

Correlation coefficients of brine freeze-thaw loss with the other individual parameters in Table 4 are highest for sulfate soundness loss, insoluble residue >200 and insoluble residue <200. The brine freeze-thaw test is being considered by INDOT as a replacement for the sulfate soundness test and a high correlation has been found between them by INDOT (Hoover, 1994). Based on this work, a proposed maximum allowable loss of 30% for Class A stone has been proposed for the brine freeze-thaw test. This is comparable to a value of 12% maximum allowable loss for the sodium sulfate soundness test.

Note that for the 18 dolomite sources, all brine freeze-thaw losses are well below 30%. The highest values are from 11 to 13% loss. These have fairly high sulfate soundness losses with one above the 12% maximum allowable value. They also have fairly high insoluble residue contents.

Based on the data evaluated in this study it is concluded that the brine freeze thaw test is a good measure of aggregate durability. It correlates reasonably well with sodium sulfate soundness loss and is generally more severe than the soundness test. However, for the excellent quality aggregates evaluated in this research, consistently higher losses for the brine freeze thaw test as compared to sodium sulfate loss do not always occur.

Acid insoluble residue content is directly correlated to brine-freeze thaw loss. A high percentage of clay in carbonate rocks increases the brine freeze thaw loss and the sodium sulfate soundness loss.

Pavement Deterioration Studies

As a final portion, three Indiana pavements were examined that had undergone deterioration soon after being placed in service. Problems developed during the first winter for the new pavement surfaces. It was postulated that deicing salts caused accelerated deterioration of the surface pavements through aggregate damage. An evaluation of these pavements was accomplished in conjunction with the laboratory study.

Deterioration of the pavements occurred by a spalling off of aggregate and bituminous binder to yield elongated depressions in the wheel pathway. Isolated sections of the road were affected in this manner.

Two of the pavements were interstate highways and the third was a dual lane federal highway, all in Indiana. Cores were obtained from all three locations. Examination of the cores indicated that the aggregate consisted of dolomite and blast furnace slag.

Table 4. Comparison, Brine F-T Loss with Petrographic Data

		Petrographic Analysis						
Sample I.D.	F-T Loss (%)	Grn Size (mm)	Median (mm)	Grain Stan Dev	Matrix (%)	Grnd mass (%)	Grains (%)	Voids (%)
1	3.54	0.13	0.085	9.9	21.5	28.4	59.9	11.7
2	1.27	0.079	0.061	3.6	2.8	30.5	59.1	10.4
3	4.006	0.059	0.042	3	21.6	42.8	49.3	7.9
4	1.54	0.194	0.053	12	26.4	41.9	48.4	9.7
5	11.359	0.069	0.052	3.8	30.9	49	45.8	5.2
7	8.325	0.074	0.075	3.7	12.3	27.4	64.4	8.2
8	0.601	0.087	0.065	4.2	5.5	14.8	71.8	13.4
9	2.683	0.066	0.063	4.9	13.2	27.2	63.5	9.3
10	2.805	0.07	0.049	3.9	2.8	28.1	61.7	10.2
11	4.683	0.058	0.041	3.5	7.9	27.8	64.7	7.5
12	5.087	0.082	0.051	4.1	7.5	30.5	64.9	4.6
13	11.93	0.057	0.043	4.9	47.5	64.8	31.2	4
15	2.543	0.116	0.085	7	24.8	36.9	58.5	4.6
17	0.609	0.119	0.114	8.7	9.4	25	68	7
18	3.019	0.063	0.058	3.6	47	60.7	35.8	3.5
19	13.121	0.059	0.046	7.4	7.4	12.6	77.6	9.8
20	12.929	0.085	0.068	4.2	13.2	34.7	55	10.3

Table 4a. Comparison, Brine F-T Loss with Chemical and Physical Data.

		Compositional Analysis						Physical Tests			
Sample I.D.	F-T Loss (%)	X-ray Diff % Dol	Elem Mg (%)	Sib-Dol % Cal	From Mg % Dol	Insol Res >200, (%)	Insol Res <200, (%)	Absorp (%)	Specif G	Sulf Soun % Loss	L.A. Abra % Loss
1	3.54	98.3	12.4	4.2	94.1	1.23	0.48	0.79	2.732	0.36	25.53
2	1.27	93.23	12.9	1.4	97.9	0.02	0.69	2.39	2.599		28.53
3	4.006	96.87	11.3	11.1	85.7	2.07	1.06	2.64	2.605	9.57	27.05
4	1.54	82.72	11.3	12.2	85.7	0.11	2	6.25	2.39	9.86	29.36
5	11.359	90.18	11.6	2.2	88	3.55	6.28	2.2	2.616	7.3	29.19
7	8.325	98.66	12.9	0.8	97.9	0.21	1.13	3.05	2.583	4.79	32.43
8	0.601	99.78	13.1	0.4	99.4	0.03	0.2	2.38	2.594	0.76	31.73
9	2.683	84.53	1.07	11.4	81.2	4.4	3.02	1.21	2.671	5.59	22.47
10	2.805	95.84	12.6	0.2	95.6	1.81	2.35	3.76	2.489	3.29	36.98
11	4.683	93.69	12.2	1.1	92.6	2.71	3.59	2.02	2.632	5.99	24.75
12	5.087	92.35	11.9	2.1	90.3	3.39	4.26	2.74	2.588	2.24	29.5
13	11.93	76.4	10.1	17.8	76.6	0.78	4.78	4	2.48	13.18	30.28
15	2.543	99.5	12.8	2.4	97.1	0.27	0.22	1.03	2.669	2.56	29.76
17	0.609	99.65	12.9	1.8	97.9	0.03	0.32	1	2.718	0.31	25.69
18	3.019	97.85				0.94	1.21	2.48	2.619	3.62	27.25
19	13.121	87.65				4.45	2.21	1.59	2.648	6.45	30.75
20	12.929	94.1				3.14	2.76	2.95	2.59	5.33	28.53

Typically, the dolomtie pieces were more fractured than the slag. Some of the dolomite showed worn grooves. Fractures in the bituminous matrix was observed.

Based on the petrographic examination of the cores it was concluded that there was no clear evidence that the aggregate pieces had performed poorly in the pavement. No significant deterioration of aggregates from weathering was observed. There also was no indication that the slag had caused any deterioration to the dolomite or to the bituminous binder.

Conclusions

From the evaluation of the pavement cores, it was concluded that there was not obvious indication of aggregate deterioration in the distressed pavements. Also the laboratory analysis on the dolomite sources showed that they consisted of sound materials. The overall conclusion is that the pavement deterioration does not appear to be linked to poor aggregate performance, but instead, is due to some other mode of failure.

Acknowledgements

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The Effect of Bedrock Depth on Pavement Response

49th Highway Geology Symposium, Arizona, September 10-14, 1998

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Abstract

Bedrock depth has a great impact on the accuracy of back-calculated layer moduli. There are ways to determine the bedrock depth, such as by augering, GPR, and DCP tests. However, to determine the bedrock depth from FWD data is the easiest and least expensive way. The effects of bedrock depth on FWD responses for an in-service pavement in Jacksboro, Texas are documented. In addition, a comparison between the bedrock depths determined from augering and DCP tests with an algorithm using FWD maximum surface deflection data is discussed in the paper. It was found that the bedrock depth has a great impact on the surface deflection data, particularly for third to sixth sensors. For example, for a given load, there is over 145% increase in surface deflection (r3 and r4) when the bedrock depth increases from 28.5 to 103 inches. For deflections r5 and r6, an increase of over 550% was observed. The time-deflection history patterns were drastically different between shallow and deep bedrock. The shallow bedrock (28.5") allowed little oscillation after the first peak. Also, for shallow bedrock, there is no significant increase in r5 readings when loads were increased from a 6-kip to a 16-kip load. In contrast, an increase of over 215% was observed for a bedrock depth of 103". With 5 different known bedrock depths, the equation in the MODULUS program using FWD maximum surface deflection data gave a reasonable estimate, with a maximum error of 35%.

Introduction

Falling Weight Deflectometer (FWD) data has been used extensively by state highway authorities to determine pavement layer characteristics. The Texas Department of Transportation owns a total of 15 FWD devices. The cost of each FWD device is approximately $160,000. The main purpose of the FWD test is to determine the layer moduli through a backcalculation process. However, when shallow bedrock is present, the accuracy of back-calculated layer moduli is greatly affected. Uddin et al. [1988] stated that the subgrade modulus may be significantly overestimated if a semi-infinite subgrade is falsely assumed, when actual bedrock exists at a shallow depth. Two inexpensive ways to confirm the depth to bedrock are (1) using an extended Dynamic Cone Penetrometer (DCP); and (2) low-frequency Ground Penetrating Radar (GPR). Though the extended DCP test is reasonably accurate, it is a destructive, time-consuming and labor-intensive task. In addition, not many state highway authorities have the ability to conduct low frequency GPR tests. Therefore, it is preferable to assess the bedrock depth from the FWD test data, if possible. Algorithms have been proposed by researchers to estimate the depth to bedrock from the FWD test data. For example, Rohde and Scullion [1990] proposed an algorithm to predict the depth of bedrock from FWD maximum surface deflection data. This algorithm was

incorporated into the most popular back-calculation program MODULUS. Chang, et al. [1992] use the complete time history of deflection from FWD test data to estimate bedrock depth. Roesset, et al, [1995] extended the methodology proposed by Chang, et al. [1992] to account for the effects of stiffness and degree of saturation. This paper attempts to compare the depth to bedrock determined from augering and DCP tests with the proposed algorithm using FWD maximum surface deflection data.

Pavement Section

The section is an in-service pavement and is located on US281 near Jacksboro of the Fort Worth District. US281 is a two-lane highway in each direction. The first asphalt layer of the test section was constructed in 1957. There were four major overlays/rehabilitations that were completed in 1971, 1976, 1986, and 1995. Figure 1 shows the complete pavement history. The last major rehabilitation was done in 1995, with 2 inches of recycled ACP, using the Remix process [Wimsatt et al. 1998]. Prior to that, in 1986, there was a major rehabilitation using 3+ inches of lightweight aggregate ACP. The total asphalt and base thicknesses are approximately 7.25 and 15 inches, respectively.

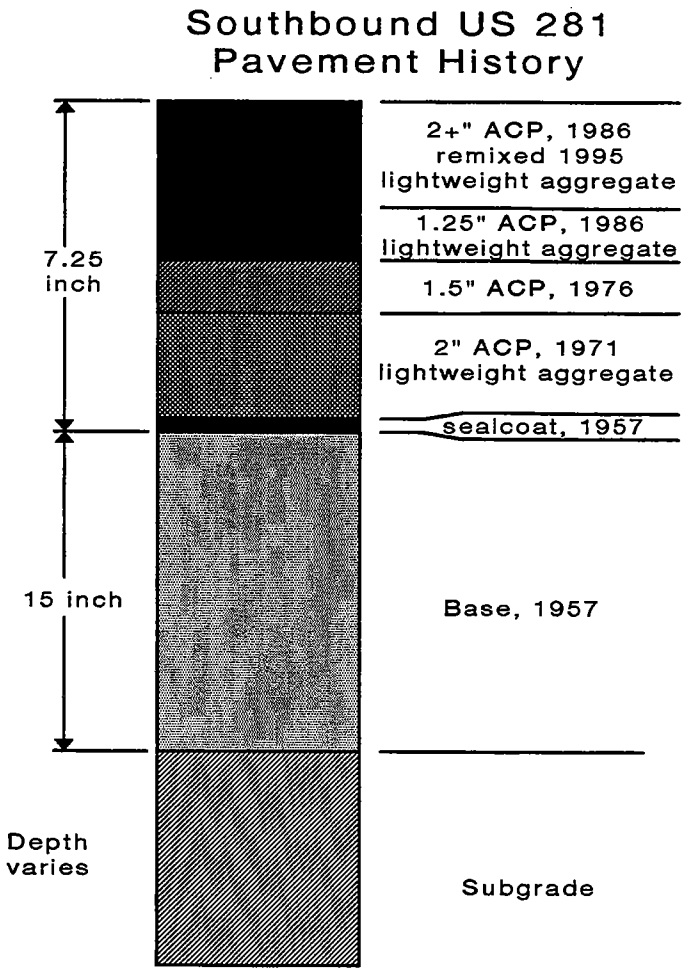

Fig. 1 – Southbound US281 Pavement History

This pavement section provides an opportunity to validate the proposed algorithms for determination of the bedrock information obtained from the FWD tests. From GPR, coring, augering and extended DCP tests, it was found that within a 400 ft section, the bedrock depth varies from about 28.5 to 105 inches (the pavement is located in a fill section). Figure 2 shows the measured (by DCP and auger) bedrock depths, and how the DCP tests and the FWD tests were carried out at the same locations. For this test, the FWD load plate was placed at the DCP locations, with geophones pointed south along the wheel path. Note that the bedrock depth may vary within the length of one FWD (6ft).

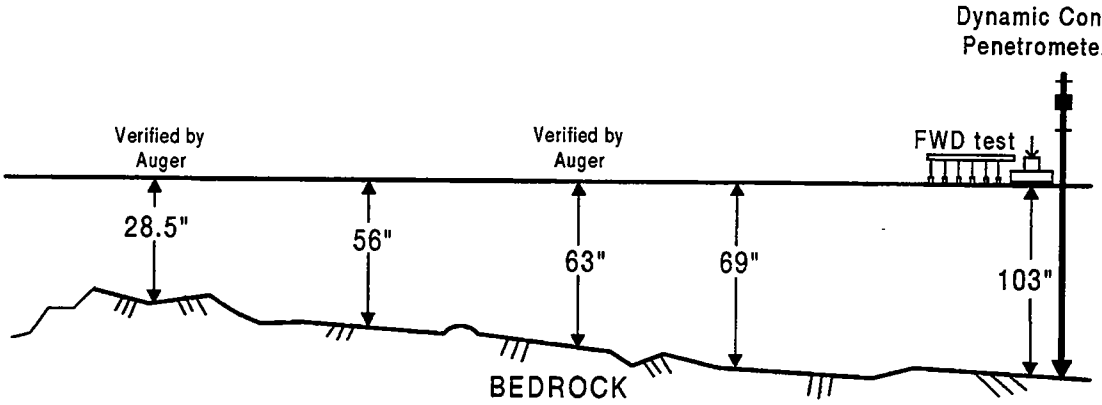

Fig. 2 – Measured Bedrock Depths, FWD Test and DCP

Test Procedure

Several FWD tests were carried out on US 281 near Jacksboro, Texas. An extended DCP (along with an auger to penetrate the ACP) was used to measure the depth to bedrock where the FWD tests were conducted. The extended DCP works the same as an ordinary DCP, except that extra sections can be added to the penetrating rod as it is being driven into the subgrade. After the DCP hit bedrock, it was marked, removed, and all extensions reattached so that the total depth to bedrock could be measured.

In addition, a outside contractor was hired with a regular auger rig to verify two bedrock depths determined from extended DCP. The bedrock is defined as a rigid layer exceeding 1ft thick. The bedrock depth (and thickness) at 28.5 and 63 inches were verified by the drilling operation.

Figure 3 illustrates typical DCP test data. Note that the data starts with the base layer. A hole was drilled through the ACP, and the DCP was seated with 2 blows prior to collection of the penetration data. It is clear that no advancement was made with 30 blows after the DCP hit the bedrock, at approximately 28 inches.

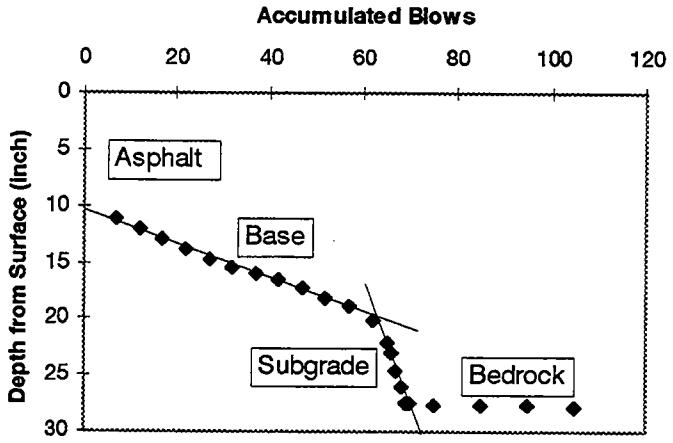

Fig. 3 -- Typical DCP Test Data

The time-history FWD data (deflection vs. time) was recorded every 0.2 milliseconds, as shown in figures 4, 5 and 6. These figures show two typical profiles from the first and seventh FWD geophones, recorded at the center of the load plate and six feet from the center, respectively. Figure 4 shows a deflection profile recorded over shallow (28.5" deep) bedrock, with the FWD load of approximately 16 kips. Figures 5 and 6 shows the r1 and r7 profiles taken from locations with deeper bedrock (63 and103", respectively). It was found that when the bedrock is deep, there is oscillation after 30 milliseconds, and the 7th reading is of a magnitude large enough so that the sensitivity of the geophone does not become a problem at any FWD load. However, when the bedrock is shallow, there is little oscillation, and the 7th reading is too small to be reliable. Several other observations can be made from these figures.

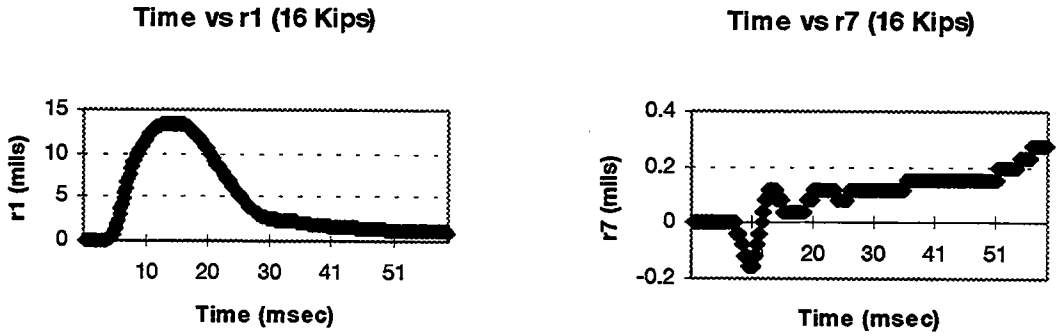

Fig. 4 -- Typical r1 and r7 for shallow (28.5") bedrock

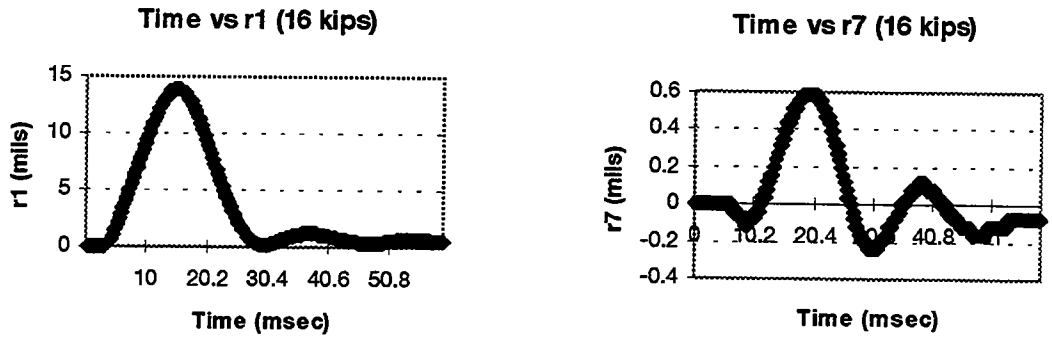

Fig. 5 -- Typical r1 and r7 for Medium (63") bedrock

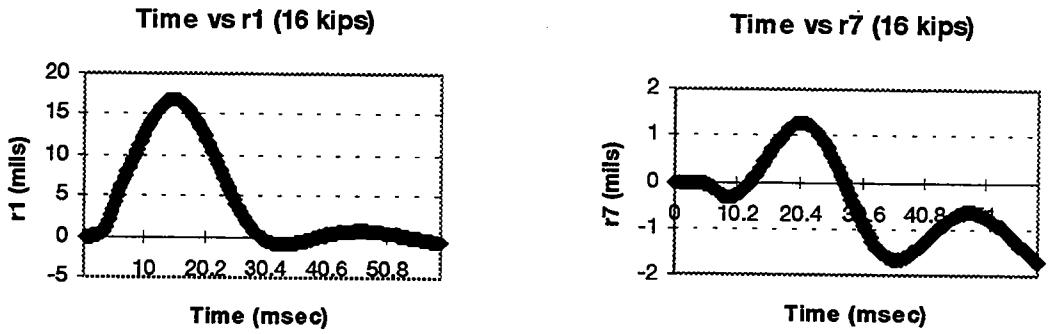

Fig. 6 -- Typical r1 and r7 for deep (103") bedrock

With shallow bedrock, the pavement oscillates upward (negative deflection values) before the deflection peak. For deeper bedrock (as in figure 6), the rebound occurs after the deflection peak. Second, note the small magnitude of the deflections in figure 4. The deflection peaks at only 0.12 mils. The data seems to make discrete jumps of .04 mils, the smallest change in deflection the FWD will detect. Third, when testing over shallow bedrock, the FWD may record a false maximum deflection 6 feet from the load center. The deflection seems to drift upward after the surface wave has gone. For this drop, the maximum deflection recorded by the FWD is nearly 0.3 mils, and probably would have been higher had deflection been recorded for a longer time.

Figure 7 presents comparison of deflection profiles (from r4 through r7) for the 4th drop load normalized to 15 kips. The curvature is higher for deeper bedrock, as expected. The deflection profile for bedrock at 28.5" has higher r7 than r6 reading, which is abnormal. This is believed due to reflection. An empirical equation to estimate the depth to bedrock may include deflections as far from the load as 5 feet, (r6) which remain sensitive to bedrock depth. The effects of bedrock depth on r4 and r5 readings under various levels of load are presented in figure 8. It was observed that the deeper the bedrock, the greater the influence on r4 and r5 readings. Specifically, the increasing loads caused no significant increase in r5 readings when bedrock was shallow (28.5" from surface). Thus, it reconfirms that the

responses of outer sensors are more closely related to the subgrade properties. When the rigid layer is encountered at a shallow depth, the higher stiffness will prevent a higher load from significantly increasing the deflections r4 and r5.

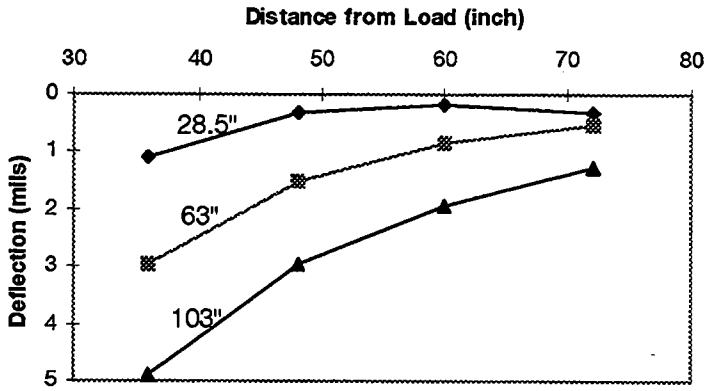

Fig. 7 -- Comparison of Deflection Profiles for Load Level Normalized to 15 kips

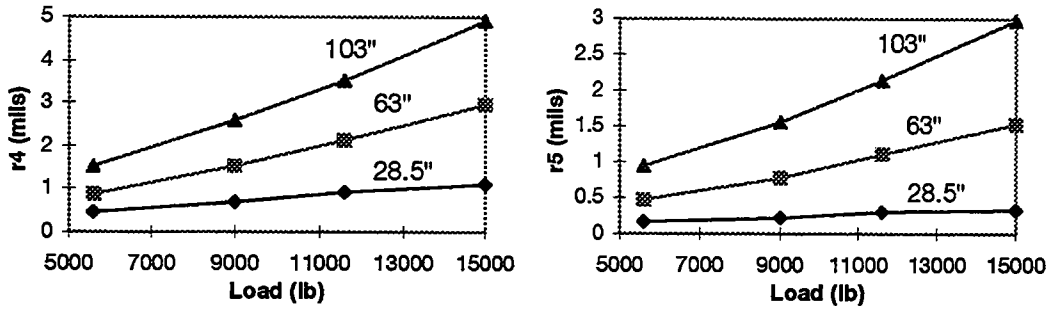

Fig. 8 -- Comparison of r4 and r5 Deflections at Various Bedrock Depths

Table 1 shows the FWD deflections recorded 3 feet (r4) from the load center and normalized to four loads. Deeper bedrock generally leads to higher r4 deflections because of lower stiffness in the subgrade layer . In particular, the r4 deflection ratio between 103" and 28.5" increases from 2.37 (at 5600 lb.) 3.43 (at 15000) because of more stress penetration into subgrade from the higher level of loads.

Table 1 – Normalized r4 deflections for various bedrock depths

bedrock depth\load	5600 lb	9000 lb	11600 lb	15000 lb
shallow (28.5")	0.46	0.71	0.91	1.10
moderate (63")	0.83	1.45	2.03	2.79
deep (103")	1.55	2.61	3.53	4.90
% increase, shallow to deep	238%	265%	289%	343%

Table 2 compares the deflections r1 through r7 recorded over shallow and deep bedrock. It is observed from Table 2 that the bedrock depth has a great impact on the surface deflection data, particularly for the third to sixth sensors' data . For example, for a given load, there is over 145% increase in surface deflection when the bedrock depth increases from 28.5 to 103 inches. For r5 and r6, an increase of over 550% was observed. For bedrock at 28.5", the r7 deflection is higher than the r6 deflection, which is abnormal. Note that the deflections closer to the load (r1 and r2) increase by a much smaller percentage than r3 through r6, because of the same properties in asphalt and base layers. The major differences from r3 to r6 are due to the effects of bedrock depth, as expected.

Table 2 – Comparison of normalized deflections (11600 lb.) for shallow and deep bedrock

bedrock depth\deflection	r1 (mils)	r2	r3	r4	r5	r6	r7
shallow (28.5")	9.80	5.60	2.38	0.91	0.29	0.19	0.32
deep (103")	12.40	9.39	5.89	3.53	2.13	1.32	0.84
% increase, shallow to deep	27%	68%	148%	289%	628%	576%	162%

Algorithm to Predicted Bedrock Depth

The FWD surface deflection is the result of the deformation of the pavement layer in the stress zone. In other words, if the pavement layer encounters no stress or the pavement layer is a rigid layer, there should be no deformation contribution from that layer. Rohde and Scullion [1990] proposed an algorithm based on the hypothesis that the position of zero surface deflection will be strongly related to the depth in the pavement where no deflection occurs (i.e., where it encounters the rigid layer). In addition, if a rigid layer was encountered at some depth, no surface deflection will occur beyond the offset where the stress zone and the stiff layer intercept. Rohde and Scullion [1990] presented several formulas which estimate bedrock depth. The formula given for ACP greater than six inches thick is as follows:

$$1/B = 0.0409 + 0.5669\, d_o + 3.0137\, d_o^2 + 0.0033\, BDI - 0.0665 \log (BCI)$$

where:

B = the estimated depth to bedrock (ft)

d_o = intercept determined by the following steps:

1. plot deflection in mils on the Y axis vs. the inverse of the offset (distance from the load center, in feet) on the X axis
2. extrapolate the steepest section of the curve down to the X axis (the units of d_o are 1/ft)

$BDI = r_2 - r_3$ (Base Damage Index)

$BCI = r_3 - r_4$ (Base Curvature Index)

r_i = surface deflection (in mils) normalized to a 9000 lb. load at an offset i-1 feet.

The bedrock depths compared to Rohde and Scullion's [1990] formula are given in Table 3. The formula proposed by Rohde and Scullion [1990] is used in MODULUS 4.0. Five different bedrock depths were used in the comparison (28.5, 56, 63, 69, and 103 inches). In view of Table 3, this method overestimates bedrock depth which is relatively shallow, and underestimates deeper bedrock depths. The maximum error was found to be 35%. The authors believe the degree of accuracy may be increased if the parameters r5 and r6 were included in the equation. Research is in progress to incorporate r5 and r6 in the equation. It can be concluded that with 5 different known bedrock depths, the equation in MODULUS for ACP thickness greater than six inches gives reasonable estimates with a maximum error of 35%.

Table 3 – Comparison of estimated and measured bedrock depths

Actual depth	Predicted Depth	% Error
28.5"	35.8"	-12%
56"	36.6"	35%
63"	53.7"	15%
69"	60.0"	13%
103"	77.5"	25%

Conclusion

The results presented in this paper document the effects of bedrock depth on FWD responses of an in-service pavement in Jacksboro, Texas. In addition, a comparison between the bedrock depths determined from augering and DCP tests with an algorithm using FWD maximum surface deflection data is discussed. The conclusions are given as follows:

1. A pavement section with various bedrock depths is successfully identified. This pavement section could be used to verify and calibrate models proposed by researchers.
2. The bedrock depth has a great impact on the surface deflection data, particularly for third to sixth sensors' data . For example, for a given load, there is over 145% increase in surface deflection when the bedrock depth increases from 28.5 to 103 inches. For r5 and r6, an increase over 550% was observed.
3. The time-deflection history patterns were drastically different between shallow and deep bedrock. The shallow bedrock (28.5") caused no oscillation after first peak.
4. There is no significant increase in r5 readings for shallow bedrock when load is increased from 6kips to 16 kips. In contrast, an increase of over 215% was observed for bedrock depth of 103".
5. Based on 5 different known bedrock depths, the equation used in MODULUS is found to give reasonable estimates, with a maximum error of 35%.

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Influence of Bedrock Type and Depth on Highway Deflections

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ABSTRACT

Geologic maps depict the distribution of rock types at and near the land surface. Falling-weight deflectometer (FWD) data collected on Texas highways appear to be correlated to bedrock type, particularly at the outermost sensors. To examine this apparent bedrock influence, we compared FWD data with mapped geologic units from six roadway segments in four major physiographic regions of Texas. This analysis revealed statistical differences in FWD response among regions that are likely to be related to systematic differences in either bedrock depths or physical properties of geologic units that range from Precambrian to Holocene in age and include many different types of sedimentary, igneous, and metamorphic rocks. At the W7 detector (6 ft offset), average normalized deflections are highest for areas where roads are underlain by siliciclastic sedimentary rocks (sandstones, mudstones, and shales) and unconsolidated alluvial sediments. Lowest normalized W7 deflections are measured in areas underlain by Precambrian igneous and metamorphic rocks that include granites, schists, and gneisses and in areas underlain by chemically precipitated sedimentary rocks such as limestone.

Discrimination of specific rock types is better obtained from ratios calculated from deflections measured at the W2 (1-ft offset) and W7 detectors than from W7 deflections alone. W2:W7 ratios vary regionally somewhat but are highest for rigid rock types such as granites, gneisses, and schists (ratios between 17 and 40); are intermediate for limestones (between 10 and 27); and are relatively low for sandstones, mudstones, and unconsolidated sediments (between 6 and 14). These results suggest that (1) geologic maps can be used to estimate outermost FWD sensor response for highway segments and (2) rock type might be predicted from FWD data, allowing the FWD to be used in applications such as geologic mapping and sinkhole detection.

Geologic maps can also be used to qualitatively estimate bedrock depth, a critical parameter in FWD data analysis. Field experiments demonstrate that we can modify the FWD to collect seismic refraction data along with conventional FWD data, to improve the estimate of bedrock depth. Ongoing development of this hybrid technique will test its applicability and practicality in representative geologic and highway settings.

INTRODUCTION

The purposes of this study are to examine whether (1) geologic maps can be used to interpret roadway deflection data collected by Falling Weight Deflectometers (FWD) and (2) seismic refraction data, perhaps collected along with FWD data, can be used to estimate bedrock depths accurately enough to improve FWD analyses.

Trailer-mounted FWD's consist of a falling weight and seven calibrated detectors at distances of 0, 1, 2, 3, 4, 5, and 6 ft from the falling weight. The height of the weight drop can be selected to

produce downward-directed seismic impulses of varying strength. The vertical detectors, in contact with the pavement as the weight falls, measure pavement deflection following weight impact. The most commonly used FWD data are the maximum deflections at each detector, normalized for drop load. In general, deflections measured close to the source are most affected by pavement conditions, and deflections measured at the farthest offsets are more affected by deeper layers such as fill, soil, and bedrock. Physical properties of roadway layers that can be calculated from FWD data also depend on depth to bedrock (depth to "rigid" layer), which is generally not known. Rather than drilling boreholes to measure bedrock depth directly, we wish to determine the precision with which geological and geomorphological information can be used to estimate bedrock depths. We also want to examine whether geophysical methods, such as seismic refraction, can be adapted to rapidly and accurately estimate bedrock depths at an FWD site.

There is an empirical relationship between rock type and FWD deflections, particularly at the longest offsets. Average W7 deflections (6-ft source-to-detector distance) calculated on a county-by-county basis (fig. 1) show a strong resemblance to a simplified geologic map of the state (fig. 2). Largest W7 deflections are observed along the Texas coast and in the Panhandle, where the geologic units are relatively young. Smallest W7 deflections are observed in central Texas, where old igneous and metamorphic rocks and younger limestones are mapped. Outcrop trends of individual geologic units match average W7 deflection trends visible across many counties, including (1) the increased average deflections in east Texas on the Miocene Fleming and Oakville Formations and the Pliocene Willis Formation; (2) low deflections on the Cretaceous Trinity, Fredericksburg, and Lower Washita Groups in central Texas; and (3) increased deflections that follow the Cretaceous Austin, Eagle Ford, Woodbine, Upper Washita, Navarro, and Taylor Groups in northeast Texas (figs. 1 and 2).

Physiographic Regions and Bedrock Types

The relationship between far-offset FWD data and geologic units supports the subdivision of Texas into regions that have similar FWD response. Many earth scientists have recognized physiographic regions that reflect differences in elevation, topography, geologic structure, and bedrock types (Wermund, 1996). These seven principal physiographic regions (Gulf Coastal Plains, Edwards Plateau, Central Texas Uplift, Grand Prairie, Basin and Range, North-Central Plains, and High Plains) provide a framework for grouping rock types that influence FWD response.

Bedrock types differ in each of the seven principal physiographic regions. On the Gulf Coastal Plains, unconsolidated and semiconsolidated sands, silts, and clays deposited along rivers and shorelines during the Cenozoic Era (within the last 66 million years [Ma]) form relatively weak highway substrates. Relatively young bedrock is also found in the High Plains of the Texas Panhandle, where unconsolidated to moderately cemented eolian (windblown) and alluvial (river-borne) sand and silt formed the Blackwater Draw Formation during the Quaternary Period (<2 Ma) and the Ogallala Formation during the Miocene to Pliocene Periods (24 to 2 Ma). Limestone and dolomite deposited during the Cretaceous Period (144 to 66 Ma) underlie the Edwards Plateau in central Texas, forming strong substrates that are resistant to erosion. Sandier, calcareous deposits of similar age underlie the Grand Prairie, the northern extension of the Edwards Plateau. Westward-dipping limestone, sandstone, and shale dating to the late Paleozoic Era (320 to 245 Ma) are found in the North-Central Plains. The oldest rocks in Texas are found in the Central Texas Uplift and the Basin and Range region of west Texas. In the Central Texas Uplift, mechanically strong, late Precambrian Era (2,000 to 1,200 Ma) igneous and metamorphic rocks and Paleozoic Era (570 to 245 Ma) sandstone, limestone, and shale crop out. In the Basin and Range, faulting formed a series of basins

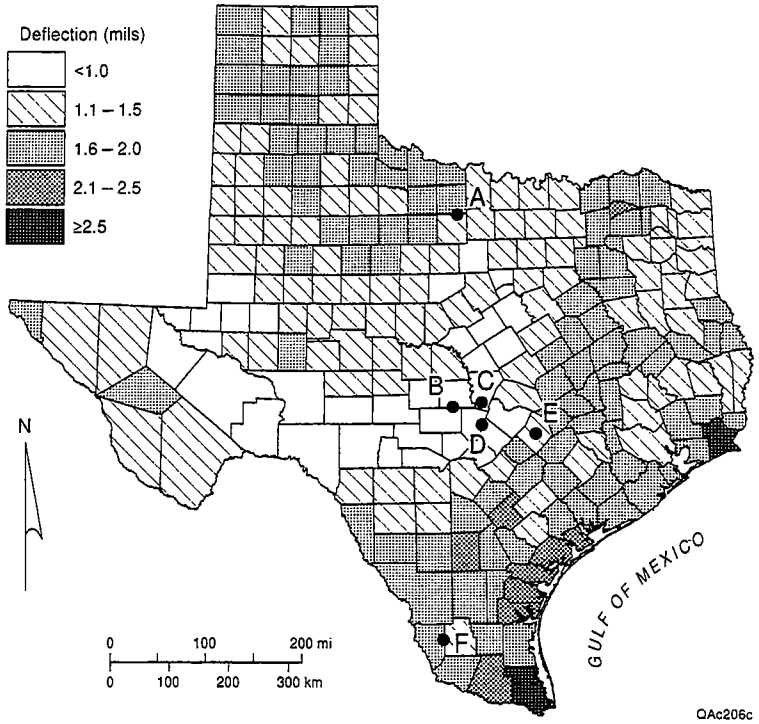

Figure 1. Average roadway deflection by county measured at Falling Weight Deflectometer detector W7. Also shown are locations of highway segments at sites A through F.

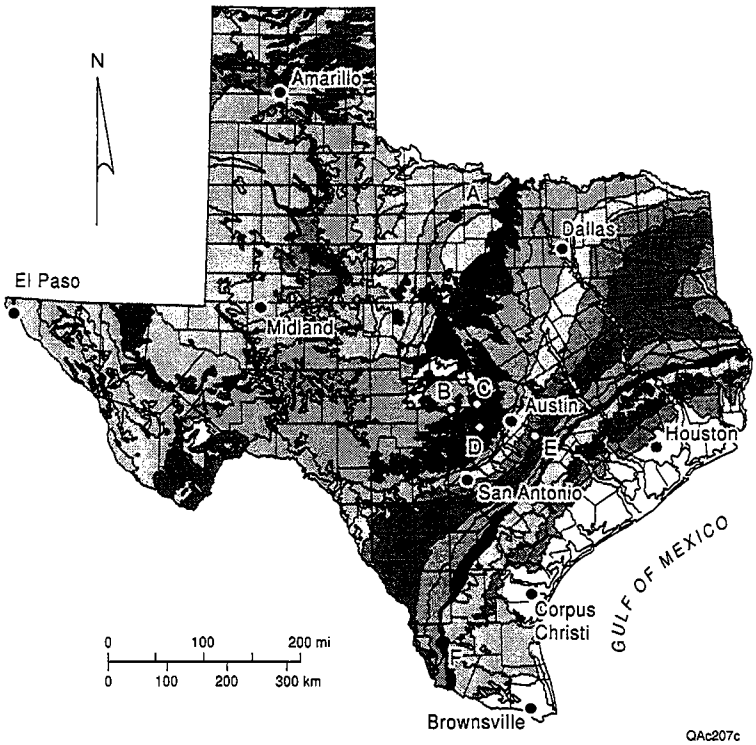

Figure 2. Simplified geologic map of Texas. Adapted from Bureau of Economic Geology (1992).

and ranges. The ranges, cored by strong igneous, metamorphic, and sedimentary rocks, are separated by basins that have been filled by younger sedimentary deposits that are generally weaker than the range-forming rocks.

More detailed information on the distribution of geologic units is obtained from geologic maps produced at various scales. For a statewide study, the most useful map series is the Geologic Atlas of Texas. This series consists of 38 maps that cover the entire state at a scale of 1:250,000 and have been published over the last several decades by the Bureau of Economic Geology. Soil surveys, published by the Soil Conservation Service of the U.S. Department of Agriculture, exist for most Texas counties. The information on soil distribution, grain size, soil depth, and surface slope contained in the maps and tables that make up these surveys, more detailed than that shown on most geologic maps, may also be useful in the interpretation of FWD data. Soil maps, published at a scale of 1:20,000, show many more units and subdivisions than do most geologic maps but are difficult to place in a statewide or regional context that would lend itself well to FWD analysis.

METHODS

Our approach to understanding the relationship between FWD response and bedrock type and depth was to examine in detail several road segments in different physiographic regions. The six highway segments analyzed are located in the North-Central Plains (one), the Central Texas Uplift (two), the Edwards Plateau (one), and the southern (one) and interior (one) Gulf Coastal Plains. We are also acquiring seismic refraction data along selected highway segments to investigate the effectiveness of this method in determining bedrock depths to maximum depths of about 20 ft.

Bedrock Type and FWD Response

To determine whether there is a quantifiable relationship between bedrock and FWD response, we (1) normalized FWD response to a common 9,000-lb load, (2) plotted FWD locations on U.S. Geological Survey 7.5-minute quadrangle maps, (3) determined elevations for each FWD site, and (4) determined from 1:250,000-scale geologic maps (Brown and others, 1972, 1976; Proctor and others, 1974; Brewton and others, 1976; Barnes, 1981; Hentz and Brown, 1987) which geologic unit underlies the highway at each FWD site. These data were entered into a data base that includes highway name, reference marker, geologic unit, elevation, and normalized deflections for each FWD site.

We then analyzed the data base to understand better how bedrock influences FWD response. Plots of elevation, rock type, and deflection versus distance along the highway show how deflections relate to different geologic units beneath the highway and to changes in elevation and relief. When the deflection data are sorted by rock type, we can calculate the average deflection bowl for a given bedrock type, determine how much the deflection bowls vary, and decide whether bedrock types have distinctive deflection bowls. If deflection bowls have different slopes, we can calculate deflection ratios for near- and far-offset detectors to further discriminate rock types.

Seismic Refraction

Seismic refraction is a relatively simple and well-established geophysical method (Telford and others, 1976; Milsom, 1989) to determine compressional-wave velocity structure at depths as shallow as 1 ft to as deep as several miles. In the shallow subsurface, seismic refraction is commonly used to measure depth to the water table or to bedrock (rigid layer beneath soil and weathered bedrock). Compressional-wave velocities increase downward in most geologic settings, where relatively dry

soil (compressional-wave velocities ranging from 1,000 to 2,300 ft/s) is underlain by saturated soil at the water table (compressional velocities of about 4,900 ft/s) or by unweathered bedrock (compressional velocities commonly more than 6,500 ft/s, depending on rock type). These typically abrupt, downward increases in wave velocity refract surface-generated seismic waves back to the surface, where they are detected by motion sensors (geophones). The time delay between seismic source impact and first seismic arrivals at known geophone distances allows us to calculate compressional velocities and thicknesses of near-surface layers, which in turn allows us to estimate a depth to the water table or to bedrock. In general, exploration depth increases with distance between the source and detector. For shallow investigations, the detector spread should extend from within a short distance of the source to a maximum distance of four or more times the desired maximum exploration depth. This allows us to measure enough arrivals of both the direct wave (traveling in the surface layer only) and the critically refracted wave (traveling along the water table or at the interface between the surface layer and bedrock) to calculate accurate compressional-wave velocities for these layers.

We recorded seismic refraction data using 48 40-Hz geophones, a 48-channel seismograph, and several seismic sources (including the FWD) at the J. J. Pickle Research Campus at The University of Texas at Austin and at the Mobile Load Simulator site in north-central Texas. Spread length, geophone spacing, and seismic source selection depend on target depths, ambient seismic noise, ground conditions, and desired lateral resolution. For typical highway settings, a sledge hammer, a modified soil-probe hammer, or the FWD itself can be suitable sources. Spread lengths were 47 ft (48 geophones, each 1 ft apart). We picked first compressional-wave arrivals using the Seismic Processing Workshop software package. We calculated true seismic velocities, layer thicknesses, and apparent dip angles using the slope-intercept method (Palmer, 1986).

RELATIONSHIP BETWEEN BEDROCK TYPE AND FWD RESPONSE

Bedrock units are one of three major rock types: igneous (solidified from molten rock), sedimentary (particles deposited by wind, water, or gravity flow), and metamorphic (plastically deformed igneous or sedimentary rock). Physical properties of these rock types, including density, wave-propagation velocities, and elastic parameters, have been shown in numerous field and laboratory experiments to vary widely. For geologic maps to be useful in the interpretation of FWD data, deflections should show some relationship to mapped rock type. From the similarity of county deflection averages to a simplified map of Texas, we infer that bedrock type and FWD response are related (figs. 1 and 2). Whether this relationship is caused by a similarity in bedrock depths of a given bedrock type or by a similarity in physical properties of a given bedrock type is unknown.

Although data were examined for all six sites in four physiographic regions, we present in detail only those from site B in the Central Texas Uplift. This region has a range in rock types that is similar to that observed over the entire state.

Site B (Central Texas Uplift)

The Central Texas Uplift is underlain by Precambrian igneous and metamorphic rocks, Paleozoic and Mesozoic sedimentary rocks, and unconsolidated Quaternary sediments. County average deflections in this region of typically rigid bedrock types are among the lowest in Texas, ranging from less than 1 to 1.5 mils (fig. 1). Site B is a segment of State Highway 16 that begins in southern Llano County and extends 38 mi southward into Gillespie County. It is underlain mostly by Precambrian metamorphic (Packsaddle Schist and Valley Spring Gneiss) and igneous

(Town Mountain Granite) rocks and the Cretaceous Hensell Sand (fig. 3). A few occurrences of younger granites, Cambrian Hickory Sandstone, Cretaceous Fort Terrett Limestone, and Quaternary stream deposits are mapped along the highway (fig. 3). Younger geologic units are found at the relatively high elevations on the south part of the segment; older igneous and metamorphic rocks are found at relatively low elevations on the north part of the segment (fig. 3). Average W7 deflections for both Llano and Gillespie Counties are less than 1.0 mils (fig. 1), reflecting the abundance of rigid bedrock in the Central Texas Uplift.

We analyzed FWD data from 69 locations along this highway segment. This segment has the highest average W1 deflection of any of the six study sites (34.7 mils) but the third-lowest average W7 deflection (0.93 mils). When the data are grouped by geologic unit (fig. 4), they show that (a) sites having large near-offset deflections generally also have large far-offset deflections and (b) there is more variation within a geologic unit than there is between average deflections of each rock type. The average W7 deflections calculated for the Hensell Sand are higher than those for the Town Mountain Granite and the Valley Spring Gneiss, but the range in individual W7 deflections observed for these rock types is similar: 0.4 to 2 mils for the Hensell Sand, 0.3 to 1.1 mils for the Town Mountain Granite, and 0.3 to 1.8 mils for the Valley Spring Gneiss (fig. 4).

Statistically, Precambrian igneous and metamorphic rocks have low average W7 deflections that range from 0.53 mils for younger granites to 0.87 mils for the Packsaddle Schist (figs. 5 and 6a). Higher average W7 deflections, ranging from 1.11 to 1.24 mils, are calculated for Cretaceous and

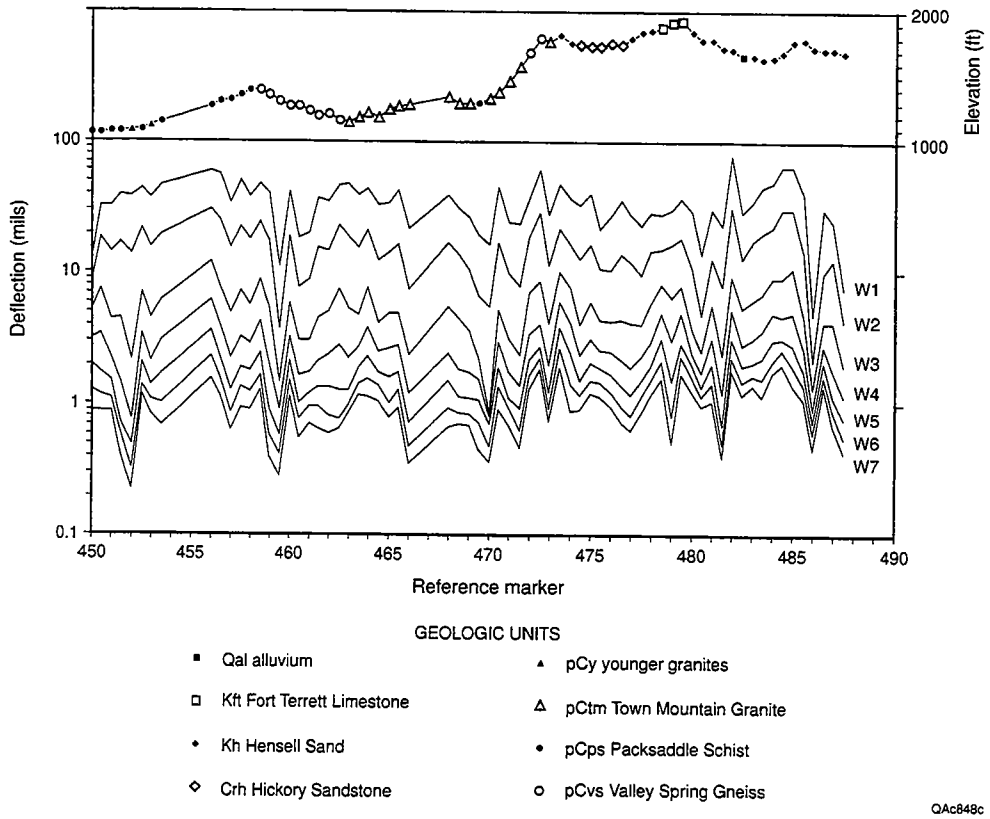

Figure 3. Geologic units, elevation, and W1 through W7 deflection along State Highway 16, Llano and Gillespie Counties, Central Texas Uplift.

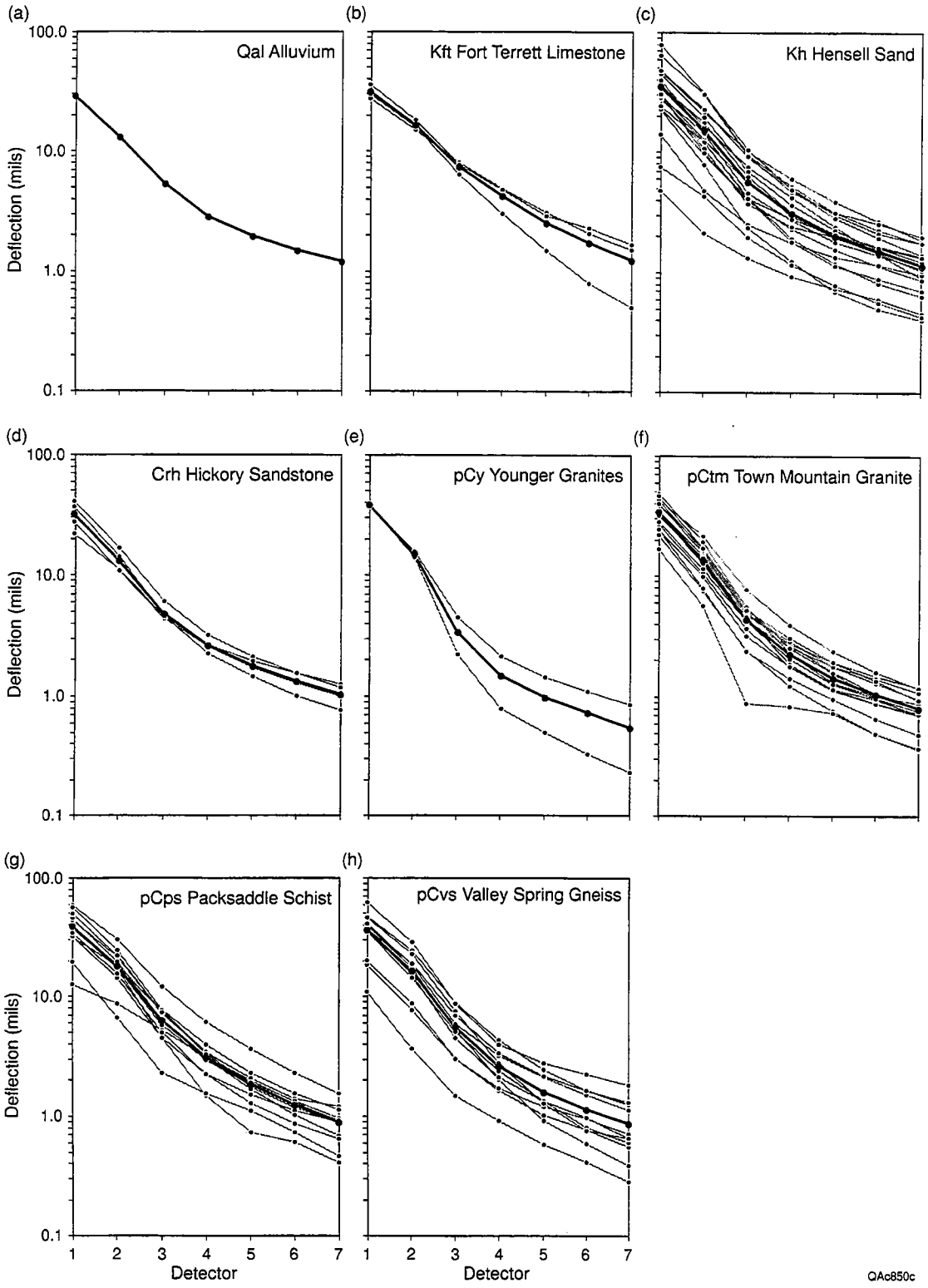

Figure 4. Average (solid line) and individual (shaded line) deflections for formations mapped along State Highway 16.

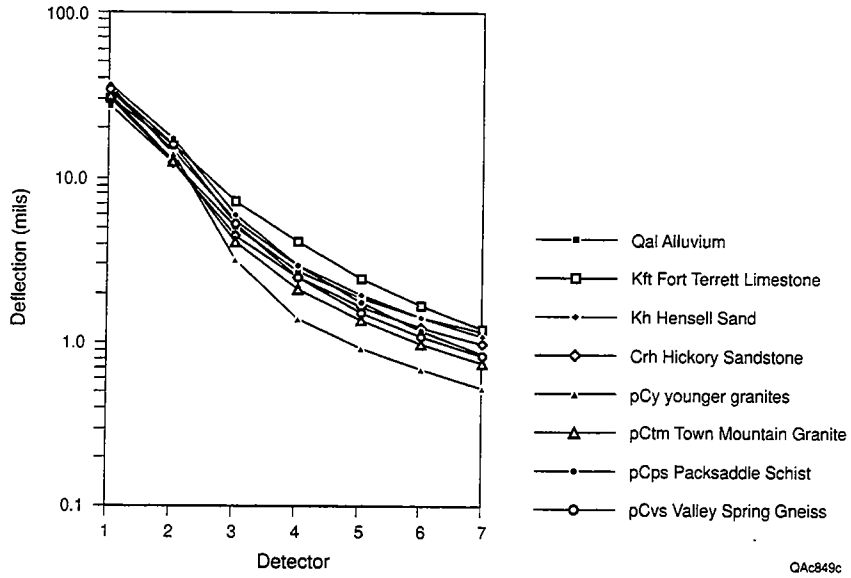

Figure 5. Average deflections for rock units mapped along State Highway 16.

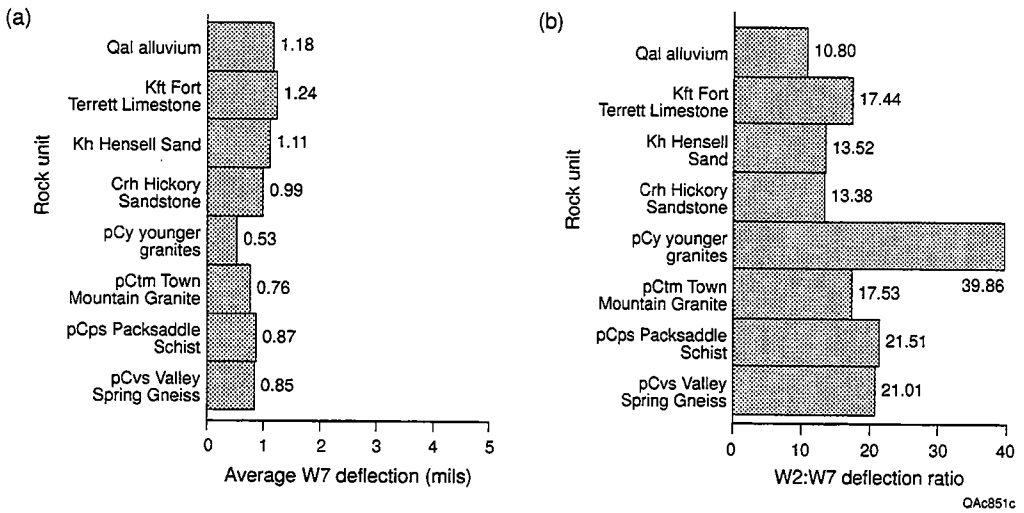

Figure 6. Average W7 deflection (a) and W2:W7 deflection ratio (b) by rock type along State Highway 16.

younger sedimentary units. W7 averages for geologic units at site B are similar to those calculated for geologic units in the North-Central Plains.

The W2:W7 ratio provides better discrimination of rock type for this highway segment. Very high ratios are calculated for the rigid rock units (fig. 6b): between 17 and 40 for granites, metamorphic rocks, and the Fort Terrett Limestone. Lower ratios are calculated for the younger sedimentary units, ranging from 13 to 14 for the Hickory Sandstone and Hensell Sand and 11 for Quaternary stream deposits. Ratios for the most common units encountered along State Highway 16 are higher than those observed at the North-Central Plains and Gulf Coastal Plains sites and are similar to ratios calculated for the Edwards Plateau site.

Deflection by Rock Type

Average FWD response for principal rock types can be examined by grouping individual geologic units into these basic types regardless of physiographic region. Principal rock types mapped in the Central Texas Uplift, North-Central Plains, Edwards Plateau, and Gulf Coastal Plains regions are (1) unconsolidated sedimentary deposits, (2) sandstones, (3) mudstones, (4) limestones, (5) granites, and (6) metamorphic gneisses and schists. Each geologic unit mapped at FWD sites along the six highway segments examined can be classified as one of these geologic types.

W7 averages and W2:W7 ratios for each individual rock type define ranges of observed values for the principal rock types (fig. 7). FWD response along roadways built over igneous and metamorphic rock types such as granite, gneiss, and schist have low average W7 deflections (0.5 to 0.9 mils) and high to very high W2:W7 ratios (more than 17) compared with that of other rock types. Siliciclastic sedimentary units such as sandstone and mudstone have similarly low W2:W7 ratios (between 9 and 15) but sandstone tends to have smaller average W7 deflections than does mudstone. Unconsolidated sediments, including Quaternary alluvium and older, uncemented sand and gravel, exhibit a wide range of W7 deflections (0.6 to 2.3 mils), along with very low W2:W7 ratios (6 to 14) that are similar to those observed for lithified siliciclastic rocks. Although sandstone, mudstone, and unconsolidated sedimentary deposits have similar W2:W7 ratios, average W7 deflections, which are generally highest for unconsolidated deposits and lowest for sandstones, provide some basis for discriminating between these types.

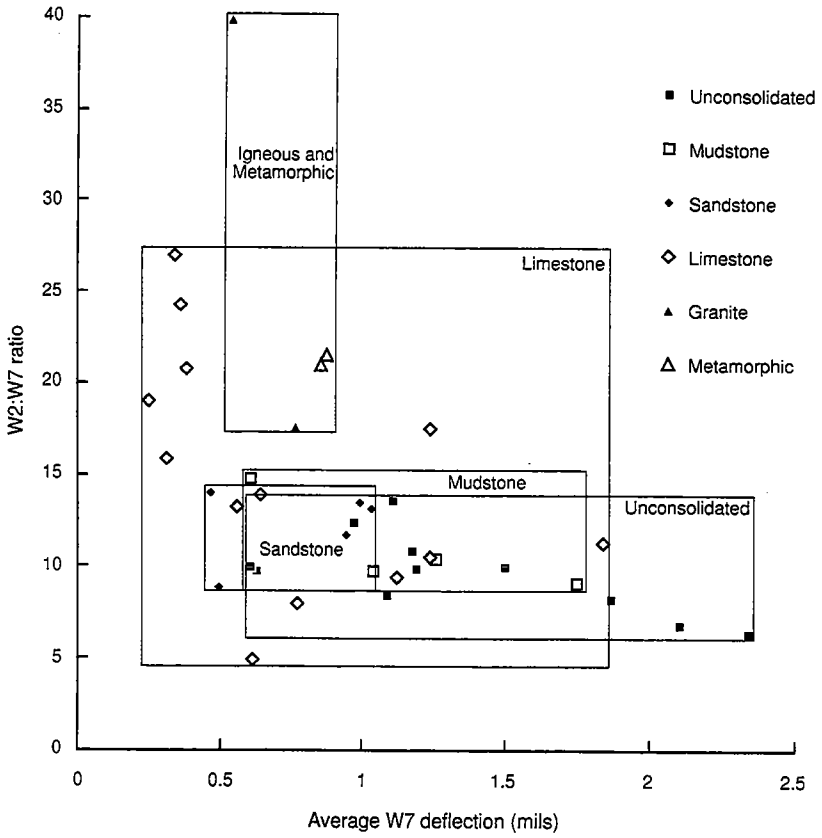

Figure 7. Average W7 deflection and W2:W7 deflection ratio for individual rock types mapped at the six test sites.

FWD response along highways underlain by chemically precipitated limestones is perhaps the most variable of all the principal rock types (fig. 7). Some limestone units have the lowest observed W7 deflections (below 0.5 mil) and high W2:W7 ratios (between 15 and 27); other limestone units have W7 deflections as much as 1.8 mils and W2:W7 ratios as low as 5, values that are comparable to those of siliciclastic units. These higher W7 deflections and lower W2:W7 ratios probably indicate common clay-rich units within larger sections of limestone, weathered limestone, or greater depth to bedrock.

Seismic Refraction Testing

FWD sensors can record a time series of pavement movement and thus might be used to detect critically refracted arrivals if the source-to-sensor lengths were extended beyond the current 6-ft limit. We are developing and testing a hybrid seismic refraction–FWD system by deploying a longer spread of geophones behind an FWD, recording FWD data and using the same impulse to record seismic refraction data along the highway, and calculating bedrock depths for the FWD site from the seismic refraction data.

We recorded seismic impulses generated by an FWD at the J. J. Pickle Research Campus of The University of Texas at Austin (fig. 8). This 60-ms shot record, acquired by 48 geophones at distances between 1 and 47 ft from the FWD source, shows clear direct and critically refracted compressional-wave arrivals that are needed to estimate depth to rigid layer. Additional arrivals visible in the

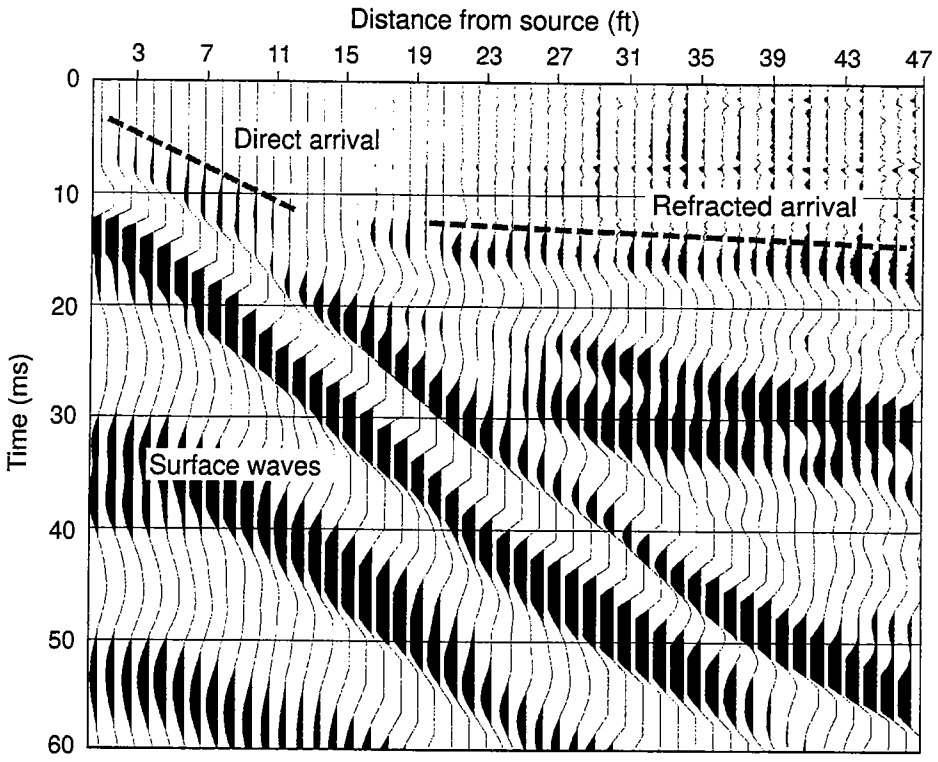

Figure 8. Seismic response recorded with 48-geophone spread using FWD source. Visible phases include the direct arrival, a critically refracted arrival from the underlying rigid layer, long-wavelength, low-frequency surface waves, and reflected compressional waves. Data recorded on a flexible-pavement road on the Pickle Research Campus.

seismic record include long-wavelength, high-amplitude surface waves and reflected waves between 20 and 47 ft and 20 to 40 ms arrival time. Spectral analyses of ground motion detected by near-source geophones indicate that the FWD produces an impulse with frequencies between about 20 and 200 Hz, which is a useful range for seismic refraction and shallow reflection investigations.

By moving the source to the opposite end of the geophone spread, we acquired reversed seismic refraction data that allowed true seismic velocities and bedrock depths to be calculated. At the Pickle site, near-surface velocities calculated from the direct arrival are 1,040 ft/s. This velocity is within the range expected for the dry fill and roadbase below a thin layer of asphalt and gravel pavement. The velocity of the critically refracted wave is 9,900 ft/s, a value that is typical of the Cretaceous limestone that crops out nearby. Refraction data yield a depth-to-rigid-layer estimate of 1.7 m. This rigid depth can be used to analyze the FWD deflection data.

DISCUSSION AND IMPLEMENTATION

The relationship between bedrock and FWD response at long offsets that is suggested by average deflections by county is supported by more detailed analysis of FWD response along individual highway segments in four of the major physiographic regions of Texas. Although we have shown that rigid rock types such as limestones and igneous and metamorphic rocks have statistically lower deflections at the outermost detector than do less rigid sedimentary rock types, there is enough scatter in FWD deflections for a given rock type to make it difficult to predict rock type reliably from the outermost deflections alone. Better discrimination of rock types is obtained by comparing deflection ratios that tend to normalize road-construction differences; in this study, W2:W7 ratios were highest for rigid rock types and lowest for materials with low stiffness, such as sandstones, mudstones, and unconsolidated sediments.

It is unclear whether lower W7 deflections and higher W2:W7 ratios for highways over rigid rocks versus highways over soft rocks are due to differences in material properties of the geologic units or to systematic differences in depth to bedrock among rock types. Remaining work will help distinguish between these potential explanations. We will be collecting FWD data and seismic refraction measurements at sites that represent a variety of geologic units and roadway types. These data will allow us to directly compare observed deflection, depth to bedrock, and mapped geologic unit and determine the relative influence of bedrock rigidity and bedrock depth on FWD response. It may be that bedrock rigidity accounts for the differences in average deflection calculated for the outermost FWD detector and that site-to-site variations in bedrock depth over a given rock type account for the large variance observed for that rock type along a roadway.

Despite the fact that we have not yet established whether bedrock rigidity or bedrock depth is more important in determining bedrock influence on FWD response, some bedrock influence is clear. Some possible implementations of the geology–FWD relationship include:

1. Constructing a road data base containing geologic unit, rock type, average W7 deflection, and the observed range in W2:W7 deflection ratio. This data base would allow users to enter a highway name and mileage marker and obtain the rock type for that location, as well as the expected FWD deflections for that rock type. The highway engineer would then have a preliminary check on new FWD data and an approximation of bedrock rigidity that would help in interpreting site-specific FWD data.

2. Using existing FWD data to define averages and ranges of FWD response for all mapped geologic units, then using these data in the design and construction of new highways. Once a highway alignment is known, the statistical FWD response for the geologic units can be used to suggest bedrock properties, control test-boring locations, and help make highway design decisions.

3. Collecting FWD data at closely spaced intervals along highways over geologic units that are susceptible to sinkhole development or other rapid, lateral change in physical properties. W7 deflections should increase and W2:W7 ratios should decrease from a hard limestone substrate to sinkholes where limestone has been eroded and holes filled with softer materials. This technique would also be useful in detecting alternating soft marls and hard limestones that might be below the resolution of geologic maps.

4. Modifying the FWD to allow collection of deflections (as is currently done), along with seismic refraction data. Whereas it is difficult to separate the influences of bedrock rigidity and bedrock depth by using FWD data alone, tests that use seismic refraction equipment employing an FWD source suggest that bedrock depth and bedrock rigidity can be estimated from refraction data. These parameters, in turn, will allow better quantitative analysis of FWD data.

5. Using available FWD data, or hybrid FWD–refraction data, to improve geologic data. Doing so will benefit all users of geologic information, including those in transportation, construction, land-use planning, mining, education, and materials exploration. Deflections, bedrock depths, and seismic velocities of bedrock and overburden obtained from a combined FWD–refraction system would allow comparisons of observed values with expected values for mapped rock types. These comparisons may (1) indicate areas where current mapping may need revision, (2) produce a better understanding of the distribution of rock types over large areas where surface exposures are limited, and (3) delineate individual rock types that commonly compose a mapped geologic unit.

CONCLUSIONS

Preliminary conclusions are that:

- FWD deflections at the outermost sensors correlate to mapped geologic units at the county average scale;
- there is a statistical relationship between mapped geologic unit and FWD response at six test highway segments in four major physiographic regions;
- variance in FWD response within individual geologic units is large, making interpretation of geologic unit from FWD response alone uncertain;
- ratios of inner- and outer-detector deflections, such as W2:W7, partly normalize road construction differences and allow better rock-type discrimination than that obtained from W7 data alone;
- in general, highway segments constructed over relatively rigid bedrock units such as limestone, granite, and metamorphic rocks, exhibit low average W7 deflections and high W2:W7 ratios, whereas highways constructed over softer bedrock units such as mudstone, sandstone, and unconsolidated sediments have high average W7 deflections and low W2:W7 ratios;
- from FWD data and mapped geologic units alone, we cannot determine whether the bedrock influence on FWD response is due to bedrock rigidity or to systematic changes in bedrock depth among geologic units; and
- seismic refraction methods, particularly when integrated into FWD data collection, may provide reliable bedrock depths and rigidities that will allow better analysis of FWD data.

Continuing work will focus on quantifying the relationship between mapped geologic units, bedrock depth and rigidity, and FWD response by our acquiring FWD data and seismic refraction data at sites that represent a variety of geologic and pavement types. Our goal is to determine the best way to obtain bedrock depths from geologic maps, FWD data, and seismic refraction methods.

ACKNOWLEDGMENTS

This project (7-2990) is funded by the Texas Department of Transportation (TxDOT) through the Center for Transportation Research (CTR) at The University of Texas at Austin. Frank McCullough, Director of CTR, has provided guidance on the types of geologic information that would be useful in highway design, testing, and performance. TxDOT staff at the Design Division, Pavements Section, provided FWD data and have helped modify the FWD for use as a seismic source for refraction measurements. Publication edited by Lana Dieterich, designed by Jamie H. Coggin, and approved by Noel Tyler, Director, Bureau of Economic Geology.

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Percolation Theory Approach to Estimating Earthwork Factors in Weathered Granites

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Abstract

Principles of Percolation Theory (PT) are applied to characterize a modulus-porosity relationship for weathered granites. The relationship is shown to be consistent with test results of weathered granites reported in the literature. Earthwork factors for a highway project involving extensive excavation in weathered granites are estimated using this relationship with laboratory testing of recovered core samples and refraction seismic measurements of the in-situ rock mass.

INTRODUCTION

As part of the geotechnical characterization for improvement of State Route 260 east of Payson, Arizona to a four lane divided highway, earthwork factors are needed for significant roadway cuts and fills in weathered granites in the Preacher Canyon area. About 1.4 million cubic yards of excavation, including cuts with centerline depths in excess of 100 feet, and fills up to about 50 feet in thickness are anticipated. The highway project is located in the Tonto National Forest where few potential waste material sites are present, and those sites available are of limited area. Thus, realistic estimates of cut and fill quantities are needed to optimize volumes in design and minimize waste material. With the exception of initial field stationing, english units have been used on this project and in this paper. Modulus in units of GPa is used to be consistent in certain relationships.

With the exception of thin mantles of alluvium, colluvium and residual soils, the geology of the project area consists almost entirely of Precambrian granite in various stages of weathering. Geotechnical exploration for the project to date has included rock core drilling and sampling in borings advanced to depths of 20 to 110 feet. The granite encountered in the borings generally is decomposed to moderately weathered, with infrequent zones of slightly weathered rock. It is medium to coarse grained and pinkish to grayish brown. It commonly is soft to moderately hard with some hard to very hard zones. Degrees of weathering and hardness are strongly correlated. Rock mass discontinuities are very closely to closely spaced, and are generally spaced from less than 0.2 to 1.0 foot. Fracture surfaces are slightly rough to rough, and are typically stained and/or partially filled with silt or clay. The rock quality designation (RQD), defined as the percentage of rock core per run which is relatively sound and exceeds 4 inches in length, typically ranges from 0 to 35, with occasional core runs having RQD runs of 60 to 100. Core recoveries of less than 100 percent were very common and typical in

Fig 1 Westbound Deep Cut Profile with
Weathering, RQD & Seismic Velocities

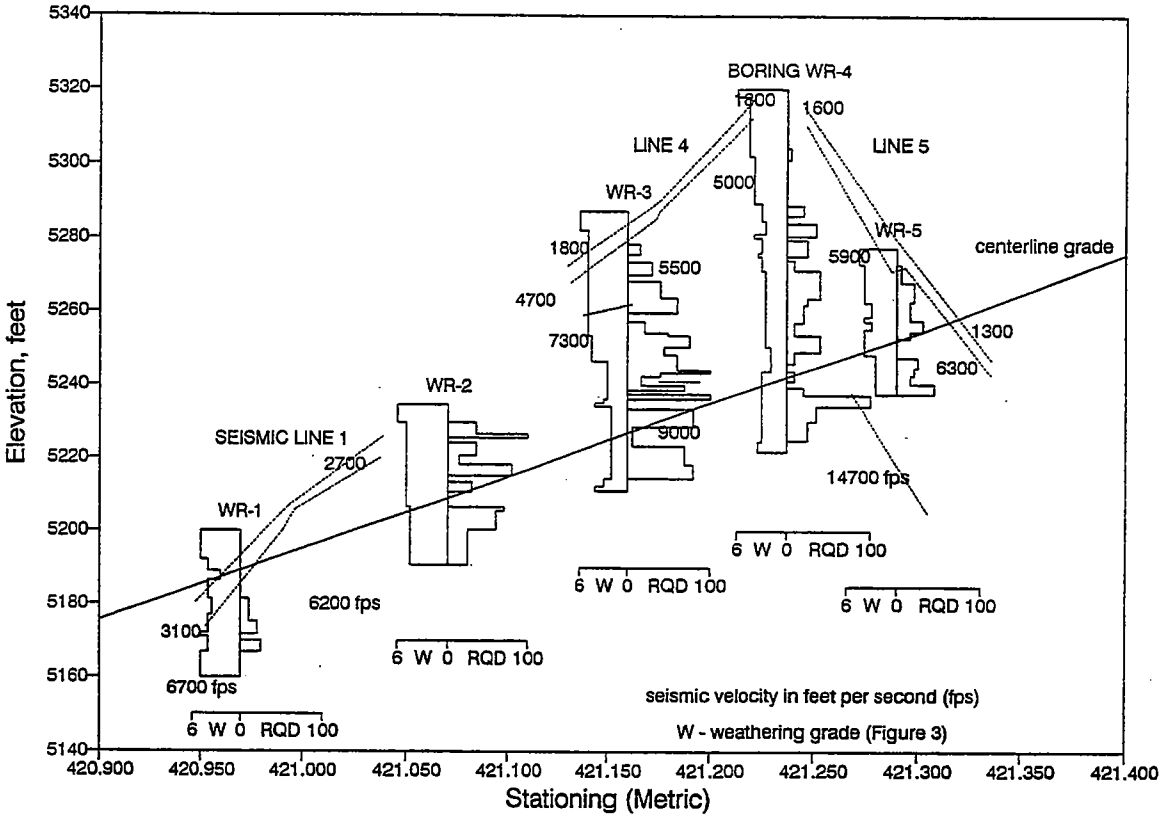
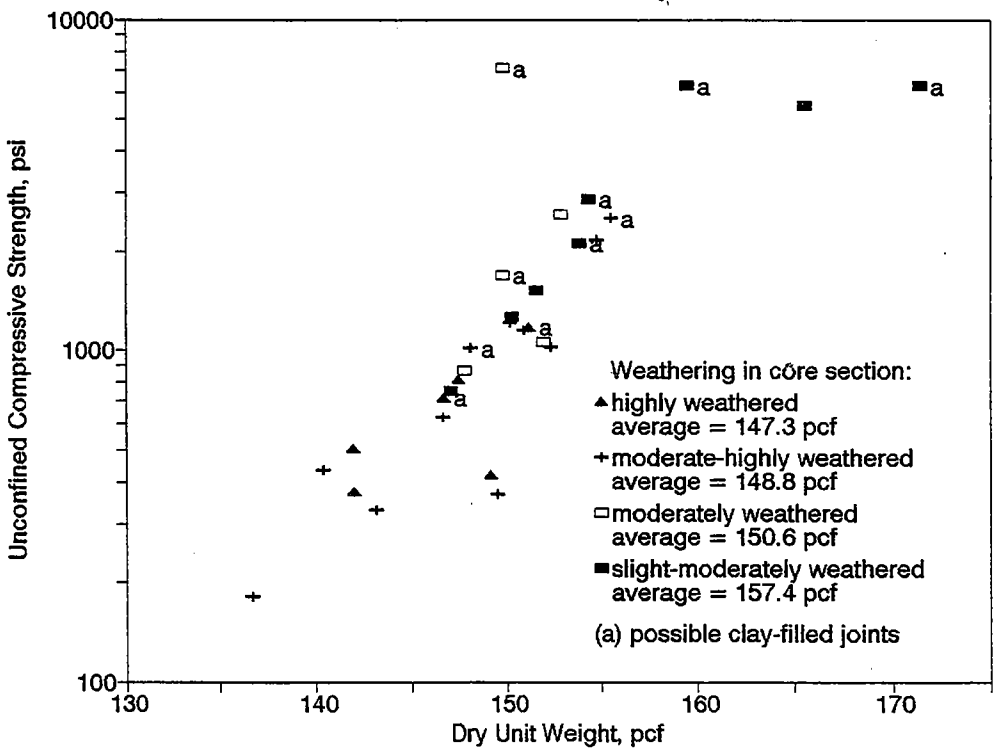


Fig 2 Results of Tested Core Samples,
Weathered Granites - Preacher Canyon AZ

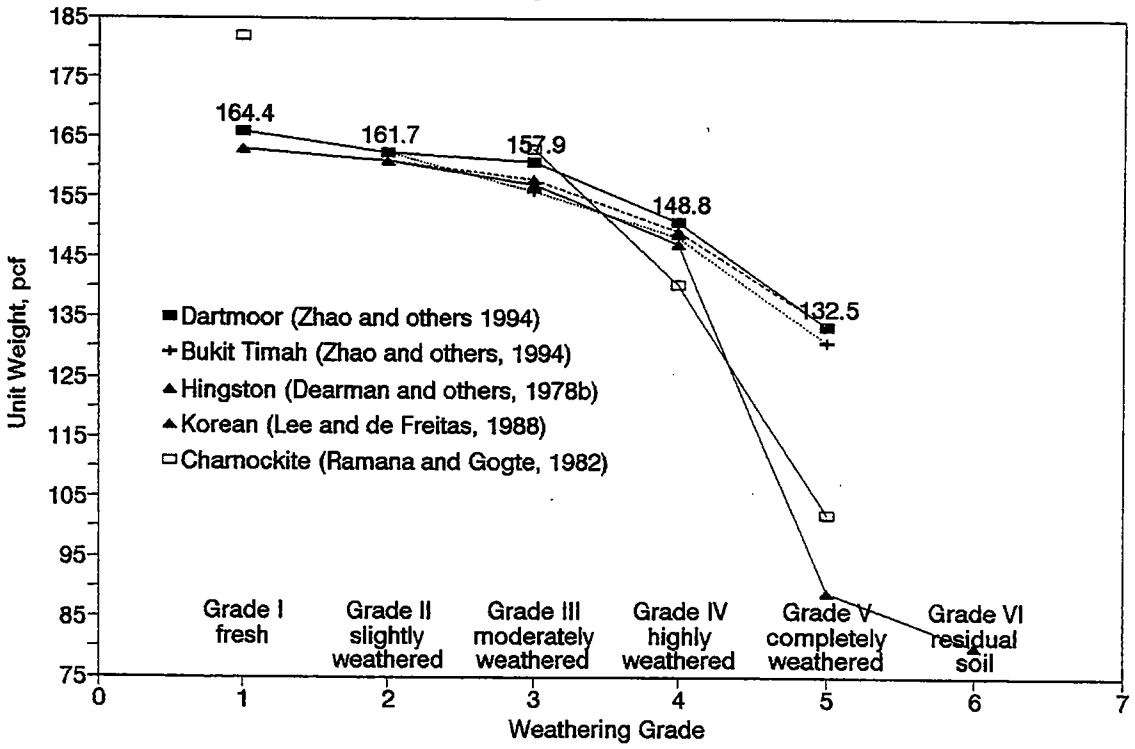


highly weathered or decomposed material. Refraction seismic lines completed along the alignment were either 300-feet or 120-feet in length, depending upon the depth of excavation anticipated at each line location. A 12-channel signal enhancement seismograph with a sledgehammer energy source was used in the refraction seismic work. A summary of weathering and RQD data from borings and p-wave velocities from refraction seismic lines along part of a deep cut are presented in Figure 1. Figure 2 presents results of 31 unconfined compression tests performed on recovered core samples, and includes unit weights and the weathering condition of the section of core from which the samples were obtained.

UNIT WEIGHTS OF WEATHERED GRANITE

Granites generally become less dense and have more porosity as weathering increases. Figure 3 summarizes typical unit weights for various weathering grades (Dearman and others, 1978a) as reported in the literature. Reported unit weights and weathering grades are reasonably consistent among the Dartmoor and Hingston granites in England, Bukit Timah granites in Singapore and slightly to highly weathered Korean granites. Reported unit weights for Charnockites in India (except when moderately weathered) and decomposed Korean granites have significantly different unit weights compared to the other data. Average unit weights for slightly-moderately to highly weathered core samples from the Preacher Canyon (Figure 2) project are typically lower than the values presented in the literature. Furthermore, where core recovery was less than 100 percent or the core disintegrated during drilling and handling, a unit weight could not physically be obtained.

Fig 3 Typical Unit Weights of Granites
by Weathering Grade from Literature



It is anticipated that porosity, which influences the unit weight of a granitic rock mass, occurs both in the intact rock micro-fabric (grain boundary microcracks, primary porosity) and in the macro-fracture aperature spaces (secondary porosity). Effects of primary porosity can be tested in recovered intact rock core. Since recovered core normally breaks apart at fractures, especially open fractures, effects of secondary porosity cannot be tested from core drilling. Estimating a rock mass unit weight might be feasible using measured core sample unit weights and the values or trends presented in Figure 3 when that rock mass does not have considerable fracturing (minimal secondary porosity) or significant portions of highly weathered or decomposed material. Even when some portions of the rock mass are highly weathered to decomposed, a reduction in total rock mass unit weight might be estimated sufficiently using Figure 3 relationships. However, when a rock mass is heavily fractured (significant secondary porosity) or a large portion is highly weathered to decomposed, some means of in-situ measurement may be necessary to improve estimates of rock mass unit weight.

PERCOLATION THEORY (PT) OVERVIEW

In PT, percolation describes when a system becomes macroscopically open to a given phenomenon (Sahimi, 1994). Percolation is a simple model of a disordered system based on randomness. Percolation operates on networks with sites that are or are not occupied, or bonds between sites that are or are not connected. The percolation behavior is a function of the probability of sites being not occupied or bonds being not connected. For purposes of the analyses presented in this paper, that probability is equal to the sum of the primary and secondary porosity (total porosity) in the geologic material mass. The percolation threshold (Pc) is a transition point between an open and a closed system. Stauffer and Aharony (1992) present the theoretical background and development of PT, and Sahimi (1994) presents a variety of PT applications, including applications in rock fracture and fluid flow in porous media.

The form of the PT relationship for elastic modulus is

$$E = k (Pc - P)^{f}$$

where E is elastic modulus and k is a proportionality constant. Having an exponent which is characteristic for a given phenomenum implies universality in percolation phenomema.

Sahimi (1994, Chapter 11) discusses and summarizes percolation-based behavior of modulus in terms of the exponent f. He describes the behavior of modulus in terms of two basic types of gels, physical gels and chemical gels. In general, modulus in chemical gels is dominated by stretching or central forces acting through the elements of the network. Modulus in physical gels is additionally influenced by bond bending forces, which are similar to mutually touching particles rolling on top of one another during deformation of the network. Sahimi reports that f is about 1.9 to2.2 for chemical gels and about 3.75 for physical gels.

PT AS A METHOD TO ESTIMATE GEOLOGIC MASS UNIT WEIGHT

Total porosities within some in-situ geologic material deposits can be estimated by measuring modulus (such as from refraction seismic p-wave velocities) using a PT approach. Total porosity can provide an estimate of unit weight applicable to a whole rock mass, including portions that are not recovered when coring, and includes the effects of secondary porosity in joints and fractures. Elastic modulus, as quantified by PT, varies universally as a function of the difference between the material porosity and a threshold porosity described as the percolation threshold Pc. In-situ material deposit elastic

moduli are estimated from in-situ seismic p-wave velocities, and porosities are estimated from particle distributions and measurements or estimates of in-situ material deposit densities.

Previous work by the author interpreted PT relationships for well-graded cohesionless granular streambed materials in the arid and semi-arid Southwestern U.S. (Rucker, 1998; 1996). In-situ dynamic low-strain moduli were calculated from refraction seismic p-wave velocities in a depth range of 3 to 10 feet (to control the influence of overburden confinement) and assumed Poisson's ratio of 0.33. Porosity was estimated from cohesionless alluvial material dry density estimates using a method of Burmister (1963) and assuming a specific gravity of 2.67. The critical percolation threshold porosity providing the best-fit to the data was Pc = 0.42 with a physical gel f of 3.75. For a typical Poisson's ratio of 0.33 for soil, the constant k was estimated to be 200,000,000 for low-strain Young's modulus in units of psi.

PT POROSITY-MODULUS RELATIONSHIP IN WEATHERED GRANITES

Dynamic low-strain modulus and porosity relationships for several data sets reported in the literature are presented in Figure 4. The scales in Figure 4, and also Figures 6 and 7, are logarithmic with modulus on the y-axis. The x-axis plots the difference between the percolation threshold Pc (assumed to be 0.42) and the porosity of a sample. With these logarithmic x- and y-axis scales, the PT modulus-porosity relationship plots as a straight line (power law). The constant used in the PT equation was modified slightly to reflect anticipated differences in cohesionless soil and weathered rock. A representative Poisson's ratio of 0.22 was assumed for rock, and the constant k was recalculated to be 152,000,000 for modulus in psi with this lower Poisson's ratio. The resulting PT modulus-porosity trend is plotted in Figure 4.

Low-strain Young's moduli for the Charnockites (Ramana and Gogte, 1982) are calculated for individual core samples from reported densities, laboratory p- and s-wave velocities using standard formulas (Viskne, 1976). Typical moduli and porosities per weathering grade are reported by Zhao and others (1994) for Dartmoor and Bukit Timah granites. A single sample of sonic-measured modulus (39 GPa) and porosity (0.9%) is presented by Haupt and others (1992). Galle (1994) reports a p-wave velocity (1910 m/s) and porosity (15%) for a core sample; modulus is estimated by assuming Poisson's ratio of 0.2. Modulus-porosity data from the literature cluster about the PT trend in Figure 4 once porosities become greater than just a few percent. This is consistent with rock mass behavior becoming controlled by fracturing once fracturing is sufficiently dense.

ESTIMATING LOW-STRAIN MODULUS & POROSITY FROM LABORATORY TESTING

Low-strain moduli of the Preacher Canyon weathered granite core samples initially presented in Figure 2 are estimated from the measured unconfined compressive strengths as shown in Figure 5. Static moduli are estimated using the constant times unconfined compressive strength concept of Deere and Miller (Bieniawski, 1989). Dynamic moduli are then estimated from the static moduli using a modification of the method of van Heerden (1987). The estimating relationships used are

$E_{static} = UCS * 400$ and

$E_{static} = 0.055 * E_{dynamic}^{1.61}$ (units of GPa)

Fig 4 Weathered Granite Relationships
based on Porosity, Modulus & PT Trend

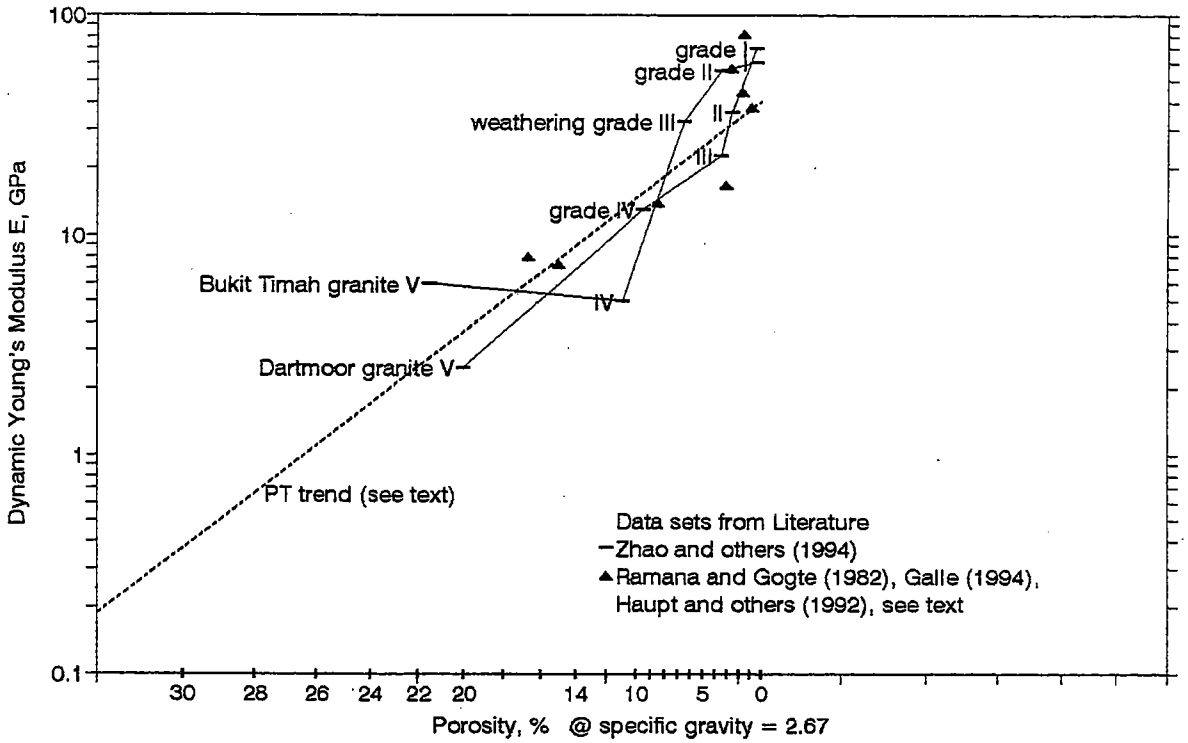
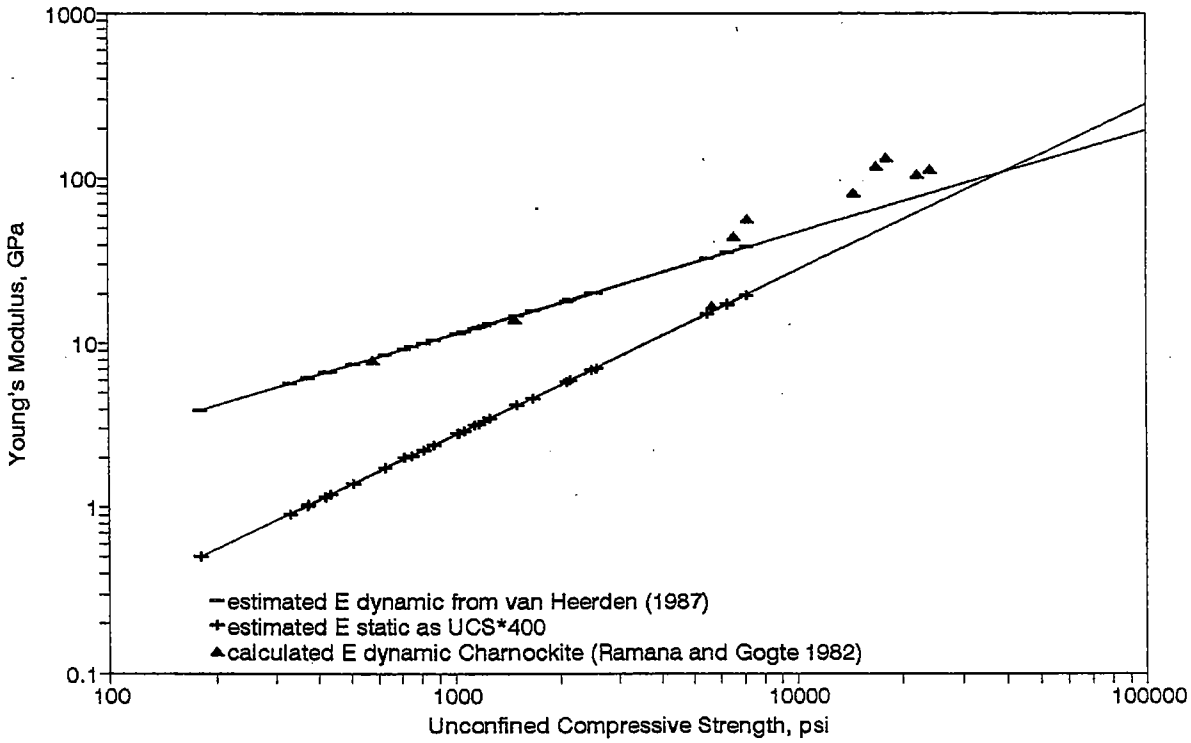


Fig 5 Granite Modulus Relationships
Measured and Estimated from UCS Tests



where UCS is unconfined compressive strength, E_{static} is the static modulus and $E_{dynamic}$ is the dynamic low-strain modulus. Calculated dynamic moduli for Charnockites (Ramana and Gogte, 1982) are also presented in Figure 5. An excellent correlation of the two lowest modulus UCS Charnockite data points with the estimate trend for the lower range of UCS indicates the usefulness of this procedure in weathered granites.

Densities of weathered granite core samples are obtained by measurement during unconfined compression testing of the cores. Knowing dry unit weight and assuming a specific gravity of 2.67, porosities are calculated. The samples represent unsaturated conditions in this semi-arid climate. The resulting plots of estimated modulus and density for the Figure 2 Preacher Canyon weathered granite data superimposed on the Figure 4 results are presented in Figure 6. The estimated modulus-porosity trend for the Preacher Canyon weathered granites is consistent with both the PT trend and weathered granite data in the literature.

It should be noted that one Preacher Canyon data point lies considerably above the general trend in both Figures 2 and 6. That relatively low density piece of core has come from a rock mass zone with clay filled joints whose cohesive nature tends to bond the core pieces together. It's relatively high UCS and estimated modulus is consistent with tested core samples of limestone, sandstone and welded tuff. These materials, being cemented or welded together and typically lacking the grain boundary microcracking characteristic of weathered granites, behave as a chemical gel rather than a physical gel. A chemical gel PT model of massive, unfractured deposits may also be able to model modulus-porosity relationships. However, a fractured large-scale rock mass behaving as a physical gel but composed of rock whose individual small-scale pieces behave as a chemical gel may not be able to be characterized on the basis of laboratory testing of those small-scale pieces.

ESTIMATING IN-SITU MODULUS & POROSITY, EARTHWORK FACTORS

Rock mass low-strain modulus is estimated from field refraction seismic p-wave velocities. Given that the rock mass is unsaturated in this semi-arid climate, a typical Poisson's ratio of 0.22, which results in s-wave velocities of 0.6 times the p-wave velocities, is assumed. The relationship of seismic velocity to dynamic Young's modulus is then added as another y-axis to Figure 6 to obtain Figure 7. Field seismic velocities may then be used to estimate porosity and dry density as shown in Figure 7.

Estimates of densities of in-place materials in cuts and existing embankments were made to assist in determining earthwork factors. Four Proctor curves were determined from bulk field samples. Two typical maximum dry densities, about 121 and 114 pcf, were indicated from these tests. It is assumed that the existing embankments include a considerable portion of rock which is removed during Proctor tests. Assuming 30 percent rock retained on the No. 4 sieve and a bulk specific gravity of 2.4 (150 pcf, typical from Figure 2), corrected total maximum dry densities for the tested bulk samples are increased to about 125 and 120 pcf, respectively. Greater portions of this oversize rock would be expected to increase the dry densities further; at 50 percent retained on the No. 4 sieve, maximum dry densities would be about 128 and 124 pcf, respectively. Two refraction seismic lines were completed on existing embankments within the project limits. Representative p-wave velocities of 2,600 and 2,200 fps were obtained from these lines. Representative in-place dry densities for these embankments were estimated to be 123 and 120 pcf, respectively, as shown in Figure 7.

Fig 6 Weathered Granite Relationships
with Preacher Canyon Core Test Data

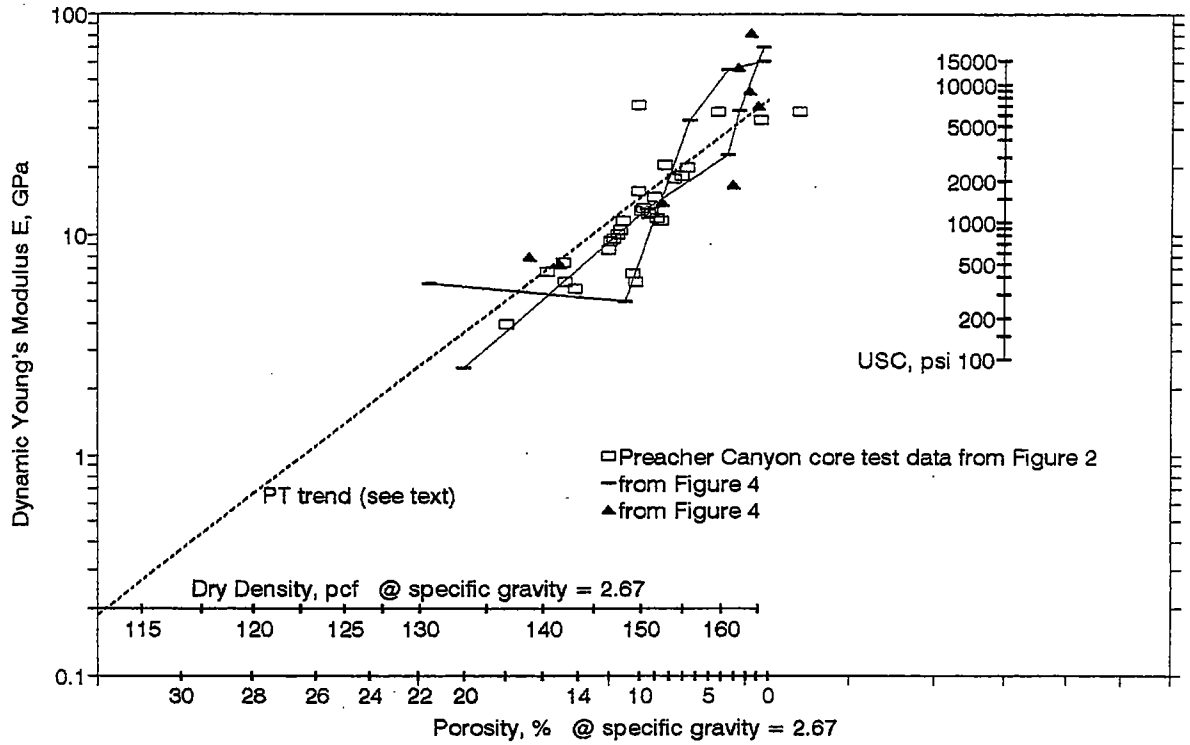
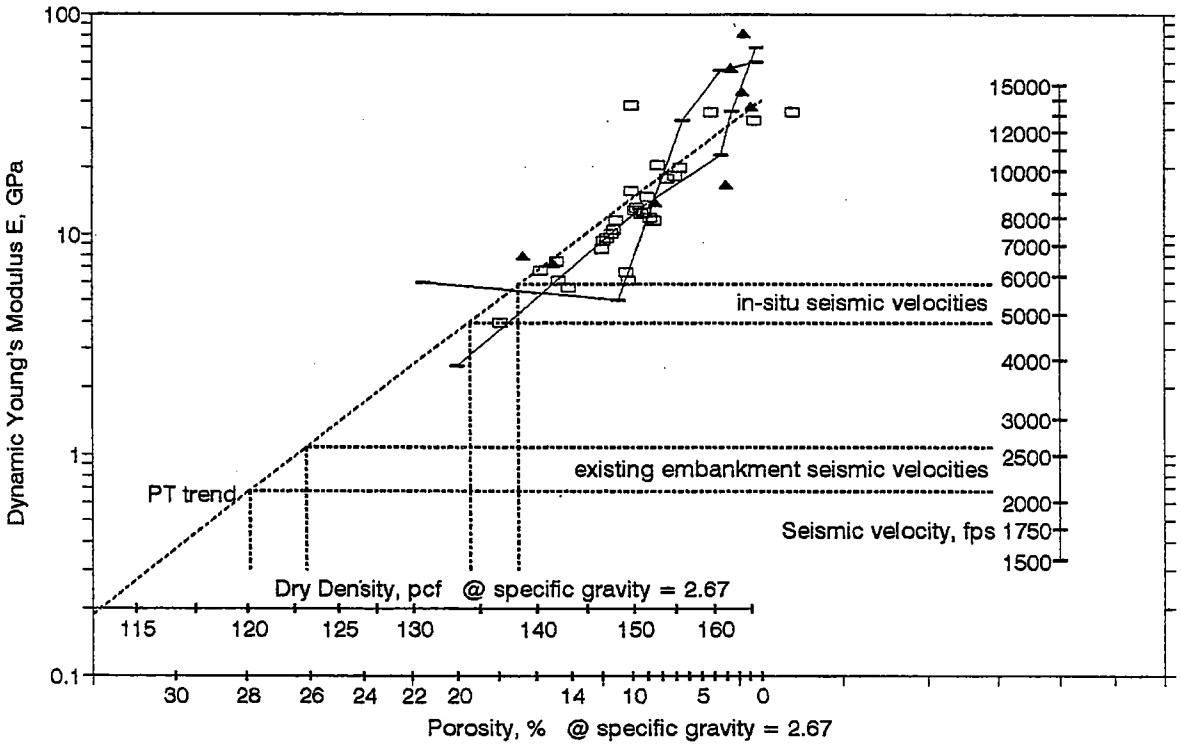


Fig 7 Weathered Granite Relationships
with Shrink/Swell from Seismic Velocity



Representative p-wave velocities from project refraction seismic data were used to estimate in-place representative dry densities of the cut materials. Between Stations 420+900 and 421+400 (metric), the representative p-wave velocity was about 5,800 fps and the estimated dry density was 138 pcf. Between Stations 421+400 and 422+200 (metric), the representative p-wave velocity was about 4,800 fps and the estimated dry density was 134 pcf. Assuming that lower maximum dry densities correlated to greater weathering, earthwork factors were estimated to be 138/123=1.12 and 134/120=1.12, or about 12 percent swell.

CONCLUSIONS

A Percolation Theory approach to characterizing the density of a weathered granite rock mass has been demonstrated. The PT low-strain modulus and porosity relationship in weathered granites is shown to be consistent with data presented in the literature. This relationship is further demonstrated using low-strain modulus estimated from unconfined compressive strengths and measured unit weights of recovered core samples from the Preacher Canyon highway project. Seismic p-wave velocities obtained from refraction seismic surveys provide the means to estimate low-strain modulus for the in-situ rock mass, from which dry density (unit weight) is estimated through the PT relationship. This provides a theoretical basis for and possible improvement over previous empirical relationships (Caltrans, 1978) using field seismic velocity to estimate earthwork factors.

As presently derived and with proper calibration, this PT method is anticipated to have application in weathered, fractured crystalline rock masses. This method is not expected to provide good results in very low porosity, widely fractured to unfractured crystalline rock masses, or fractured sedimentary formations. Sedimentary or pyroclastic volcanic rock masses which are essentially unfractured might be effectively modeled using a PT approach where the cemented or welded nature of the rock mass is taken into account.

ACKNOWLEDGEMENTS

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RECYCLED AGGREGATES IN ROAD CONSTRUCTION—AN ECONOMIC ASSESSMENT

David Wilburn[1] and Thomas Goonan[2]

ABSTRACT

Recycled materials are increasingly being used to supplement natural aggregates derived from crushed stone, sand, and gravel in road construction. An understanding of the economics and factors affecting the level of aggregates recycling is useful in estimating the potential for recycling and in assessing the total supply picture of aggregates. The U.S. Geological Survey conducted an analysis of the supply sources, technology, costs, incentives, deterrents, and market relationships associated with the production of recycled aggregates. Results from cash flow analyses indicate that under certain conditions aggregates derived from construction and demolition waste can have economic applications.

While recycled aggregates are used in a variety of cement and asphalt road construction applications, product variability and strength characteristics usually limit their use to road base, backfill, and asphalt pavement. Quality is often source dependent, and indiscriminant blending may lead to quality degradation. Careful feed monitoring, testing, and marketing can broaden the use of recycled aggregates into other applications.

Aggregates recycling is most likely to be successful where transportation dynamics, disposal and tipping fee structures, resource supply/product markets, and municipal support are favorable. Costs for three representative fixed site operations of different sizes were modeled in this study. Under study conditions, all were found to be profitable and highly dependent upon local tipping fees and market prices, which can vary significantly by location. Smaller operations were found to have different operational dynamics, often requiring creative marketing or incentives to maintain profitability.

Consumption of recycled aggregates from crushed concrete increased 170% between 1994 and 1996, but comprised less than 0.4% of total aggregates consumed in 1995. Construction waste supply is regional, and is determined by local infrastructure decay and replacement rates. Aggregates recycling rates are greatest in urban areas where replacement of infrastructure is occurring, natural aggregate resources are limited, disposal costs are high, or strict environmental regulations prevent disposal. Consumption is expected to grow as construction contractors recycle as a means of saving on transportation, disposal, and new material costs and aggregate producers include recycled material in order to prolong the life of their reserves and improve product mix. In some locations, the amount of material available for recycling is insufficient to meet present industry demand. While recycled aggregates can be a supplement for natural aggregates in selected road applications, their use should be evaluated locally based upon relative cost, quality, and market factors.

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Recycled Aggregates in Road Construction—An Economic Assessment

Construction materials in general, and aggregates in particular, are important components of infrastructure. Development and extraction of natural aggregate resources (primarily crushed stone and sand and gravel) are increasingly being constrained by urbanization, zoning regulations, increased costs, and environmental and societal concerns, but use of recycled materials from roads and buildings is growing as a potential supplement to natural aggregates in road construction. Recycling has the potential to extend the life of natural resources by supplementing resource supply, reduce environmental disturbance around construction sites, and enhance sustainable development of our natural resources. This paper assesses the economic factors of aggregates recycling, and indicates conditions that are necessary for aggregates recycling to be most successful, as part of an ongoing program of the U.S. Geological Survey to evaluate natural resource options.

Transportation distances and costs are a significant part of the dynamics that define the use of construction aggregates within a region, but they normally do not directly affect operational profitability of the fixed site recycler, because costs for transportation are typically incurred by the contractor of a construction project, who supplies feed for recycling, rather than the recycler. The contractor is, however, concerned with the cost associated with transportation. The amount of material that the supplier will make available to the recycler is based in part on a calculation that compares the relative costs of delivering and paying a tipping fee to the recycler, the costs of transporting construction debris to competitors, or the cost of disposing of this material in a landfill. The distance that aggregates can be hauled economically varies regionally; however, each kilometer that a ton of aggregates is hauled can add $0.10 to $0.20 to its cost (Socolow, 1995), if trucks return empty to get more aggregates. Backhauling of material from the delivery site can reduce delivery cost by as much as 50 percent (R. Hawkins, Vice President, Allied Recycled Aggregates, oral commun., 1997). Costs are in turn dependent upon local aggregates supply and market conditions. For mobile recycling operations, the cost of moving equipment from site to site must be considered.

Costs for producing recycled aggregates were developed from Denver area data collected from published literature, personal contacts, and site visits. The methodology used in this study allows for variation in costs and revenues to be evaluated and analyzed. Costs in other regions of the United States may differ due to raw material supply, operational, competition, or demand variations. Production cost information for three representative fixed-site recycling operations are presented. Cost models were built to represent small (110,000 tons/year), medium (253,000 tons/year), and large (312,000 tons/year) capacity aggregate recycling operations.

Figures 1, 2, and 3 show costs for the three size models evaluated in this study. Capital costs, operating costs, and revenues are represented for each model.

From these data, the costs associated with equipment (such as equipment capital, equipment maintenance, and recovery of capital) clearly are a significant contributor to total production costs for a recycling operation, particularly a smaller size facility. Recycling requires initial capital

Figure 1. -- Estimated 1996 costs for a 110,000 t/yr recycled aggregates operation.

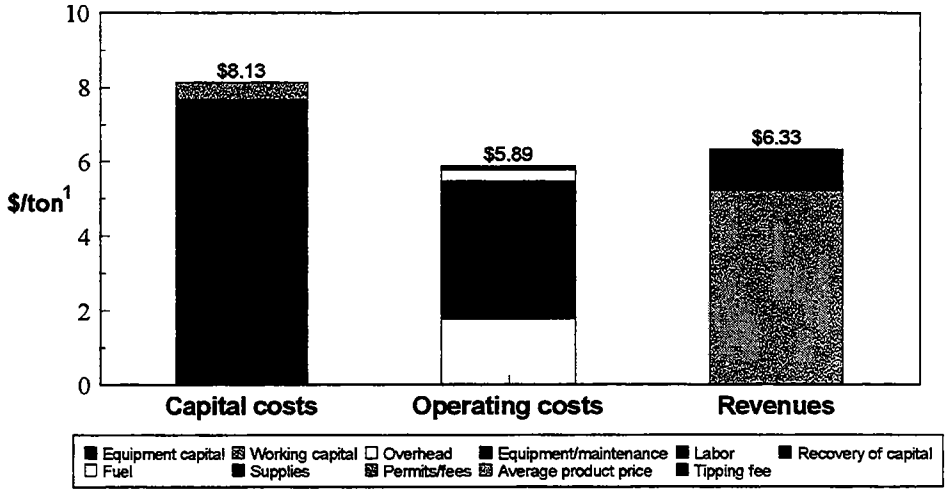


Figure 2. -- Estimated 1996 costs for a 253,000 t/yr recycled aggregates operation.

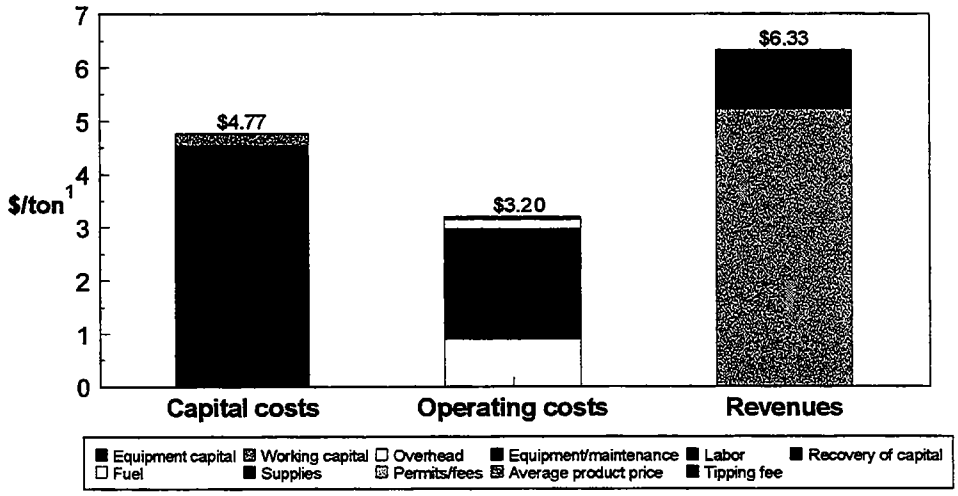
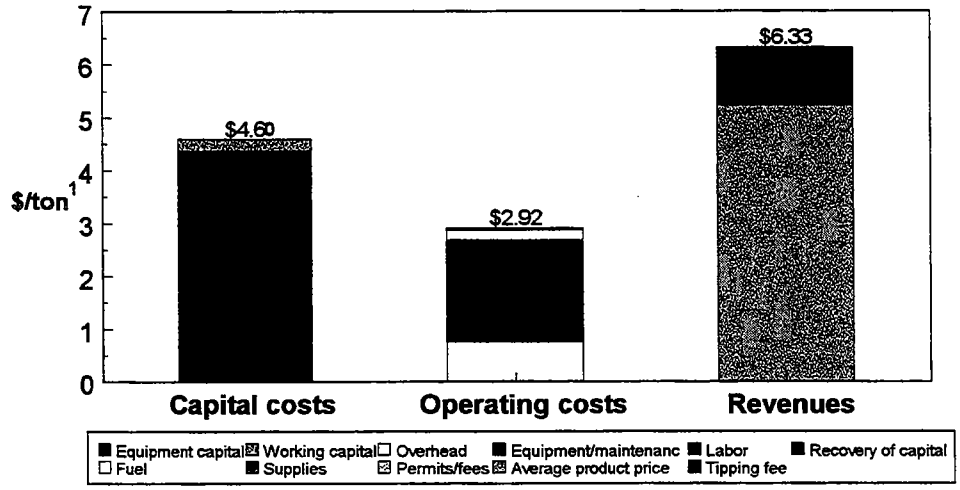


Figure 3. -- Estimated 1996 costs for a 312,000 t/yr recycled aggregates operation.



[1] All costs are expressed in terms of U.S. dollars per ton (2,205 pounds).

expenditures of approximately $4 per ton of production capacity for the large recycling operation to about $8 per ton for the small operation due to economies of scale. Costs reflect the purchase of new equipment; used equipment may be an option in some cases. Capital recovery of equipment (depreciation) constitutes an additional 14 - 20 percent of total operating cost, and is highest for the larger operation. During the period of capital recovery, equipment maintenance constitutes 22 - 24 percent of the total unit operating cost.

The largest component of total unit operating cost for a recycling facility is fixed overhead, including major expense items like management and clerical salaries, building and land rental costs, advertising expense, and property and real estate taxes. Overhead ranges from 26 percent of total unit operating cost for the large operation to 30 percent for the small operation. Land lease costs represent a significant portion of operational overhead, although land requirements for recycling operations are typically small (generally 2-6 hectares). Many operations lease land rather than purchase it. Based upon reported lease fees[1] for comparable industrial land in the Denver area, an average annual land lease cost of approximately $19,000 was assumed for the small operation, $43,000 for the medium operation, and $53,000 for the large operation. Supply and permitting costs together account for less than 1 percent of the total unit operating cost.

Labor costs contribute about 20 - 23 percent to total unit operating cost. For the capacities assessed in this study, labor requirements vary little on a percentage of total cost basis, but vary significantly on a unit cost basis. Most recycling plants require pickers to ensure that feed supply is as free as possible from deleterious material. Large-sized recycling plants require only one or two additional equipment operators to handle a significant capacity increase. Consequently, the large plant, with a capacity of about three times the small plant, incurs a labor cost per unit of product that is 42 percent of the small operation. A unit cost savings is achieved. Productivity of the large plant is also higher. At full capacity, productivity estimates for this study range from 52,000 tons per person for the large operation to 22,000 tons per person for the small operation.

Fuel costs average 6 - 7 percent of total unit operating cost, mainly for diesel fuel and electricity used to provide power for mobile and stationary equipment. Energy, supplies, and permitting fees generally constitute less than 10 percent of the total unit operating cost. A mobile plant would have a higher fuel cost, to account for transportation of equipment from site to site.

A range of different size products is typically produced by recycling operations to meet the varying needs of local markets. Prices for each product can vary regionally due to demand and market considerations. An average price of $5.23 per ton was assumed for this study, based upon an assumed throughput ratio of asphalt to cement concrete of 60:40 and a weighted average of reported 1996 prices for known aggregate products in the Denver area. Products containing different proportions of cement/asphalt concrete would generate different prices depending upon the prices of these products for the area of the United States in question. Recovery of byproducts such as rebar from recycling operations was not considered in the evaluations.

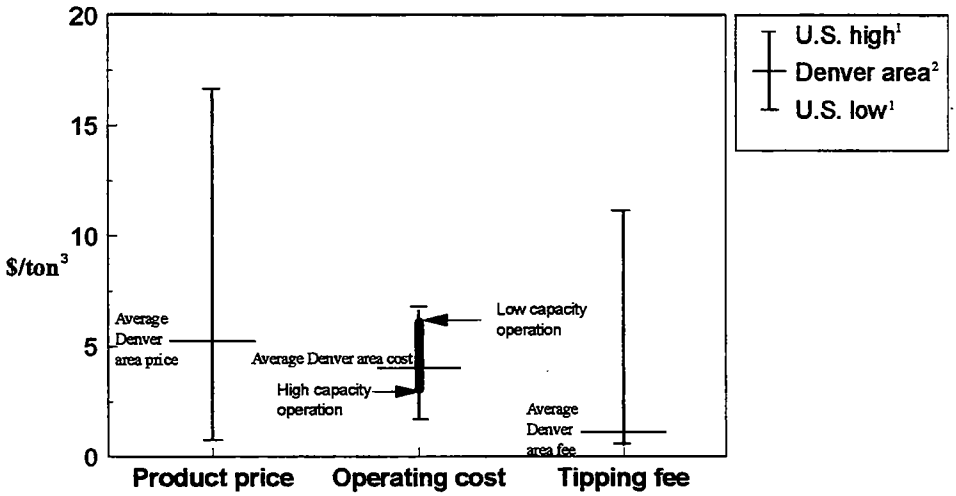
The cost figures suggest that recycling operations are influenced by production rate, equipment capital, and operating costs. However, revenues generated by product prices and

[1] Based upon data provided by Fuller and Company, Denver, Colo., August 31, 1997.

tipping fees are the principal factors affecting the economics of the operation. Production capacity determines equipment and labor requirements, and such requirements are not greatly influenced by regional variations. Once an operation's capacity has been determined and equipment selected, an operator is limited in what it can do to significantly change equipment and labor costs. Fixed operating costs also offer limited flexibility. However, a recycling operation is affected by regional variation in product prices and local tipping fees.

Figure 4 relates the costs reported in this study with reported U.S. costs, prices, and tipping fees (Deal, 1997).

Figure 4. -- Comparative cost and revenue estimates of recycled aggregates.



[1] Adapted from Deal (1997).
[2] Estimated from this study.
[3] All costs are expressed in terms of U.S. dollars per ton (2,205 pounds).

The wide variation in product price is a result of the variable nature of recycled aggregate products and regional markets. Highly specialized products such as painted landscape rock may sell for as much as $15 per ton, while poor quality fill material might sell for less than $1 per ton. The price spread for road base, the principal market for recycled aggregates, is much narrower; the reported Denver price of $5.23 per ton fits well with reported U.S. sale prices for road base which range from $2.76 to $6.61 per ton.

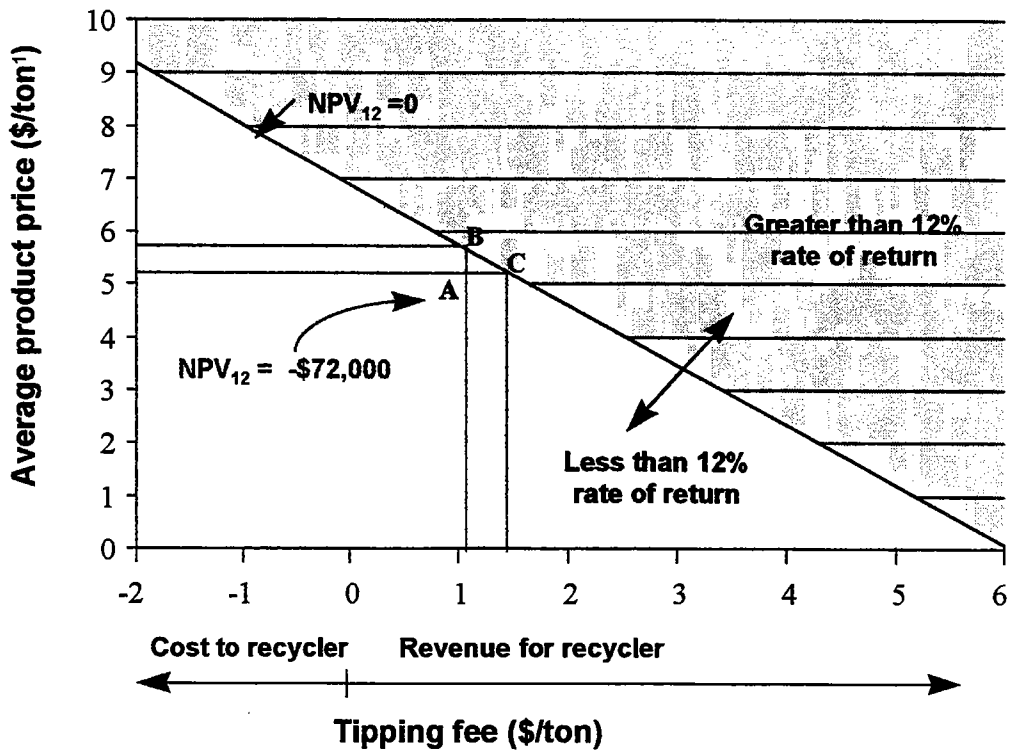
The reported range in U.S. tipping fees for recycling operations is likewise quite broad. The assumed Denver tipping fee of $1.10 per ton falls on the low end of this range. Aggregates from both natural and recycled sources are readily available in the Denver market, and local landfill charges for construction debris are relatively low. A low recycling tipping fee is expected where source material is readily available and costs of alternatives are low.

Operating costs for the three model operations fit well within the U.S. range reported by Deal (1997). Cost variation across the United States is much smaller than variation in either product price or tipping fee.

Both product price and tipping fees vary significantly from region to region, as each region has different market conditions, specifications, and aggregate sources. The price of available crushed stone or sand and gravel often determines the local market price for recycled aggregates, because crushed stone and sand and gravel contribute a far greater volume of source material to the aggregates market than recycled material in most regions. In a similar manner, local tipping fees are set based upon the volume of waste material, the availability of disposal sites, local demand for aggregates, relative transportation distances and costs, and State and regional legislation.

In order to evaluate the effect of product price and tipping fee on profitability, a series of cash flow analyses were performed at different tipping fee levels, each calculating the product price that would have to be realized in order to produce a discounted cash flow net present value (NPV) equal to zero. NPV is commonly defined as the present value of all revenues less the present value of all costs, including initial capital costs (Stermole, 1980). A NPV is associated with a discounted rate of return (a 12 percent rate of return was considered the necessary rate of return for operations to cover the opportunity cost of capital plus risk). Output from these calculations is presented as a set of product price and tipping fee pairs, each yielding $NPV_{12}=0$ (at a 12 percent DCFROR). Figures 5, 6, and 7 are line graphs of product price versus tipping fee generated based on cash flow analyses of the three models.

Figure 5. -- Profitability of a 110,000 t/yr recycled aggregates operation.
(Represented by combinations of product price and tipping fee which result in a $NPV_{12} = 0$)



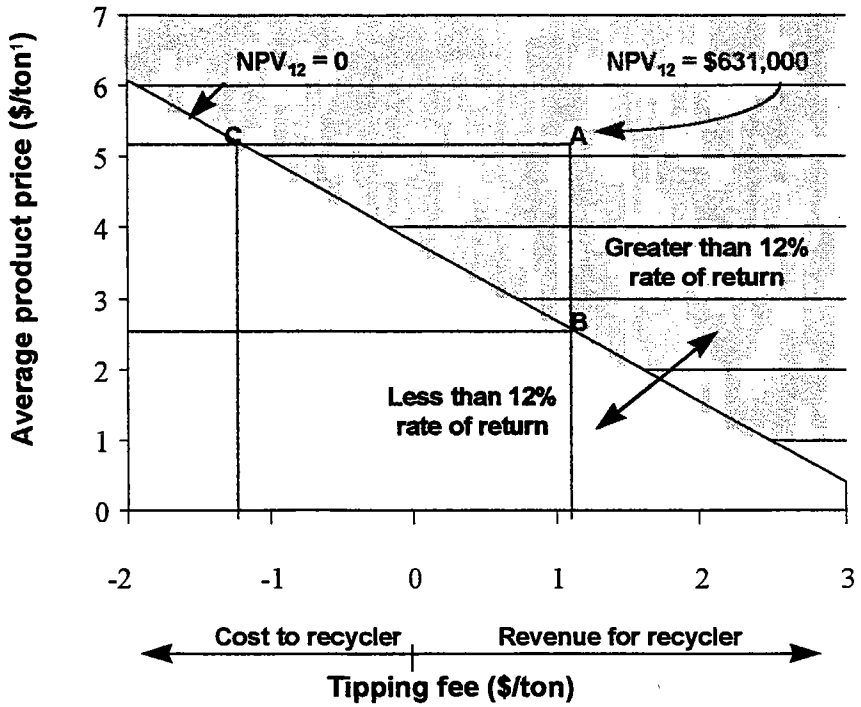
[1] All costs are expressed in terms of U.S. dollars per ton (2,205 pounds).

The solid line on the graph represents combinations of product price and tipping fee yielding a zero NPV (break even point) with a 12 percent discount rate. The area above this line represents recycling operations which earn a rate of return greater than 12 percent. Below the line, in the yellow shaded area, all combinations of product price and tipping fee yielding a negative NPV, indicating operations that earn less than a 12 percent rate of return.

The horizontal axis reports a range of tipping fee values. Positive tipping fee values reflect the most common situation for recyclers where a fee is paid to the recycler by the supplier to receive and process construction debris. Such fees represent revenue to the recycler. A negative value reflects the situation where the recycler may pay for construction debris; usually this would occur only when the recycler is in short supply of feed material and thus must pay for additional material in order to meet the demand for established contracts. This represents a cost to the recycler somewhat analogous to mining costs incurred by natural aggregate producers.

For each of the three graphs, the NPV representing base case conditions (product price of $5.23 per ton, tipping fee of $1.10 per ton) is plotted as point A. The distance that the plotted point is above (below) the $NPV_{12} = 0$ line represents the degree of profitability or loss for the assumed rate of return of 12 percent. For a small (110,000 tons per year) recycling operation, figure 5 shows that such an operation earns less than a 12 percent rate of return, and has an estimated negative NPV_{12} of -$72,000 (the point falls below the NPV_{12}=0 line). Such an

Figure 6. -- Profitability of a 253,000 t/yr recycled aggregates operation.
(Represented by combinations of product price and tipping fee which result in a NPV $_{12} = 0$)

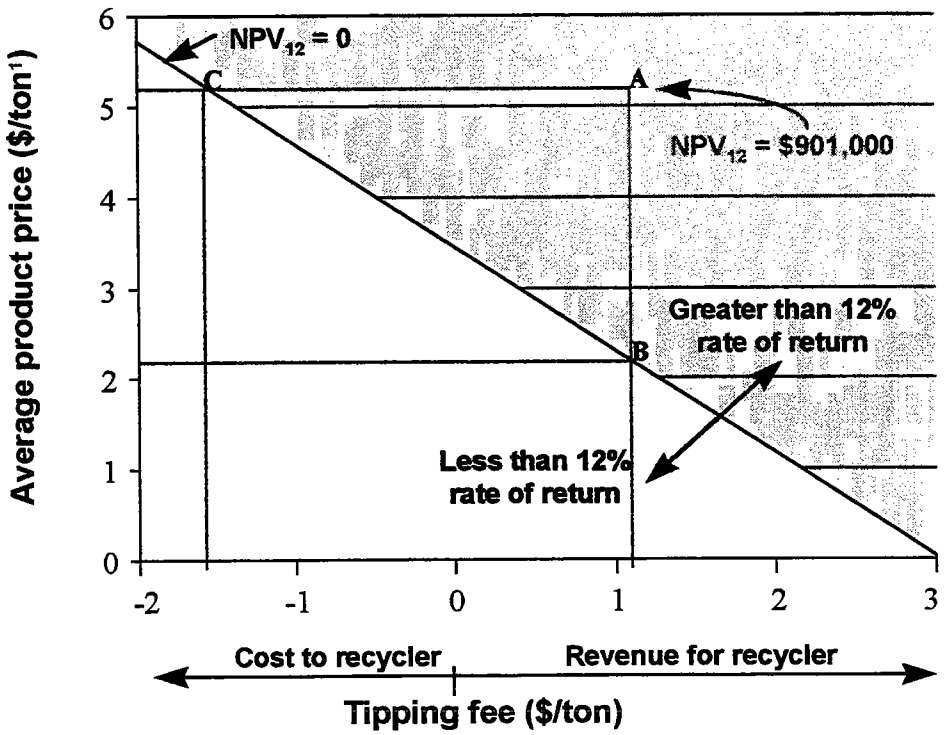


[1] All costs are expressed in terms of U.S. dollars per ton (2,205 pounds).

operation would require an average market price of $5.77 per ton of product (point B), or an increase in tipping fee from the assumed $1.10 per ton value to about $1.71 per ton (point C), or a combination of these to achieve a rate of return of 12 percent.

The medium size (253,000 tons per year) model, however, plots above the NPV_{12}=0 line (NPV_{12}= $631,000), showing a level of profit above the assigned 12 percent rate of return. Figure 6 indicates that for this model, the average product price could fall to $2.69 per ton (point B), and the operation would still achieve a rate of return of 12 percent (assuming a charge of $1.10 per ton tipping fee). Alternatively, the tipping fee could be eliminated (at a product price of $5.23 per ton) and the recycler could pay up to $1.38 per ton of material (point C) and still achieve a rate of return of 12 percent. A combination of these would also result in achieving a rate of return of 12 percent.

Figure 7. -- Profitability of a 312,000 t/yr recycled aggregates operation.
(Represented by combinations of product price and tipping fee which result in a $NPV_{12} = 0$)



[1] All costs are expressed in terms of U.S. dollars per ton (2,205 pounds).

Figure 7 indicates that under the base conditions, the large (312,000 tons per year) operation model would be more profitable, with an estimated NPV_{12}= $901,000. For the large operation, the average product price could fall to $2.31 per ton (point B) before the operation would no longer achieve a rate of return greater than 12 percent (at $1.10 per ton tipping fee). Alternatively, the tipping fee could be eliminated (at a product price of $5.23 per ton) and the recycler could pay up to $1.69 per ton of material (point C) and still achieve a rate of return of 12

percent. A combination of these would also result in achieving a rate of return of 12 percent. Therefore, larger sized operations have more leeway than smaller sized operations to absorb product price decreases and (or) tipping fee decreases before becoming unprofitable.

This analysis indicated that under assumed conditions (product price of $5.23 per ton and a $1.10 per ton tipping fee), neither the medium nor the large model operations needed a $1.10 per ton tipping fee to achieve a rate of return of 12 percent. For the 253,000 ton-per-year operation, the operating cost could increase by $2.55 per ton, and the operation would still achieve a 12 percent rate of return. For the 312,000 ton-per-year operation, the operating cost could increase by $2.92 per ton under the stated conditions and still achieve a 12 percent rate of return.

Although these figures were developed based on Denver area costs, product prices, and tipping fees, they can be used to represent the product price/tipping fee combinations reported in other areas of the United States to provide an indication of profitability of a recycling operation. Once an appropriate size model is chosen, a particular combination of price and tipping fee can be located on the graph representing that model. For a recycling operation, a positive tipping fee would normally be selected. The intersection point of these two factors would indicate a relative level of profitability one could expect from that operation, depending on whether the point fell above or below the $NPV_{12} = 0$ line and the proximity of that point to the line. If the point fell below the $NPV_{12} = 0$ line, the modeled operation would likely not be profitable, but if the point fell above the line, it would indicate the likelihood of earning at least a 12 percent rate of return.

Note that these graphs are based on an assumption that the facility would operate at 88 percent of maximum capacity. Typically, recycling operations seldom operate at a sustained full capacity level, because of periodic shutdowns to remove deleterious material or (if using a mobile plant) to relocate the plant to a new site. Consequently, the level of reported profitability reported by figures 5 through 7 may vary if productivity differs significantly from the assumed rate of 88 percent. In order to determine the relative impact of productivity variation on operational economics, reduced capacity runs for the large size operation were generated. These showed that a large operation running at about 72 percent of capacity produced the same NPV_{12} = $587,000 value as a mid-size operation operating at 88 percent of capacity. This suggests that the closer to its rated production capacity a recycled aggregates facility operates, the higher rate of return it could earn, all other things being equal.

Consequently, one factor affecting the viability of an aggregates recycler is the availability of feed material. If construction debris or other sources of feed are not consistently available, or if there is some seasonality to the availability of local feed material that limits the ability of the recycler to operate at or near capacity, this would reduce the profitability of an operation. The amount of material available for recycling is limited by the quantity and changing conditions of local construction debris. There is often not enough recycle supply to meet demand for aggregates, so natural aggregates production continues to be the primary source of aggregates in road construction in applications where they can substitute. At best, the contribution of recycled material will grow gradually until all of the available supply is consumed.

Product pricing is often controlled by factors outside the direct control of the recycler. The amount of material presently available from natural aggregate deposits often substantially overshadows the amount of material available from recycling. In the Denver area, for example, recycled aggregates only account for about 1 - 2 percent of the aggregates market and about 20 percent of the road base market; consequently, prices for road base or asphaltic concrete, the principal end uses of recycled aggregates, are often largely a function of the amount of natural aggregates locally available.

Product quality and uniformity can also pose a risk to the potential recycler. Natural aggregate producers continue to supply the bulk of the material for building and road construction because they are able to supply sufficient high quality material for a wide variety of higher valued product applications. Product variability and strength characteristics usually limit the use of recycled aggregates to road base, backfill, and asphalt pavement. Quality is often source dependent, and indiscriminant blending may lead to product degradation. Unless the recycler has established long term contracts for consistent, high-quality feed material, it may be difficult to maintain a predictable revenue stream because of uncertainty related to future feed availability and quality or market price fluctuations. Careful feed monitoring, testing, and marketing can broaden the use of recycled aggregates into other applications.

The analysis suggests that a small operation (operating at or below a level of 110,000 tons per year) might have difficulty operating profitably as a fixed-site operation (NPV_{12} = -$72,000 under market conditions of product price = $5.23 per ton and tipping fee = $1.10 per ton). Smaller operations appear to be more constrained by negative changes in productivity or market conditions, so must adjust their methods of operation to increase their chances of success. The following examples illustrate methods recyclers have used to do this:

1. They can increase tipping fees or charge higher product prices to increase the amount of revenue generated in markets that can absorb these higher amounts.

2. They can geographically reposition themselves to gain a transportation cost advantage over their fixed-site competitors.

3. They can act as subcontractors to the principal contractor on a job site, operating on land that does not require payment for use. This reduces the operating cost for the recycler and reduces the disposal cost for the principal contractor, thus allowing the contractor to pay a higher fee for recycling. The job-site recycler may then be able to offset the additional costs of relocation and set-up.

4. While operating on the site of the prime contractor, the small operator could take on supplementary business on an ad hoc basis, which would spread the costs over a larger output, improve efficiency, and increase revenue to the recycler.

5. When recycling is mandated by States or municipalities (as in California after recent earthquakes) the guaranteed market for the recycler's product removes some of the risk of doing business. In this situation, a contractor must use a specified proportion of recycled

material. This may result in higher prices for recycled aggregates than would otherwise be the case.

Increasingly, recyclers are developing creative cooperative agreements with municipalities to ensure a stable supply of material or establish long term markets. Contractual arrangements with construction contractors and natural aggregate producers is becoming more commonplace as the economic benefit for both parties is recognized.

In summary, the trend towards urbanization in the United States has provided, and probably will continue to provide, a strong demand for high-volume, low-cost aggregates material for repair and development of additional infrastructure. Consumption of recycled aggregates from crushed concrete increased 170 percent between 1994 and 1996, but comprised less than 0.4 percent of total aggregates consumed in 1995. Construction debris supply is regional, and is determined by local infrastructure decay and replacement rates. Aggregate recycling rates are greatest in urban areas where replacement of infrastructure is occurring, natural aggregate resources are limited, disposal costs are high, or strict environmental regulations prevent disposal. Consumption is expected to grow as construction contractors recycle as a means of saving on transportation, disposal, and new material costs and aggregate producers include recycled material in order to prolong the life of their reserves and improve product mix.

The relative costs and charges (tipping fees) of recyclers, their competitors, and landfills determine the amount of material ultimately available for recycling. The cost of transportation has a significant impact on the economics of construction operations. It is not surprising, therefore, that mobile, job-site recycling is becoming common for larger construction projects, as a means of avoiding high transportation, disposal, and new material costs. Even so, the amount of material available nationally for recycling is insufficient to meet present industry demand. On a national basis, it is unlikely that recycling will ever completely replace natural aggregates as road base in road construction.

Aggregates recycling is most likely to be successful where transportation dynamics, disposal and tipping fee structures, resource supply/product markets, and municipal support are favorable. It is apparent that success varies with location. Based upon economic considerations alone, aggregates recycling should continue. Product pricing and tipping fees are the most significant factors influencing the competitiveness of a recycling operation. Under the conditions specified in this analysis, both medium and large recycling operations can operate profitably because of the tipping fee and lower overall costs. Even so, a combination of economic, social, and legislative factors tend to restrict the use of recycled material to lower valued product applications in road construction.

Slight variations in product revenues or tipping fees can affect operational profitability significantly. Equipment capital, operating, and maintenance costs and plant layout and efficiency also have significant impact on economic performance. Operator experience in crushing technology, material handling, and efficient plant design are important to success. Although land requirements are typically small for recycling operations, land costs can have a significant impact on economics, particularly for smaller operations.

Aggregate recycling operations often must overcome risks associated with feed and product availability, pricing, and quality. A facility that is able to maintain a high level of sustained production and secure consistent, long term sources of feed material has a greater chance of success. Similarly, a facility that produces consistent, quality products suitable for diverse markets has a greater opportunity for success Recycling operations that would normally not be profitable as fixed-site operations must use creative methods to establish a market niche for themselves

In spite of these risks, the economic benefit for construction contractors to consider the use of recycled aggregates is substantial. Recycled material can supplement natural aggregates supply. Aggregates recycling appears to be profitable, and in many cases can meet specifications for lower value products such as road base, thereby potentially freeing up higher quality material for higher value applications. The use of recycled aggregates should be evaluated locally based on relative cost, quality, and market factors.

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ABSTRACT

An analysis of the substitution of crushed cement concrete for natural construction aggregates uses a materials flow diagram that tracks all material flows into and out of the cement concrete portion of the products made with cement concrete: highways, roads, and buildings. Crushed cement concrete is only one of the materials flowing into these products, and the amount of crushed cement concrete substituted influences the amount of other materials in the flow. Factors such as availability, as well as physical properties that can affect stability and finishability, influence whether crushed cement concrete or construction aggregates should be used or predominate for a particular end use

Crushed Cement Concrete Substitution for Construction Aggregates - A Materials Flow Analysis

Thomas Kelly[1]

INTRODUCTION

The substitution of crushed cement concrete for natural aggregates can effectively be investigated by applying a materials flow analysis. The amounts of substitution occurring for each end use can be explained by examining the factors that direct the flow of crushed cement concrete into its end uses, that is, the physical-chemical properties of crushed cement concrete, its availability, and transportation costs.

Construction aggregates are natural mineral and rock materials used by the construction industry in portland cement concrete, bituminous concrete pavement, roadbase, construction fill, railroad ballast, rip rap and jetty stone for waterway construction, landscaping and other construction uses. They are used as an inexpensive component in portland cement concrete, roadbase and fill materials where they take up space (bulk) while providing the compressive strength necessary in the final product. In bituminous concrete pavement, construction aggregates provide resistance to creep at elevated temperatures as well as bulk. In portland cement or bituminous concrete pavements, construction aggregates must have the surface properties to allow the physical and chemical bonds which hold these products together. The physical properties required of construction aggregates for their various uses are: bulk, weight, durability, compressive strength, porosity, permeability, inert chemistry, uniformity of composition, and special features such as shape, color, and texture. The natural sources of construction aggregates are crushed stone and sand and gravel. Crushed cement concrete that substitutes for natural aggregates must have a similar combination of physical properties required by the end use to compete effectively.

IMPORTANCE OF CONSTRUCTION AGGREGATES TO THE U.S. ECONOMY

Crushed stone is, by weight, the major raw material used by the construction industry, and sand and gravel aggregate is the second most used material. Together their use is an indicator of the economic well-being of the Nation, because of their close connection to the construction products, highways, roads, and buildings. Figure 1 is an area chart showing the U.S. consumption of these commodities from 1900 to 1995 with a projection of trends in future consumption from the year 1995 to 2020 (Tepordei, 1997, p. 2).

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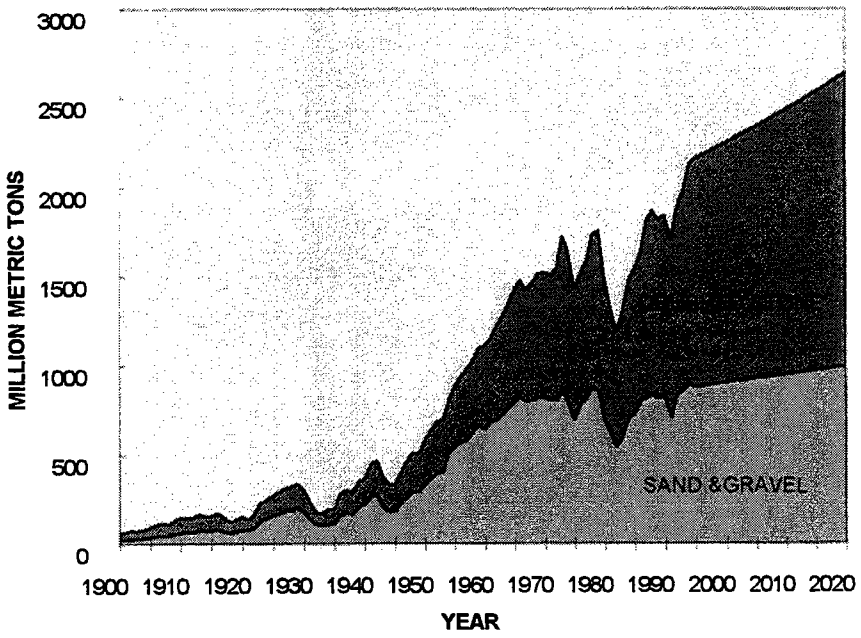
NATURAL AGGREGATE CONSUMPTION IN THE UNITED STATES (HISTORICAL DATA WITH PROJECTIONS)

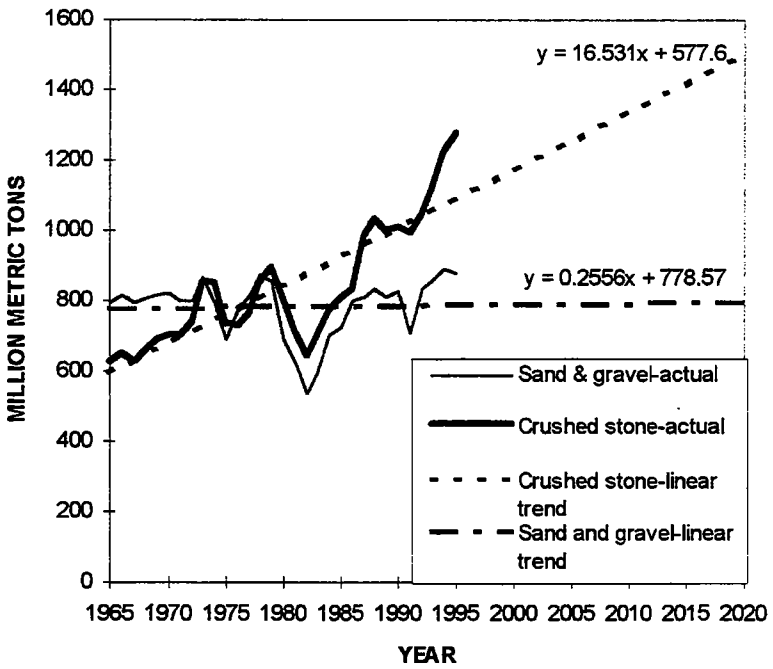


Source: U.S. Geological Survey
Figure 1

Figure 1 indicates that the consumption of natural aggregates increased over the years with a rapid rise in consumption occurring between the years 1945 and 1965. The construction of the interstate highway system and the postwar construction boom were the primary causes for this rise. During the years 1965-1995 two noticeable changes in consumption occurred. The consumption of natural aggregates decreased in slope and three large dips in consumption are visible. The dips in aggregate consumption reach a low point during the recession years 1975, 1982, and 1991. These dips indicate that the consumption of natural aggregates is tied to economic growth in general and growth in the construction industry in particular. Figure 1 projects the consumption for both crushed stone and sand and gravel to increase until the year 2020. From the slope of these projections it is assumed that crushed stone will increase in consumption at a greater rate than sand and gravel.

Another method of illustrating these diverging trends in both materials' consumption is to linearly graph and project the consumption of crushed stone and sand and gravel. A line graph of consumption from the years 1965 to 1995, figure 2, following, indicates that crushed stone consumption surpassed sand and gravel consumption in 1974, traded places back and forth until 1979, and then increased in consumption relative to sand and gravel until 1995.

PROJECTIONS OF U.S. CONSUMPTION OF CRUSHED STONE AND SAND AND GRAVEL



Source: U.S. Geological Survey
Figure 2

A study published by Subhash B. Bhagwat of the Illinois State Geological Survey states that the increased use of crushed stone in Illinois can be explained by two factors: the increased use of limestone and dolomite for pollution control in coal-burning power plants, and the change in quality specifications for construction aggregate materials during the past two decades (Bhagwat, 1997, p. 2). William Langer, Natural Aggregate Resource Geologist for the U.S. Geological Survey, states that another factor influencing this trend towards increased reliance on crushed stone is caused by greater competition from other land uses for areas underlaid with sand and gravel. Linear projections (linear equations shown on graph) of these trends on a national basis, figure 2, highlight the diverging consumption of crushed stone and sand and gravel. This is an important consideration because the substitution of crushed cement concrete for natural aggregates occurs for end uses more often fulfilled by the use of crushed stone than sand and gravel.

THE FUTURE OF CRUSHED CEMENT CONCRETE SUBSTITUTION

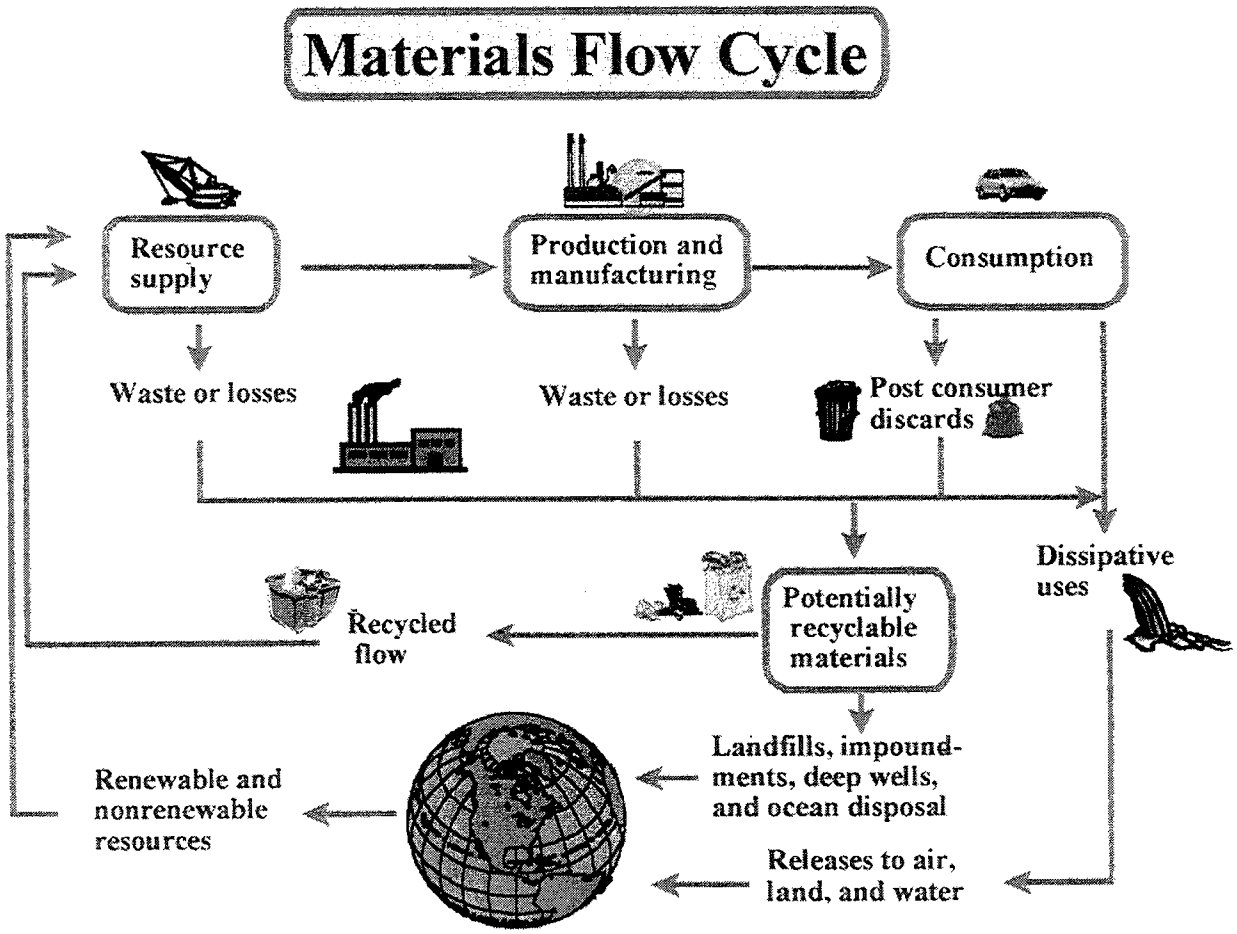
The markets for construction aggregates are largely dependent on population density, because populated areas require more highways, roads, and buildings than do sparsely populated areas (McCarl, 1994, p. 289). This has created a paradoxical situation, whereby the physical placement of construction products may diminish the availability of natural aggregate resources because potential sites of future resources are being used for present construction. Construction aggregates are a high-volume, low-unit-value commodity, making transportation cost a determining factor in competing sources of natural aggregates. Thus location of resource is an important factor for a substitute material such as crushed cement concrete. The use of crushed cement concrete is helping to resolve the problem of diminishing resource availability for two reasons: the populated regions of the country are also the location of an "urban deposit" of concrete (Wilburn and Goonan, 1998 p. 12), and the location of the resource, waste concrete from construction demolition, is near the site of new construction, lowering the transportation cost.

Currently, the annual amount of substitution of crushed cement concrete for construction aggregates is less than 0.4 percent in the United States. Approximately 7 million metric tons of crushed cement concrete

substitutes for construction aggregates, which are consumed at approximately 2 billion metric tons per year. But the use of crushed cement concrete is rapidly increasing. Between 1994 (Collins and Ciesielski, 1994, p. 19) and 1996 (Deal, 1997, p. 10) the estimated amounts of crushed cement concrete used to substitute for construction aggregates increased by more than 160 percent. Natural aggregate producers, who represent only part of crushed cement concrete producers, increased their production of recycled concrete by 37 percent between 1995 and 1996 (Bolen and Tepordei, 1996, tables 16 & 22). Several factors presented herein show why this substitution should continue to increase. A method of tracking this increase is to analyze the flow of construction aggregates through the materials flow cycle.

THE MATERIALS FLOW CYCLE

The materials flow cycle is illustrated in Figure 3.



Source: U.S. Geological Survey
Figure 3

Materials are extracted from the earth in a crude form, refined, and purified, so that they can be manufactured into products. At the end of a product's lifetime, the materials are either discarded or recycled. At every step of a material's lifecycle there are potential material losses. These losses may occur through the discard of wastes or through dissipative losses of material into the environment. Material substitutions can be analyzed by using the materials flow cycle.

MATERIAL SUBSTITUTIONS

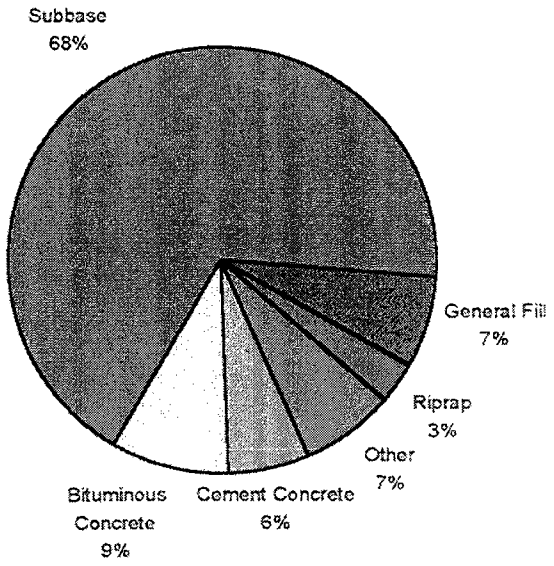
Material substitution occurs when an alternate material is substituted for a traditional material in a product. Substitution often involves more than one commodity, because most products are a combination of several minerals, metals, or plastics. Thus the substitution process must be analyzed at the product level to be understood. Examples of recent substitutions include the material changes in motor oil cans and construction framing studs: plastics have replaced metal and paper in oil cans; steel studs require an alternative form of insulation—foam panels—to produce an exterior residential house wall, a product traditionally insulated with fiberglass. An analysis of material substitution should include flows of all materials into a product, because use of an alternative material may require changes in quantity or types of other materials in the product. Material substitutions may also affect post-consumption flows of the materials to recycling or disposal, because material changes upstream may change the type or quantity of materials released downstream at the end of a product's lifecycle.

A product may contain several end uses for the same material. For example, construction aggregates are used in a loose form as base material for a road and in a bound form for the bituminous or cement concrete pavement at the surface of a road. At the point of substitution, in the product, the flows of all materials to each end use should be given a flow direction and quantity in order to analyze the effect that the current amount of substitution has on the U.S. economy. For crushed cement concrete consumption the relevant economic issues are waste reduction, increased longevity of our natural aggregate resources, environmental impact, and conservation of energy resources.

THE END USES FOR CRUSHED CEMENT CONCRETE

Substitution of crushed cement concrete for construction aggregates occurs for several end uses. The majority of these end uses are part of the products: highways, roads, and buildings. Figure 4 illustrates these end uses and quantifies the percentages flowing to each end use.

END USES OF CRUSHED CEMENT CONCRETE



Source: ***C& D Debris Recycling***, September/October 1997

Figure 4

Uses of crushed cement concrete illustrated in Figure 4 are all substitutions for construction aggregates derived from sand and gravel and crushed stone. Subbase is the aggregate material used for the bottom 25-36 cm in highway construction. Bituminous concrete is the pavement layer of highways that uses approximately 4.5-6 percent by weight asphalt binder mixed with 94-95.5 percent aggregate material for the road surface. Cement

concrete used for highway and building construction is the combination of portland cement, water, and 60-75 percent aggregate. General fill is aggregate used to provide drainage for building foundations, leach fields, or as pipe bedding. Riprap is large chunks of aggregate used for shore protection along rivers or harbors. The "other" category includes end uses such as, landscape rock, and railroad ballast. The breakout in figure 4 is not inclusive of all the end uses for construction aggregates, because crushed cement concrete is not capable of replacing all end uses for construction aggregates. Furthermore, these uses are only part of the materials in roads, highways and buildings; these products require other materials besides construction aggregates for their production.

THE MATERIALS FLOW DIAGRAM FOR SUBSTITUTION

Figure 5 represents the flow of materials involved in the substitution of crushed cement concrete for construction aggregates.

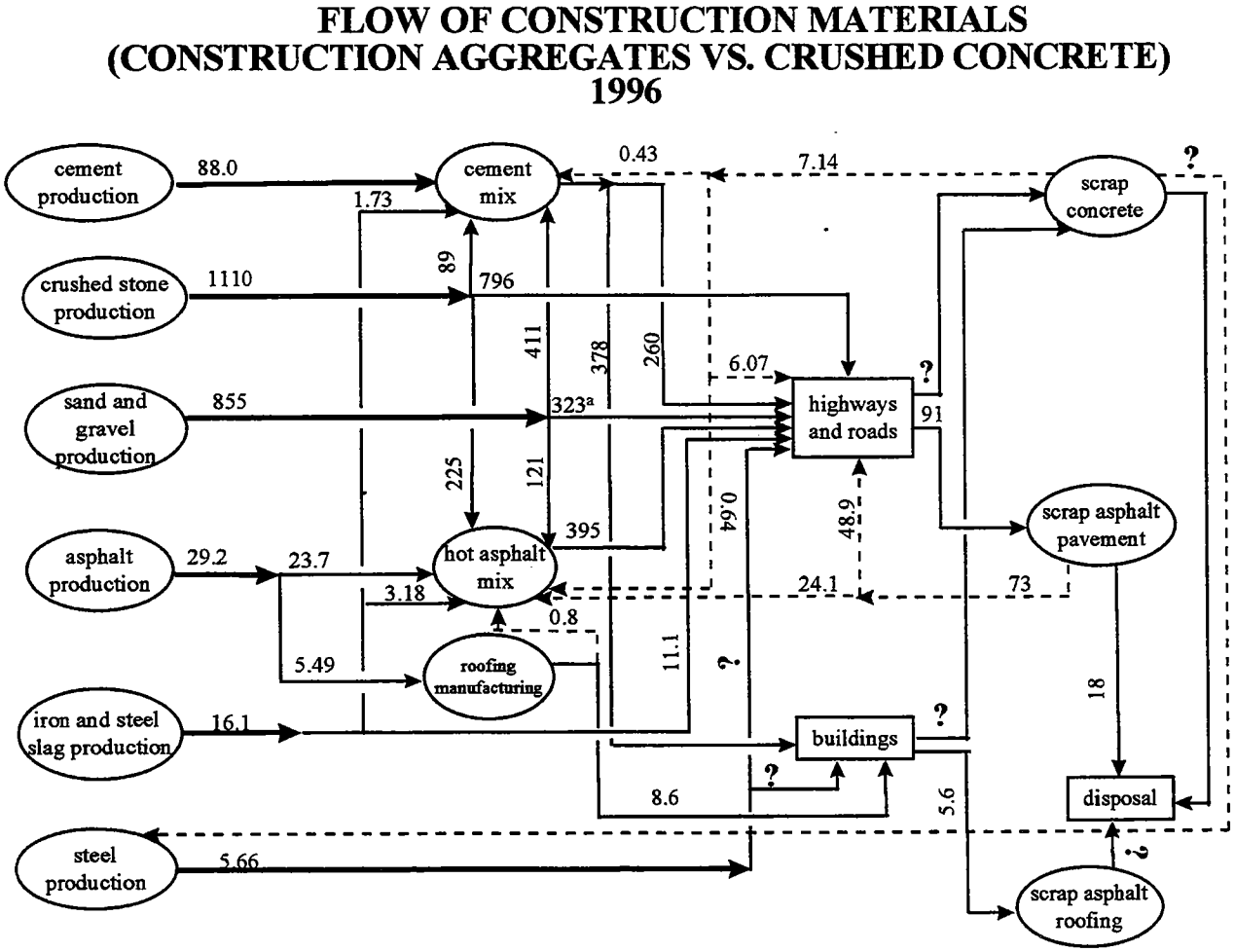


Figure 5

This flow diagram is analogous to the materials flow diagram, figure3, in that it tracks the flow of materials through the material cycle. The figure represents data or estimates for 1996. (See the appendix of the complete paper, Crushed Cement Concrete Substitution for Construction Aggregates: A Material Flow Analysis, for a discussion of this issue. The paper is available online at http://minerals.er.usgs.gov/80/minerals/pubs/) Figure 5 represents the flow of all materials involved in the use of construction aggregates for highways, roads, and buildings, not just the flow of a single commodity. The inclusion of all materials is necessary because the analysis is done from the perspective of a substitution process. As previously mentioned, the substitution of an alternative

material for a traditional material should be analyzed at the product level, because of the possible changes in material flows into and out of a product when a substitution occurs.

This materials flow diagram is designed to investigate the substitution process of crushed cement concrete for construction aggregates in two products, "highways and roads" and "buildings." Therefore, those uses of construction aggregates for which crushed cement concrete will not substitute are eliminated. For example, crushed limestone used to produce cement cannot be replaced with crushed cement concrete, so this flow is not included on the diagram and this amount of crushed stone is not included in the crushed stone commodity flow. Maintaining this exclusive consideration of only those materials involved in the substitution establishes a quantitative relationship between the amount of materials flowing into and out of the products. The substitution process is quantified so that the amount of material that can be replaced, construction aggregates used in highways, roads, and buildings, can be compared to the amount of material available for substitution, crushed cement concrete.

The construction aggregates used this year will, of course, not be available for recycling as crushed cement concrete for many years. To indicate the time dependency of the data, ovals and rectangles are used. All of the data flowing into or out of the ovals (processes) span a year's time, whereas the data flowing into and out of a rectangle (stock) span more than a year's time. By subtracting the known inflows from a known outflow, an unknown inflow may be estimated in a process oval, or by subtracting known split flows from a total flow an unknown split flow may be calculated. (Split flows occur downstream of a total commodity or recycled flow.) It is necessary that all flows except one into or out of an oval are known in order to do a material balance on a process oval. This condition is met for the ovals "cement mix" and "hot asphalt mix" but not for "roofing manufacturing." Material balances were performed on the "cement mix" and "hot asphalt mix" ovals to calculate the amounts of crushed stone flowing into each process, and the total of these two flows was subtracted from the total crushed stone flow to calculate crushed stone used for "roadbase and others." The ability to calculate unknown flows is one example of the usefulness of this materials flow diagram.

Commodity flows are indicated with heavy black arrows which split into individual flows, thin black arrows, that lead to end uses in the products. Smaller downstream flows from the products are indicated with thin black arrows also, and recycling flows are indicated with dashed arrows. Recycling flows can also split into smaller quantity flows so that they can be directed to the particular end use for which they substitute. The details of the calculations performed and data sources used in order to produce this materials flow diagram are included in the appendix of the complete paper as mentioned previously.

Some lumping of end uses was done to simplify the flow diagram, figure 5. The categories subbase, general fill, riprap, and other in figure 4 are combined into a category called "roadbase and others." This lumping of categories is illustrated in figure 6 following. This lumped flow is directed into the product or stock, "highways and roads" on the flow diagram for several reasons. It is impossible to indicate to which product some of the end uses flow because the data are unavailable. For example, construction fill is not always associated with roads or highways. Buildings also use some aggregate for construction fill, and some construction fill, such as pipe bedding may not be accurately associated with either "highways and roads" or "buildings." However, the amount of flow to "highways and roads" as roadbase material is so much greater than the flows to the other end uses, "bituminous concrete" and "cement concrete," that less error is introduced by associating all small categories with "highways and roads." Moreover, the physical properties required for roadbase and the other small categories are more alike than the physical properties required of the other two categories, "bituminous concrete" and "cement concrete."

END USES OF CRUSHED CEMENT CONCRETE

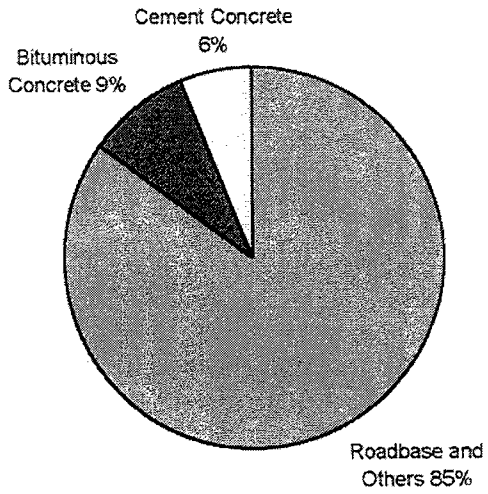
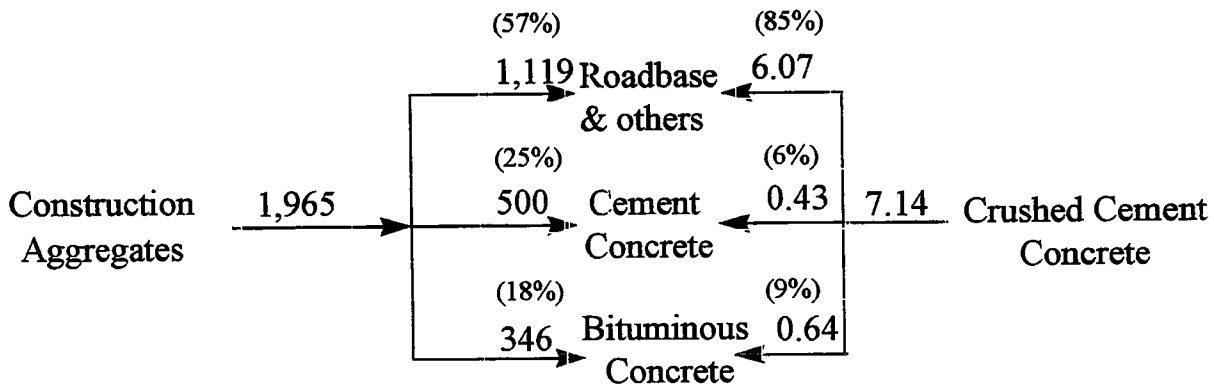


Figure 6

The amounts of crushed cement concrete flowing to these three end uses shown in Figure 6 are compared to the amounts of construction aggregates flowing to these same end uses in Figure 7.

FLOWS INTO END USES
CONSTRUCTION AGGREGATES VS. CRUSHED CEMENT CONCRETE



Flows in million metric tons
1996

Figure 7

The construction aggregates that flow to these end uses are those reported for sand and gravel and crushed stone in the U.S. Geological Survey (USGS) *Minerals Yearbook* (Bolen and Tepordei, 1995, table 1). The amount

of crushed cement concrete that flows to these end uses is reported in a joint survey taken by Vanderbilt University and C&D Debris Recycling, an industry association magazine. (Another survey is currently being done by the Environmental Council of Concrete Organizations, and results are expected in late Spring of 1998.) Comparison of the total flow quantities of construction aggregates versus crushed cement concrete indicates that substitution is occurring at less than 0.4 percent per year, (7.14/1,965 = 0.0036). Also noticeable is that 85 percent of the crushed cement concrete flows to the end use, "roadbase and others," versus 57 percent of the construction aggregates. This can be explained partially by availability of crushed cement concrete and transportation costs. However, the physical properties of crushed cement concrete such as its absorption and specific gravity also contribute to this preferred direction of flow.

AVAILABILITY AND TRANSPORTATION COST FACTORS IN USE AS ROADBASE

Eighty-five percent of concrete recyclers have the ability to go to the job site in order to crush concrete (Deal, 1997, p. 10). Only 24 percent are actually going to the job site to crush concrete, but the ability to locate a yard near a job site can also be a consideration in the costs of recycling. Because transportation cost is a major competitive factor among sources of aggregate, portability of the crushing operation can provide the cost advantage to crushed cement concrete over other aggregate sources at a particular job site. This is especially true for roadbase, because the material can be crushed and set in place on site. Crushed cement concrete for use in bituminous and cement concretes, however, may require transportation from the site of demolition to a crushing operation then to a mixing plant and finally back to a construction site. The advantage in transportation costs for roadbase materials can be viewed as an advantage in availability as well as an advantage in transportation cost. It is as if the mine site were located on the construction site.

Cement concrete pavements in highways, roads, parking lots, airport runways, and sidewalks contain less steel than the concrete demolished from buildings. Steel is generally not placed in pavements layers, because it can cause cracking during freeze-thaw cycles. This also increases the recyclability of the crushed cement concrete from pavements, because it is easier to produce a product without the complication of having to remove and recycle or dispose of the steel byproduct of the crushing operation.

EFFECTS OF PHYSICAL PROPERTIES OF CRUSHED CEMENT CONCRETE ON ITS USE

The difference in physical properties between natural aggregates and crushed cement concrete is strongly influenced by the inclusion of the cement paste surrounding the aggregate in crushed cement concrete, according to Dr. Stephen W. Forster, geologist for the Federal Highway Administration (FHWA). Physical properties of this paste are markedly different from the properties of natural aggregates. The specific gravity of crushed cement concrete is commonly lower than that of the natural aggregate used to make the concrete, and the absorption of water is always higher. The cement paste component of crushed cement concrete produces more fines than when stone is crushed. These fines have an absorption as much as 8 times greater than fines produced by crushing stone. As a result the use of crushed cement concrete in hot asphalt mix requires the use of more asphalt binder than does the use of crushed stone, which then increase the cost of bituminous concrete because asphalt is the most expensive component of bituminous concrete pavement.

If crushed cement concrete is being used for aggregate in cement concrete mix, specific gravity, absorption, fineness, and angularity are all important physical properties. Crushed cement concrete's absorption, when substituted in cement concrete, requires that more water be used in the cement mixing ratio than when natural aggregate is used. Furthermore, the irregular surface of crushed cement concrete has more nooks and crannies than does natural aggregate and this causes more cement to be used to fill these small holes (Wilson,1993, p. 4). Cement is the most expensive component of cement concrete, so this adds to the cost of using crushed cement concrete. The angularity of crushed cement concrete, especially in the fine sizes, affects concrete finishability and workability. Experience indicates that up to 75 percent natural aggregate fines may be necessary to obtain a workable cement concrete mix that is capable of being finished, according to Dr. Forster. This is a problem with the use of crushed stone sand also, and it is resolved by the use of natural sand (McCarl, 1983, p.72). The availability and transportation costs of natural sand may affect the amount of crushed cement concrete used in a cement mix.

For the category "roadbase and others" crushed cement concrete must have the appropriate physical properties that permit servicing that particular end use. For both roadbase and fill material, the fines may be a problem where drainability is required. Washing the crushed cement concrete to remove fines may be required, but this adds to the cost of using crushed cement concrete. Dissolution of the crushed cement concrete can occur over time in a fill or roadbase application, so the soundness of the concrete may be an important factor for these end uses. Soundness is a test that measures the aggregate's general resistance to environmental exposure, including heating and cooling, wetting and drying, and freezing and thawing. If dissolution occurs, a possible consequence is an elevation of ground water pH by the leaching of calcium hydroxide from the cement paste. If this ground water contacts air, calcium carbonate will precipitate out of the solution, possibly clogging drainage systems. Contact between vegetation and the high-pH ground water may cause vegetation damage (Snyder, 1995, p. 3). Dense roadbases will be less susceptible to leaching, and, in fact, may even become more tightly cemented together by the chemical activity occurring when water passes through the material. (This fact was noted by Stephen W. Forster of the Federal Highway Administration (FHWA) on October 5, 1997 in a talk given at a Construction Materials Recycling Seminar in Minneapolis.) Whether the use is for roadbase or fill, crushed cement concrete must be able to support the load applied to it over time, so the ability to retain compressive strength as measured by soundness, is a property that must be considered.

For other uses in the "roadbase and others" category, (riprap or jetty stone or any use in which weathering may have an effect on the useful life of the product), soundness may be an important consideration. Aesthetic properties are a consideration for use as landscaping rock. There have been attempts to paint crushed cement concrete to increase its aesthetic appeal. This extra treatment adds to the cost of using crushed cement concrete.

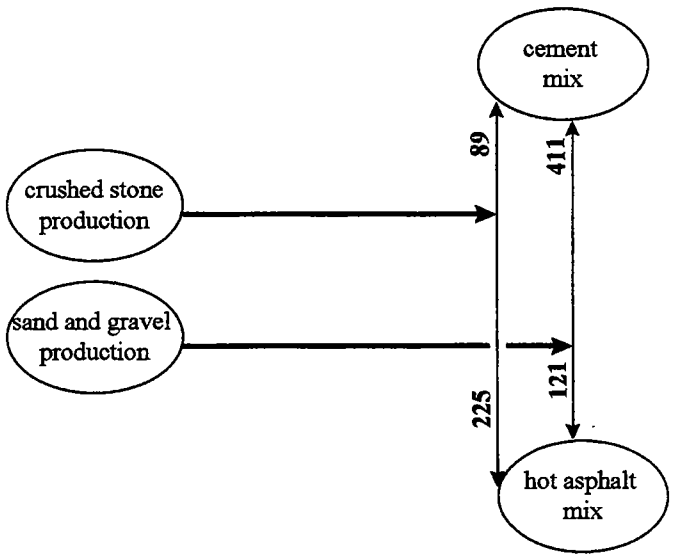
In summary, crushed cement concrete must meet the physical specifications required by the end use in order to compete with construction aggregates from natural sources. It is able to meet these specifications for all three categories of end uses, "roadbase and others," "bituminous concrete," and "cement concrete." For use as roadbase material, transportation cost is an important factor in the preference exhibited in the material flow of crushed cement concrete to this end use. The higher costs of using crushed cement concrete in bituminous concrete or cement concrete are due to its increased absorption, requiring more asphalt or cement—a clear example of a materials flow being changed by a substitution process. Note that for these two end uses the materials flow is changed in the quantity, not the type of materials required. However, because construction aggregate is a mix of coarse and fine aggregates, the use of crushed cement concrete may also change the mix requirements of the aggregate, depending on the end use. If crushed cement concrete is used for cement concrete, the natural sand component of the mix may need to be increased, to give finishability and workability to the cement mix.

OTHER APPLICATIONS FOR THE MATERIALS FLOW DIAGRAM

The materials flow diagram, figure 3, is a first step in tracking a material through its lifecycle from resource production through manufacturing processes, consumption and disposal or recycling. Laying out this flow clarifies the processes affecting a material's flow. These processes can then be studied in greater detail so that estimation can be made of the material losses occurring in each process. The same knowledge can be obtained from a materials flow diagram designed to study a material substitution such as figure 5. Moreover, figure 5 establishes a quantitative relationship between flows for several materials which can be tracked over time to measure changes in the substitution process. But, as figure 5 also demonstrates, one needs more than a simple comparison of the substitution of crushed cement concrete for construction aggregates to understand the flow of construction aggregates. Construction aggregates are recycled from waste bituminous pavements, asphalt roofing materials, and iron and steel slag. All these forms of recycling help meet demand for construction aggregates. By presenting the known data in an organized manner as in figure 5, a greater understanding of the processes involved in several materials' lifecycles is achieved because both the flow direction and quantity of material flow are illustrated.

Another example of an increased understanding of the flow of materials in the construction industry is given by examining the preferential flows of sand and gravel to "cement mix" and of crushed stone to "hot asphalt mix." Figure 8 is an excerpt of the materials flow diagram, figure 5.

FLOW OF NATURAL AGGREGATES
1996



Flows in million metric tons

Figure 8

Figure 8 shows that sand and gravel flow preferentially to cement mix and crushed stone flows preferentially to hot asphalt mix. The reason for the preferential flows is the different shapes of the two materials. Construction aggregates are a mixture of fine and coarse aggregates and the mixture may vary depending on the product for which the aggregate is to be used. Larger sizes of coarse aggregate are used for more massive structures. In areas where natural sand is scarce, stone may be crushed to produce a stone sand used for the fine portion of the aggregate mix. A mixture of crushed stone and sand and gravel may be used to give the aggregate mix the properties necessary for a particular product. Sand and gravel mix has sphericity in both the coarse and fine aggregate components and crushed stone has a sharp angular shape. The sphericity of fine aggregate in sand and gravel provides cement mix the ability to flow into forms, workability, and to be finished with a smooth surface, finishability. The coarse aggregate, gravel, in a sand and gravel mix does not provide resistance to creep in bituminous concrete pavement because the round edges of the gravel slide past one another with less resistance than the sharp angular faces of crushed stone. Thus crushed stone flows preferentially to hot asphalt mix. Furthermore, the fine aggregate portion of crushed stone, crushed stone sand, does not give the workability and finishability provided by the fine aggregate component of sand and gravel. Thus a mix of materials may be used to give the properties required for the product. This is another example of the usefulness of the materials flow diagram. Analyzing all materials used to make the products "highways and roads" and "buildings", highlights differences between material flows.

The crushed cement concrete flows illustrated in figure 7 also show this preference in flow. Crushed cement concrete has a shape closer to crushed stone than does sand and gravel mix. It flows preferentially to "bituminous concrete" in comparison to flow towards "cement concrete." Crushed cement concrete is more angular than sand and gravel and thus can provide resistance to creep in asphalt pavement made with bituminous concrete.

A STATIC REPRESENTATION OF A DYNAMIC PROCESS

Figure 5 is a static representation for the year 1996 of the flow of materials through their lifecycles. In order to more accurately use this information to predict the future for the substitution process, a dynamic model would have to be developed. Looking at the changing quantitative relationships between materials in the past would be a first step at developing this model. Figure 1 is a historical look at the use of natural aggregates in the United States since 1900. The rapid rise in consumption of natural aggregates from 1945 to 1965 may indicate a

future rapid rise in availability of demolished concrete. To predict the timing of this release of crushed cement concrete requires an estimate of lifetimes of construction products. Studying the consumption patterns in the past may lead to an understanding of those factors which will affect the lifetimes of construction products. These factors could be included in a dynamic model so estimates of future substitution rates could be made based on a projection of these factors into the future. For example, the ratio between building construction, and road and highway construction may have an effect on the amount of concrete demolished in the future, because highways and roads may have shorter lifetimes than buildings. Knowing the factors which influence the ratio in the past may allow more accurate predictions of concrete available to be crushed in the future. With so many variables to consider, the problem of predicting quickly becomes complicated, making a dynamic model necessary. Factors expected to influence the substitution process could be evaluated by performing several runs on the model analyzing the sensitivity of the model to the factors.

Even this simple static model contains many different data sources and assumptions and the author realizes that it is only a first step in analyzing the material flows in figure 5. If another source of information is perceived as more accurate than that used by the author, a contact will be greatly appreciated.

CONCLUSIONS AND IMPLICATIONS FOR THE FUTURE

This paper demonstrates that a materials flow analysis examining a material substitution can lead to a greater understanding of the flow of several materials through their lifetimes. The materials flow diagram, figure 5, provides an understanding of both the direction of materials' flows and the quantity of these flows. This information can be used to provide an overview of the use of these materials in the construction industry, or a section of the diagram can be isolated and investigated for the relationships between just a few material flows. An example of an overview is the comparison made in studying the substitution of crushed cement concrete for construction aggregates. This was the original goal of this paper and it led to designing the materials flow diagram, figure 5, with specified constraints on the data included. Once the diagram was produced, however, more relationships among materials' flows became evident. An example of this is the preferential flows of sand and gravel and crushed stone illustrated in figure 8, an isolated portion of the total flow diagram. The overview of flow relationships provided by figure 5 can lead to new questions concerning the use of materials in the construction industry. For example: How much substitution will occur in the future? The large difference between concrete put in place in 1996 and concrete recycled indicates that a future supply of material is potentially available; will it meet the specifications for the end uses? The preferential flows illustrated in figure 8 take on greater importance when they can be compared through time; will sand and gravel continue to decline in use compared to crushed stone? The relationships between material use and substitution in 1996 provide insights into demand for natural materials and waste reduction in the future; should this pattern of materials flows shown in figure 5 be tracked over time?

Finally, some other advantages to organizing the data into a flow diagram are that material balances can be used to calculate unknown flows and to reveal data gaps. In this paper, crushed stone flows were calculated by material balance, and the important data gaps concerning materials flowing from demolition of highways, roads, and buildings were noted.

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THE ULTIMATE ROADSIDE GEOLOGY: GEOLOGICAL HERITAGE SITES

Terry S. Maley

ABSTRACT

Geological heritage conservation means the planned and careful management of geologically significant features and landscapes to protect their natural, scientific, historic and aesthetic values for the benefit of future generations. Geological heritage sites may have scientific, historic, and aesthetic significance. Scientifically significant sites consist of geological features, rock or mineral types, paleontological resource, and landscapes that represent the best example of their kind or are so uncommon that they have great significance for science education and scientific research. Historically significant sites are places where cultural activities or events occurred because of some geological feature or landscape. Categories of historically significant sites include: (1) geological sites significant to the history of geology; (2) sacred geography of Native Americans; (3) old mining districts; and (4) geological features associated with western exploration and settlement. Aesthetically significant sites include landscapes that are visually appealing because of a particular geological setting. Many geological heritage sites have been lost or degraded by human action. Common causes for the degradation of geological sites include extraction under the mining law, flooding, vandalism, tourism, criminal theft, air pollution, and commercialization.

INTRODUCTION

Earth Heritage Programs

Public land management agencies in the United States need a systematic and well-defined geological heritage conservation program to recognize and manage geological heritage sites for their scientific, historic and aesthetic values (Maley and Randolph, 1993 and 1994). In the past, recognition of heritage sites has been haphazard, incomplete and biased towards resources that have spectacular scenic or other attractive qualities. The term "heritage" as applied to geologically significant features and landscapes means we received these features as part of our birthright and have the opportunity to benefit from them during our lifetime; but, we also have the responsibility to pass them to future generations undiminished in quality and quantity. The term "conservation" refers to the planned and careful management of these natural features and landscapes to protect their natural and scientific values. For the most part, small-scale (less than 1 km^2) geologically significant features and landscapes, which do not have special scenic qualities, have been ignored.

Geologic features and landscapes may be significant for their scientific, historic, and aesthetic values. In some cases a single geologic site such as Yellowstone National Park may be significant for all three types of values.

Scientifically Significant Sites

Scientifically significant sites consist of geologic features, rock or mineral types, paleontological resources and landscapes that (1) represent the best example of their kind; (2) are textbook examples; and (3) are unique or rare. Such sites generally have great significance for science education and scientific research.

Historically Significant Sites

Cultural activities, events or places occur because of some geological resource. As Savoy (1992, pg. 218) observed "geologic and geographic setting of regions—the physical landscape—has influenced or controlled patterns of human exploitation and settlement as well as the life ways of peoples and their cultural landscape." Categories of historically significant sites include: (1) geological sites significant to the history of geology; (2) sacred geography of Native Americans, including cultural landscapes, migration and trade routes, meeting places, hunting areas, hunting camps and winter camps; (3) old mining districts; (4) geologic features associated with western exploration and settlement; and (5) major historic battlefields where geology played a significant role in the outcome.

The Silent City of Rocks in south-central Idaho, located approximately 7 km north of the Idaho-Utah border, is an excellent example of a scientifically significant site that also has significant historical values. Although jointing controls the general form of outcrops in the City of Rocks, spheroidal weathering with granular disintegration is the agent responsible for creating the bizarre and remarkable shapes that characterize the area. Case hardening of the outer surface of the granitic forms is also an important mechanism in creating unusual forms such as caves, niches, arches, bath tubs and sinks, hollow boulders and toadstools.

The Silent City of Rocks is recognized as both a natural and historic landmark and has been included into the Federal park system because of its historical and geological significance. The junction of the California and Salt Lake-California connection trails is located near the City of Rocks, and the California Trail, which passed through the City of Rocks, was established in 1843. Joseph Walker led a wagon train off the Oregon Trail at Raft River 80 km to the northeast, through Almo, then through the City of Rocks and on to California. Immigrants were fascinated by the City of Rocks and those who maintained diaries recorded their impressions. Surprisingly, it was not until 1988 that the Silent City of Rocks finally received Congressional protection through designation as the City of Rocks National Reserve (Public Law 100-696).

Aesthetically Significant Sites

Many landscapes are visually appealing because of a particular geologic setting and have been withdrawn by Congressional action as wilderness areas, National Recreation Areas, National Parks, National Monuments and Wild and Scenic Rivers. Outstanding examples of aesthetically significant geological sites are Glacier National Park, Yosemite National Park and the Grand Canyon National Park. Generally, this type of site has been recognized and protected.

Categories of Geological Features and Landscapes

Geological heritage sites include a variety of features and landscapes with a great diversity of size, shape and geologic history. The following examples of geological features and landscapes are given to illustrate some, but by no means all, of the possible categories:

1. The place where a geologic feature, rock type, type specimen of a fossil was first recognized and described.

> *Examples*: **Snake River Flood Gold** (Wyoming and Idaho)— the 1200-km-long Snake River, from the headquarters near Yellowstone National Park to Lewiston Idaho, is the type locality for flood gold occurrences in the United States; **Mount Monadnock** (New Hampshire)—the type locality of a monadnock, or isolated mountain remnant; **Como Bluff** (Wyoming)—first and best discoveries of certain Jurassic mammals including 80 new vertebrate species; **Morrison Fossil Area** (Colorado)—First site of discovery of giant dinosaur fossil bones in North America.

2. Historically significant sites where original contributions to the understanding of geologic processes or principles were inspired.

> *Example*: **Hutton's Geological Sites in Scotland**. Geological sites such as Siccar Point and Salisbury Crags had a major effect on the evolution of geologic thought. Though these sites are outstanding examples of their kind, they are primarily significant for their historical role as the specific sites where a scientific breakthrough was accomplished.

3. Textbook examples of geological features and processes.

> *Examples*: **Ship Rock** (New Mexico)—an outstanding example of an exposed volcanic neck with spectacular radiating dikes; **Spanish Peaks** (Colorado)—one of the best exposed examples of igneous dikes; **Valley of Fire** (Nevada)—an outstanding example of overthrusting with associated fold.

4. Paleontological sites and other sites that contain scientifically significant stages of biological evolution in the fossil record.

> *Examples*: **Dinosaur Trackway** (Colorado)—one of the largest known exposures of dinosaur tracks on a single bedding plane with some 1,000 tracks; **Hagerman Fossil Beds** (Idaho)—the world's greatest variety and numbers of animals from Upper Pliocene; **Indian Springs Trace Fossil Site** (Colorado)—the best trace fossil locality in North America that shows the markings of Ordovician animal life; **Mammoth Site** (Hot Springs, South Dakota)—one of the largest concentrations of mammoth remains in the United States; **Rancho La Brea** (California)—natural asphalt tar pits in which Pleistocene animals became entrapped.

5. Features created by wind (dunes, hoodoos, arches), water (canyons, bedrock forms), ice (cirques, aretes, moraine, etc.), weathering (hoodoos, arches, colored rocks) and mass wasting (landslides, rockfalls, talus, etc.).

Examples: **Nebraska Sand Hills** (Nebraska)—the largest sand dunes complex in the western hemisphere; **Bruneau Sand Dunes** (Idaho)—the westernmost dune is reported to be the largest single sand dune in North America, standing about 160 m above the level of a nearby lake; **Malaspina Glacier** (Alaska)—the largest piedmont glacier in North America; **Mount Katahdin** (Maine)—outstanding example of glacial features such as eskers, drumlins, kames, kettleholes and moraines; **Moss Island** (New York)—the best exposure of glacial age potholes eroded by meltwater floods in the eastern United States.

6. Caves and karst topography.

Examples: **Craighead Caverns** (Tennessee)—a cavern system with the largest known underground lake in the United States; **Cumberland Cavern** (Tennessee)—one of the largest cave systems in the country; **Wyandotte Cave** (Illinois)—one of the outstanding cave system within the karst region of the east-central United States; **Grassy Cover Karst Area** (Tennessee)—one of the best examples of karst development and underground drainage in the United States.

7. Hot springs, artesian springs and aquifers.

Examples: **Ichetucknee Springs** (Florida)—an example of a large artesian spring group and the Floridian aquifer from which Florida's great springs emerge; **Thousand Springs and the Snake River Plain Aquifer** (Idaho)—the Snake River Plain aquifer is a remarkable aquifer of great resource and economic significance. For about 160 km downstream from Twin Falls an estimated volume of 5.8 km^3 of water per year enter the Snake River from gigantic springs on the north side of the canyon making it one of the world's most productive ground water systems (Whitehead and Covington, 1987; Malde, 1991). According to Meinzer (1927) and Malde (1991) this 160 km reach of the canyon has 11 of the 65 springs in the United States with an average discharge exceeding 2.8 m^3/s.

8. Geologic features that offer classic research or educational opportunities.

Example: **The Great Rift and Associated Volcanic Features** (Idaho). The Great Rift system (Russell, 1902; Murtaugh, 1961; Prinz, 1970) consists of a series of north-northwest-trending fractures, which extend 85 km from the northern margin of the eastern Snake River Plain to the Snake River. Craters of the Moon National Monument is an outdoor museum of volcanic features. Features such as crater wall fragments, cinder cones, mini-shield volcanoes, lava bombs, spatter cones, lava tubes, tree molds and rifts are among the best examples you can observe anywhere in North America.

9. Outstanding examples of significant stages in the earth's evolutionary history (paleosols, stratigraphic sections, varves, etc.).

> *Examples*: **Red Mountain Expressway Cut** (Alabama)—exposes a remarkable combination of structural and stratigraphic features that document the Paleozoic development of the southern Appalachian fold belt; **The Burgess Shale Fauna**. The world-famous Burgess shale locality in the Canadian Rockies contains remarkably well-preserved soft-bodied animals and represents one of the most important paleontological discoveries ever made. Charles D. Walcott, an American specialist in the Cambrian discovered the fossils in 1909 when his pack horses tripped over a block of shale with an exposed fossilized bedding surface. Although it was once believed that the Burgess Shale contained a restricted local fauna, new discoveries of Burgess Shale-type fossils in the Cambrian of North American, north Greenland and South China established that soft-bodied animals were widely distributed in Cambrian seas (Cowen, 1995).

10. Variety of related and significant geologic features within a small geographic area; even though any one feature may not be worthy of special recognition, the combination of related features in proximity may be unique and geologically significant.

> *Example*: **Catastrophic Bonneville Flood Features.** The Bonneville Flood is one of the largest floods known from the geologic record. Gilbert (1878, 1890) recognized the evidence for a catastrophic flood during field study of Pleistocene Lake Bonneville between 1876 and 1879). The size of the Bonneville Flood is only second to the Missoula or Bretz Flood which flooded the northwest numerous times (Malde, 1968). Pluvial Lake Bonneville once covered a vast area in northwest Utah to levels almost 335 m above the present level of the Great Salt Lake. Approximately 14,400 years ago (Cerling, 1990), 4,700 km^3 of water (O'Connor, 1993) flowed over Red Rock Pass in southeast Idaho and then flowed westward, following the path of the present Snake River. The surface features of this flood are very similar in form and scale to those of the Missoula Flood.

11. Archeological and historical sites that have geological significance such as mining districts, sacred geography, battlefields, obsidian flows used for tool making, and rock shelters.

> *Examples*: **Devils Tower** (Wyoming)—sacred geography for 20 tribes; **Wallowa Lake** (Oregon)—sacred geography for Nez Perce tribe; **Gettysburg National Military Park** (Pennsylvania)—geologically significant battlefield; **Chinese Placer Mines** (Idaho)—about the turn of the century and until at least 1912, Chinese miners followed paystreaks into placer gravels deposited by Salmon River.

12. Geological curiosities such as erratics, meteorites, nonvolcanic craters, hoodoos, etc.

> *Examples*: **Odessa Meteor Crater** (Texas)—one of only two known meteor sites in the United States; **Barringer Meteor Crater** (Arizona)—the largest impact crater discovered in the United States; **Madison Boulder** (New Hampshire)—the largest known glacial

erratic in North America; **Natural Bridge** (Virginia)—a karst bridge that also serves as a highway bridge.

13. Unique or uncommon rock or mineral sites.

    Example: **Palisades of the Hudson** (New York)—one of the best examples of a diabase sill known in the United States; **Square Butte** (Montana)—one of the best examples of banded magmatic rock in the United States; **Orbicular rocks near Shoup** (Idaho)—outcrop of orbicular rock with exceptionally well formed orbicules (Maley, 1975).

14. Geological features, formations and landscapes that have exceptional natural beauty.

    Examples: Most of the national parks and monuments, particularly in the western United States, fall in this category. Some of outstanding examples include Grand Canyon National Park, Glacier National Park, Yosemite National Park, Yellowstone National Park, Death Valley National Park and Crater Lake National Park.

THREATS TO EARTH HERITAGE SITES

Many geologically significant landscapes have been lost or degraded by human action. Many old photographs are the only record of the way these landscapes appeared before such loss or degradation. Many river valleys have been obliterated by gold dredging or dams. Common causes for the degradation of geological sites include (1) extraction under the mining law, (2) flooding, (3) vandalism, (4) tourism and vandalism, (5) criminal theft, (6) air pollution, and (7) commercialization.

Extraction Under the 1872 Mining Law

Since 1866 the mining laws of the United States have authorized removal of valuable deposits of minerals from public lands. For example, no one will ever be able to study the bonanza native silver lodes that once cropped out on the mountain ranges of Nevada. It is also well established in case law that mineral specimens, gem stones and gem minerals may be appropriated and extracted from the public lands under the Mining Law.

Flooding

Celilo Falls, a magnificent falls on the Columbia River east of the Dalles, Oregon, served as the ancestral fishing grounds for native Americans. The Dalles hydroelectric dam flooded the area and inundated the falls during the 1950s. The sacred fishing site was used for numerous intertribal ceremonies of the northwest, including those related to fertility, thanksgiving and Salmon (Walker, 1988).

Vandalism and Tourism

A combination of tourism and vandalism can cause degradation to a site. In addition to improvements such as access roads and support facilities, removal of "specimens" from a small source of unique rock or mineral can seriously degrade or ruin a site. Even if the scientific values remain, the cumulative effect of many samples taken by visitors may cause serious attrition to a site. The Belleview erratic is an excellent example of the threat "tourist attrition" poses to geologic attractions of a limited size.

The Belleview erratic is the largest of more than 300 known ice-rafted boulders carried from glacial Lake Missoula in Montana to the Willamette Valley in western Oregon by one of the catastrophic Spokane Floods about 14,000 years ago (Allison, 1935). This argillite boulder from the northern Rocky Mountains weighed about 160 tons when it was first measured in 1950. However, by 1980 "tourist attrition" had carried off approximately 70 tons or about 2.3 tons per year. Although the boulder is protected by a small state park established to recognize it, as Allen and others (1986, p. 182) observed:

> If well-meaning but uninformed tourists continue to take "souvenirs" at this rate, this unique geologic phenomenon will have disappeared by the year 2019.

Criminal Theft

In the Columbia National Wildlife Refuge, located about 10 miles northwest of Othello, Washington, a unique outcrop of columnar basalt was exposed by the Spokane Flood. The general area has also been designated the Drumheller Channels National Natural Landmark interpretive site because of a remarkable landscape left by the catastrophic Spokane Floods that swept across southeast Washington numerous times between 12,000 and 17,000 years ago. These gigantic floods, each containing more than 500 cubic miles of water, scoured the channeled scablands across Washington as they moved southwestward following the drainage of the Columbia River. In the vicinity of the Drumheller Channels, outstanding scabland features such as anastomosing coulees and empty-rock basins were carved and are now occupied by numerous lakes and form a unique landscape and ecosystem. The uppermost flow of Columbia River Basalt was partly peeled away by the violent flood waters; however several remnants of the flow remain as mesas. This basalt flow is characterized by remarkably well-developed columnar jointing, with individual columns about 0.5 meter in diameter and 7 to 10 m tall. On the downstream side of one of the mesas, the basalt columns were separated and pulled away from the rock wall by the force of current and stand as isolated rock towers separated by up to several meters from the mesa rim. There may be no other example in the world of large five and six-sided columns of basalt completely separated from the outcrop.

In 1994 an individual illegally entered the Refuge with large equipment and removed four of the columns and damaged several others in the extraction process. Needless to say he removed the best examples and the damage is irreparable because the columns cannot be restored to their original position. The refuge manager published a notice in the local newspaper in an effort to learn the identity of the culprit. Several people came forward with the needed information and

the four stolen columns were located. Two of the columns had been installed in a home and the other two were on a flatbed trailer awaiting their planned use as an entry structure to the property.

Air Pollution

Enjoyment of viewing scenic landscapes can be impaired by air pollution. For example, views at Craters of the Moon National Monument are jeopardized by proposed nuclear waste storage and cleanup plans at the Idaho National Engineering Laboratory, 1995). The INEL laboratory is only 24 km away from the monument. Several decades ago, visitors could see the Grand Tetons, about 240 km from the site. DOE plans to build three or four more processing plants to incinerate low-level wastes.

Commercialization

Shoshone Falls and Thousand Springs are two geological features along the Snake River in south-central Idaho that were significantly degraded by commercial development. The 65 m-high Shoshone Falls are spectacular when the river is flowing. However, since 1905 almost the entire flow of the Snake River has been diverted for irrigation by Milner dam a few miles upstream (Malde, 1987). The Thousand Springs area along the Snake River in Idaho has 65 of the largest springs in the United States (Whitehead and Covington, 1987). Many of the most spectacular springs are on private lands and are conveyed through flumes and pipelines rather than gushing out of the aquifer and down the canyon wall. Until the early part of this century when the water was developed for power and irrigation, the springs were a spectacular scenic feature. Springs poured from outlets along the canyon forming an almost continuous wall of water over the north side of the canyon extending for hundreds of meters (Russell, 1902).

National Parks and Monuments

Although Abraham Lincoln signed legislation protecting Yosemite Valley in 1864, Yellowstone National Park, established by an act of Congress on March 1, 1872, represents the first major effort in the United States and possibly the world to conserve and protect an outstanding geological area. Much of the credit goes to Ferdinand Vandiveer Hayden, a geologist who is well known for the Hayden Survey. When Hayden surveyed the Yellowstone area in 1871, it was the only remaining area in the coterminous United States left to explore. With a special appropriation of $40,000 from Congress to support the expedition, Hayden set off in June of 1871 with 34 men, including a mineralogist, a topographer, two artists and a photographer. Based on 38 days of field work in the Yellowstone wilderness Hayden prepared a 500 page report accompanied by Jackson's photos, and Moran's sketches and paintings to show Interior officials, Congressmen, Senators and other interested and influential parties. His efforts finally culminated in the simultaneous introduction of a bill in the House and the Senate on December 18, 1871. It is remarkable that, using the transportation and resources available at the time, Hayden could have completed the field expedition of a remote area in the west, prepared a 500 page report as well as many other publications, convinced his superiors in Interior and the Congress, had a bill introduced and had it enacted into law—all in less than 9 months.

President Grant signed the bill into law on March 1, 1872, creating the first national park over a region of spectacular geologic features and landscapes. This was the nations first effort to preserve an area of outstanding beauty for the enjoyment of future generations and was accomplished only a few months before enactment of the 1872 Mining Law (Act of May 10, 1872; 17 Stat. 91).

Each national park has been withdrawn through a specific act of Congress. National monuments are established by Congressional action and Executive authorization. Many national monuments have been established by presidential proclamation under the Antiquities Act of June 8, 1906 (34 Stat. 225; 16 U.S.C. 431). The purpose of the national parks and monuments "is to conserve the scenery and the natural and historic objects and the wild life therein and to provide for the enjoyment of the same in such manner and by such means as will leave them unimpaired for the enjoyment of future generations" (16 U.S.C. 1).

National Natural Landmarks Program

The National Natural Landmarks program is managed by the National Park Service; it is based on the legal authority of the Secretary of the Interior under the Historic Sites Act of 1935 (Public Law 74-292; 49 Stat. 666; 16 U.S.C. 461 et seq.) and two other statutes that refer to national natural landmarks (16 U.S.C. a-1 through a-7; 16 U.S.C. 1908). Implementing regulations for the National Natural Landmarks program, adopted in 1982, are contained in Part 62 of Title 36 of the Code of Federal Regulations.

The purpose of the Natural Landmarks program is "to identify and encourage the preservation of nationally significant examples of the full range of ecological and geological features that constitute the nation's natural heritage." 36 CFR 62.1. Sites on the registry are selected for both their geological and ecological values so they cannot be considered strictly geological or ecological sites (Spicer, 1987). Although thousands of potential sites have been identified, about 587 sites in 48 states, 3 territories and the commonwealth of Puerto Rico have been designated as National Natural Landmarks.

The national natural landmark program applies to all land in the United States regardless of ownership; this is important because many landmarks involve mixed ownerships of federal, state and private land. As a general rule the National Park Service prefers to obtain the consent or approval of all the owners before a designation is made. The designation of a landmark does not withdraw the land from other uses and the National Park Service has no regulatory authority to manage or preserve the sites, other than getting commitments from the various owners to do so.

American Indian Religious Freedom Act and Sacred Geography

Many unique or scientifically significant geologic attractions and features represent sacred geography to American Indian religious beliefs. Sacred geography may be represented by an imposing mountain such as a volcano, waterfall, cave, colorful rock formation, hot spring, or an unusual landform (Walker, 1988). Sacred geography is a universal and integral part of American Indian religions. The natural landform or feature represents a sacred altar or church where

people may communicate with spirits and acquire spiritual powers (Walker, 1988). Degradation or loss of access to such sites may restrict or preclude the opportunity or right to practice a crucial part of religion.

Sacred geologic features require religious activities at specific times such as calendrical rituals at the spring and fall equinoxes or the winter and summer solstices. So we have many sacred sites that are particularly sacred at a specific time (Walker, 1988). Excellent examples of these geologic sites include Celilo Falls (discussed above), Wallowa Lake, a beautiful glacial lake in northeastern Oregon and Devils Tower, a spectacular volcanic neck in northeastern Wyoming. Wallowa Lake, a Nez Perce vision quest and ceremonial ground was the home of Chief Joseph. His descendants meet at the lake to memorialize his life, death and their tribal history (Walker, 1988). Unfortunately, the aesthetic aspects of this site are degraded by the construction of many lake-front homes. Devils Tower is a 382-m-high erosional remnant of a volcanic neck that shows spectacular columnar jointing (Karner and Halverson, 1987). Devils Tower, which lies within Devils Tower National Monument, designated America's first national monument by President Teddy Roosevelt in 1906, is an important landmark of the northern Great Plains. It is also a sacred feature for more than 20 tribes who claim cultural relationship to the feature. They use the site for religious ceremonies, particularly during the summer solstice. In June many tribes celebrate on the land surrounding Devils Tower with Sun Dances, prayer offerings, sweat lodge ceremonies and vision quests. Recently, conflicts have occurred between rock climbers and the tribes so the National Park Service started a voluntary climbing ban which is apparently successful.

CONCLUSIONS

Starting with the establishment of Yellowstone National Park in 1872, the United States has successfully conserved large-scale geological landscapes through a statutory-based national park system. Not only does the western United States have a remarkable number of world-class geological heritage sites, but it has managed to preserved them from development. Millions of foreigners travel to the United States every year for the primary purpose of visiting the national parks.

Perhaps the best opportunities to conserve those sites that have dropped through the cracks can be found on the Federal lands. The total area of the 50 states is 2.3 billion acres. Of this area, approximately 662 million acres, or 30 percent of the total area, is still in Federal ownership. Most of this Federal land is administered by the U.S. Forest Service and the Interior agencies such as the Bureau of Land Management, the National Park Service, the Fish and Wildlife Service and the Bureau of Reclamation.

Each geological heritage site should be evaluated in terms of the appropriate uses of the site and the management options for development and protection. The management plan for each site, which should be integrated the agency land use plan for the area, would identify threatened sites and developmental potential for purposes such as earth science education and research and recreation programs with interpretative centers and trails. Also, a national listing could be established and maintained on a computer database with information abstracted from each

geologic site report. This would be a convenient method for external users such as educators and scientists to access geological resources information on Federal lands.

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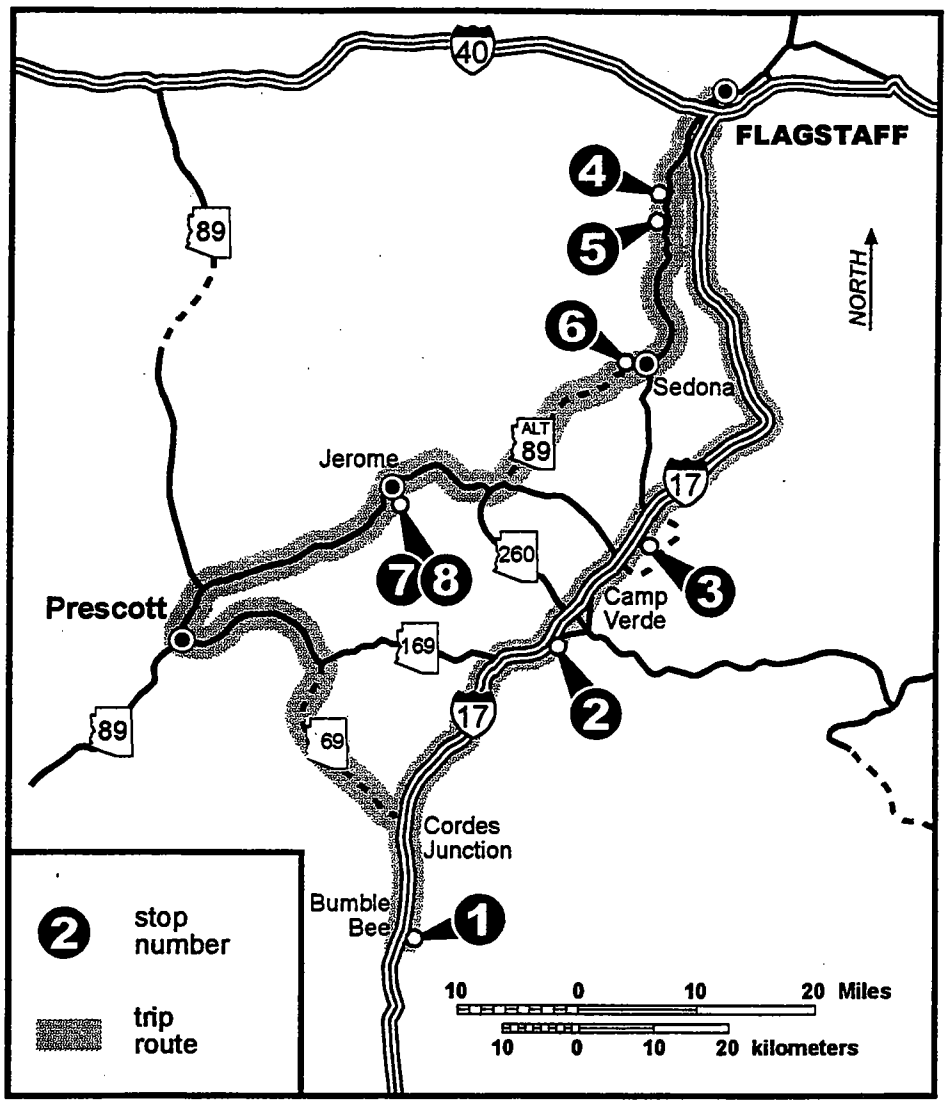
Highway Geology and Road Log Through Arizona Transition Zone

Prescott Region, Black Canyon Hwy., Verde Valley, and Redrock Country

State Route 69 & 89A
Interstate 17

Yavapai and Coconino Counties
Arizona

September 12, 1998



Field Trip Route

Highway Geology Symposium

September 1998

Highway Geology
Prescott, Black Canyon, Verde Valley, And Redrock Country

Larry D. Fellows, Arizona Geological Survey
Nicholas M. Priznar, Arizona Department of Transportation
J. Dale Nations, Northern Arizona University
Paul A. Lindberg, Consulting Geologist
Beverley A. Everson, Prescott National Forest

Introduction

Arizona includes parts of the Colorado Plateau and the Basin and Range geologic provinces, the contact of which is gradational over a distance of about 40 miles. This gradational area, commonly referred to as the Transition Zone, has characteristics of both the Colorado Plateau and the Basin and Range provinces. Stops 4 and 5 are along the boundary between the Transition Zone and the Colorado Plateau. This boundary is called the Mogollon (pronounced Muggy-own) Escarpment or Rim. All other field trip stops are within the Transition Zone.

The boundary between the Basin and Range province and the Transition Zone is within a mile or two on either side of milepost 231, about ten miles south of the Rock Springs exit of I-17, the southernmost point reached on this field trip. If you drove to Prescott from Phoenix along I-17, you started in the Basin and Range province and crossed into the Transition Zone near milepost 231.

The purpose of this field trip is to observe geologic features and their impacts on highway construction and maintenance. The first part of the trip -- from Prescott to Cordes Junction to Rock Springs – roughly parallels the old "Black Canyon Highway," an historic road that connected Prescott, the Territorial capital, with Phoenix.

The Black Canyon Highway/1

Black Canyon Highway (I-17) is now the primary north-south corridor in central Arizona. The former route (SR69) ran from Prescott through Prescott Valley, Dewey, Humboldt, Mayer, Cordes, Black Canyon City, Rock Springs, and New River to the metropolitan Phoenix area. .

The route had a humble beginning as an Indian trail through Black Canyon 150 years ago. With the discovery of rich mineral deposits in the Bradshaw Mountains, a primitive road developed to access the mines southeast of Prescott in the 1850s and 1860s. After 1863, when the Arizona Territory was established, the road was improved slightly and extended southward to Phoenix. The first recorded use of this extended route (Black Canyon Road) was in 1870. In 1878 a stagecoach line was established and made regular passenger and mail runs from Prescott to Phoenix. The road was rough, the ride took several days, and robberies were common.

A railroad was completed between Phoenix and Prescott in 1895 and soon became a competitor with the Black Canyon road. After 1898, stagecoaches ran less frequently, but the stagecoach era lasted until about 1912. In the early 1900s, as many as 200,000 sheep and goats were herded up the Black Canyon road from Phoenix to Cordes every spring, where they were dispersed to summer pastures. They made the return trip in the fall to get to winter pastures.

Automobiles began to use the road in the 1910s and in 1915 the Cordes store began to sell gasoline. In 1916 one could travel from Prescott to Phoenix by automobile in less than 18 hours, including time to change many flat tires. (It's a two-hour drive today!) Even so, mainly wagons and stagecoaches used the Black Canyon road until 1917.

Few improvements were made to the road until the late 1930s. In 1936, the Black Canyon road was designated as State Highway 69 and became eligible for Federal highway funds. It wasn't until 1946, after World War II, that surveying began for a new, re-routed paved road. The new route climbed up out of Black Canyon, crossed Black Mesa, and completely bypassed Agua Frio, Big Bug, Bumble Bee, Canon, Cordes, and Gillette. The road skirted around the edges of other communities such as Dewey, Humboldt, and Mayer. Construction and paving of the new Black Canyon Highway was completed in 1956. The communities of Prescott Valley and Cordes Junction developed after the old road was paved. The Black Canyon Freeway (I-17) from Phoenix to Cordes Junction parallels the old Black Canyon road, as does State Route 69 from Cordes Junction to Prescott.

Overview of the Geology of the Prescott Area

Prescott and Prescott Valley are in the Transition Zone, within which some of the oldest Precambrian rocks in the State crop out. The Proterozoic Prescott Granodiorite crops out on Granite Mountain (northwest) and the Granite Dells (north). Metavolcanic and metasedimentary rocks of the Yavapai Supergroup (1.75 to 1.8 Ga) are exposed throughout the Prescott area and in the Black Hills to the east. Exhalative massive sulfide deposits associated with the metavolcanics have been mined for base and precious metals in the Black Hills and the Bradshaw Mountains south of Prescott.

Rocks of mid-Tertiary age (21 to 27 Ma) are unconformable on the Precambrian rocks. Basalt interbedded with gravel, tuff, and tuffaceous sediment of the Hickey Formation (10.4 to 15.4 Ma) are at the surface throughout the area.

Basin fill in Prescott and Prescott Valley includes Pliocene lacustrine clay and carbonates interbedded with alluvial gravel, sand, and silt. This sediment, which can be seen in roadcuts along SR 69 throughout Prescott Valley, are correlative with the Perkinsville formation in Chino Valley and the Verde Formation in the Verde Valley. Pleistocene-age sediment includes organic-rich clay, silt, and sand with interbedded stream gravel. Fossils of Mammoth, camel, bison, and horse have been found in Pleistocene deposits in several drainages in Prescott Valley and Chino Valley.

Road Log

Miles	Comments
0.0	Leave Prescott Resort parking lot.
0.5	State Route (SR) 69. Turn left. Precambrian rocks exposed in roadcuts for next 5 miles.
5.7	Prescott Valley Town Limit
	Prescott Valley is built on an alluvial surface that slopes uniformly toward a former depositional basin to the east. The area from Dewey northward was the site of a shallow lake before an integrated drainage system developed. Gravel deposits shed from the west, such as those beneath us, grade eastward into lakebed sediment. After a drainage system developed, the stream and its tributaries began eroding the lacustrine deposits. Remnants of the lakebed sediment are exposed in roadcuts ahead.
9.5	Leave Prescott Valley. Basin fill gravel is exposed in cuts along north side of road.
12.2	Lacustrine deposits exposed for next 5 miles.
14.0	Enter town of Dewey.
15.0	SR 169 intersects from left (east). Stay on SR 69.
	Near mile post 278.5, we will be passing the reddish brown tailings pile of the Iron King Mine. Between 1903 and 1969, the Iron King produced over 6 million tons of ore. Gold, silver, copper, lead, and zinc were all found within this massive sulfide deposit. The tailings pile (approximately 4.3 Million tons of materials) is currently being processed into a natural soil supplement (fertilizer), which is distributed internationally.
18.1	Vertically dipping Precambrian rocks exposed on right and for next 10 miles.

In the vicinity of mile post 271.4 the new divided highway passes over an extensive cave formation in Tertiary limestone /travertine. During construction of this section, numerous irregularly shaped voids ranging form 2 to 20 feet in diameter and from 3 to 40 ft below highway grade were encountered. The voids were selectively over excavated and back filled. The travertine (locally called "Onyx Marble") is a calcium carbonate, hot spring deposit, approximately 75 feet thick. It is easily identified by it's colorful alternating white, green brown and red, horizontal banding.

25.1 Cross Big Bug Creek.

25.2 Town of Mayer on right.

28.9 Basalt in roadcut on left. Between here and Stop 1 (15 miles ahead) basalt of Tertiary age is unconformable on rocks of Precambrian age. In this immediate area the Precambrian rocks are granitic.

32.6 Precambrian granodiorite exposed for next 12 miles.

33.1 SR 69 intersects with I-17 at Cordes Junction. Take I-17 South.

33.9 Cross Big Bug Creek. Weathered Precambrian granodiorite is exposed in cuts.

39.7 Badger Springs exit. Stay on I-17 S and begin climb to top of Black Mesa, which is capped by Tertiary basalt.

43.1 Sunset Point Exit. Leave I-17 S.

43.9 **Stop 1: Sunset Point Rest Area**

This rest area is on top of Black Mesa, which is capped by basalt (10.6 my) that came from Joe's Hill, three miles due east. The Agua Fria River has cut its canyon through the basalt between here and Joe's Hill.

Look down (west) into Black Canyon. The old stagecoach and wagon road was in the valley below. What was once Bumble Bee, a stagecoach stop, was along this road below us. The mountains to the west are the Bradshaw Mountains. In this area they are composed of the Crazy Basin Quartz Monzonite (a variety of granite) that is of Precambrian age, about 1.7 billion years old. This granite is the youngest of several intrusive bodies in the Bradshaws. The lower hills between here and the Bradshaws are a composed of north-striking, vertically dipping, metamorphosed Precambrian-age volcanic rocks. We drove through them in the Mayer vicinity on the way to this stop.

44.4 Return to I-17 S.

44.9 Begin descent into Black Canyon.

45.3 Basalt on left, first saguaro on right.

For the next several miles exposures in roadcuts consist of variably dipping Tertiary basalt, nearly vertical Precambrian metamorphics, and nearly horizontal lacustrine deposits. Lacustrine deposits are exposed at the top of the 20-15 Ma section of interbedded basalt, tuff, and fluvial-lacustrine sedimentary rocks. Because of the soft lacustrine deposits beneath the basalt, this area has experienced, and is still experiencing, downslope movement.

Sunset Point Rest Area

Saturday September 12, 1998

PHYSIOGRAPHY

- *(W) Bradshaw Mountains -- Black Canyon -- Black Mesa -- Agua Fria River -- Perry Mesa (E)*

EARLY PROTEROZOIC GEOLOGY (1800-1700 Ma)

- To the west of Sunset Point, the **Crazy Basin monzogranite** pluton (1699 Ma) forms much of the rugged eastern side of the Bradshaw Mountains.
- The north-trending Early Proterozoic **Shylock fault zone** separates the monzogranite and older metasedimentary and metavolcanic rocks from granodioritic and metavolcanic rocks to the east. The Shylock fault zone is a major Early Proterozoic structure of intense vertical foliation that averages about 2 km in width for 60 km. The complex deformational history represented by this zone of highly strained rocks is controversial, but mostly reflects the 1700-1690 Ma **Yavapai Orogeny**.
- Several granodioritic plutons and minor metavolcanic rocks underlie much of the area east **of Black Canyon**, from Cordes to Black Canyon City. These units include the **Bumblebee granodiorite**, **Badger Spring granodiorite**, **quartz diorite of Bland** (1720 Ma), and various granodiorite and tonalite phases of the **Cherry Springs batholith** (1740 Ma).
- Localized granodioritic Laramide intrusions (64 Ma) are present in the Crown King, Walker, and Poland Junction areas (Anderson and Blacet, 1972). Many of these are mineralized.

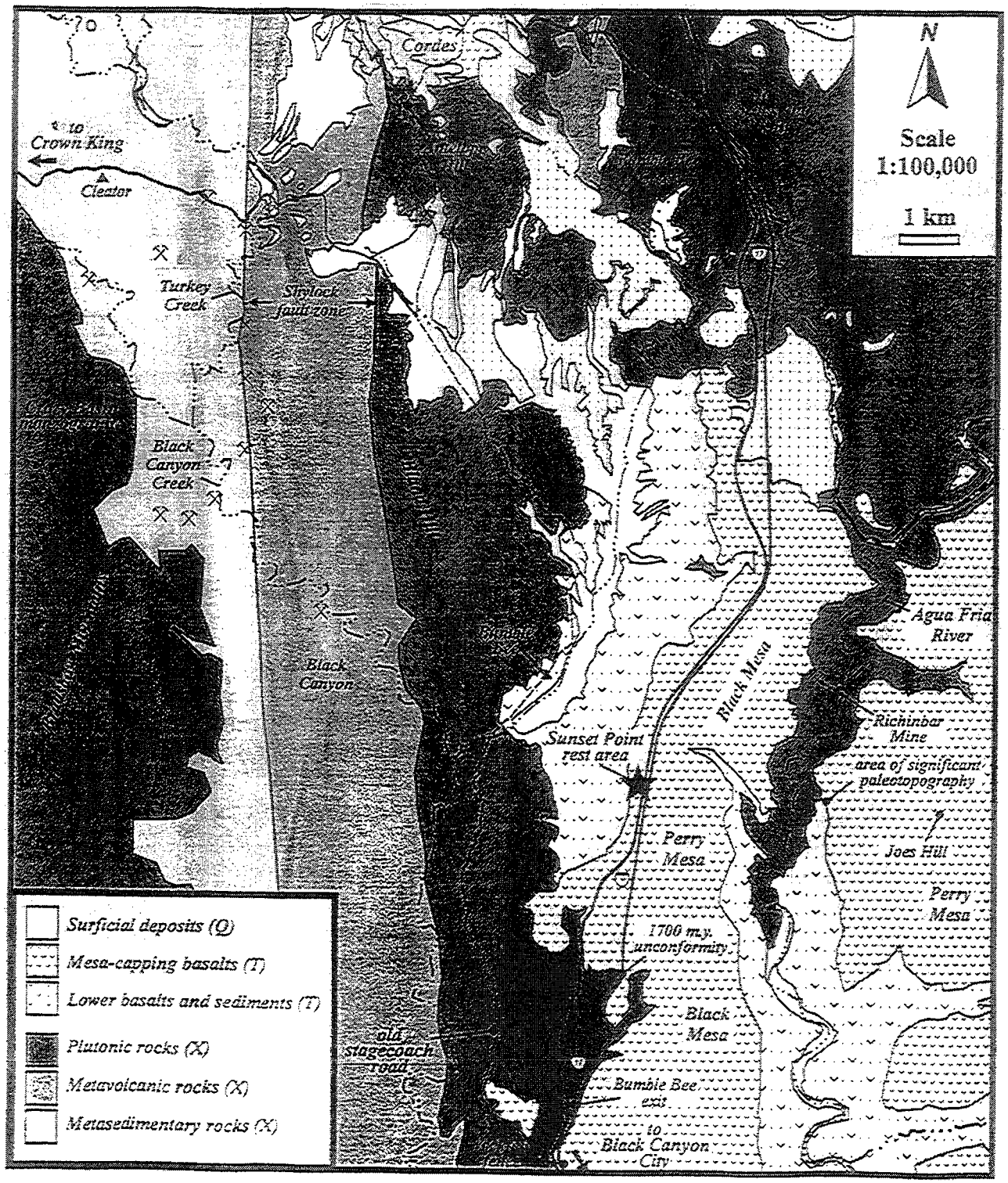
TERTIARY GEOLOGY

- To the east of the **Bradshaw Mountains**, interbedded Tertiary sedimentary and basaltic rocks are present beneath the **Hickey Formation** basaltic lavas (14-10 Ma).
- The older sediments and lavas were probably confined to channels and small paleobasins developed in the Proterozoic basement, whereas the upper basalt flows have a relatively large areal extent and cap many of the broad plateaus.
- **Black Mesa** represents the western margin of the **Perry Mesa** lava sheet that has been dissected by the Agua Fria River and Black Canyon Creek. A sequence of multiple basalt flows (<100 m thick) cap Black Mesa, with thin sandstone beds, scoria, agglomerate, and basaltic tuff present locally.
- The source for the large volumes of mesa-capping lavas in the area may have been local fissures and/or small shield volcanoes. **Joes Hill** may be a shield volcano, produced by basaltic plains volcanism, that is similar to shield volcanoes of the Snake River Plain (Greeley, 1982).

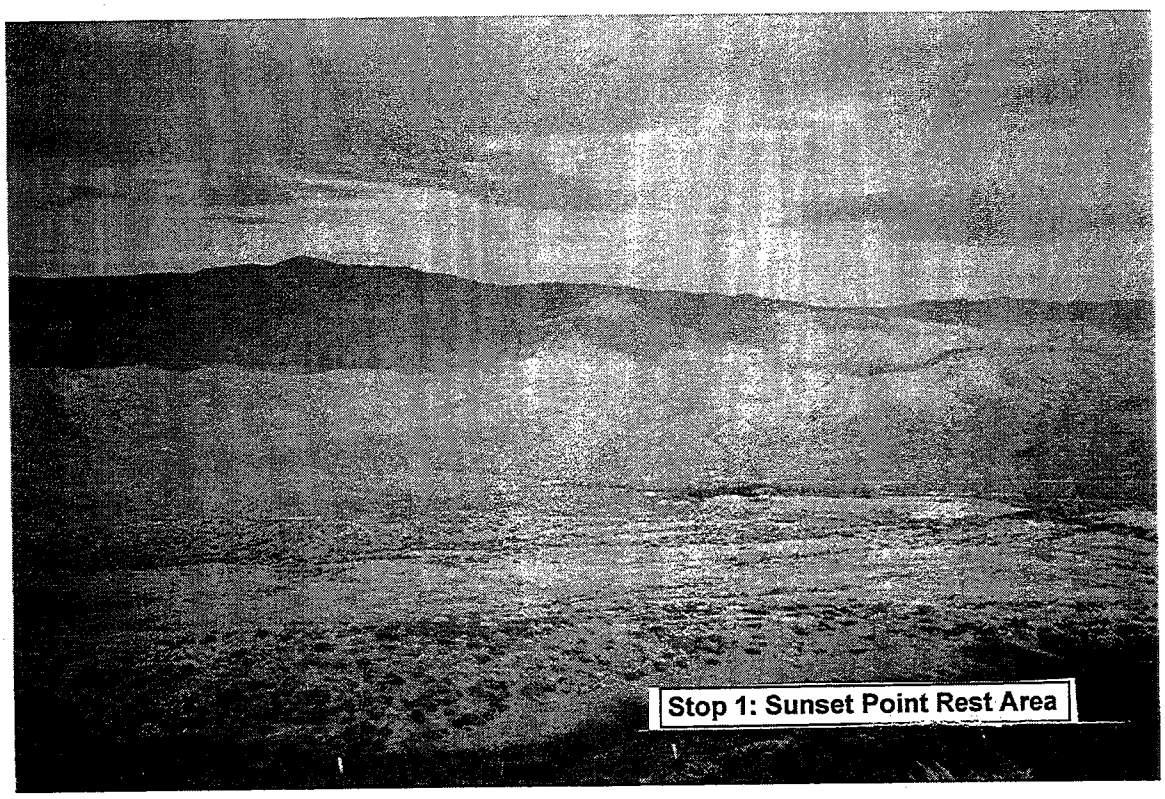
HISTORY OF THE BLACK CANYON AREA

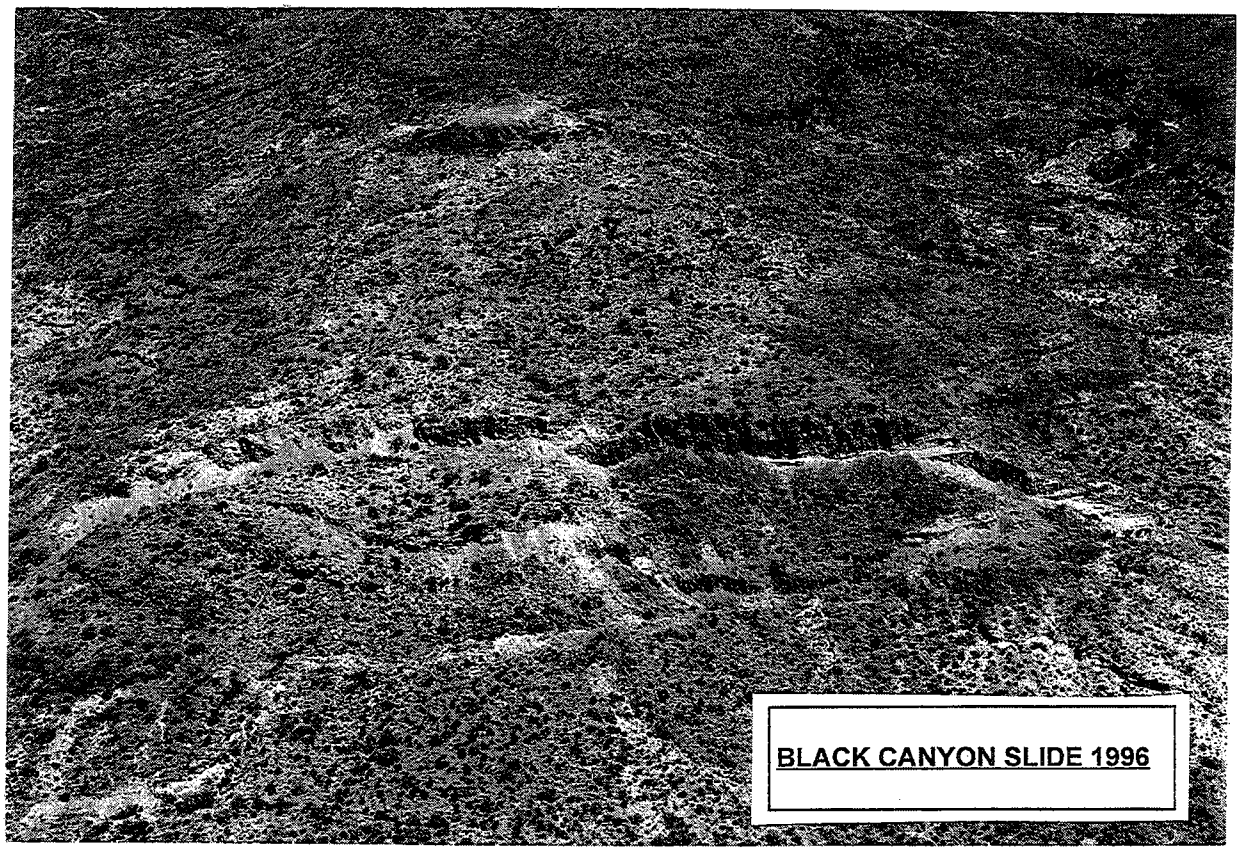
- Yavapai Indians occupied the area before the Civil War.
- 1863 - Gold discovered. The Bradshaw Mining District included massive sulfide, vein, and placer deposits that yield Ag, Au, and Cu ore from many mines (e.g., Richinbar, Golden Turkey, DeSoto, Silver Cord, etc.). The Lost Pick Mine is a fabled 1870's gold strike lost in the recesses of the Agua Fria River canyon.
- 1871 - Bradshaw City (near Crown King) included at least 5000 people until the 1880's.
- 1870's - Stagecoach era. The stagecoach road was as a primary link between Prescott and Phoenix.
- 1902 – Completion of Frank Murphy's "Impossible Railroad" - a narrow-gauge railroad from Mayer to Crown King.
- The boom times lasted until the end of World War I.

Compiled by Bob Leighty



Geology of the Black Canyon - Black Mesa area (modified from Jerome, 1956; Anderson and Blacet, 1972b; Leighty, unpub. mapping).


Stop 1: Sunset Point Rest Area


BLACK CANYON SLIDE 1996

1958

1996

52.2 Agua Fria River.

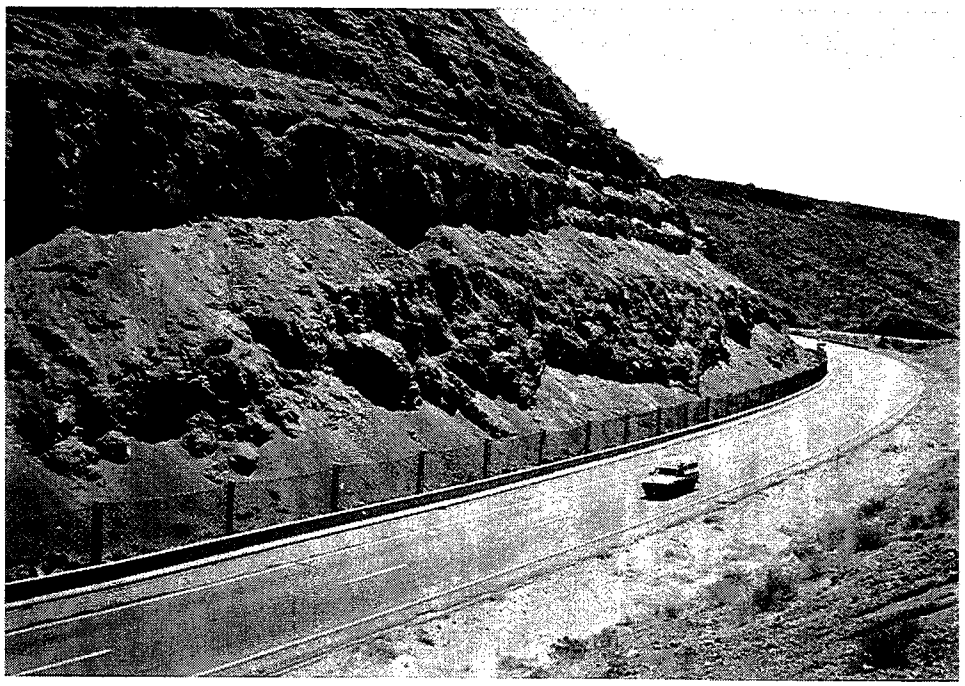
53.3 Exit 242 – Rock Springs. Leave I-17 S, turn left at end of exit ramp and cross over I-17, turn left and enter I-17 N.

Landslide scar straight ahead. A large mass of rock has pulled away and is moving slowly toward the south along a nearly flat surface of soft clay. The landslide, which is visible on air photos flown the 1930s, was not caused by construction of I-17 and is not a threat to I-17. Movement along the slide is being monitored. This entire area of soft, clay-rich lakebed deposits overlain by basalt is potentially unstable.

53.8 Enter I-17 N.

54.8 Agua Fria River

58.9 Conspicuous cut on right side of highway. Tertiary-age basalt flows and interbedded tuffaceous sediment are unconformable on steeply dipping Precambrian-age metamorphic rocks. The basalt dips south–southwest, roughly perpendicular to the highway alignment. Rapid weathering of the tuffaceous beds undermines the overlying basalt, which has columnar jointing, causing it to fall onto the highway.



A 9-foot-tall Brugg fence, rated at 60 ft/tons capacity, was constructed in 1991 to reduce the hazard to the travelling public. Since then the fence has endured numerous impacts from basalt boulders and helped keep the highway open and safe.

60.7 As we preoceed up the grade keep your eyes open for a textbook unconformity! Nearly flat-lying Tertiary basalt (10.6 my) is sitting on vertical beds of Precambrian age (1,699 my) in roadcut on right. These rocks are part of the Bland Quartz Diorite pluton that is probably 1,740 to 1,720 m.y. old. The Crazy Basin Quartz Monzonite in the Bradshaw Mountains to the west is 1,699 m.y. old.

61.2 Another good view of unconformity, but not as spectacular as the previous one.

61.8 Top of Black Mesa.

63.2 Sunset Point Rest Area on left.

73.1 Big Bug Creek.

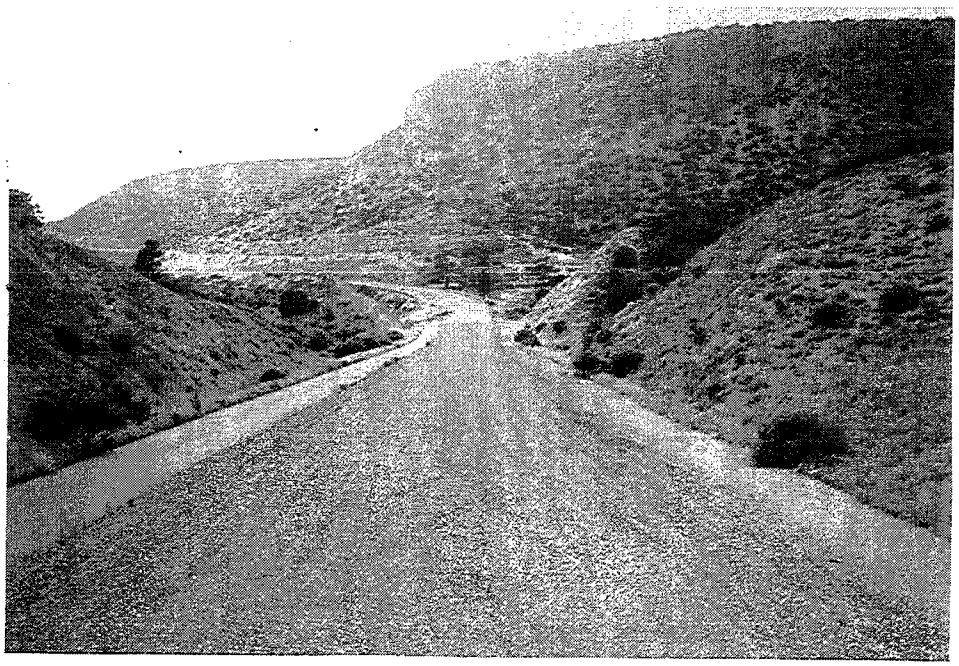
73.2 Exit 262 -- Cordes Junction. SR 69 to Prescott. Stay on I-17 N.

Tertiary basalt is exposed along the highway for the next 18 miles. We are crossing the western edge of the Estler Peak volcanic field, which includes many Hickey Formation basaltic flows, pyroclastic beds, and volcaniclastic sedimentary deposits. Numerous variably eroded cinder cones and vent deposits in the area represent sites of volcanic activity that occurred between 10 and 12 m.y. ago. Hickey Formation fluvial sediments cover much of the area west of the highway.

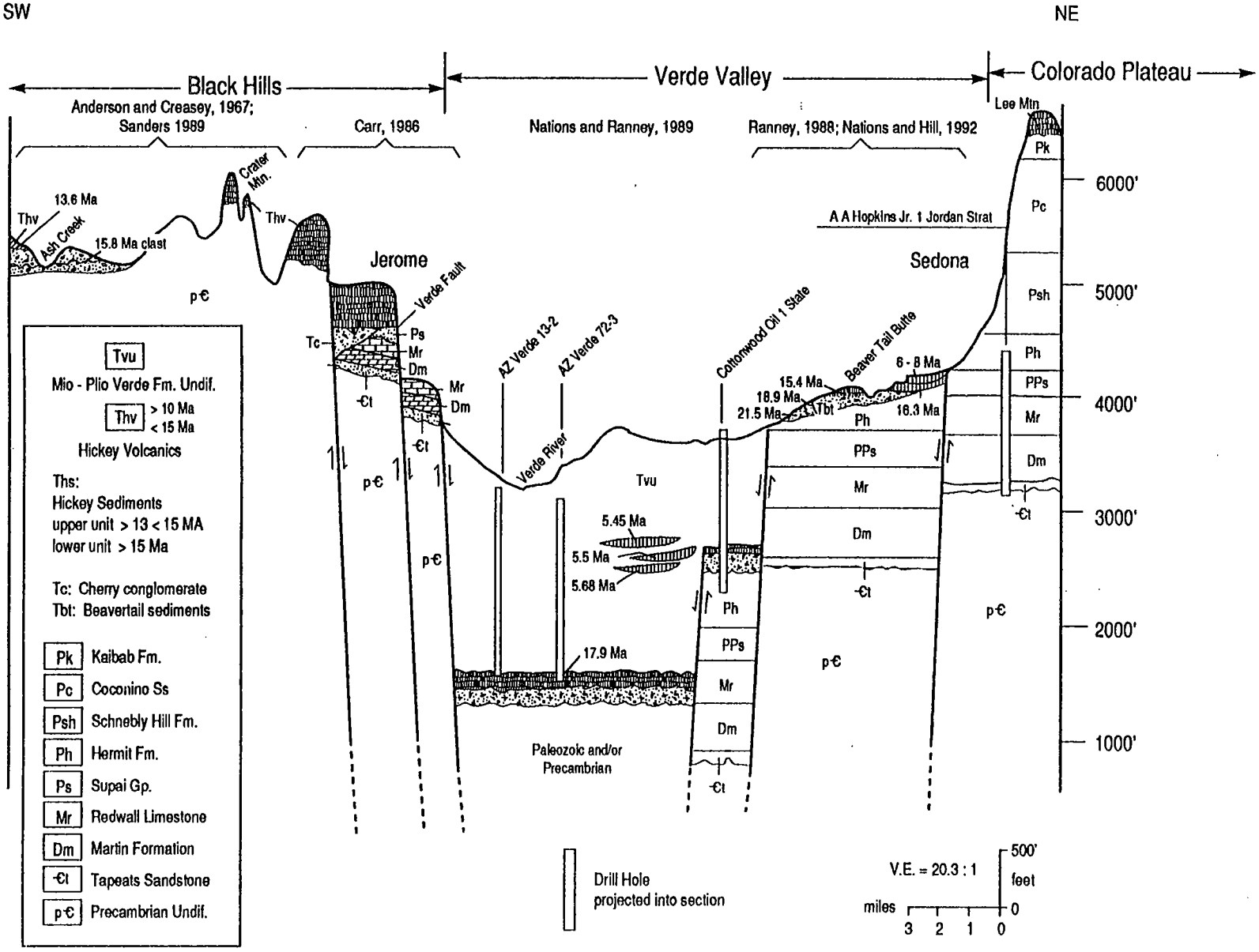
89.0 Exit 278 -- Cherry Road. SR 169 to Prescott. Stay on I-17 N.

91.5 Begin descent into Copper Canyon and the Verde Valley. Basaltic lavas, volcaniclastic deposits, and sedimentary beds of the Hickey Formation are exposed in this area. A distinctive tuffaceous sandstone horizon is exposed south (right) of the road below the uppermost basaltic rocks. An andesite flow is exposed high above the road on the north (left) side. The Middle Tertiary Cherry conglomerate is exposed above Precambrian basement rocks in the vicinity of the runaway truck ramp. This sequence of roadcuts is on the upthrown side of the Verde fault zone, which consists of a series of subparallel faults that drop downward toward the northeast and form the southwest side of the Verde basin. None of the faults are clearly exposed.

94.1 Runaway truck ramp. Massive quantities of river gravel and terrace deposits were screened near Beaver Creek to produce the very well rounded +/-3/4 inch diameter arrestor beds in the median. The gravel you see represents approximately 7% of all the material that was processed.



Cross-Section, Colorado Plateau to Black Hills



Cross Section through the Black Hills and the Verde Valley showing the relationship of the Hickey sediments to the Cherry conglomerate and the Beaver Tail Butte sediments (Modified from Sanders, 1989).

Nations, J.D., 1992

95.7 Greenish clay beds of the Verde Formation on left. We are now on the downthrown side of the Verde fault.

95.8 **Exit 285 – General Crook Highway. Turn left and park.**

Stop 2: Verde Basin

The Verde basin began to form about 8 million years ago, when the northwest-trending Verde fault zone developed. This is the fault we crossed about three miles before we stopped here. Other northwest-trending faults developed near Sedona, although there was not as much displacement along them as along the Verde faults. This faulting created a northwest-trending structural basin about 12 miles wide and twice as long. Displacement along the Verde fault near this stop was only several hundred feet. At Jerome, however, there was about 6,000 feet of displacement.

Runoff flowed into the Verde basin, forming a shallow lake that was present until about 2 million years ago. Sediment eroded from the adjacent mountains was deposited in the lake. Deposition into the lake kept pace with recurrent movement along the faults.

Coarse-grained sediment was deposited around the margins of the lake and finer deposits settled out farther from the shoreline. Salt, which was mined decades ago, gypsum beds, which are still being quarried, and marly limestone beds were deposited in the lake. Lava flows spilled into the lake in several areas, as is evidenced by basalt layers that are interbedded with lakebed deposits. More than 3,000 feet of sediment were deposited into the lake.

Water eventually found an outlet from the lake and the Verde River and its tributaries developed. The river is now busily eroding the soft sediment that was deposited in the lake.

Return to I-17 N.

97.8 Exit 287 – SR 260 to Cottonwood and Payson. Stay on I-17 N.

98.8 Verde River. Verde Formation straight ahead.

100.5 Exit 289 – Montezuma's Castle. Stay on I-17 N.

Green-colored lakebed deposits are exposed on both the north and south sides of this exit.

103.1 Limestone beds of the "Verde Formation", have been "let down" into what was once a cave or part of a cave system.

104.0 Exit 293 – McGuireville and Montezuma's Well. Stay on I-17 N.

107.4 **Stop 3: Rest Area**

Pit stop, arm-waving, and discussion of Verde Formation, Mogollon Rim, ramp basalts, and other items of major geologic importance.

109.6 Exit 298 – SR 179 to Sedona. Stay on I-17 N.

111.1 Basalt exposed for next 11 miles. This is part of a series of stacked basalt flows that flowed from the Colorado Plateau down into the Verde Valley about 6.4 million years ago.

116.9 Exit 306 – Stoneman Lake. Stay on I-17 N.

131.1 Exit 320 – Schnebly Hill Road. This is a narrow graveled road that goes to Sedona. It is passable by automobile. Stay on I-17 N.

146.3 Kaibab Limestone (Permian age) in cut on left.

147.8 Exit 337 – Pulliam airport. Exit I-17 N, turn left at end of exit ramp, and cross over I-17.

148.2 Intersection with SR 89A. Turn left (south).

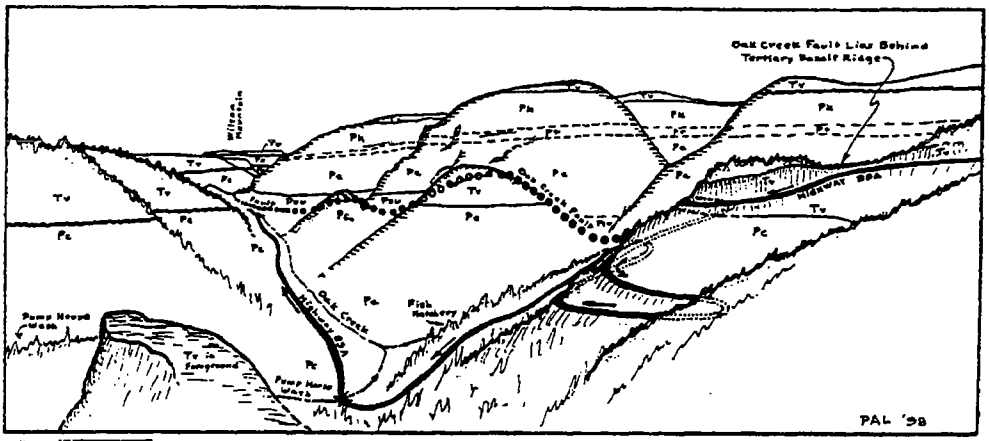
156.9 Oak Creek Vista. Turn left and park. Restrooms are available. Walk south to the overlook.

Stop 4: Oak Creek Vista

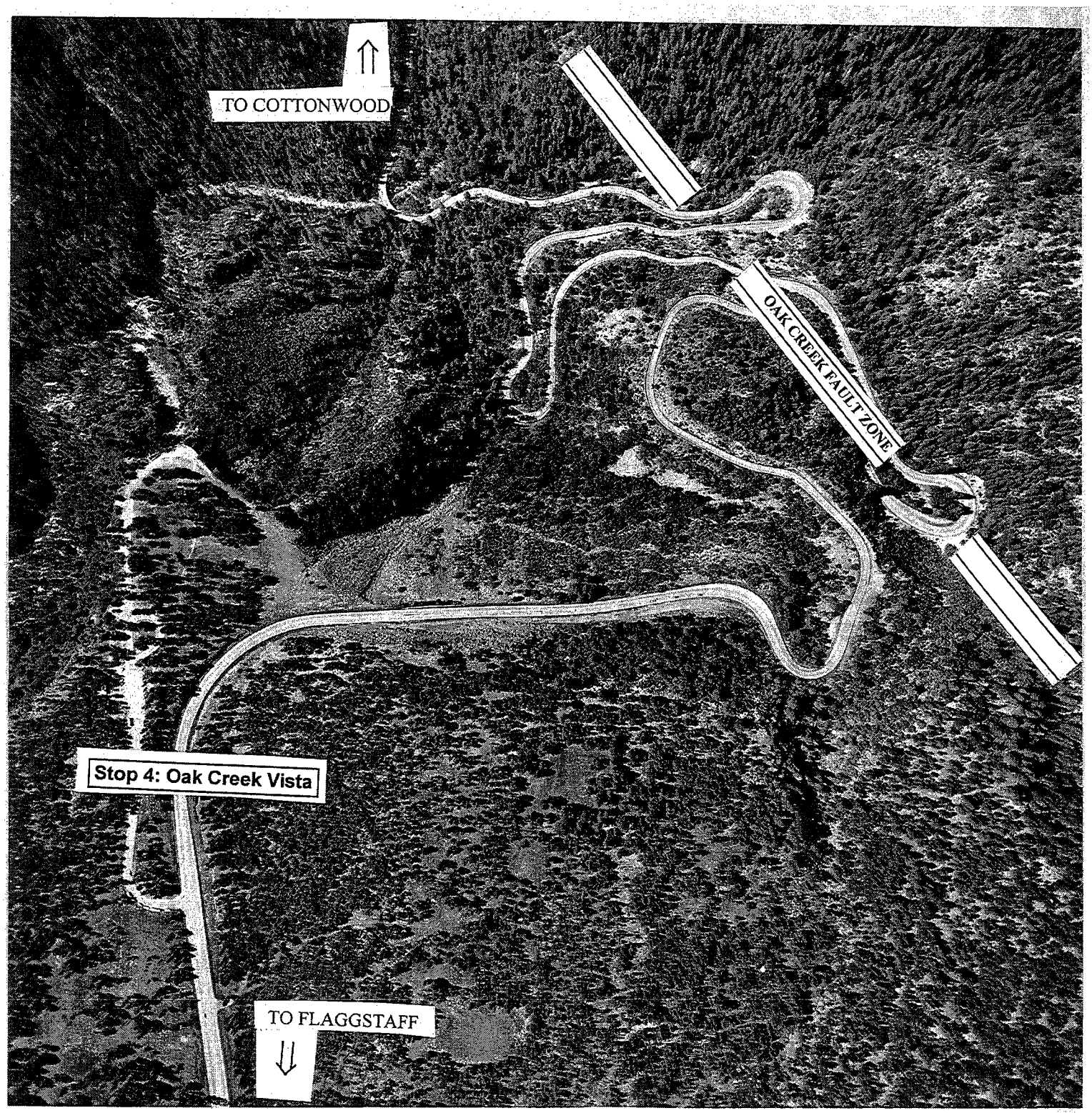
Oak Creek Canyon closely follows the Oak Creek fault, which has had two episodes of movement. The first was during the Laramide orogeny 80-50 million years ago. Crustal compression caused the rocks to rupture and the ancestral Oak Creek fault, a high-angle reverse fault, to form. Rocks on the east side of the fault were shoved upward relative to those on the west side. Erosion subsequently removed a greater thickness of rock from the structurally higher east side than the west. Then about 6 to 8 million years ago basalt lava flows covered the erosion surface, including the network of gravel-filled channels that had developed on it.

Crustal stretching (tension) caused the second episode of movement along the Oak Creek fault. This time rock on the east side of the fault, including the basalt on which we are standing, dropped down 300-400 feet relative to the west. The same basalt can be seen on the western skyline at an elevation that is 300-400 feet higher than where we are standing.

The basalt is unconformable on Paleozoic sedimentary strata on both sides of the canyon. On the west side of the canyon, the basalt rests on the Kaibab Limestone, Toroweap Sandstone, and Coconino Sandstone. On the east side, however, the basalt is in contact with the Coconino Sandstone. The Kaibab and Toroweap were eroded before the basalt spilled onto the surface.



Stop 4, Panoramic view from the head end of Oak Creek Canyon. The view looks toward the south from the Highway 89A viewpoint. Tv=Tertiary basalt lava flows, Pk=Kaibab Limestone, Pt=Toroweap



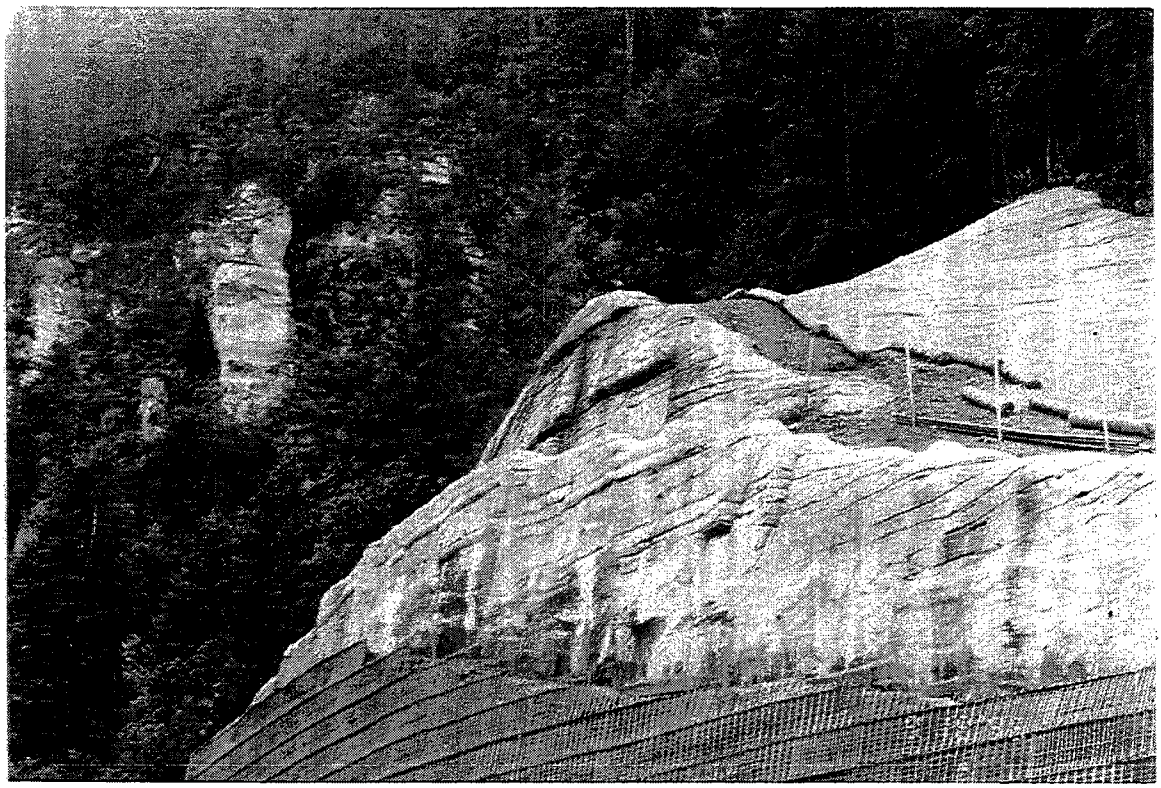
158.1 Oak Creek fault. As the highway descends below the dark-colored basalt, beds of tan Coconino Sandstone are present. At this point the fault has crushed the sandstone and destroyed traces of bedding. Water moving along the fault zone has bleached the sandstone into a white, friable mass. Watch for similar appearing exposures, which indicate a fault or fault zone, as we proceed south toward Sedona. The highway crosses

the Oak Creek fault three times. For most of this distance the highway is on the down-dropped side of the fault.

158.9 Oak Creek Fault again.
Between here and Sedona the highway drops down in the stratigraphic section through the Coconino, Schnebly Hill, Hermit and Esplanade formations.

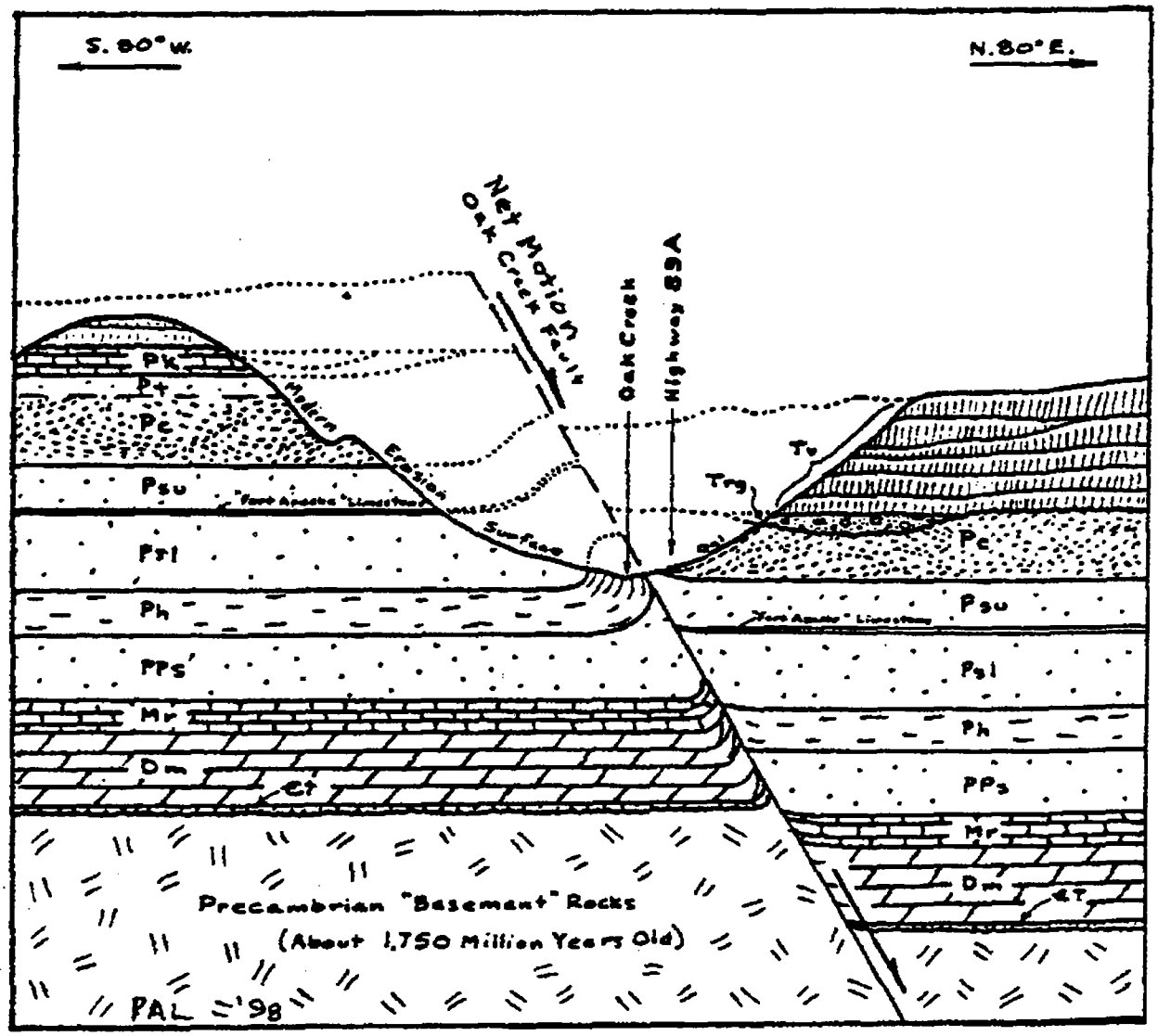
162.2 Stop 5: Call of the Canyon.

The "rock" exposed along the east side of the highway appears to be Coconino sandstone. It is actually a synthetic outcrop constructed for ADOT. Erosion of the existing steep cut in colluvium caused periodic rock spalling onto the highway during precipitation events. A tiered retaining wall was selected to stabilize the cut face. In order to get approval from the U. S. Forest Service and aeceptance by local residents, ADOT chose to make the retaining wall look as natural as possible. Our Coconino look-alike seems to have accomplished this purpose.



165.5 Cross Oak Creek. The bridge that we cross was actually built along side of the highway on scaffolding. After the bridge was assembled it took two days to jack it into its permanent position.

165.6 Slide Rock State Park on right. During the summer as many as 2,000 people visit this area daily to enjoy the cold water and sun.



Cross section of Oak Creek Canyon at Halfway Picnic area, looking north. High-angle reverse faulting on the ancestral phase of the Oak Creek fault occurred during Laramide Uplift when the east side of this area was raised higher than the west side. Prolonged Tertiary erosion wore down the landscape to a NE-draining peneplane and removed more of the raised Paleozoic stratigraphic section on the eastern side of the ancestral fault zone than on the western side. The Tertiary erosion surface contains channelized Rim Gravels and was then capped by Late Miocene basalt flows. After the basalt flows had been erupted crustal extension took place. The east side of Oak Creek fault then dropped to its present elevation. Modern erosion exposes all of these phases. Geologic codes are as follows:

Tv,	Late Miocene, basalt flows
Trg,	Tertiary, Rim Gravels
Pk,	Permian, Kaibab Limestone
Pt,	Permian, Toroweap Sandstone
Psu,	Permian, Upper Schnebly Hill formation
Psl,	Permian, Lower Schnebly Hill formation
Ph,	Permian, Hermit formation
PPs,	Pennsylvanian-Permian, Supai Sandstone
Mr,	Mississippian, Redwall Limestone
Dm,	Devonian, Martin Dolomite
Ct,	Cambrian, Tapeats Sandstone

170.8 Midgely Bridgely. That's what the sign says! This 375 feet steel arch bridge was constructed in 1939. The red rocks upon which the bridge is founded are part of a late Paleozoic sedimentary sequence locally referred to as the Supai Group.

172.1 Enter Sedona.

172.6 Intersection of SR 179 and 89A. Keep right on SR 89 A. Hermit formation exposed in roadcuts.

173.6 Airport Mesa Road. Turn left, drive to top of mesa, and park at overlook point.

Stop 6: Airport Mesa Overlook

From this vantage point one gets an excellent view of the Permian-age sedimentary rocks exposed in the Mogollon Escarpment. Basalt (10-6 ma) caps the cliffs and is underlain by the Kaibab, Toroweap, Coconino, Schnebly Hill, and Hermit Formations. Airport Mesa is capped by basalt.

A thick sequence of carbonate rocks – the Redwall Limestone (Mississippian age) and the Martin Dolomite (Devonian age) – lies about 650 feet below the land surface here. Solution has formed caverns in the Redwall into which the younger overlying rocks occasionally collapse. One such sink, locally known as the Devils Kitchen, is just a couple miles to the northeast.

Early records indicate that there was a major collapse of the sinkhole in the 1880s. Another was documented in 1989. Cracks have developed parallel to the margin of the sink. When will the next collapse occur?

Return to SR 89 A.

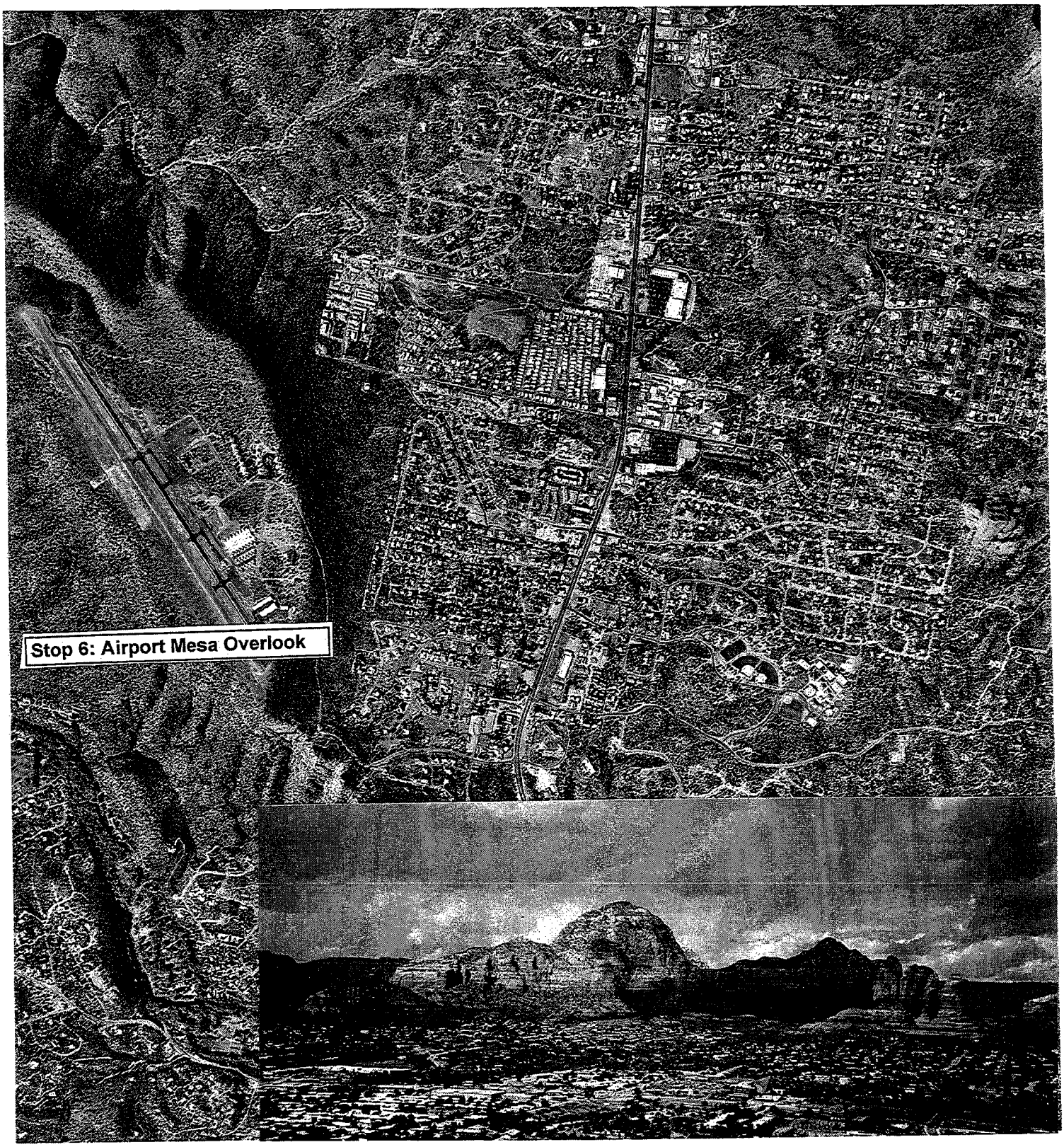
175.7 Leave Sedona.

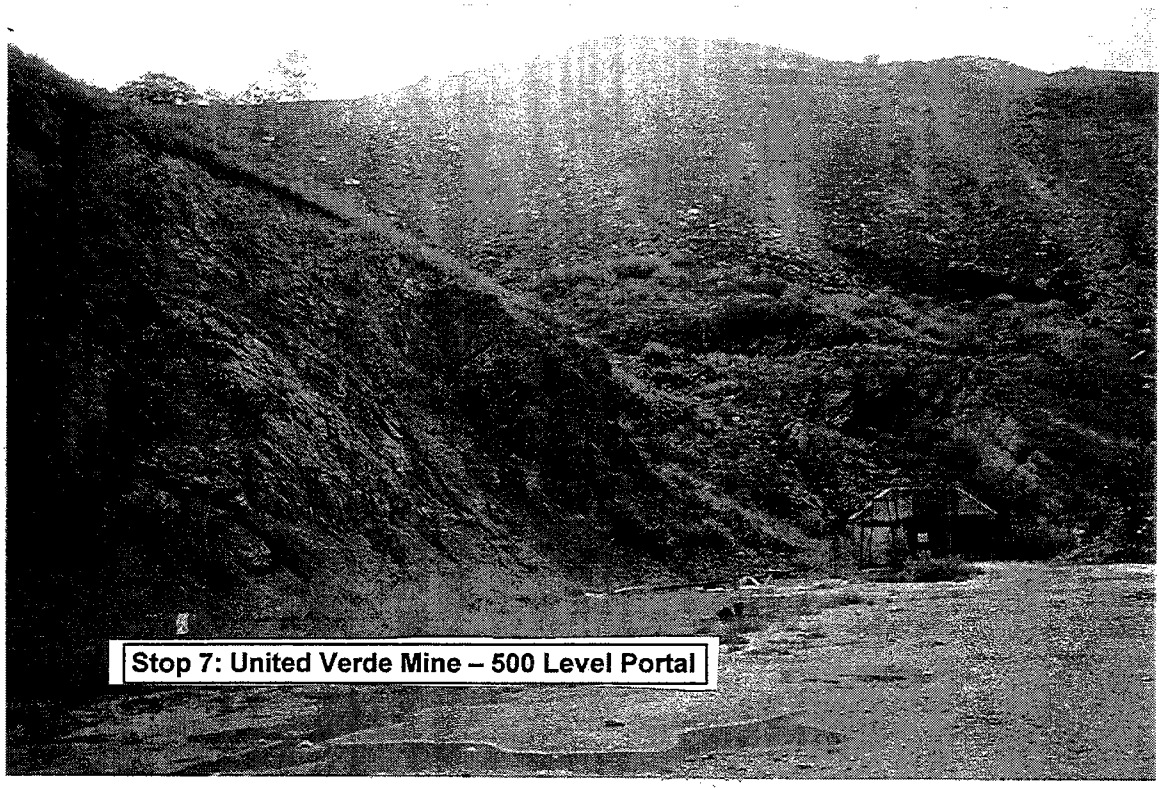
Faults that form the northeastern margin of the Verde basin are present in this general area. Outcrops of "red rock" are predominant until just beyond the bridge over Dry Creek. There, basalt, dropped down to the southwest, is in fault contact with the redbeds.

180.7 Black Hills and Mingus Mountain, straight ahead, on the upthrown side of the Verde fault zone.

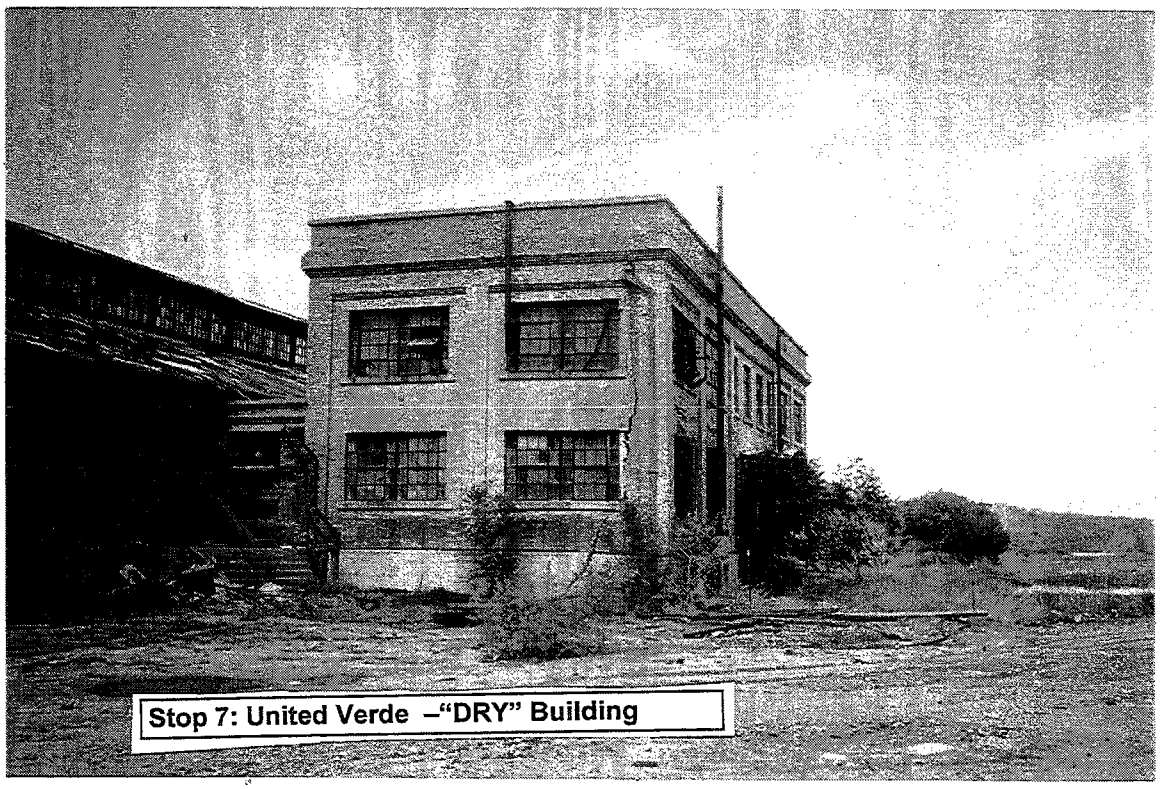
190.3 Cross the Verde River and enter Cottonwood. The bus will be passing a conspicuous outcrops of the Verde Formation in this area. Go straight through the first stoplight. Turn left at the second stoplight. Next, watch for a four-way stop at a Texaco station. Turn left to go to Jerome.

The Phoenix Cement Plant is to the north. The Redwall Limestone is quarried, mixed with shale, volcanic ash, and other ingredients and made into cement. As you drive up the mountain toward Jerome, the road crosses the Verde fault zone, a series of down-to-the-valley faults with a combined displacement of about 6,000 feet.


Stop 6: Airport Mesa Overlook



Stop 7: United Verde Mine – 500 Level Portal



Stop 7: United Verde –"DRY" Building

199.9 Enter Jerome.

200.9 **Stop 7: United Verde Mine – 500 Level Portal**

One of the faults in the Verde fault zone is well exposed here. Redwall Limestone (Mississippian age) has dropped down along the fault and is in contact with Precambrian-age volcanic rocks.

The large building, long abandoned, was called a "dry." This is where miners changed into work clothes before going underground and cleaned up before going home. The building could handle 1,000 men at one time.

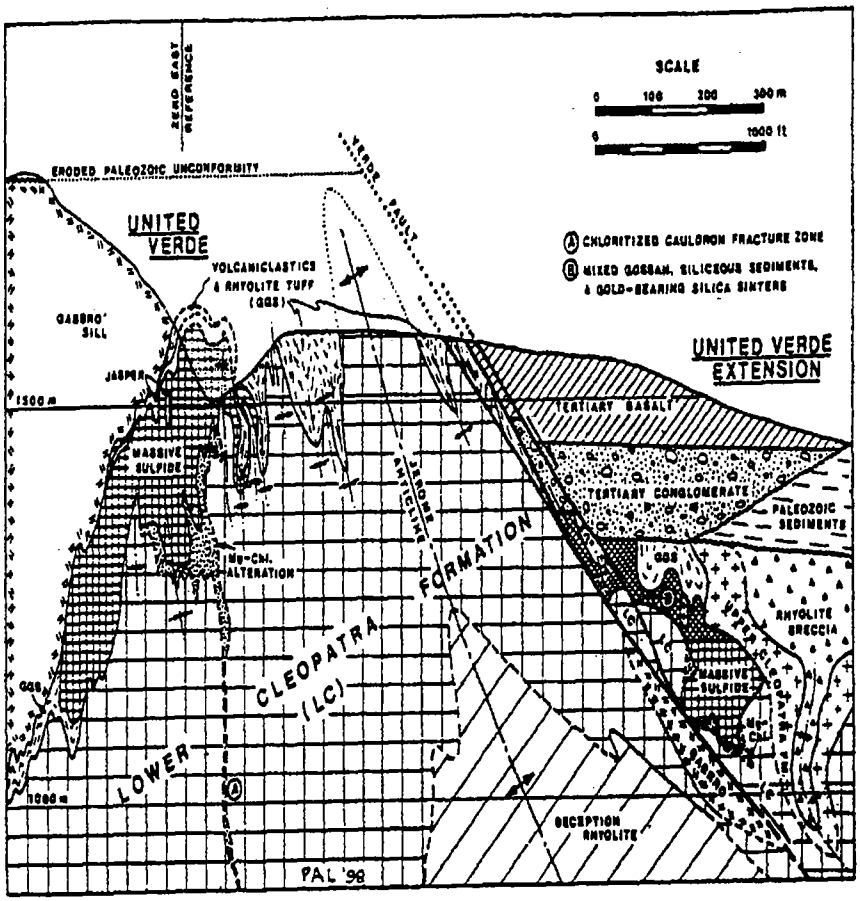
Stop 8: United Verde Mine – Open Pit

It is probable that malachite and azurite pigments were mined from this site in prehistoric times. Spanish explorers, led by Hopi guides, described surface deposits here in 1585, but "modern" mining did not begin for another 300 years. Between 1895 and 1975 the two most important mines at Jerome, the United Verde and the United Verde Extension (UVX), produced more than 1.8 million tons of copper metal. That's enough to provide every individual, living in the United States today, with 14 pounds of refined copper. In addition, the mines produced 57 million ounces of silver and more than 1.5 million ounces of gold as byproducts from the copper ores. Zinc, never extracted during the main period of mining, remains as a reserve. Cominco American, Inc. is currently evaluating the zinc potential in the United Verde mine under a joint-venture agreement with Phelps Dodge Corporation.

The ore deposits formed on a sea floor contemporaneously with the submarine extrusion of volcanic rocks 1,750 million years ago. Black-smoker massive-sulfide mounds and chimneys, identical with those that have been found on the modern sea floor at submarine vent sites, are present in the United Verde deposit. These rocks were later subjected to intense deformation during a period of mountain building. All rocks in the district have been strongly folded.

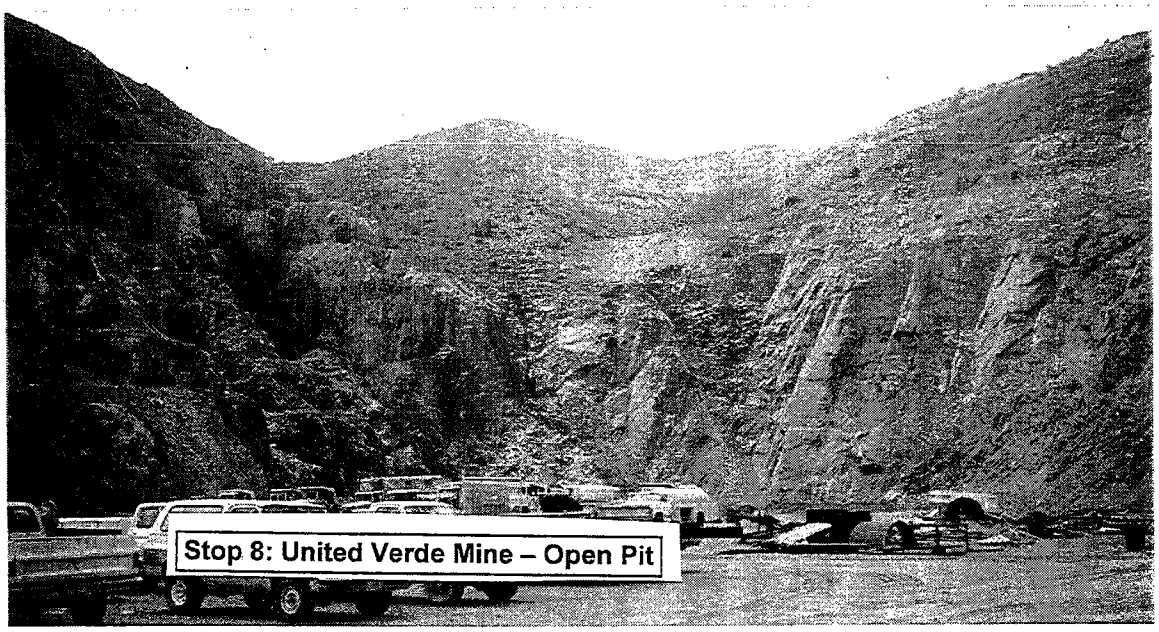
The United Verde and UVX, world-class ore bodies, are separate massive sulfide deposits that were emplaced on the uppermost surface of the Cleopatra Rhyolite. Because of folding, the ore bodies are now on opposite limbs of a large, north-northeast-plunging anticlinorium. Mining took place down to the 4,500-foot level, or 4,200 feet below the 300-level bench that leads into the open pit.

Early in the century, stubborn mine fires began to burn underground in exposed areas of the massive sulfide. The only way to contain the fire was to dig it out completely. This open pit, begun in about 1919, produced a great deal of additional ore that was missed by earlier underground mining. Its life was over by the end of World War II.



STOP 7 & 8 East-west geologic cross section through the Jerome anticlinorium showing the location of the United Verde and UVX volcanogenic ore deposits. The view looks due north. The United Verde deposit cropped out on the erosion surface but the UVX deposit to the east remained undiscovered until late 1914.

Stop 8:
Stop 7:


Stop 8: United Verde Mine – Open Pit

202.4

Jerome Retaining Walls

.As we depart the mine and return to SR 89A (near mile post 344), the trace of the Verde Fault is sub-parallel to the highway. In this area the fault is obscured by city buildings and the renovated highway. The Hickey Formation underlies the highway in the vicinity of the hairpin turn, (mile post 344.2) and the city park. Traveling uphill from this location we cross onto the fault line.

Another local structure is the Hull Fault. It trends nearly perpendicular to the Verde Fault, and occupies a wash, (dry drainage channel), north of the old elementary school. Locally this structure dips 70 degrees to the south-east.

The Hull Fault exposes Pre-Cambrian Quartz-Porphyry to the northwest, and the Deception Rhoyolite, to the southeast. The Deception Rhyolite is easily identified by its reddish buff to cream color and pronounced (roughly) orthogonal jointing.

The Tapeats Sandstone has been locally quarried and incorporated into the construction of many of the structures in Jerome. This material resisted erosion and is easily broken into rectangular shapes. The highway was originally constructed of local random rubble (breccia), clay rich soil, and fault gouge. Many of the stone retaining walls were augmented with reinforced structural concrete retaining walls in the 1930's by the WPA.

This area has a long history of geotechnical problems that include: differential settlement, creeping soils, landslides and deteriorating residential and municipal foundations. Newspaper accounts document mass movement phenomena dating back to February 1928. At that time, the Verde Copper News describes a "crack in the pavement of Main street, crevices in office buildings, jagged irregular cracks in brick and concrete structures, the collapse of a church building and ominous bulges in walls." This event severely damaged an entire block of buildings on the east side of Main street.

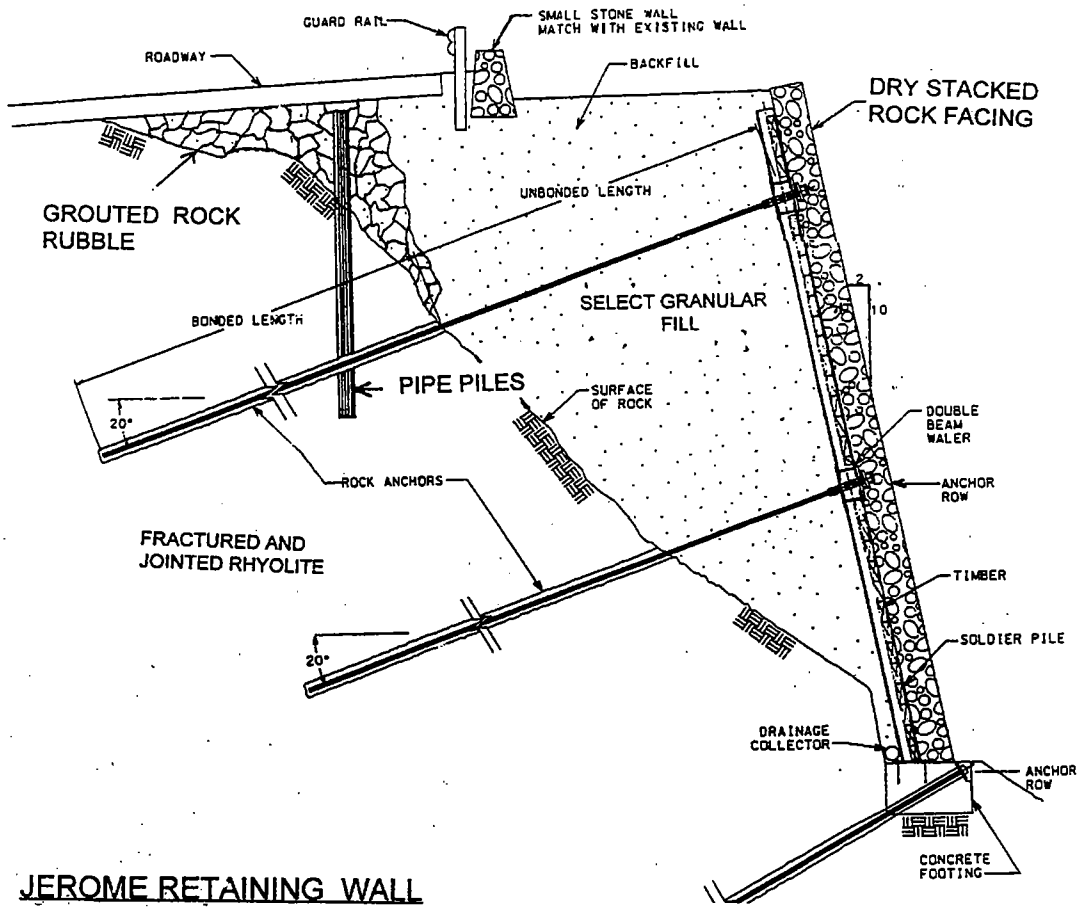
Within the past 15 years, many of the original retaining structures in this area, have been replaced, with a series of cantilevered retaining walls with select back-fill and good drainage. These walls have been constructed with stone facades to mimic the 1930's architecture.

In the winter of 1993, a severe winter storm initiated a landslide at the edge of town, eroding bouldery material from the embankment, and destroying one lane of the highway. In order to keep the highway open, a number of stability techniques were employed. First the remaining rock rubble was stabilized by installing small pipe piles just behind the scarp face. These piles were grouted in place along with the remaining rock rubble. Next, an H-beam column and lagging retaining wall was rock bolted into position and anchored the entire mass into the Deception Rhyolite. Finally a dry stacked stone wall was constructed from on site waste material. The overall effect was to create a modern retaining structure with a look of "turn of the century" construction techniques.

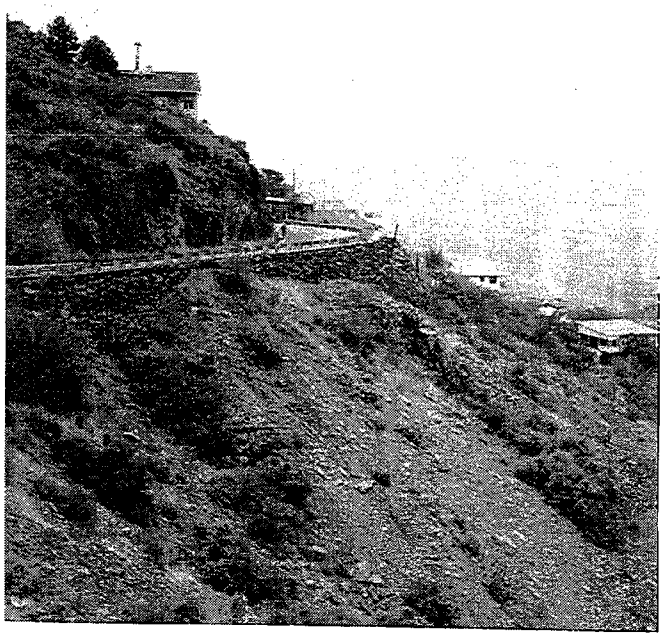
202.5 Leave Jerome

207.9 Pre-basalt gravel channel on right.

209.5 Pass over Mingus Mountain. Watch for Hickey Formation basalt flow overlying baked gravel. Make descent through sequence of Paleozoic carbonate rocks. The highway



JEROME RETAINING WALL
CROSS - SECTION



crosses the Coyote fault at the base of the mountain. This fault, which separates the Black Hills from Prescott Valley and Chino Valley, produces occasional small earthquakes. Displacement along the fault ranges from less than a meter at the northern end of the black Hills to more than 360 meters in southeastern Prescott Valley.

Between mileposts 227 and 228, the highway crosses Coyote Wash, one of several drainages in the area along which teeth and bone fragments of Ice Age mammals have been found.

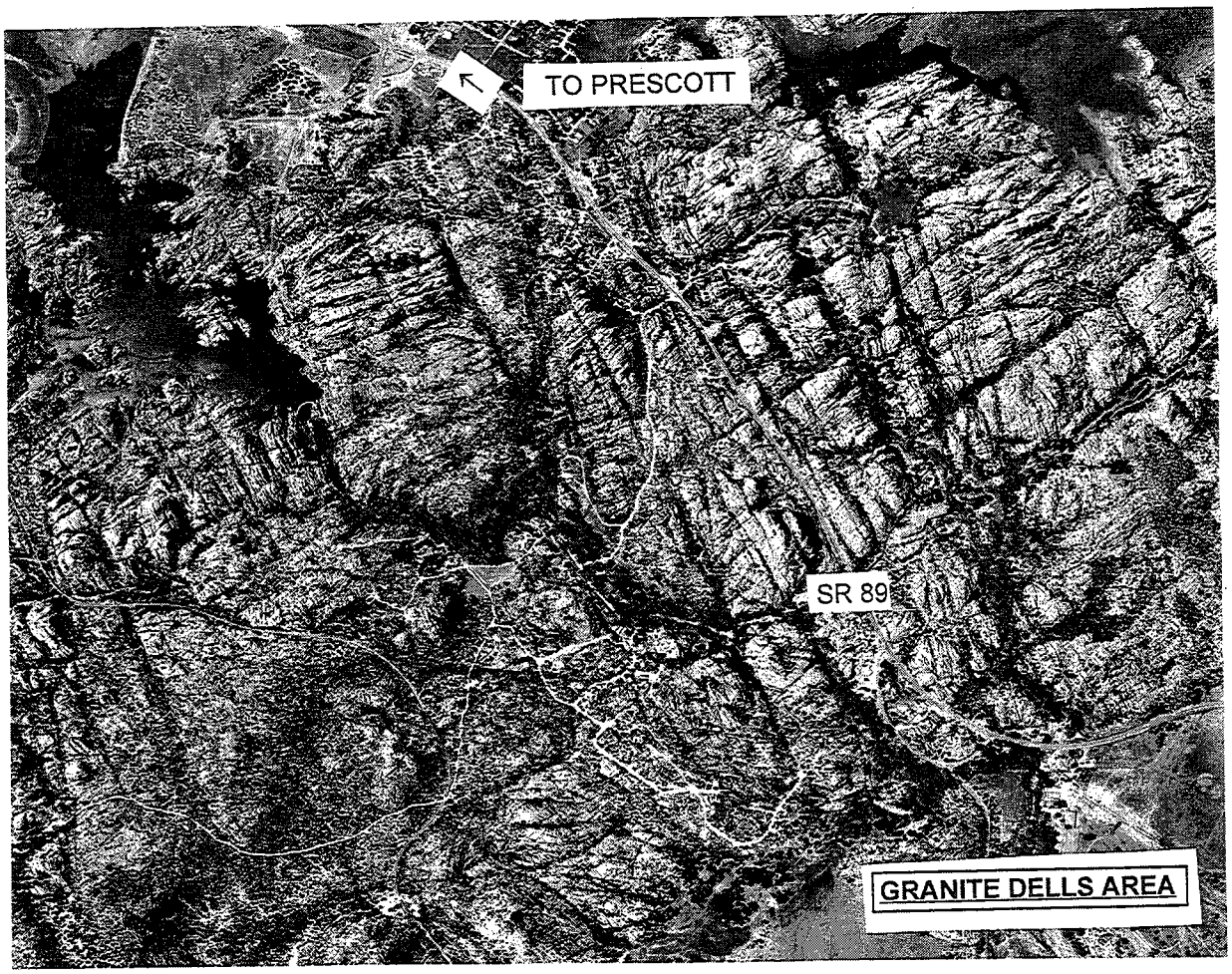
221.0 Enter Prescott Valley. Stay on SR 89 A.

228.1 Intersection with SR 89. Turn left (south) on SR 89.

228.2 Granite Dells. This is a Precambrian-age granite that was deeply weathered and subsequently buried by younger rocks. The granite is now being exhumed, exposing the ancient weathered and eroded land surface, including spheroidal boulders and well-developed joint sets. Color banding has developed parallel to the major joints and gives a striking appearance to the granite. The granite has elevated concentrations of uranium-bearing minerals that generate radon gas as one of their daughter products.

233.3 Enter Prescott. Turn left on SR 69. Turn left at first light and return to Prescott Resort.

End of trip.



REFERENCE:

1 Jeb Stuart Rosebrook, 1994, The Black Canyon Highway: Highway to History, 1863-1949, MS Thesis, Arizona State University

Appendix A: Cumulative Index of HGS Proceedings

CUMULATIVE INDEX

HIGHWAY GEOLOGY SYMPOSIUM PROCEEDINGS

VOLUME I - First, Second, Third and Fourth Highway Geology Symposia

First Annual Symposium on "Geology As Applied To Highway Engineering"
April 14, 1950 - Department of Highways - Richmond, Virginia.

Parrott, W. T., "Synopsis Of First Highway Geology Symposium."
(Papers presented not published).

Second Annual Symposium on "Geology As Applied To Highway Engineering"
February 16, 1951 - Virginia Department of Highways - Richmond, Virginia.

Cornthwaite, A. B., "The Adhesion Of Bituminous Films To Highway Aggregates."
Cooper, B. N., Dr., "Geological Enterprise In Virginia - Present And Future."
Whitmore, Frank C., Jr., "The Importance Of Geology In Military Highway Construction."
McConnell, Duncan, Dr., "Detrimental Minerals In Concrete Aggregate."
Granger, A. T., "What Does The Engineer Expect Of The Geologist?"
Hicks, L. D., "The Use Of Plate Bearing Tests In The Thickness Design Of Flexible Pavements."
DeBuchananne, George D., "The Control Of Groundwater In Consolidated Rocks."
Woolf, D. O., "The Identification Of Rock Types."
Wheeler, Frank W., "The Construction Of Highway Bridges And Separation Structures In Unconsolidated Sediments."

Third Annual Symposium on "Geology As Applied To Highway Engineering"
February 29, 1952 - Virginia Department of Highways and Virginia
Military Institute - Lexington, Virginia.

Dobrovolny, Ernest, "Application Of Geology To Highway Work As Practiced By The States, A Questionnaire Summary."
Nicol, Allen H., "Studies Of Aggregates On Okinawa."
Nesbitt, Robert H., "Problem Minerals In Concrete Aggregates Of The Southeastern States."
Anderson, Rex S., "Geology And Highway Engineering - A Continuing Mission."
Baker, R. F., "The Design Of The Slopes Of Highway Rock Excavations In West Virginia."
Hadley, Jarvis B., "The Geology Of A Highway Slide At Gatlinburg, Tennessee."
Scroggie, Everette, "The Application Of Geology To Bridge Foundations."
Parrott, W. T., "Geological Problems In The Design And Construction Of Highways In Virginia."

Fourth Annual Symposium on "Geology As Applied To Highway Engineering" February 20, 1953 - Morris Harvey College and West Virginia State Road Commission - Charleston, West Virginia.

Gregg, L. E., and Havens, James H., "Applications Of Geology To Highway Engineering In Kentucky."
Marshall, Harry E., "Some Experiences Of The Department Of Highways With Landslides In Ohio."
McNeal, John D., "The Application Of Geology To Highway Subdrainage In Kansas."
Woods, K. B., Johnstone, J. C., and Yoder, E. J., "Some Engineering Problems Associated With The Preglacial Marrietta River Valley."
Price, Paul H., "The Landslide Problem."
Cavendish, Ray, "Landslides Affecting West Virginia Roads."
Eckel, Edwin B., "Contributions By The U. S. Geological Survey To Highway Engineering Research."
Philbrick, Shailer S., "Design Of Deep Rock Cuts In The Conemaugh Formation."

VOLUME II - Fifth and Sixth Highway Geology Symposia

Fifth Annual Symposium on "Geology As Applied To Highway Engineering" March 16, 1954 - The Ohio State University and the Ohio Department of Highways.

Melvin, John H., "Correlation Of Geological Studies By Various Governmental Agencies In Ohio."
Hyland, John R., "Aids Of The Ohio Geologic Survey For Highway Engineers."
Warrick, W. A., "Application Of Geology To Highway Engineering As Seen By A Highway Engineer."
Mason, Neil E., "Ohio's Experience In The Use Of Geophysical Methods In Subsurface Exploration."
Norris, Stanley E., "Importance Of Ground Water Studies To Highway Engineering."
Lewis, D. W., "Effect Of Coarse Aggregate On Concrete Durability."
Mintzer, Olin., "Geology In The Engineering Curriculum, The Highway Engineer's Viewpoint."
Supp, Carl W. A., "Geological And Soils Engineering On Ohio Turnpike Project No. 1."
Barger, Lewis, "Three Dimensional Aspects Of Landslides."

Sixth Annual Symposium on "Geology As Applied To Highway Engineering" February 18, 1955 - Johns Hopkins University and The Maryland State Roads Commission.

Singewald, Joseph T., Dr., "The Role Of The Geological Surveys In Geology As Applied To Highway Engineering."
Barber, Edward S., "Geology And Foundations."
Smith, Preston C., "Use Of Aerial Photographs In Engineering - Soil Mapping."
Melville, Phillip L., "Concrete Aggregate Reaction In Virginia."
Mobley, Arthur B., "Earth Resistivity Indicates Subsurface Geology."
Supp, Carl W. A., "Engineering Geology Of The Chesapeake Bay Bridge."
Allen, Alice S., "Sources Of Information On Ground Conditions."

VOLUME III - Seventh, Eighth, Ninth and Tenth Highway Geology Symposia

Seventh Annual Symposium on "Geology As Applied To Highway Engineering" February 24, 1956 - N. C. State Highway and Public Works Commission - Raleigh, North Carolina.

Baker, Robert F., "Engineering And Landslides."
Mather, Bryant, "Application Of Petrographic Procedures To Highway Engineering."
Deuterman, Martin, "Investigation Of Bridge Foundations."
Laursen, Emmett M., "River Bed Scour At Bridge Foundations."
Laurence, Robert A., "Geologic Features Of The Eastern States As Related To Highway Engineering."
Bickel, J. O., "Equipment Used In Geological Engineering."
McCullough, Charles R., "Aerial Photos And Highway Engineering."

Eighth Annual Symposium on "Geology As Applied To Highway Engineering" February 15, 1957 - The Pennsylvania State University and Commonwealth of Pennsylvania - Department of Highways.

Eckel, Edwin B., "New Developments In The Study Of Landslides."
Gary, Carlyle, Dr., "Geological Information In Pennsylvania For The Highway Engineer."
Engel, Harry J., "Bridge Foundation Experiences."
Weeden, Harmer A., "Pedology Helps The Highway Engineer."
Bird, Paul H., "Experience In Designing Rock Slopes In New York State."
Baker, William O., "Photogrammetry In Practice."

Ninth Annual Symposium on "Geology As Applied To Highway Engineering" February 21, 1958 - University of Virginia and Virginia Department of Highways.

Gay, Archer B., "Highway Geology And The Contractor."
Legget, Robert F., "Geology And Transportation Routes."
Maner, Alfred W., "Soils And Geological Engineering Partners, Not Competitors."
Rice, James M., "A Rapid Method For Determining The Resistance Of Ledge Rock To Freezing."
Dole, George J., "Ammonium Nitrate As An Explosive."
Tuttle, Curtis R., "Application Of Seismology To Highway Engineering Problems."

Tenth Annual Symposium on "Geology As Applied To Highway Engineering" February 20, 1959 - Georgia Institute of Technology and Georgia State Highway Department - Atlanta, Georgia.

Felix, George D., "Geology As An Aid To Right-Of-Way."
Furcron, A. S., Dr., "Distribution And Character Of Stone For Aggregate In Georgia."
Marshall, Harry E., "Design Considerations In The Treatment Of Soft Foundations."
Upham, Charles M., "The Use Of Geological Investigations In Foreign Consulting Work."
Seeger, Ralph W., "Highway Material Survey In West Virginia."
Fletcher, G. A., "Geology In Foundation Engineering."
Belcher, Donald J., "Applied Geomorphology."

Eleventh Annual Highway Geology Symposium - February 26, 1960 - Florida State University - Tallahassee, Florida.

Vernon, Robert O., "The Geological Distribution Of Highway Base Course Material And Aggregate In Florida."
Mather, Bryant, "Petrology Of Concrete Aggregate."
Carey, W. N., Jr., and Schmidt, W. J., "The AASHO Road Test: A Progress Report."
Michener, P. Z., "Preliminary Subsurface Investigation Of The Chesapeake Bay Crossing."
Fritz, Axel M., Jr., "The MD Engineering Seismograph And Its Application To Highway Engineering."
Bruun, Per, "Beach Erosion And Protection In Florida."
Lynch, S. A., and Weaver, Paul, "Pavement Disruption By Recent Earth Movements."
Radzikowski, Henry A., "Rock And Earth In The Highway Program."

Twelfth Annual Symposium on "Geology As Applied To Highway Engineering" - February 10-11, 1961 - Engineering Experiment Station, Bulletin Number 24 - The University of Tennessee, Knoxville.

Young, Norman C., and Pierce, T. R., "Principal Highway Engineering Characteristics Of Some Tennessee Formations."
Goodwin, William A., "Evaluation Of Pavement Aggregates For Non-Skid Qualities."
Nichols, Donald R., and Yehle, Lynn A., "Highway Construction And Maintenance Problems In Permafrost Regions."
Officer, Charles B., "Use Of Continuous Seismic Profiler (Sparker) In Geologic Investigations For Vehicular Tunnel And Bridge Crossings."
Bailey, Reed W., "Madison River - Hebgen Lake Earthquake And Highway Problems."
Stuart, W. Harold, "Geological Conditions Complicating Highway And Railroad Relocations In The Northwest."
Moore, R. Woodard, "Observations On Subsurface Explorations Using Direct Procedures And Geophysical Techniques."
Dorman, C. W., "The Economics Of Natural Resource Valuation."
Gutschick, Kenneth A., "Altering Physio-Chemical Characteristics Of Clay-Bearing Soils With Lime."

Thirteenth Annual Highway Geology Symposium - March 16, 1962 - Phoenix, Arizona.

Hall, Bruce M., "Status Of Geological Registration."
Gallaher, B. J., "Desert Materials - Types And Uses."
Eckel, E. B., "Use Of Engineering Geology Maps."
Smith, P. C., "Purpose Of The Highway Materials Inventory."
Sergent, B. D., "Soil Mechanics In The Southwest."
Mitchell, Stanley N., "Problems Facing The Engineer And The Geologist In A Highway Department Employing An Engineering Geologist."
Kersch, George A., "Regional - Aerial Geological Investigations In Highway Geology."

Fourteenth Annual Highway Geology Symposium - March 22, 1963 - A. & M. College of Texas - College Station, Texas, Texas Highway Department, and the Texas Bureau of Economic Geology.

Pelzner, Adrian, "Applications Of Agricultural Soil Surveys To Highway Geology."
Laughter, C. N., "Correlation Of Culvert Performance And Soil Conditions."
Henry, H. A., "Engineering Geology Operations In The Texas Highway Department."

Benson, Gordon R., "Geology - A Vital Part Of Subsurface Engineering In Illinois."
Welp, Theodore L., "Materials Geology, Co-ordinator Of The Aggregate Inventory."
Lemish, John, "Carbonate Aggregate Research."
Moore, Richard T., "The Effect Of Fractured Ground On Highway Structure Design."
Leith, C. J., and Gupton, C. P., "Some Geologic Factors In Highway Slope Failures In North Carolina."
Drew, E. D., "The Development And Utilization Of Engineering Geology In The California Division Of Highways."

Fifteenth Annual Highway Geology Symposium - March 19, 1964 - Missouri Geological Survey and Water Resources - Rolla, Missouri.

Helmer, R. A., "The Duties And Training Of A Geologist In The Oklahoma State Highway Department."
Grimes, Walter W., "Geology And Foundation Problems Of Glacial Drift In Eastern South Dakota."
Migliaccio, Ralph R., "Engineering And Construction Problems In The Valdez District, Alaska."
Landrum, J. D., "A Foundation Investigation Of Cherokee Cave Under Route I-55, City Of St. Louis."
Lounsbury, W. R., and Schuster, R. L., "Petrology Applied To The Detection Of Deleterious Materials In Aggregates."
West, T. R., and Aughenbaugh, N. B., "The Role Of Aggregate Degradation In Highway Construction."
Heagler, John B., Jr., "Mineralogy And Soil Stabilization."

Sixteenth Annual Highway Geology Symposium - March 25-26, 1965 - University of Kentucky - Lexington, Kentucky.

Goodwin, W. A., "The Application Of Geology In The Benefication Of Aggregates."
Laughlin, G. R., Scott, J. W., and Havens, J. H., "Freeze-Thaw Characteristics Of Aggregates."
Lounsbury, R. W., and West, T. R., "Petrography Of Some Indiana Aggregates In Relation To Their Engineering Properties."
Sherwood, W. Cullen, Dr., "The Role Of Aggregate Type In Pavement Slipperiness."
Smith, Preston C., "Landslide Research."
Struble, Richard A., "Shallow Subsurface Exploration Utilizing Airphoto Interpretation And Geophysical Techniques."
Dobrovolny, Ernest, "The Highway Research Board And Its Committee On Engineering Geology."

Seventeenth Annual Highway Geology Symposium - April 21-23, 1966 - Iowa State University - Ames, Iowa, Iowa State Highway Commission.

Burgat, Virgil A., "Engineering Geology In Kansas Highway Construction."
Mitchell, Robert E., "Seismic And Resistivity Matured."
Michael, Robert D., "Techniques Of Engineering Geology In Evaluation Of Rock Cores For Construction Materials Sources."
Blattert, Robert E., "The Effects Of Peat Deposits On Highway Design In Iowa."
Eversoll, Duane A., and Burchett, Ray, "Nebraska Geology And Highway Engineering Procedures."
Bruce, Richard L., "Landslides In The Pierre Shale Of South Dakota."
Kneller, William A., "A Study Of Chert Aggregate Reactivity Based On Observations Of Chert Morphologies Using Electron Optical Techniques."
Fritz, Axel M., "Engineering Geophysics: Its Use And Abuse."
Ledbetter, John F., "Aspects Of Using A Borehole Deflectometer To Diagnose An Unstable Rock Slope."
Welp, T. L. and Others, "Panel Discussion On Chemical And Physical Reactions Of Carbonate Aggregates In Concrete."
McElherne, T. E., "Introduction."
Lemish, John, "Background On Carbonate Aggregate Behavior In Concrete."
Mather, Katharine, "Test Methods Used In Investigating Alkali - Carbonate Reaction."
Gillott, J. E., "Concrete Performance As Related To The Behavior Of Carbonate Aggregates."
Newlon, Howard, "Chemical And Physical Reactions Of Carbonate Aggregates In Concrete."
Axon, Ely O., "Concrete Performance As Related To The Behavior Of Carbonate Aggregates."
Mather, Katharine, "Waterways Experiment Station Experience With Alkali - Carbonate Reaction."
Newlon, Howard, "Physical Test Methods Used In Investigating Aggregates."
Dunn, James, "Distress Of Aggregate By Absorbed Water."
Lemish, John, "Concrete Weathering Studies."
Panelists and Audience, "Questions And Answers."

Eighteenth Annual Highway Geology Symposium - April 20-21, 1967 - Purdue University and Indiana Highway Commission - Lafayette, Indiana.

Woods, K. B., "Some Highway Problems Of The United States Correlated With Physiographic Provinces."
Miles, R. D., "Anyone Can Interpret Soil Groups From Aerial Photographs."
Struble, R. A., and Mintzer, O. W., "Combined Investigation Techniques For Procuring Highway Design Data."
Berliant, R. F., and Sanborn, A. F., "Comprehensive Investigations Facilitate Design Of Interstate Highways Over Bottomland Soils."
Johnson, Robert B., "The Use And Abuse Of Geophysics In Highway Engineering."
Vineyard, Jerry D., and Williams, James H., "A Foundation Problem In Cavernous Dolomite Terrain."
Fredericksen, Walter, "Stabilization Of Abandoned Mine Under An Interstate Highway."
Hotler, C. F., "Soil Survey Practices In Indiana."
French, Robert R., and Carr, Donald C., "Geologic Factors Affecting The Exploration For Mineral Aggregates In The Indianapolis Area."
Malott, D. F., "Shallow Geophysical Exploration By The Michigan Department Of State Highways."
Panel Members, Panel Discussion: Preliminary Exploration For Highways With Emphasis On Local Problems."

Nineteenth Annual Highway Geology Symposium - May 16-17, 1968 - West Virginia Geological and Economic Survey, State Road Commission of West Virginia, West Virginia University Department of Geology, and West Virginia University College of Engineering.

Donaldson, Alan C., "Geology Of West Virginia With Special Reference To The Field Trip Area."
Hayes, Russel R., and Worrell, Donald T., "Geologic And Engineering Implications Of Poor Quality Sandstone From Simulated Highway Tests."
Leach, Richard C., " The Problem And Correction Of Landslides In West Virginia."
Thompson, Berke L., and Long, Donald C., "A Controlled Fill Over Sediments Of Ancient Lake Monongahela Near Fairmont, West Virginia."
Seeger, Ralph W., "Geological Investigations For A Trans-Andean Highway."
Smith, James D., "Geology Its Relation To The Design Of The East River Mountain Tunnels."

Cooper, Byron N., "Geology Of Big Walker Mountain Tunnel On Interstate Route 77, Wythe And Bland Counties, Virginia."
McGrain, Preston, and Dever, Garland R., Jr., "The Geometry Of Limestone Aggregate Sources In Kentucky's Appalachian Region."
Arkle, Thomas, Jr., and Brock, Samuel M., Jr., "The Geology Of Construction Sand And Gravel Resources In West Virginia."

Twentieth Annual Highway Geology Symposium - April 16-19, 1969 - University of Illinois - Urbana-Champaign, Illinois.

Thornburn, T. H., and Liu, T. K., "Soils Of Illinois And Their Engineering Characteristics."
Brownfield, Robert L., "A Geotechnology Profile In Jo Davies County, Illinois."
Keene, Kenneth R., "Problems With Highway Cuts In Loess Near East St. Louis, Illinois."
Harvey, Richard D., "Fracture Surfaces Of Carbonate Aggregates: A Scanning Electron Microscope Study."
Gamble, James C., Hendron, A. J., Jr., and Way, Grover C., "Foundation Exploration For Interstate 280 Bridge Over Mississippi River Near Rock Island, Illinois."
Ferland, Jack A., "Seismic Mapping Of Cavities And Voids."
Thompson, M. R., "Properties Of Lime-Treated Soils."

Twenty-First Annual Highway Geology Symposium - April 23-24, 1970 - The University of Kansas - Lawrence, Kansas, State Highway Commission of Kansas, State Geological Survey of Kansas.

Wilson, Frank W., "Highway Problems And The Geology Of Kansas."
Goodfield, Alan G., "Rock Falls And Landslides Along State Highway 79 At Clarksville, Missouri."
Hammerquist, D. W. and Hoskins, Earl, "Correlation Of Expansive Soil Properties And Soil Moisture With Pavement Distress In Roadways In Western South Dakota."
Taylor, Charles L., "Geometric Analysis Of Rock Slopes."
Sennett, Robert B., "Geologic Factors In Design Of Excavated Rock Slopes."
Jenkins, Gomer, Jr., "Hydraulic Borrow Materials In Urban Areas."
Eversoll, Duane A., "Amphibious Drilling Rig."
Whitfield, John W., and Williams, James H., "Underground Transportation Routes And Quarry Practices In The Kansas City, Missouri-Kansas Area."
West, Terry R., "Application Of Remote Sensing To Highway Locations."
Stallard, Alvis H., "Remarks On Kansas Highway Research Project In Remote Sensing For Soils And Geologic Mapping."
Hartley, Alan, "The Influence Of Geological Factors Upon The Mechanical Properties Of Road Surfacing Aggregates."

Twenty-Second Annual Highway Geology Symposium - April 22-23, 1971 - The University of Oklahoma - Norman, Oklahoma, Oklahoma Geological Survey, Oklahoma Department of Highways.

Mankin, Charles J., and Johnson, Kenneth S., "Geology of Oklahoma - A Summary."
Hayes, Curtis J., "Engineering Classification of Highway - Geology Problems In Oklahoma."
Laguros, Joakim G., and Kumar, Subodh, "Predictability Of Shale Behavior."
Deere, Don U., and Gamble, James C., "Durability - Plasticity Classification Of Shales And Indurated Clay."
Bayless, Glen, "Post-Construction Performance Of P-18 Bridge Abutments And Approach Fills, Eastern Oklahoma."
Parrott, William T., "Control Of A Slide By Vertical Sand Drains, U. S. Route 220, Alleghany County, Virginia."
Merten, Fred K., "Straight Creek Tunnel Construction, Route I-70, Colorado."
Ivey, John B., "Highway Geology Feasibility Study, Luluabourg To Mbuji-Mayi Republic Of Congo (Kinshasa)."
Cleaves, Arthur B., "The Camino Marginal, Peru."
Gedney, David S., "A Design Approach To Rock Slope Stability."
Thompson, Berke L., "The Use Of Air As A Drilling Medium For Subsurface Investigation."
Johnson, B. J., "Penetrohammer, The Penetrometer Machine And Its Application."

Twenty-Third Annual Highway Geology Symposium - April 27-28, 1972 - Virginia Highways Research Council, Virginia Department of Highways, Virginia Division of Mineral Resources, Old Dominion University - Hampton, Virginia.

Jimenez, John, "Embankment Failures On The Tijuana - Ensenada Turnpike In The Lower California Peninsula, Mexico."
Branthoover, Gerald L., "Geological Engineering Investigation Of Talus Slopes In Lewiston Narrows, Pennsylvania."
Herbold, Keith, "Cut Slope Failure In Residual Soil."
Vogelsand, W. H., and Munyan, A. C., "Engineering Geology Of The Semi-Indurated Strata Of The Virginia Coastal Plain."
Thomas, Carl O., "Remote Sensing Applications To Near Surface Geology."
Mitchell, D. A., and Brack, C., "Ecological Impact Of Hydraulic Construction Methods In Georgia."

Whitlow, B. S., "Investigation Of Deterioration In Concrete Roadway Slab Of The Robert E. Lee Bridge, Richmond, Virginia."
Baldwin, J. S., and Dawson, J. W., "Effects Of Angular Sands On
Sennett, R. B., "Engineering Geology, Environmental Geology And Land Use."

Twenty-Fourth Annual Highway Geology Symposium - August 9-10, 1973 - Wyoming State Highway Department - Sheridan, Wyoming.

Miller, Daniel, "Wyoming Geology" (Abstract Only).
Royster, David L., "Highway Landslide Problems Associated With The Escarpment Areas Of The Cumberland Plateau In Tennessee."
Wisehart, Richard M., and Wagner, J. Ross, "Slope Stability And Engineering Consequences Of A Stream Capture, Bluff Creek, Northern California."
Edwards, Larry J., "Engineering Geologic Map Units For Highway Planning."
Patty, Tom S., "Accelerated Polish Test For Coarse Aggregate."
McKittrick, David P., and Gedney, David S., "Reinforced Earth For Highway Applications."
Bukovansky, Michael, "Engineering Geology And Rock Mechanics Help To Design A Freeway In Northern Spain."
Gilmore, John B., "Landslide Problems On The West Approach To Eisenhower (Straight Creek) Tunnel, Colorado."
Everitt, Martin C., and Holland, T. W., "The Shell Canyon, Wyoming Landslides."
Bauer, Edward, "Application Of Geology To Highway Construction In Mountain Terrain, Lovell-Burgess Junction, Wyoming."
Patty, Tom S., "Investigation Of Failing Concrete In Houston, Texas, Caused By Unsound Cement."

Twenty-Fifth Annual Highway Geology Symposium - May 23-24, 1974 - N. C. Department of Transportation and Highway Safety, N. C. Department of Natural and Economic Resources, and N. C. State University - Raleigh, North Carolina.

Keene, Kenneth R., "An Evaluation Of Sand Drain Installation Methods In Recent Alluvium."
Conrad, Stephen G., "Introduction Of The Geology And Mineral Resources Of North Carolina."
Royster, David L., "Construction Of A Reinforced Earth Fill Along I-40 In Tennessee."
Brown, L. F., Jr., and Fisher, W. L., "Environmental Atlas Of The Texas Coastal Zone And Its Role In Land Use Planning."
Sowers, George F., "Geological And Seismic Engineering For Nuclear

Power Plants In The Southeast."
Welby, Charles W., "ERTS And Multispectral Photography."
Goughnour, Roger D., and Mattox, Robert M., "Subsurface Exploration State Of The Art."
Sams, Clay E., and Gardner, Charles H., "Engineering Geology Of I-26 Landslides, Polk County, North Carolina."
Eades, James L., Dr., "Lime Stabilization."
Paper Not Published

Twenty-Sixth Annual Highway Geology Symposium - August 13-15, 1975 - Idaho Transportation Department, Division of Highways - Boise, Idaho.

Bond, John G., "General Geology Of Northern Idaho And Western Montana And Its Implication On Highway Construction."
Mathis, Don H., "Application Of Geotechnique To Design And Construction In North Idaho."
Larsen, Ronald E., "Seismic Designed Backslopes And Evaluation In A Structurally Disturbed Basalt Section."
Welch, J. David, "Environmental Geology Survey, Regional Land Use And Intermodal Transportation Planning, Northeastern Pennsylvania."
Steward, John E., "Use Of Woven Plastic Filter Cloth As A Replacement For Graded Rock Filters."
Garner, L. E., "Aggregate Resource Conservation In Urban Areas."
Patty, Tom S., "Petrography As Related To Potential Skid Resistance Of Paving Aggregates Used On Texas Highway Projects."
Bailey, Allen D., "Rock Types And Seismic Velocities Versus Rippability."
Johnson, Edgar L., "Traffic Safety Instrumentation, Foundation Landslide, Oregon."
Farnham, Paul R., "In-Situ Measurement Of Shear-Wave Velocities For Engineering Applications."
Bartholomew, Charles L. and Ireland, Herbert O., "The Use Of Organic Topsoils As Construction Materials."
Kumar, S.; Annamalai, M.; and Laguros, J. G., "A New Analytical Approach To Split Tensile Strength Of Pavement Materials."
Thronton, Sam I., and Arulanandan, Kandiah, "Collapsible Soils, State-Of-The-Art."
Weatherby, David E., and Chapman, Ronald K., "Design And Installation Of An Earth Tieback Support System."
Denness, Bruce, and Smith, Robert M., "A Landslide In Pleistocene Deposits: Columbia (S. Am.)."
Wilmarth, Verle R.; Kaltenbach, John; Lenoir, William B., "Skylab Explores The Earth."

Twenty-Seventh Annual Highway Geology Symposium - May 19-21, 1976 - Florida Department of Transportation and University of Florida -

Orlando, Florida.

Wisner, William A., "Florida Geology And Sinkholes."
Fountain, Lewis S., Sr., "Subsurface Cavity Detection: Field Evaluation Of Gravity, Radar, And Earth Resistivity Methods."
Morey, Rexford M., "Detection Of Subsurface Cavities By Ground Penetrating Radar."
Brooks, H. K., "Offshore Foundations Evaluation By Seismic Techniques" Paper Not Published
Omes, Gildas, "Applications Of Geophysical Methods To The Detection Of Shallow Karstic Cavities."
Benson, Richard C., "Applications And Economics Of Engineering Geophysics."
Moore, Harry L., "Understanding The Phenomenon Of Piping."
McGrain, Preston, "Lapis-Type Features In Kentucky Karst Region."
Williams, Don E., "Underground Construction In The Development Of Trails In A Large Cavern System In The Southern Ozark Mountains Karst Terrain."
Strohm, W. E., Jr., "Shale Deterioration Related To Highway Embankment Performance."
Kumar, Subodh, "Shrinkage Factor For Fill Construction Iowa."
Hammond, Tom, and Huckle, Horace F., "A Resume' Of A Study Between The Soil Conservation Service And The National Aeronautics And Space Administration To Determine If Remote Sensing Could Be Technically And Economically Developed As A Beneficial Tool To Assist Soil Scientists In Selecting Spots For Field Observations."
West, Terry R., "Evaluation Of Gravel Deposits Using Remote Sensing Data, Wabash River Valley North Of Terra Haute, Indiana."

Twenty-Eighth Annual Highway Geology Symposium - August 10-12, 1977 -
South Dakota Department of Transportation, Geological Survey, and South Dakota School of Mines & Technology - Rapid City, South Dakota.
Carrigan, Mark C., and Shaddrick, David, "Homestake's Grizzly Gulch Tailings Disposal Project."
Lovell, C. W., Bailey, M. J., and Wood, L. E., "Point Load Strength And Hardness Measures Of Shale."
Youell, James R., "Angle Hole Drilling Approach For Highway Engineering Data Collecting."
Burgat, Virgil A., "Cut Slope Design Based On Stability Characteristics."
Bauer, Edward J., "Reinforced Earth Fill On Steep Mountain Terrain, Highway 14a, Big Horn County, Wyoming."
Hanna, Bruce E., "The Use Of Reinforced Earth Walls As Bridge Abutments."
Meadors, G. S., Jr., "The Geology And Construction Techniques Of The Second Hampton Roads Bridge Crossing, Norfolk, Virginia."
Grimes, Walter W., "Geotechnical Investigation For The Des Plaines River System Tunnels And Shafts."
Gardner, Charles H., and Tice, J. Allen, "A Study Of The Forest City

Creep Slide On The Oahe Reservoir, South Dakota."
Glass, F. R., "Horizontal Drains As An Aid To Slope Stability On I-26, Polk County, North Carolina."
Royster, David L., "Some Observations On The Use Of Horizontal Drains In The Correction And Prevention Of Landslides."

Twenty-Ninth Annual Highway Geology Symposium - May 3-5, 1978 - Maryland State Highway Administration and Geological Survey - Annapolis, Maryland.

Cleaves, Emery T., "Geologic Contrasts Across The Fall Zone."
Rabchevsky, George A., and Brooks, David J., "Landsat Overview Of Fall Line Geology."
Langer, William, H., and Obermeier, Stephen F., "Relationship Of Landslides To Fractures In Potomac Group Deposits, Fairfax County, Virginia."
Winter, Ernest, and Beard, Brian, "The "O" Street Slide And Its Geologic Aspects, Washington, D. C."
Supp, Carl W. A., "Engineering Geology Of The Chesapeake Bay Bridges."
Hoover, Earl G., "Health And Environmental Assessment - A Major Geologic Concern On The Fall Line."
Frohlich, Reinhard K.; Maloney, James P.; and Lowry, Bruce E., "Control Of Vibrations From Commercial Blasting In Urban Areas."
Witort, Edward Anthony, Jr., "Effect Of Fall Line Geology On Design Of US-64, Rock Mount, North Carolina."
Guilbeau, Leonard H., "Design Alternatives For Construction Over Compressible Materials."
Monahan, Edward J., "Weight-Credit Foundation Construction Using Foam Plastic As Fill."
Lovell, Charles William, and Lovell, Janet Elaine, "Measure The Voids, Not The Solids."
Baker, Wallace Hayward, "Compaction Grouting Methods For Control Of Settlements Due To Solt-Ground Tunneling."
Brahma, Chandra S., "Significance Of Groundwater And Methods Of Ground Control In Tunneling."
Collison, Gary H., "Exploration And Geochemical Reporting For The Rock Tunnels And Station Of The Mondawmin Section Of The Baltimore Region Rapid Transit System."
Siegel, Ronald A.; Kovacs, William D.; Lovell, Charles William, "New Method Of Shear Surface Generation For Stability Analysis."
Brahma, Chandra S., and Ku, Chih-Cheng., "Geotechnical Perspective On Slurry Wall System."

Thirtieth Annual Highway Geology Symposium - August 8-10, 1979 - Office of Federal Highway Projects, Federal Highway Administration - Portland, Oregon.

Chassie, Ronald G., "Landslide Tests Reinforced Earth Wall."
Mullarkey, J. Ray, "Fabrics In The Highway: The State Of The Art In Civil Engineering Applications."
Watkins, Reynold K., "Structural Performance Of Buried Corrugated Polyethylene Tubing."
Hatheway, Allen W., "Revision Of The 1967 AASHTO Manual On Foundation Investigations."
Jackson, Newton C., "Summary Of Use Of Sawdust For Highway Fills."
Ward, Timothy J., "Modeling Erosion And Sedimentation From Roadways."
Plum, Robert L., "Decision And Risk Analysis As A Practical Tool For Geotechnical Engineers And Geologists."
Kuenzli, James R., "Stabilization Of The Upper Portion Of The Hat Creek Landslide."
Royster, David L., "Landslide Remedial Measures."
West, Terry R., "Petrographic Examination Of Aggregates Used In Bituminous Overlays For Indiana Pavements As Related To Their Polishing Characteristics."
Miller, Henry J., "Geophysical Investigations Of Hampton Roads For Crossing Of Route I-664."
Pope, David H., "A Demonstration Project For Deicing Of Bridge Decks."
Lovell, C. W., "Compaction Prestress Makes A Difference."

Thirty-First Annual Highway Geology Symposium - August 13-15, 1980 - Bureau of Economic Geology and The University of Texas at Austin in Cooperation with Texas Department of Highways and Public Transportation at Austin, Texas.

Moore, H. L., "Karst Problems Along Tennessee Highways: An Overview."
Simpkins, W. W.; Gustavson, T. C.; Alhades, A. B.; Hoadley, A. D., "Impact Of Evaporite Dissolution And Collapse On Cultural Features In The Texas Panhandle And Eastern New Mexico."
Sansom, J. W., Jr., Shurbet, D. H., "Microearthquake Studies In Texas."
Abeyesekera, R. A., and Lovell, C. W., "Characterization Of Shales By Plasticity Limits, Point Load Strength And Slake Durability."
Wilson, John, "The Texas Natural Resources Information System."
Lo, T. Y. K., and Lovell, C. W., "The Geotechnical Data Bank."
Patty, T. S., "Engineering Petrography: Highway Applications."
Allen, P. M., "Evaluation Of Channel Stream Bank Erosion In Urbanizing Watersheds In The Blackland Prairie, North-Central Texas."

Yelderman, J. C., Jr., "The Type Area Concept: A Practical Method Of Integrating Natural Resources With Planning, Development, Maintenance, And Landscaping Of Transportation Systems."
Whittecar, G. R., and Simpkins, W. W., "Drumlins At Potential Sources Of Sand And Gravel In Glaciated Regions."

Thirty-Second Annual Highway Geology Symposium - May 6-8, 1981 - Tennessee Department of Transportation, Division of Soils and Geological Engineering - Gatlinburg, Tennessee.

Glass, F. R., "Unstable Rock Slopes Along Interstate 40 Through Pigeon River Gorge, Haywood County, North Carolina."
Tice, J. Allan, "The Hartford Slide - A Case History."
Aycock, James H., "Construction Problems Involving Shale In A Geologically Complex Environment, State Route 32 - Appalachian Corridor "S", Grainger County, Tennessee."
Mathis, Henry, "Temporary Landslide Corrective Techniques Avert Catastrophe."
Wright, E. M., "Remedial Corrective Measures And State Of The Art For Rock Cut Slopes In Eastern Kentucky."
Watts, C. F., and West, T. R., "A System For Rapid Collection And Analysis."
Hale, B. C., and Lovell, C. W., "Prediction Of Degrability For Compacted Shales."
Wilson, Charles; Pope, David; and Sherman, William, "A Review Of The Progress Of The Wyoming Heat Pipe Program."
Jones, Don H.; Bell, Bruce S.; and Hansen, Jack H., "The Application Of Induced Polarization In Highway Planning, Location, And Design."
Byerly, Don W., and Middleton, Lloyd M., "Evaluation Of The Acid Drainage Potential Of Certain Pre-Cambrian Rocks In The Blue Ridge Province."
Winchester, P. W., Jr., "Some Geotechnical Aspects Of Early Planning Along Corridor K, Appalachian Development Highway; Section Between Andrews And Almond, North Carolina."

Thirty-Third Annual Highway Geology Symposium - September 15-17, 1982 Colorado Geological Survey, USDA Forest Service, FHWA, Colorado Department of Highways - Vail, Colorado.

Bennett, Warren, "Experimental Compaction Of Collapsible Soils At Algodones, New Mexico."
Of State Highway 91, Climax Mine Area, Summit County, Colorado."

Holmquist, Darrel V., "Slope Stability Consideration Of The Colorado State Highway 91 Relocation."
Pakalnis, Rimas, and Lutman, T., "Application Of Vacuum Horizontal Drainage."
Wyllie, Duncan C., and Wood, David F., "Stabilization Of Toppling Rock Slope Failures."
Robinson, Charles S., and Cochran, Dale M., "Engineering Geology Of Vail Pass I-70."
Hynes, Jeffrey L., "Geology Of The Glenwood Canyon Along I-70."
Bell, J. R.; Barrett, R. K.; and Ruckman, A. C., "Geotextile Earth Reinforced Retaining Wall Tests."
Pell, Kynric, and Nydahl, John, "Geothermal Heating Of The Bridges And Tunnels In Glenwood Canyon."
Liang, Y., and Lovell, C. W., "Predicting The Strength Of Field Compacted Soil From Laboratory Tests."
Teme, S. C., and West, T. R., "Determination Of Friction Angle Values For Rock Discontinuities In Regard To Stability Of Highway Cuts."
Turner, A. Keith, "Computer Generated Maps."
Benson, Richard C., "Evaluation Of Differential Settlement Of Collapse Potential."
Sherman, William F., "Geotechnical Applications In Maintenance And Reconstruction Of The Existing Highway System."
Thornton, Sam I., "Fly Ash Leachate In Highways."

Thirty-Fourth Annual Highway Geology Symposium - May 2-4, 1983 - Georgia Department of Transportation, Georgia Geological Survey, FHWA - Atlanta, Georgia.

Dickerson, Robert T., "Investigation, Evaluation, And Quality Control Of Aggregate Sources In Georgia."
Bailey, Warren F., "Georgia Stabilized Embankment Wall Construction."
West, T. R., and Fein, M. R., "Geologic And Economic Aspects Regarding The Development Of An Underground Limestone Mine, Indianapolis, Indiana."
Leary, Robert M., and Klinedinst, Gary L., "Retaining Wall Alternates."
Nicholson, Peter J., "Innovations In Anchored Retaining Walls."
Abramson, Lee W., "Geotechnical Instrumentation Of Modern Retaining Wall Designs In An Urban Setting."
Barksdale, Richard D., and Dobson, Tom, "Improvement Of Marginal Urban Sites Using Stone Columns And Rigid Concrete Columns."
Trettel, Charles W., "Blasting Vibrations In An Urban Environment."
Lambrechts, James R., "Southwest Corridor Project, Boston, Massachusetts."

Gruen, H. A., and Lovell, C. W., "Preloading Peat For Foundation Use."
King, John W., "Measurement Of Construction Influences On Adjacent Structures."
Sharma, Sunil, and Lovell, C. W., "Strengths And Weaknesses Of Slope Stability."
Collison, Gary H., "Geotechnical Data Collection For Design Of The Cumberland Gap Pilot Bore."

Thirty-Fifth Annual Highway Geology Symposium - August 15-17, 1984 - California Department of Transportation, and the Department of Geology, San Jose State University - San Jose, California.

Williams, John W., "Geotechnical Setting Of The San Jose Area, California."
Sparrowe, Thomas A.; Vassil, Vasiliki B.; and Young, Douglas T., "1984 Inventory Of Foothill Landslides, Santa Clara County, California."
Cotton, William R., "Engineering Geology Of The Carmel Valley Road Rockslide, Monterey County, California."
Higgins, Jerry, "Characteristics Of Mudflows: Some Examples From The 1980 Mount St. Helens Eruptions."
Berkland, James O.; Dahlin, Alan; and Remillard, Richard, "The Congress Springs Landslide Updated."
Durgin, Phillip, "Failure By Subsurface Stormflow In Melange Terrane."
Orr, William, "Correction Of Sycamore Draw Landslide, South Of Big Sur, Monterey County, California."
Holzhausen, Gary R., "Slope Stability Monitoring In The Digital Age."
Alt, Jack, "Geologic And Seismic Considerations For Proposed Highway Bridge Sites Near Quito, Ecuador."
Griggs, Gary B., "Highway Protection And Maintenance At Waddell Bluffs, Santa Cruz County - Problems In An Active Geologic Setting."
Smith-Evernden, R. K., "Wave Erosion Of State Highway 1 Along The San Gregorio Fault Between Davenport And Pescadero, California."
West, Terry, "Detailed Office, Field, And Laboratory Analysis To Discern Rock Slope Stability, Interstate Highway 287, Northeastern New Jersey."
Chapman, K. Ronald, "Contracting For And Using Tiebacks For Landslide Stabilization."
Chen, Fred Y. M., "Geotechnical Design Parameters For Cut-And-Cover Stations And Tunnel Segments Of L. A. Metro Rail Project."
Schoeberlein, Elizabeth, and Slaff, Steven, "Overcoming Difficulties Encountered During Geotechnical Field Investigations Along Urban Transportation Corridors."

Sorensen, Mike, "Earthquake Ground Response Study For The Century Freeway, Los Angeles, California."
Hannon, Joe, and Walsh, Tom, "Final Results Of Embankment Performance At Dumbarton."

Thirty-Sixth Annual Highway Geology Symposium - May 13-15, 1985 - Indiana Department of Highways, Kentucky Transportation Cabinet, School of Civil Engineering at Purdue University - Clarksville, Indiana.

Gray, Henry H., "Outline Of The Geology Of The Louisville Region."
Mathis, Henry; Wright, Earl; and Wilson, Richard, "Subsidence Of A Highway Embankment On Karst Terrain."
Moore, Harry, "The Pellissippi Parkway Extension - Geotechnical Engineering Karst Terrain."
Drumheller, Joe C., "Exploration And Repair Of Limestone Sinkholes By Impact Densification (abs)."
Amari, Dominick and Moore, Harry, "Sinkholes And Gabions: A Solution
Killey, Myrna M. and Dumontelle, Paul B., "Illinois Landslide Inventory: A Tool For Geologists And Engineers."
Quinlan, James F., "Who Gets Sued When You Sink Or Swim, And Why: Liability For Sinkhole Development And Flooding That Affects Homes, Roads And Other Structures."
Anderson, Thomas C. and Munson, William E., "Tieback Walls Stabilize Two Kentucky Landslides."
Reeves, Ronald B. and Weatherby, David E., "Electrical Isolation Of Tieback Anchorages."
Richardson, David N., "Relative Durability of Shale - A Suggested Rating System."
Munson, William E., "Evaluation Of Geotechnical Designs For Shale
Schuhmann, Mark J. and Schmitt, Nicholas G., "Use Of New Albany Shale For Subgrade And Pavement Stabilization."
West, T. R. and Hummeldorf, R. G., "Use Of Sonic Logs In Evaluating Roof-Rock Strength For An Underground Coal Mine."
Pfalzer, William, "Wick Drains."
Bleuer, N. K., "The Nature Of Some Glacial And Manmade Sedimentary Sequences And Their Downhole Logging By Natural Gamma Ray."
Lienhart, David A. and Stransky, Terry E., "Laboratory Testing As An Aid In The Design Of Cable Anchor Systems For Rock Reinforcement."
Nwabuokei, S. O. and Lovell, C. W., "Predicting Settlements Within Compacted Embankments."
Bachus, Robert C., "The Effects Of Sample Disturbance On The Stress-Deformation Behavior Of Soft Sandstone."
Nieto, Alberto S. and Matthews, Peter K., "Moment-Driven Deformation

In Rock Slopes."

Thirty-Seventh Annual Highway Geology Symposium on "Geotechnical Aspects of Construction in Mountainous Terrain" - August 20-22, 1986 - Montana Department of Highways and Montana Division, FHWA - Helena, Montana.

Berg, Richard S., "Geology of Montana".
Jones, Walter V., and Stilley, Alan, "Geotechnical Design Considerations For Road Construction Of An Active Talus Slope."
Torbett, C. Allen, and Ryan, Patrick T., "Statistical Analyses Of Factors Related To Rock Slope Stability In Eastern Tennessee."
Wright, Earl M., and Bukovansky, Michael, "Stability Problems Of Rock Cuts, US-23 In Eastern Kentucky."
Abramson, Lee W. and Daly, William F., "Analysis And Rehabilitation Of Aging Rock Slopes."
Miller, Stanley M., "A Time-Based Model To Help Evaluate Future Stability Of Cut Slopes."
Ciarla, Massimo, "Wire Netting For Rockfall Protection."
Watters, Robert J., Karwaki, Lyn, "Rockfall Mitigation As A Function Of Cost Benefit And Probability Assessment."
Wilde, Edith M., and Bartholomew, Mervin J., "Statewide Inventory And Hazard Assessment Of Deep Seated Landslides In Montana."
Moore, Harry L., "The Construction Of A Shot-In-Place Rock Buttress For Landslide Stabilization."
Turner, A. Keith, "Application of Personal Computer Models For The Stability Analysis Of Three Land Slides Near Vail, Colorado."
Thomaz, J. E. and Lovell, C. W., "General Method For Three Dimensional Slope Stability Featuring Random Generation Of Three Dimensional Surfaces."
Cowell, Michael J., Anderson, Ron and Anderson, Bob, "Polymer Geogrid Reinforced Soil Slopes Replace Retaining Walls."
Reeves, R. Bruce, "Design And Specification Of Tied Back Walls."
Aiyer, A. Kullathu, "Performance Of Internally Reinforced Soil Retaining System."
Franceski, John A., "Roadway Stabilization Using A Tieback Wall."
Thornton, Sam I., and Elliott, Robert P., "Resilient Modulus - What Is It?"

Thornton, Sam I., and Elliott, Robert P., "Resilient Modulus - What Does It Mean?"
Schulte, Michael P., "Dynamic Pile Monitoring And Pile Load Tests In Unconsolidated Sands And Gravels, Wyoming."
Olson, Larry D. and Church, Edward O., "Survey Of Non-Destructive Wave Propagation Testing Methods For The Construction Industry."

Norris, Gary, "Evaluation Of Nonlinear Stabilized Rotational Stiffness Of Pile Groups."
Ludowise, Harry, "Refraction Seismic Study To Explore A Borrow Source In A Remote Area."
Remboldt, Michael D., "Use Of Computer Spread Sheets In Geotechnical Design And Review."

Thirty-Eighth Annual Highway Geology Symposium on "Highway Construction In Unstable Topography" May 11-13, 1987 - Pennsylvania Department Of Transportation and Engineers' Society of Western Pennsylvania - Pittsburgh, Pennsylvania.

Adams, William R., Jr., "An Empirical Model To Be Used In Evaluating The Potential For Landsliding In Allegheny County, Pennsylvania."
Brossard, Elizabeth A., and Long, Michael T., "Exploration And Analysis Of A Proposed Roadway Over Organic Soils In Western Oregon."
Young, Brian T., and Shakoor, Abdul, "Stability Of Selected Road Cuts Along The Ohio River As Influenced By Valley Stress Relief Joints."
Leech, Thomas G.; Diviney, John G.; Janik, Charles T., "Landslide Stabilization In Hilly Urban Terrain."
Ackenheil, Alfred, "Ft. Pitt Tunnel North Portal Cut Slopes Revisited."
Watts, Chester F., and Frizzel, Earl, "A Preliminary Look At Simple Back Analysis Of Rock Slope Stabilities Utilizing Micro Computers."
Newman, F. Barry and Gower, T. R., "Geological Hazards Along Dorcon Road And LR 1094."
Bruce, Donald A., Dr., and Boley, Dennis L., "New Methods Of Highway Stabilization."
West, Terry R., "Highway Construction In The Lake Bed Deposits, Southwestern Indiana."
Hamel, James V., "Geological And Geomophological Investigation For Cultural Resource Evaluations."
Olson, Larry D., Church, Edward and Wright, Clifford C., "Nondestructive Testing And Evaluation Methods For Investigating The Condition Of Deep Foundations."

Bachus, Robert C., "Lessons Learned From European Practice On The Use Of Stone Columns For Site Improvements."
Diviney, John G. "Ground Modification Of Highway Embankment Foundation By Dynamic Compaction."
Hayden, Myron; Bloomburg, D.; Upchurch, S. B.; Williams, Ronald C., "Cone-Penetrometer Exploration Of Known Sinkholes."

Sheahan, James M., "Cut Slope Design For A Major Urban Highway In The Pittsburgh Area."
Moore, Harry L., "Karst vs. Highway Ditchlines In East Tennessee."
Thornton, Sam I., Kirkpatrick, W. E., "Cures For Slope Failures In Arkansas."
Leichner, Charles H., "Anchored Solutions For Unstable Topography."
Miller, S. M. Orbach; Canavan, William; and Kochel, R. Craig, "Assessment Of Landslide Potential Along Route 3, Southern Illinois."
Bonaparte, R.; Berg, R. and Butchko, S., "The Use Of Geosynthetics To Support Roadways Over Sinkhole Prone Areas."
Stokowski, Steven J., Jr., "Ground Magnetic Studies In Appalachian Valley Karst."
Voytko, Edward P.; Scovazzo, Vincent and Cope, Neil, "Rock Slope Modification Above The North Portal Of The Mt. Washington Tunnel."
Hazen, Glenn A., and Sargand, Shad, "The Effect On Highways Of Surface Subsidence Resulting From Longwall Coal Mining."
`Wilshusen, J. Peter; Inners, Jon D.; Braun, Duane D., "Rock Slide On I-81, Northeastern Pennsylvania."
Markunas, Bernard, "Roadway Relocation Through Abandoned Municipal Dumps: A Case Study Near Hershey, Pennsylvania."

Thirty-Ninth Annual Highway Geology Symposium on "Construction To Minimize Environmental Impact" - August 17-19, 1988 - Brigham Young University, Utah Department Of Transportation, Utah Geological And Mineral Survey - Park City, Utah.

Doelling, Hellmut H., "A Brief Overview Of The Geology Of Utah."
Harty, Kimm M., "Geologic Hazards Of Utah."
Abramson, Lee W. and Hansmire, William H., "Geologic Engineering On The New Interstate H-3 In Hawaii."
Norrish, Norman I., and Lowell, Steve H., "Aesthetic And Safety Issues For Highway Slope Design."
Murtha, Geri Q.; Tiedemann, Robert B.; and Green, Richard W., "Construction Constraints - Wetlands, Runoff, Contamination."
Bruce, D. A., Dr., "Urban Engineering And The New Technologies."
Sotir, Robbin B., and Moore, William L., III, "Fill Slope Repair Using Soil Bioengineering Systems."

Curtin, Thomas J., and Tharp, Thomas M., "Stability Investigation Of Mt. Carmel Tunnel By Physical And Finite Models."
Rippere, K. H.; Williams, R. S.; and Funkhouser, M. R., "Investigation & Stabilization Of A Developing Landslide At The Intersection Of US-89 And The D&RGW Tracks East Of Thistle, Utah."
Fan, J. C., and Lovell, C. W., "The Measured Slope Steepness Factor &

Its Theoretical Analysis For Predicting Soil Erosion On Highway Slopes."
Verduin, J. R., and Lovell, C. W., "Reliability Analysis With PCSTABL5M."
Rana, G. M.; Smith, Jim; and Irani, Khodi, "Ground Water Influence On Highway Fill Slope Stability."
Moore, Harry L., "Oriented Pre-Split For Controlling Rock Slides."
Coffin, James L., "Installation Of An Underdrain System For Slope Stability."
Karably, K. B., and Humphries, R. W., "Talus Slope Stability Using Tie Back Anchors In Provo Canyon, Utah."
Leonard, Matthew; Plum, Robert L.; and Kilian, Al, "Considerations Affecting The Choice of Nailed Slopes As A Means Of Slope Stabilization."
Against Rockfall & Debris Flow & All Other Purposes Of Protection."
Mitchell, David A., "4000 Bridge Foundation Investigations."
Capaul, W.; Wyllie, D.; Dunsmore, R.; Smith, R.; Winger, J.; Paroni, A.; Draeger, J., and Perfect, J., "Construction Of A Tied Back Soldier Pile Rock Retaining Wall Along I-90 In Northern Idaho, A Case History."
West, Terry R., "Construction Of A New Interchange For The Indiana Toll Road, Complicated By Poor Soil Conditions And Presence Of Sanitary Land Fill, Gary, Indiana."
Pihl, Roger, and Bowen, Tim, "Design And Construction Methodology For
Miller, Stanley M., "Modeling Shear Strength At Low Normal Stresses For Enhanced Rock Slope Engineering."
Burk, Robert L., and Moser, Kenneth R., "Spirit Lake Memorial Highway-Geologic Investigations In A Zone Of Natural Aesthetic Change."
Levine, Edward N.; Genson, Gordon, R.; Dye, Ronald R., and Slifer, James C., "From Geophysics To Design In An Environmentally Sensitive Area."
Scott, James H.; Burdick Richard G.; and Ludowise, Harry, "Interpretation Of Seismic Refraction Data On A Microcomputer."
Thronton, Sam I., and Elliott, Robert P., "Rapid Shear As An Evaluation For Base Course Material."

Fortieth Annual Highway Geology Symposium on "Symposium Venue" - May 17-19, 1989 - Alabama Highway Department - Birmingham, Alabama.

Bearden, Bennett L., "General Overview Of The Geology And Natural Resources Of Alabama."
Walker, Thomas H., "Engineering Geology And Geotechnical Engineering For A Preliminary Route Alignment Study For 25 Miles Of Arizona State Route 87."
Huang, Wei-Hsing, and Lovell, C. William, "Suitability Of Bottom Ash For Indiana Highway Construction."

Cochrane Bridge Embankment."
Achilleos, E., and Lovell, C. William, "Update on STABL...PCSTABL5M."
West, Terry R., and Gordon, Quentin A., "Demolition And Removal Of Structures Prior To Land Reclamation."
Burns, Scott F.; Hadley, William O.; Mutchler, Jack W.; Smith, Schaun M.; and Griffin, Paul M., Jr., "Slope Failures On Highway Embankments High In Shrink-Swill Clays: Prevention And Repair."
Nelson, K. Jeff, and Selvage, John R., "Design Applications Of The Welded Wire Wall."
Sharma, Sunil, "Integrated Slope Stability Analysis Using Microcomputers."
Morales, Carlo Hugo Rivera, "Honduras Highway Geology."
Wolosick, John R., "Contract Specification Options For Retaining Wall Design And Construction: Discussion Of Construction Alternates and Case Histories."
Sharma, Sunil, and Hardcastle, James H., "Finite Element Analysis Of A Rib-Reinforced Steel Culvert."
Thommen, Robert A., Jr., "First Wire Rope Net Rockfall Protective Barrier Installed At The Grand Canyon National Park."
Wright, E. M., "Special Treatment Of Mine Openings In Rock Cut Slopes."
Humphries, Richard, and Sullivan, Randy, "Recent Highway Tunnel Projects In The Appalachian Mountains."

Forty-First Annual Highway Geology Symposium - August 15-17, 1990 - New Mexico State Highway and Transportation Department and New Mexico State University Department of Civil, Agricultural and Geological Engineering - Albuquerque, New Mexico.

Haneberg, William, "Geologic Hazards Of New Mexico."
Collins, Donley, and Swolfs, Henri, "Highway Damage Related To A Fault Near Pierre, South Dakota."
Barnes, Jamie, "Seismic Record Versus Geologic Record In The Southern Rio Grande Rift Region."
Moore, Harry, "Rockfall Mitigation Along I-40, Cocke And Cumberland Counties, Tennessee."
Watters, Robert, and Rehwoldt, Eric, "Slope Distress And Rock Fall Induced By The Presence Of Old Underground Excavations."
Restraining Nets."
Elliott, Gordon, and Rippere, Kenneth, "Performance Analysis In Rockfall Simulation."
Rector, Edward, and Lueck, Richard, "Design Of Geogrid Wall With Wick Drains In Tucumcari, New Mexico."
Thornton, Sam, and McGuire, Michael, "Geogrid-Expansive Clay

Embankment."
Cross, Richard, "Creating An Elevated Catchment Area Using A Precast Modular Wall System."
Deardorff, George, and Findley, David, "17 Miles To Mount St. Helens: Operational Aspects Of The Geotechnical Investigation."
Humphries, Richard; Elliott, Gordon; Cafarelli, Gerald; Hollenbaugh, John; and Geiger, Eugene, "Analysis And Design Of Tieback Wall No. 5 In Steubenville, Ohio."
Nicholson, Peter, and Johnston, Spark, "Alternative Methods For Retaining Walls."
Neel, Thomas, "T-Wall - Engineered For Economy."
Thornton, Sam, and Garret, Steven, "Slope Failure Probability For Layered Soils."
Chang, Chien Tan, "Federal Highway Administration's Technology Transfer Activities In Geotechnical Engineering."
Huddleson, Steven, "Data Acquisition System For Mechanical Dutch Cone Penetrometer."
Wisner, William, "Florida's Mineral Aggregate Control Program."

Forty-Second Annual Highway Geology Symposium on "Geologic Complexities in the Highway Environment" - May 28-31, 1991 - American Society of Civil Engineers, American Institute of Professional Geologists, New York State Department of Transportation, New York State Geological Survey - Albany, New York.

Bydlon, B., "Geologic Innovations on the Pennsylvania Turnpike."
Rudenko, D; Ackerman, H; and Lorence, W., "Seismic Refraction Technique Applied to Highway Design in a Strip Mined Area of Southwestern Pennsylvania."
Decker, M., and Jacobsen, G., "Evaluation of Acid Leachate Potential in Highway Construction."
Stokowski, S., "Quarry Layers - Stratigraphic Units that Serve the Public Interest."
Fischer, J., and Greene, R., "Roadways in Karst Terrane."
Mellett, J., and Maccarillo, B., "Highway Construction in Karst Terranes: Avoiding and Remediating Collapse Features."
Hardy, H., "Application of Non-destructive Testing Techniques to Slope Stability and Sinkhole Monitoring."
Brandon, S., "Rock Slope Excavation and Stabilization Methods in Highway Construction: Interstate 287 Extension, New Jersey."
Hale, L. L., III, and Gansfuss, J., "Rock Slope Investigations at Selected Hudson Valley Sites."
Burke, J., and LeFevre, S., "Rock Slope Inventory, Evaluation and Remediation for Sections Along the NYS Thruway."

Cross, R., "A Design for A Temporary Reusable Rock Catchment Barrier."
Bolton, C., "Evolution of Rock Excavation and Stabilization in New York State."
Abramson, L., "Complex Geology at Complex Sites."
Dunn, J; Banino, G; and LaGrand, D., "Treated Aggregate in an Asphaltic Concrete Road: An Apparent Success."
Hudec, P., and Achampong, F., "Improving Aggregate Quality by Chemical Treatment."
Parola, A., and Hagerty, D., "Highway Bridge Failure by Foundation Scour and Instability."
Butch, G., "Measurement of Scour at Selected Bridges in NY."
Horne, W.; Stevens, D.; and Batson, G., "Ground Penetration Radar Study of Riverbed Scour in NYS."
McGuffey, V., "Clues to Landslide Identification and Investigation."
Thornton, S., and Garrett, S., "Slope Failure Probability for Mixed Layer Soils."
Baskerville, C., "Northern New England Landslides."

Forty-Third Annual Highway Geology Symposium - August 19-21, 1992 - University of Arkansas Dept. of Civil Engineering, Arkansas State Highway and Transportation Department and Arkansas Geological Commission - Fayetteville, Arkansas.

Lin, P. S., and Lovell, C. W., "The Critical State of Debris Flow."
McManis, Kenneth , and Nataraj, Mysore, "The Influence of Post Depositional Effects on the Engineering Properties of a Marine Clay."
Stone, Charles G., "Overview of the Complex Highway Geology in West-Central Arkansas."
O'Hara, Kevin C., and West, T. R., "Evaluation of Coal Refuse For Access Road Construction at an Abandoned Mine Lands Site, Southwest Indiana."
McFarland, John D., "Landslides on Crowley's Ridge."

Annable, Jonathan and Sharum, John, "Slope Failures on Highway 71 Relocation Projects."
Munoz, Andy, Jr., "Slope Maintenance and Slide Restoration."
Graham, J. R.; Ingraham, P. C.; and Humphries, R. W., "Repairs to Rock Slopes at the US-22/SR-7 Interchange in Steubenville, Ohio."
Thornton, Sam I., and McGuire, Michael S., "Cannon Creek Embankment Instrumentation."
Lumbert, David W., and Thian, Boon K., "Design, Construction and Monitoring of a 76 Foot High Geogrid Reinforced Earth Embankment."

Moore, Harry, "The Use of Geomembranes for Mitigation of Pyritic Rock."
Lumbert, David W., and Stone, Charles G., "Highway Geology at Selected Sites in the Boston Mountains and Arkansas Valley, Northwest Arkansas."
Yarnell, Charles N., "1992 Update: Steel Wire Rope Safety Net Systems Application to Rockfall Mitigation in the U. S."
Macintosh, O. R., "Value Engineering Proposal Using Hilfiker Welded Wirewall for Soda Springs Highway Near Auburn, California."
Austin, Alvin L., and Annable, Jonathan A., "In-situ Moisture Content of Arkansas Subgrades."
Henthorne, Robert; Jones, William; and Rockers, Larry, "Investigation and Remediation of Undermined Highway."
Darrag, A. Amr; Lovell, C. W.; and Karim, A. M. K., "New Correlations for Piles Driven Into Cohesionless Soils."
Selvam, R. Panneer, and Elliott, Robert P., "Nonlinear Finite Element Analysis of Pavements Using Microcomputer."
Thornton, Sam I., and Ford, Miller C., Jr., "Aggregate Suitability (Stripping) for Asphalt Pavements."
Ruppen, Christopher A., and Baker, Michael, Jr., "When Does the Work End?"
Abstract Only

Forty-Fourth Annual Highway Geology Symposium - May 19-21, 1993 - University of South Florida Dept. of Civil Engineering and Mechanics and Florida Department of Transportation - Tampa, Florida.

Goddard, Robert E., P.G. and Wisner, William A., Jr., P.G., "Use of Aggregate in the Sunshine Skyway Bridge Project."
Khalid, Jamil, and Thornton, Sam I., "Resilient Modulus vs Strength in Cement Stabilized Base Courses."
Javed, S., and Lovell, C. W., "Use of Waste Foundry Sand in Highway Construction."
Zayed, Abla M., "Effect of Fly Ash Quality on Concrete Durability."
Spencer, Steven M., P.G., "A Roadway Problem in a Cavernous Karst Environment at the Florida Caverns State Park."
Foshee, Jon, and Bixler, Brian, "Cover-Subsidence Sinkhole Evaluation State Road 434 - Longwood, Florida."
Yovaish, Douglas J., P.E., and Law, Donald S., P.E., "Technical Related Analysis, Design, and Construction 4 Lane Highway Over 8 to 20 Feet of Peat."
Marienfeld, Mark, P.E., "Highway Reconstruction Over an Expansive Subgrade Incorportating a High Strength Geosynthetic Moisture Barrier."
Pittenger, Robert A., and West, Terry R., "Investigation for Landfill

Expansion in a Bedrock Area, Southcentral Indiana."
Behringer, David W., and Shakoor, Abdul, "A Study of Selected Landslides Along Cincinnati Roadways."
Ross, Mark A., and Vincent, Mark S., "Numerical Modeling of Sediment Erosion at Tidal Inlet Bridges."
Goddard, R. E., "44th Annual Field Trip Guide Unique Construction Projects and Problems in the Tampa Bay Area of Florida."
Carrier, III, W. David, "Mechanical Properties of Lunar Soils and Implications For a Lunar Base."
Sokol, Thomas P., P.E., and McCahan, Matthew L., "Pennsylvania Turnpike Expansion: A Reprospect."
Ahmed, Imtiaz, and Lovell, C. W., "Laboratory Study on Properties of Rubber-Soils."
Schneider, Nicholas P., and Bauer, Robert A., "Environmental Property Assessments for Highway Projects: Key Elements for Successful
Madrid, Larry D., P.E., and Smith, Ted J., P.G., "Construction of a Four-Lane Highway Embankment Over a Contaminated Landfill." (Not Published)
Kasim, Margaret F., and Shakoor, Abdul, "Predicting the Compressive Strength of Rocks From Aggregate Degradation."
Chen, W. Y.; Lovell, C. W.; Pyrak-Nolte, L. J; and Haley, G. M., "New Precursor of Stick-Slip Movement of Rock Block."
Marcozzi, Guy F., P.E., "Cement Amended Fly Ash as a Structural Fill." Abstract Only

Forty-Fifth Annual Highway Geology Symposium - August 17-19, 1994 - Portland State University, Oregon Department of Transportation, Washington Department of Transportation, Transportation Research Board - Portland, Oregon.

Koelling, Mark, "Ground Improvement Case Histories for Highway Construction."
Johnston, Robert E., and Abramson, Lee W., "Geosynthetic Reinforcing of Highway Embankments."
Bailey, Joe, "Application of MSE (Mechanically Stabilized Earth) Slopes as Replacement for Retaining Walls; a New Hampshire Case History."
Kuhne, J. C., and Glass, F. R., "Probabilistic Analysis and Design of Slopes on Interstate 26 From Asheville, NC to Tennessee."
Shaw, Lee R.; Toh, Chin Leong; and Thornton, Sam I., "Effects of Lime on Cannon Creek Embankment Soil."
Armour, Tom A.; Johnston, Mark S; and Groneck, Paul B., "Innovative Rock Anchoring at Boundry Dam, Washington."
Badger, Tom, "Rock Excavations in Steep Ground."
Flatland, Robert; Watters, Robert J.; and Cochrane, David, "Application of the Rockfall Hazard Rating System to Rock Cuts in Mountainous Terrain."

Thommen, Robert A., and Montejo, Manuel A., "Rock Retaining Wire Rope Net System Installation at Orizaba, Mexico."
Hannan, Richard; Moran, Michael; and Scofield, David, "Design, Construction, and Performance of Horizontal Drains at Bonneville Navigation Lock, Oregon."
Humphries, Rich, "Recent Geotechnical Advances in the Design and Construction of Highway Tunnels."
Huddleson, Steven M., and Lovely, Anne, Esq., "Hazardous Materials in the Roadway."
Javed, S.; Lovell, C. W.; and Eastwood, D. A.; "Waste Foundry Sand in Subgrade and Controlled Low Strength Material."
Burk, Robert L.; Norrish, Norm; and Lowell, S. M., "Investigation, Design, and Construction of the Spirit Lake Memorial Highway, Washington."
Dahill, Jim, "Investigation and Stabilization of a Developing Landslide, Highway 28, South of Lander, Wyoming."
Hager, G. Michael, and Falk, Mark, "Catastrophic Embankment Failure South of Sheridan, Wyoming, Interstate 90 - M. P. 41.1."
Kane, William F., and Beck, Timothy J., "Development of a Time Domain Reflectometry System to Monitor Landslide Activity."
Kobernik, Ricki M.; Toor, Frank N.; and Watanabe, Richard, "The Arizona Inn Landslide, Curry County, Oregon."
Peterson, Gary L; Squier, L. Radley; and Scofield, David H., "Engineering Geology and Hydrology Update, Arizona Inn Landslide, Curry County, Oregon."
Bruner, D. W.; Choi, J. C.; and West, T. R., "Evaluation of Indiana Aggregates for Use in Bituminous Highway Overlays."
Fisk, Lanny H., and Spencer, Lee A., "Highway Construction Projects Have Legal Mandates Requiring Protection of Paleontologic Resources (Fossils)."
Gaffney, Donald V., "BV-116: The Bridge by Addendum: Wetlands Dictate a Change in Design."
Selvam, R. Panneer; Elliott, Robert P.; and Arounpradith, A., "Pavement Analysis Using ARKPAV."
Sherman, William F., "The Impact of Registration on Geologists on the Professional Involved in Highway Geology."

Forty-Sixth Annual Highway Geology Symposium - May 14-18, 1995 - West Virginia Geological and Economic Survey, West Virginia Division of Highways, West Virginia University, Transportation Research Board - Charleston, West Virginia.

Kroenke, Mark A., and Shakoor, Abdul, "A Geotechnical Investigation of the Chagrin River Road Landslide Complex in the Moreland Hills

Area, Cuyahoga County, Ohio."
Reed, Benjamin C; McConnell, W. T.; and Mullen, D. M., "Potential for Slope Instability and Acidic Runoff Along a Section of the U. S. 19 Corridor; Cherokee, Graham, and Swain Counties, North Carolina."
Hamel, James V., and Hamel, Elizabeth A., "Landsliding in Pennsylvania."
Martin, A. David, "Maryland Route 31 Sinkhole."
Fischer, Joseph A., and Fischer, Joseph J., "Remediation for Highways in Karst."
Stephenson, J. Brad; Beck, Dr. Barry F., P. E.; Smoot, Dr. James L. and Turpin, Anne, "Management of the Discharge and Quality of Highway Stormwater Runoff in Karst Areas to Control Impacts to Ground Water - A Progress Report."
Hager, G. Michael, and Falk, Mark, "Landslide Repair Utilizing A Steepened Geogrid Reinforced Slope, Interstate 90, M. P. 28.9, Sheridan County, Wyoming."
Singh, Yash P.; Cavan, Bruce P.; Rhodes, Gary W., "Design of Rock Reinforcement for Tallulah Gorge Bridge Foundations."
Bigger, Joseph C., "Rock Fall Mitigation Program at Nantahala Dam Using Wire Rope Nets in Conjunction with Other Techniques."

Seider, Gary L., and Smith, Walter, "Helical Tieback Anchors Help Reconstruct Failed Sheet Pile Wall."
Ludwig, Harald P., "Permanent Highway and Landslide Walls."
Hoffman, A. G.; Melick, D. F.; Clark, D. M.; Bechtel, T. D.; "Abandoned Deep Mine Subsidence Investigation, and Remedial Design, I-70, Guernsey County, Ohio."
Toh, Chin Leong, and Thornton, Sam, "Limestone Base Course Permiability."
Lovell, C. W.; Bernal, Andres; and Salgado, Rodrigo, "Uses of Scrap Rubber Tires in Civil Engineering."
West, Terry R., "Evolution of A Technique: Petrography of Aggregates for Concrete and Bituminous Highway Pavements."
Johnston, R. Michael, and Vierling, Michael P., "A Modified Presplit Blasting Method for Use in Environmentally Sensitive Areas."
Hall, Kevin D., and Watkins, Quintin B., "Effect of Soil Horizonation on Flexible Pavement Responses."
Brown, Christopher L., and Shakoor, Abdul, "Predicting the Unconfined Compressive Strength of Carbonate Rocks from Los Angeles Abrasion Test Data."
Ruppen, Christopher A., and Eliot, Gordon M., "Mitigating Landslide/Rockfall Hazards - A Quantitative and Qualitative Evaluation of Risk Reduction."

Forty-Seventh Annual Highway Geology Symposium - September 6-9, 1996 -
Wyoming Department of Transportation, Wyoming Geological Survey,
Cody, Wyoming.

Burns, Scott, "Earthflows, Debris Flows and Roads: The Aftermath of the Winter Storm of 1996, Portland, Oregon."
Fickies, Robert, "Landslide Hazards in New York State: A Geological Overview."
Adams, Wayne, and Findley, Dave, "Peters Road Landslide."
Willems, Henry and Podniesinski, Matthew, "Rock Slope Stabilization at Dunderberg Mt., Rockland County, New York."
Priznar, Nick, and Euge, Kenneth, "Geologic Investigation of the Gordon Canyon Slide."
Turner, J. P.; Haskenkamp, R. N.; and Edgar, T. V., "Reinforced Earth Retaining Wall for Landslide Control, Snake River Canyon, Wyoming."
Wyllie, Duncan, and Kielhorn, W., "Wedge Stability Analysis - Geometric and Groundwater Enhancements."
Duskin, Priscilla, "Seismic Refraction as a Method for Determining Thickness of Organic Sediments."
Beck, Timothy J.; Kane, William F., "Current and Potential Uses of Time Domain Reflectometry for Geotechnical Monitoring."
Boundy, Bret, and Dahill, James, "Instrumentation of a Shredded Tire Fill Used for a Landslide Repair, The Burning Issue."
Gaffney, Donald V.; McCahan, Matthew L., "Integrated Geohazard Management - A Systemwide Approach."
Adams, Jr., William R., and Ruppen, Christopher, "SGH Form: Format for Early Collection of Essential Geotechnical Data."
Heinert, Kevin, "Permanent and Temporary Earth Anchoring for Highway Applications."
Ludwig, Claus, "Making a Soil Nail Wall Look Like Rock."
Moore, Harry, "The Use of Underbenching in Embankment Construction Through Mountainous Terrain, I-26, Unicoi County, Tennessee."
West, Terry, and Park, Hyuck Jin, "Rock Durability of Argillaceous Carbonate Rocks in Cut Slopes for Indiana Highways."
Lovell, C. W., "Flowable Fill Using Waste Products."
Studebaker, Irving; Patton, Susan; Studebaker, Raymond, "Subsidence Definition and Effects on Surface Construction."
Lundvall, J. F.; Turner, J. P.; Stewart, J. M.; Edgar, T. V., "Roadway Settlement Over Culverts: Causes and Cures."
Duffy, John, and DeNatale, Jay, "Debris Flow Mitigation Using Flexible Barriers."
Hall, Kevin, "Preliminary Investigations Using ALROC Potliner Sand in Asphalt Concrete Mixtures."

Forty-Eighth Annual Highway Geology Symposium - May 8-10, 1997 - Tennessee Department of Transportation, University of Tennessee: Department of Geological Sciences Institute for Geotechnology, Department of Civil Engineering, Tennessee Division of Geology Transportation Research Board, Knoxville, Tennessee.

Hager, G. Michael, and Huntington, George S., "Internal Sulfate Attack: Wyoming's Expansive Experience."
Fischer, Joseph A., and Fischer, Joseph J., "Investigation and Remediation of Service Plaza Fuel Storage Tanks in Karst."
Stephenson, J. Brad; Zhou, W. F.; Beck, Barry F.; Green, Tom S.;
Smoot, James L.; and Turpin, Anne M., "Management of Highway Stormwater Runoff Karst Areas-Baseline Monitoring and Design of a Treatment System for a Sinkhole at the I-40/I-640 Interchange in Eastern Knoxville, Tennessee."
Marks, B. Dan, "Stabilization of Karst Features on Major Engineering Projects."
Sebela, Dr. Stanka; Slabe, Dr. Tadej; Kogovsek, mag. Janja, "Motorways in Karst of Slovenia."
West, Terry R., and Pittenger, Robert, "Computer Analysis of Required Statistical Parameters for Sites with Multiple Monitoring Wells Measured for Extended Periods of Time."
Karim, A., Salgado, Rodrigo and Lovell, C. W. "Bill", "Building Embankments of Coal Combustion Fly Ash-Bottom Ash Mixtures."
Ruegsegger, L. Rick, and Lefchik, Thomas E., "Abandoned Underground Mine Inventory and Risk Assessment."
Bruhn, Robert W., Roth, Bruce L., and Chelednik, Craig, "Physical Distress Evaluation of the West Abutment of the SR-22 Bridge over the Conemaugh River."
Shakoor, Abdul, and Chu, Chein-Jen, "Physical and Chemical Evaluation of Soil-Tire Mixtures for Highway Applications."
Szturo, John F., and Duryee, Wayne, "Case Study-Drilled Shaft Foundations-Lake of the Ozarks Community Bridge."
Adkins, Grant, and Rutkowski, Nanette, "Resistivity Field Testing of Reinforced Earth Structures."
Hall, Kevin D., "Potential Effects of Superpave Implementation on the Arkansas Aggregate Industry."
Martin, A. David, "The Landslide at Waverly Street and Maryland Route 36: A Case History."
Shakoor, Abdul, and Kroenke, Mark A., "Analysis and Remediation of the Miles Road Landslide Complex, Cuyahoga County, Ohio."
Wayne, Mark H., and Wokasien, Sean, "A Case History on the Use of Bioengineering in Mechanically Stabilized Earth Slope Design."
Wright, E. M., "State of the Art for Rock Cut Slopes in Eastern Kentucky."
Arwood, Scott; Mauldon, Matthew; and Moore, Harry, "Rock Slope Stability Analysis Based on Potential Energy."
Siebert, Harry L., "The Role of a Contractor's Blasting Consultant-A Case Study."
Cross, Richard H., "Elevated Catchment Areas: A Performance Report."
Lambe, Philip C., and Sweitzer, Margaret, "Selecting Shear Strength Parameters for Weathered Metamorphic Rock."
Beard, Jerome, "Parameters Used for Design of I-26 in North Carolina."

Haston, Matt; Mauldon, Matthew; and Byerly, Don W., "Fractal Geometry for the Quantification of Rock Joint Roughness."

Forty-Ninth Annual Highway Geology Symposium - September 10-14, 1998 - Arizona Department of Transportation, Northern Arizona University: Department of Geological Sciences, Arizona Geological Survey, Prescott, Arizona.

Bieniawski, Richard Z.T., "Design Principles and Methodology for Excavations in Geologic Media."
Scovira, Scott D.; Barber, Michael F., "Final Rock Slope Blasting Techniques: Considerations in Design and Execution."
Brawner, C.O., "Case Examples for Rock Stability on Transportation Projects."
Peterson, David E.; Dahlen, Keith H. "Rock slopes and bridge foundations in a foliated rock mass along portions of State Route 69 near Mayer, Arizona."
Beck, Timothy J.; Kane, William F. "Site Characterization and Slope Stability Evaluation for the San Francisco-Oakland Bay Bridge."
Glass, F.R. "A Large Wedge Failure Along Interstate 40 at the North Carolina-Tennessee State Line."
Duffy, John D.,"Performance Review of Rockfall Mitigation Measures."
Moore, Harry, "Rockfall Control Fence Unaka Springs Road Unicoi County."
Vierling, Michael P., "The Performance of Rockfall Mitigation Measures in New York State."
Gattis, J.L.; Thornton, Sam I. "Tunnel to the 21st Century."

Humphries, Richard, "Tunnel Enlargement and Rehabilitation."
White, Jonathan L. "High Altitude Slope Reconstruction and Restoration on Colorado State Highway 82 at Independence Pass-A Continuing Case Study."
Cole, Kenneth A. "January, 1997 Storm Damage to Highway 50, El Dorado County, California."
Falk, Mark, Hager Michael, "Wolf Mountain Landslide Milepost 136.57 Snake River Canyon, Teton County, Wyoming."
Turner, John; Wolosick, John; Hasenkamp, Robert, "Construction and Performance of Reticulated Micropile Retaining Walls for Slope Stabilization, Snake River Canyon, Wyoming."
Schock, Robert E., "Three Interesting Drill Shaft Foundations Completed in Arizona."
Wright, Michael J., "Statnamic and Static Lateral Load Analysis of a Battered Pile Group vs. a Drilled Shaft Group."
Lowell, Steve M., Thommen, Robert A. II, "Testing and Application of Flexible Wire Rope Netting Barriers to Retain Mud & Debris Flow."
Siel, Barry D.; Ueblacker, Gernot; Siebert, Harry L., "Soil Nailing-Geology and Construction Considerations."
O.R. MacIntosh, "Call of the Canyon Mechanically Stabilized Earth Structure with Simulated Rock Facing."
Taylor, Gary E.; Gorski, Glen A; Alexander, Douglas E., "Compaction Grouting of a Deep Embankment Fill Along Interstate I-15 Virgin River Gorge, Arizona."
Pearthree, Philip A., "Paleoseismology and Seismic Hazard Assessment in Arizona"
Hatheway, Allen W., "Former Manufactured Gas Plants; Nemesis of Transportation Planning"

Lindberg, Paul A., "Structural Controls of Devils Kitchen Sinkhole; Sedona, Arizona."
Henthorn, Robert W., "Geolgic Investigation and Construction of the Spring River Bridge."
Slabe, Tadej, "Caves Discovered During The Construction of a Motorway Over Slovene Karst."
Fischer, Joseph A.; Fischer, Joseph J.; Ottoson, Richard S., "Geotechnical Investigation in Karst: Bent Rock Versus Flat Rock."
Benson, Richard C.; Kaufmann, Ronald D.; Yuhr, Lynn B., "Assessment, Prediction and Remediation of Karst Conditions on I-70, Frederick, Maryland."
Euge, Kenneth M. Hennon, Kerry; Williams, Jason C.; Priznar, Nick, "Ground Penetrating Radar: An Effective Tool for Locating Dry Caves Along a Portion of State Route 69 Near Mayer, Yavapai County, Arizona."
Qubain, Bashar S.; Seksinsky Eric J.; Li, Jianchao, "Design Considerations for Bridge Foundations in Karst Terrain."
Meier, W.R. Jr.; Rosner, John C., "Prospecting for Local Material Pits."
Stokowski, Steve, "Sequential Deterioration of Concrete, Congress Street Bridge, Boston, Massachusetts."
Mather, Bryant, "How to Avoid Excessive Expansion of Concrete Due to Alkali-Aggregate Reaction."
Duffy, Dennis, "Runaway Truck Ramp Testing Program"
West, T.R., "Evaluation of Crushed Stone Used for High Traffic, Bituminous Pavement Wearing-Courses in Indiana"
Chen, Dar-Hao; Bilyeu, John; Murphy, Mike, "The Effect of Bedrock Depth on pavement Response."
Paine, Jeffrey G., "Influence of Bedrock type and Depth on Falling-Weight Deflectometer Data."
Rucker, Michael L., "Percolation Theory approach to Estimating Earthwork Factors in Weathered Granites."
Beard Jerome, "400,000 Cubic Yard Slope Failure on I-26."
Wilburn, David; Goonan, Thomas, "Recycled Aggregate in Road Construction-An Economic Assessment."
Kelly, Thomas, "The Recycling of Construction Aggregates A Materials flow Analysis."

Appendix B:
HGS Proceedings Availability List

HIGHWAY GEOLOGY SYMPOSIUM PROCEEDINGS
- Availability List -

Symposium/Date	Sponsor	Availability	Price
First (1950)			
Second (1951)			
Third (1952)		The First 10 Symposia "Proceedings" are contained in a 3 volume set.	
Fourth (1953)			
Fifth (1954)			
Sixth (1955)		A Limited Number of These Volumes are available at $20.00 a set.	
Seventh (1956)			
Eighth (1957)			
Ninth (1958)			
Tenth (1959)			
Eleventh (1960)	Florida State Univ. Tallahassee, Florida	Out of Print	
Twelfth (1961)	Univ. of Tennessee Knoxville, Tennessee	Out of Print	
Thirteenth (1962)	Phoenix, Arizona	Out of Print	
Fourteenth (1963)	Texas Highway Dept. College Station, Texas	Out of Print	
Fifteenth (1964)	Missouri Geol. Survey Rolla, Missouri	Out of Print	

Symposium/Date	Sponsor	Availability	Price
Sixteenth (1965)	Univ. of Kentucky Lexington, Kentucky	Out of Print	
Seventeenth (1966)	Iowa State Univ. Iowa State Hwy. Comm. Ames, Iowa	F. R. Glass*	$6.00
Eighteenth (1967)	Purdue University and Indiana Highway Commission	Professor Terry West Civil Engineering Dept. Purdue University West LaFayette, Indiana	
Nineteenth (1968)	West Virginia Geol. and Economic Survey	W. Virginia Geological and Economic Survey P. O. Box 879 Morganton, W. Va. 26505 (Circular 10)	$20.00
Twentieth (1969)	Univ. of Illinois	Out of Print	
Twenty-First (1970)	State Highway Commission of Kansas	Out of Print	
Twenty-Second (1971)	Oklahoma Geol. Survey and Oklahoma Dept. of Highways	Oklahoma Geological Survey University of Oklahoma Norman, Oklahoma	
Twenty-Third (1972)	Virginia Hwy. Research Council, Virginia Dept. of Highways	Highway Geologist Virginia Dept. of Highways Richmond, Virginia	
Twenty-Fourth (1973)	Wyoming State Highway Department	Out of Print	
Twenty-Fifth (1974)	North Carolina Dept. of Transportation	Out of Print	

Symposium/Date	Sponsor	Availability	Price
Twenty-Sixth (1975)	Idaho Transportation Department	R. G. Charboneau Chief Geologist Idaho Dept. of Highways P. O. Box 7129 Boise, Idaho 83707	
Twenty-Seventh (1976)	Florida Department of Transportation and Univ. of Florida	F. R. Glass*	$20.00
Twenty-Eighth (1977)	South Dakota School of Mines	Out of Print	
Twenty-Ninth (1978)	Maryland State Hwy. Administration and Maryland Geological Survey	Publications Dept. Maryland Geol. Survey 711 W. 40th Street Suite 440 Baltimore, Maryland 21314	$20.00
Thirtieth (1979)	FHWA at Portland, Oregon	F. R. Glass*	$20.00
Thirty-First (1980)	Bureau of Econ. Geol. Texas Dept. of Highways	F. R. Glass*	$20.00
Thirty-Second (1981)	Tennessee Department of Transportation	F. R. Glass*	$20.00
Thirty-Third (1982)	Colorado Dept. of Highways Colorado Geol. Survey USDA Forest Service	Colorado Geol. Surv. Dept. of Nat. Resources 1313 Sherman Street Denver, Colorado 80203	$20.00
Thirty-Fourth (1983)	Georgia Department of Transportation	F. R. Glass*	$20.00
Thirty-Fifth (1984)	California Dept. of Transportation San Jose State Univ.	F. R. Glass*	$20.00

Symposium/Date	Sponsor	Availability	Price
Thirty-Sixth (1985)	Kentucky Department of Transportation School of Civil Eng. Purdue University	F. R. Glass*	$20.00
Thirty-Seventh (1986)	Montana Department of Highways Montana Div. - FHWA	F. R. Glass*	$20.00
Thirty-Eighth (1987)	Pennsylvania Dept. of Transportation Engineer's Society of Western Pennsylvania	F. R. Glass*	$20.00
Thirty-Ninth (1988)	Utah Dept. of Trans. Utah Geological and Mineral Survey Brigham Young University	F. R. Glass*	$20.00
Fortieth (1989)	Alabama Highway Department	F. R. Glass*	$20.00
Forty-First (1990)	New Mexico State Hwy. and Transportation Dept. & New Mexico State Univ. Department of Civil, Agricultural and Geological Engineering	F. R. Glass*	$20.00
Forty-Second (1991)	American Society of Civil Engineers, American Inst. of Professional Geologists, NYSDOT, New York State Geological Survey	F. R. Glass*	$20.00

Symposium/Date	Sponsor	Availability	Price
Forty-Third (1992)	Univ. of Arkansas Dept. of Civil Engineering, Arkansas State Highway and Transportation Department, Arkansas Geological Commission	F. R. Glass*	$20.00
Forty-Fourth (1993)	Univ. of South Florida Dept. of Civil Engineering Florida Dept. of Transportation	F. R. Glass*	$20.00
Forty-Fifth (1994)	Portland State Univ. Oregon Dept. of Transportation Washington Dept. of Transportation Transportation Research Board	F. R. Glass*	$20.00
Forty-Sixth (1995)	West Virginia Geological and Economic Survey West Virginia Div. of Highways West Virginia University Transportation Research Board	F. R. Glass*	$20.00
Forty-Seventh (1996)	Wyoming Dept. of Trans. Wyoming Geological Survey	F. R. Glass*	$20.00
Forty-Eighth (1997)	Tenn. Dept. of Transportation Univ. of Tennessee: Dept. of Geological Sciences Institute for Geotechnology, Dept. of Civil Eng., Tenn. Division of Geology, Trans. Research Board	F. R. Glass*	$20.00

PLEASE NOTE: The price listed does not include the charge for postage and handling. These charges will be added to the price listed. Copies of papers are $5.00 each plus postage and handling.

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