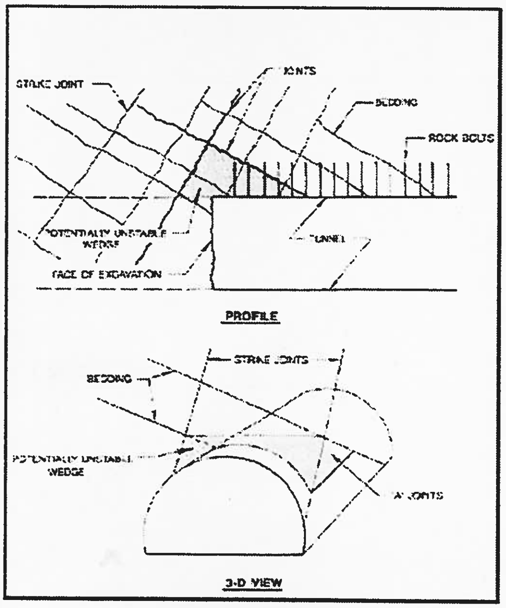
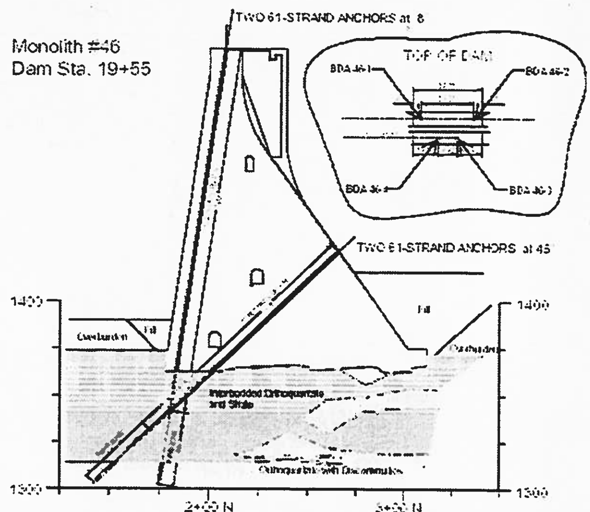
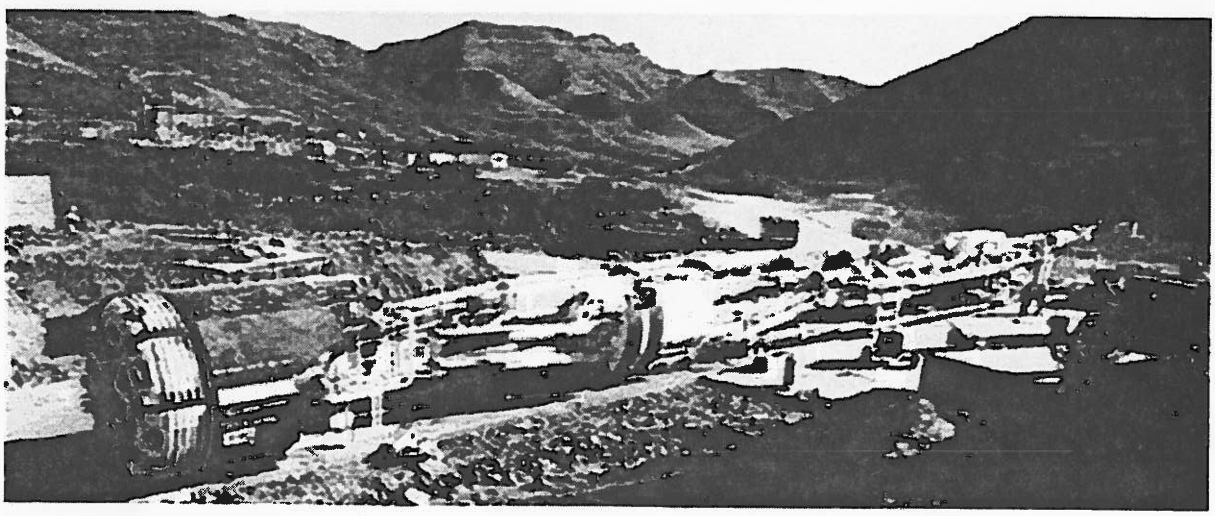
ORVSS XXXV

OHIO RIVER VALLEY SOILS SEMINAR

ROCK ENGINEERING AND TUNNELING







October 20th, 2004
Louisville, Kentucky

PROCEEDINGS OF THE THIRTY-FIFTH OHIO RIVER VALLEY SOILS SEMINAR

ROCK ENGINEERING AND TUNNELING

October 20th, 2004
Galt House
Louisville, Kentucky

Sponsored by

Kentucky Geotechnical Engineering Group

Cincinnati Geotechnical Group

University of Louisville
Department of Civil and Environmental Engineering

University of Kentucky
Department of Civil Engineering,
Office of Continuing Education,
and Kentucky Transportation Center

University of Cincinnati
Department of Civil and Environmental Engineering

University of Dayton
Department of Civil Engineering and
Engineering Mechanics

2004 ORVSS Agenda
Rock Engineering and Tunneling

MORNING SESSION:

7:30 – 8:15	Registration
8:15 – 8:30	Introductions
8:30 – 9:30	Dr. Ralph Peck – Keynote
9:30 – 10:00	Dr. D. J. Hagerty – Building a Tunnel to Leak
10:00 – 10:30	Technical Session in Exhibit Area
10:30 – 11:00	Dr. Donald Bruce – Evolution of Rock Anchor Practice over Three Decades
11:00 – 11:30	Paul Lewis – Savage Mountain Tunnel Remediation
11:30 – 12:00	Daniel Hurst – Nashville Tunnel Design and Construction
12:00 – 1:15	Lunch

AFTERNOON SESSION:

1:15 – 2:15	Dr. Ed Cording (Keynote) – Regional Tunnel Design Aspects and Construction
2:15 – 2:45	Dr. Pinnaduwa Kulatilake – Rock Slopes and Defects
2:45 – 3:15	Technical Session in Exhibit Area
3:15 – 3:45	Randall J. Essex – Innovative Rock Excavation for Highway Tunnels
3:45 – 4:15	Greg Yankey/Dr. Rick Deschamps – Bluestone Dam Numerical Rock Analysis
4:15 – 4:45	Wayne Walburton – Geotechnical Conditions of the Pine Mountain Tunnel
4:45 – 5:00	Closing Remarks
5:00 – 6:00	Social Hour

ORVSS XXXV
Planning Committee

Troy O'Neal, P.E.
Army Corps of Engineers
Louisville District
(Chairman)

Tim O'Leary, P.E.
Army Corps of Engineers
Louisville District

Tom Rockaway, PhD, P.E.
University of Louisville, Department of Civil
and Environmental Engineering

David Sawitzki, P.E.
AMEC Earth and Environmental, Inc.

Mark Schuhmann, P.E.
MATEC, Inc.

Jerry Vandevelde, P.E.
GEM Engineering

TABLE OF CONTENTS

OHIO RIVER VALLEY SOILS SEMINAR
The Olympics and Super Bowl of Regional Geotechnical Engineering Seminars?

The 2004 Ohio River Valley Soils Seminar (ORVSS) is the thirty-fifth consecutive seminar organized by the Kentucky Geotechnical Engineering Group (KGEG), Cincinnati Geotechnical Group, University of Louisville, University of Kentucky, University of Cincinnati, and University of Dayton. While affiliated with the American Society of Civil Engineers (ASCE), ORVSS has been a self-sustaining seminar, organized and operated solely by regional personnel with no national or section support from. It has been suggested that ORVSS may be the longest, uninterrupted, ASCE, annual, locally sponsored, continuing-education seminar. The seminar rotates annually between Louisville and Lexington, Kentucky and Cincinnati, Ohio. The rich history of ORVSS was summarized by Aubrey May in the 1994 Silver Anniversary Edition of the ORVSS Proceedings and supplemented by the 1999 ORVSS Planning Committee in *Remembering Thirty Years of ORVSS*. Both of these documents are reproduced on the following pages, with some minor revisions and updates. In addition, a complete listing of the ORVSS locations, dates, and topics as well as a complete bibliography of papers, is included.

A broad spectrum of presenters (academics, consultants, and contractors) with local, regional, and international experience has been assembled for this year's seminar with topics ranging from tunnels to foundations to slopes. The Planning Committee is pleased to welcome Ralph Peck and Ed Cording back to Louisville as keynote speakers to celebrate thirty-five years of ORVSS and to gather and share ideas, successes, and failures among our peers, with a uniquely regional flavor. Dr. Peck was the dinner speaker at the first ORVSS in 1970 in Lexington, Kentucky at which he was conferred the honor of Kentucky Colonel. Both keynote speakers will spin geotechnical "yarns" for all to enjoy and learn from. Dr. Peck's presentation is entitled *Tales of Tunnels I Have Known*, and Dr. Cording will tell the story of how the regional geology has impacted his previous rock tunneling projects between the Great Lakes and the Ohio River. We are also pleased to welcome Dr. Joe Hagerty, from the University of Louisville, back as a presenter. Dr. Hagerty has been an instrumental part of ORVSS since its inception and helped lay the foundation for our present success.

All great events accentuate their longevity and greatness by using Roman numerals. The Games of the XXVIII Olympiad were competed this summer, and the culmination of the upcoming professional football season will be Super Bowl XXXIX. Therefore, the ORVSS XXXV appears to be in elite company, at least numerically between the Summer Olympic Games and the Super Bowl. Seriously, thirty-five consecutive years of a locally organized specialty seminar reflects great credit upon the dedication of the geotechnical community to serve our peers, as well as civil engineers, architects, and contractors throughout the region. We look forward to many more years of successful seminars.

ORVSS XXXV Planning Committee, 2004

The History of the Ohio River Valley Soils Seminar
by Aubrey May (and contributions by Vince Drnevich and Joe Hagerty)

This being the twenty-fifth ORVSS, the organizing committee felt it appropriate to include a more detailed summary of its history. This paper summarizes the history based both on official records and personal recollections and impressions. Much of the following information was obtained from the personal recollections of seminar participants, meeting minutes from sponsoring organizations, the history of the Kentucky Geotechnical Engineering Group published in the 50th Anniversary Edition (1986) of the ASCE Kentucky Section Directory (KGEG history by R. C. Deen and V. P. Drnevich), and of course the printed ORVSS proceedings themselves.

Twenty-five seminars in uninterrupted succession is a substantial feat for a locally organized and operated series. Several of the early participants in the seminar have suggested that it may be the longest uninterrupted ASCE annual continuing education seminar, but inquiries with the ASCE National headquarters only revealed that records of the many such local seminars are not kept. We would prefer to adopt the attitude of claiming the record until proven otherwise. A listing of all the past ORVSS' and the topics are provided in Table 1 (updated to present – editor's note).

It is noteworthy that with the exception of seed funding provided by the Kentucky Section of ASCE and the University of Louisville for the first seminar, also held in Lexington, ORVSS has been a self-sustaining seminar organized and operated by regional personnel with no National or Section support. The seminar is sponsored by the Kentucky Geotechnical Engineering Group (KGEG), Cincinnati Geotechnical Group, University of Cincinnati, University of Dayton, University of Kentucky, and University of Louisville. ORVSS is held in the Lexington, Louisville, and Cincinnati areas on successive years. The alternation of seminar locations is likely one reason for the continuing success of ORVSS, with geotechnical engineers from each metropolitan area only being responsible for organizing the seminar once every three years. Regardless, the success and continuation of the seminar is a tribute to the geotechnical community in this region, and although the seminar is regionally oriented, it has always featured internationally acclaimed participants and speakers, beginning with the dinner speaker for the first ORVSS in 1970, Ralph Peck.

The first ORVSS was held in Lexington, Kentucky on October 16, 1970, but the events leading up to that first seminar extend at least to June 7, 1968 with the founding of the Kentucky Soil Mechanics and Foundations Group (KSMFG) in Frankfort, Kentucky. In addition to ORVSS, KSMFG, renamed the Kentucky Geotechnical Engineering Group on March 8, 1977, has sponsored numerous seminars and technical sessions over the years and has maintained the rigorous schedule of approximately eight to ten gatherings per year since their founding. Traditionally, at least two honorary lectures are sponsored by KGEG each year, with the lecturers being selected by the University of Louisville faculty in the Spring and the University of Kentucky faculty in the Fall. In fact, prior to its founding, the original members of KSMFG held at least two 1968 meetings that included technical sessions: the first being Soil Problems and Solutions by Nutting Engineers on February 9, 1968, and the second featuring Mobile Drilling *on Hollow Stem Augers and New Drilling Equipment held April 26, 1968. A third 1968 technical session on Pile Foundations and Their Applications by Dr. John Heer of University of Louisville was the first of the officially formed KSMFG, held September 13, 1968. Although it was not planned this way, it is appropriate that this twenty-fifth ORVSS be centered on the same topic as the first official KSMFG technical session.*

The 1968-69 schedule of KSMFG technical sessions included topics on rock core drilling, Wolf Creek Dam seepage, finite element methods, subsurface investigation, stability and consolidation, and ground freezing techniques. Joint meetings with the Kentucky Section of ASCE featured guest speakers on the study of moon samples and the Kentucky Highway Soil Exploration Program. Some of these sessions included featured speakers from the Cincinnati metropolitan area, where the Cincinnati-Dayton Soil Mechanics and Foundations Group was sponsoring similar activities. The participation of some of the members of the Cincinnati-Dayton Soil Mechanics and Foundations Group was indicative of a long-running association between the Ohio and Kentucky groups, and played a role in the early development of the partnership between the southern Ohio organizations and universities and the Kentucky groups in sponsoring all of the ORVSS' after ORVSS 1.

A review of the KSMFG annual report for 1968-69 indicated that discussion of a continuing education program and planning for a seminar was under way at that time. Woodson (Woody) McGraw, Chairman of KSMFG in its first full year (1968-69), was the Chairman of the Organizing Committee for ORVSS 1, which also included Bill Mossbarger (1970-71 KSMFG Chairman) and Joe Hagerty (1971-72 KSMFG Chairman). A summary of the committee members for succeeding ORVSS' as indicated on the proceedings is provided on Table 2 (not reproduced here – editor's note).

The Kentucky section of ASCE provided a loan of $350 and the University of Louisville volunteered to print the proceedings for ORVSS I and cover the cost of any loss incurred. The seminar was successful technically and financially; however, as the $350 was repaid along with a $150 donation to the Kentucky Section. The theme of ORVSS I was "Building Foundation Design and Construction" and included Dr. Ralph Peck, Dr. Hagerty's former Ph.D. advisor, as the speaker for the evening dinner. Dr. Peck was honored with the designation of "Kentucky Colonel" and presented with a gift of julep cups for his fine presentation. Attendance for the day's technical sessions for ORVSS I totaled 103, while attendance at the evening dinner/lecture was 149.

In the early years of the ORVSS seminars, the evening dinner session was a separate event from the day's activities. Attendance in the evening sessions often included those who had not been present for the technical sessions, and vice versa. Later, the evening dinner/lecture was included as part of the seminar, while some of the recent seminars excluded the evening dinner/lecture in favor of a social hour to permit an opportunity for catching up with old acquaintances before adjourning to allow those who drove in for the seminar sufficient time to make the long return trip to home. A complete listing of invited dinner speakers for succeeding ORVSS' could not be developed, but a summary list for those known is provided in Table 3 (not reproduced here – editor's note).

While the over 200 technical papers of the past ORVSS have always been of high quality and well received, a special part of the seminar is the breaks, lunch session, and evening social hour or dinner. While attendees come from across the continent and even from overseas, ORVSS has always been dominated by local consulting engineers from within a four hour drive of the seminar site, so the interest and knowledge of the group carries a more regional flavor. In addition, unlike the majority of technical seminars, many of the presenters at ORVSS are from a consulting environment where their success or failure is much less dependent on the findings presented, so a more relaxed atmosphere is predominant. One new attendee at the most recent seminar remarked on the closeness and camaraderie among the participants, observing that while many of those present seem well acquainted with each other, the seminar provides the primary opportunity to gather at least once a year in a neutral setting to renew friendships. This aspect of ORVSS may be as beneficial as any in maintaining state of the art geotechnical engineering in this region. ORVSS provides this region's geotechnical engineers an inexpensive, one-day forum to gather and share ideas and successes among our peers.

While the social flavor of the ORVSS has always been special, the technical content has also been very good. Clearly, the presenters put considerable effort into their papers, and many renowned geotechnical engineers are counted among those who have submitted their work through ORVSS. A review of the past ORVSS papers reveals many excellent works and the prudent geotechnical engineer would do well to review the selection. Although it is suspected that there are a number of complete sets of ORVSS proceedings, the only known complete set in public hands is the set of proceedings in the Kentucky Transportation Research Center library on the University of Kentucky campus. Completion of that set required contributions from the personal library of the late R. C. Deen of the University of Kentucky, an active promoter and participant in the most of the early ORVSS'. A complete listing of papers from past ORVSS' has been compiled and made available during the ORVSS XXV session. The listing can also be obtained from the Kentucky Transportation Research Center library. One of the speakers at ORVSS XX asked those in attendance who had attended all of the ORVSS' up to that time, and no one present spoke out, so unless an attendee at ORVSS XX was out of the room at the time the question was raised, it is unlikely there are any current "veterans" of all of the seminars.

Attendance for ORVSS over the years has varied from about 120 to 250, with an average of about 200. Attendance was as high as 305 in 1977 (ORVSS VIII). Some interesting observations while reviewing past ORVSS proceedings include editorial comments and summaries of the day's activities by Bob Deen are included in some of the proceedings from 1973 through 1976. The ORVSS logo was designed in 1978, prior to which the acronym ORVSS was not used. Both the logo and acronym caught on immediately. Photographs from some of the ORVSS' over the years are provided on the following sheets (not reproduced here - editor's note).

The Organizing Committee wishes to thank all of those who assisted in the preparation of this history. Of particular note is the contribution of photographs by Vince Drnevich and Joe Hagerty. The assistance of Aubrey May, Vince Drnevich, and Joe Hagerty in researching their records and recollections to assist in the preparation of this history is greatly appreciated.

The Organizing Committee looks forward to preparation of an updated history for the 50th ORVSS with great anticipation!

Remembering Thirty Years of ORVSS
(by the ORVSS XXX Planning Committee, 1999)

To complement the above printed history of ORVSS, and to provide some further historical background to the birth of ORVSS, it should be mentioned that, as a precursor to ORVSS, the Soil Mechanics and Foundation Division, Cincinnati Section of ASCE, and the University of Cincinnati had organized six seminars, called Soil Mechanics Symposia from 1960 to 1968. Participants in the planning, preparation and presentation of the 1960 Symposium included: F. M. Mellinger, C. K. Hoffmeyer, J. D. Kenty, R. Grayman, and G. Roberto of the U. S. Army Corps of Engineers; C. A. Witte of Vogt, Ivers, Seaman & Associates; A. H. Hunter and R. D. Blotter of A. M. Kinney Co.; C. R. Lennertz, M. F. Nethero, and F. G. Mundstock of the H. C. Nutting Co.; W. T. Zachman of the W. L. Harper Construction Co.; and Professors L. M. Laushey and R. T. Howe of the University of Cincinnati. The main purpose of these symposia was to inform contractors and architects on how geotechnical engineering relates to their profession. The topics included: Foundations for Structures; Pavements for Streets, Access Roads, Warehouses and Parking Lots; Stability of Unrestrained Slopes and Earth Retention Structures; Deep Foundations; and Earthwork.

Overall, ORVSS has been a magnificent experience for our geotechnical groups in Ohio and Kentucky. There are many ways that our members participated: some presented papers, others organized the meetings, still others invited nationally known speakers and acted as moderators. We recruited exhibitors, organized the papers for publication, handled the financial matters, negotiated with the hotels and convention halls, etc. Beside our own Vince Drnevich, Joe Hagerty, Bob Lennertz and others too many to list, many national leaders of our profession participated as speakers, keynote speakers or as evening speakers. The list is not totally complete but includes: Ralph B. Peck, Mrs. Karl Terzaghi, George Sowers, Stanley Wilson, E. D'Appolonia, T. H. Wu, Don U. Deere, G. A. Leonards, W. E. Hanson, B. Broms, Bengt H. Fellenius, G. G. Goble, Lymon Reese, E. T. Selig, Milton Harr, Joseph P. Welsh, John Dunnicliff, J. M. Duncan, Joseph S. Ward, M. R. Thompson, Clyde N. Baker, Jr., Fred H. Kulhawy, James P. Gould, Jorj O. Osterberg, F. C. Townsend and William F. Marcuson III. ORVSS VII in 1976 even featured former Senator Albert Gore, Sr. Dick Goettle, Vince Drnevich and Joe Hagerty should get most of the credit for bringing them to our meetings.

The proceedings contain a great wealth of information for the practicing engineer in this region of the country. For that reason, the bibliography of papers from previous seminars has been updated and republished. *It is our hope that in glancing at these you may discover that some topic, presented years ago, has a special meaning or relevance to a project that you are working on now or one you're anticipating in the future. In browsing, you may reminisce about a favorite past meeting - perhaps you presented a paper, or you helped organize a meeting, or first met an old friend, or business associate, or first listened to Ralph Peck. Observe the huge range of topics that the ORVSS seminars have covered; it is truly impressive! You, as members of the organizing groups, can congratulate yourself for a job well done!*

Table 1. Past ORVSS Locations, Dates, and Topics

ORVSS	Date	Topic	Location
I	Oct. 16, 1970	Building Foundation Design and Construction	Lexington, KY
II	Oct. 15, 1971	Earthwork Engineering, Start to Finish	Louisville, KY
III	Oct. 27, 1972	Lateral Earth Pressures	Fort Mitchell, KY
IV	Oct. 5, 1973	Geotechnics in Transportation Engineering	Lexington, KY
V	Oct. 18, 1974	Rock Engineering	Clarksville, IN
VI	Oct. 17, 1975	Slope Stability and Landslides	Fort Mitchell, KY
VII	Oct. 8, 1976	Shales and Mine Wastes: Geotechnical Properties, Design and Construction	Lexington, KY
VIII	Oct. 14, 1977	Earth Dams and Embankments: Design and Construction	Louisville, KY
IX	Oct. 27,1978	Deep Foundations	Fort Mitchell, KY
X	Oct. 5, 1979	Geotechnics of Mining	Lexington, KY
XI	Oct. 10, 1980	Earth Pressures and Retaining Structures	Clarksville, IN
XII	Oct. 9, 1981	Groundwater: Monitoring, Evaluation, and Control	Fort Mitchell, KY
XIII	Oct. 8, 1982	Recent Advances in Geotechnical Engineering	Lexington, KY
XIV	Oct. 14, 1983	Foundation Instrumentation and Geophysical	Clarksville, IN
XV	Nov. 2, 1984	Practical Application of Drainage in Geotechnical Engineering	Fort Mitchell, KY
XVI	Oct. 11, 1985	Applied Soil Dynamics	Lexington, KY
XVII	Oct. 17, 1986	Natural Slope Stability and Instrumentation	Clarksville, IN
XVIII	Nov. 6, 1987	Liability Issues in Geotechnical Engineering and Construction	Fort Mitchell, KY
XIX	Oct. 21, 1988	Chemical and Mechanical Stabilization of Soil Subgrades	Lexington, KY
XX	Oct. 27, 1989	Construction In and On Rock	Louisville, KY
XXI	Oct. 26, 1990	Environmental Aspects of Geotechnical Engineering	Cincinnati, OH
XXII	Oct. 18,1991	Design and Construction with Geosynthetics	Lexington, KY
XXIII	Oct. 16, 1992	In Situ Soil Modification	Louisville, KY
XXIV	Oct. 15, 1993	Geotechnical Aspects of Infrastructure Reconstruction	Erlanger, KY
XXV	Oct. 21, 1994	Recent Advances in Deep Foundations	Lexington, KY
XXVI	Oct. 20, 1995	Site Investigations: Geotechnical and Environmental	Clarksville, IN
XXVII	Oct. 11, 1996	Forensic Studies in Geotechnical Engineering	Cincinnati, OH
XXVIII	Oct. 10, 1997	Unconventional Fills: Design, Construction, and Performance	Lexington, KY
XXIX	Oct. 16, 1998	Problematic Geotechnical Materials	Louisville, KY
XXX	Oct. 1, 1999	Value Engineering in Geotechnical Consulting and Construction	Cincinnati, OH

Table 1. Past ORVSS Locations, Dates, and Topics (cont.)

ORVSS	Date	Topic	Location
XXXI	Sep. 15, 2000	Instrumentation	Lexington, KY
XXXII	Oct. 24, 2001	Regional Seismicity and Ground Vibrations	Louisville, KY
XXXIII	Oct. 18, 2002	Ground Stabilization and Modification	Covington, KY
XXXIV	Sep. 19, 2003	Applications of Earth Retaining Systems and Geosynthetic Materials	Lexington, KY
XXXV	Oct. 20, 2004	Rock Engineering and Tunneling	Louisville, KY

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Thelen, G. J. (1970). "Design, Construction and Testing of Drilled Piers on Thinly Bedded Limestone." Ohio River Valley Soils Seminar No. 1, Lexington, Kentucky.

ORVSS II – Earthwork Engineering, Start to Finish

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Rajadhyaksha, V. V. (1976). "Conventional and Unconventional Approaches Used to Locate and Eliminate Hazardous Mine Waste Dams in Ohio." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.
Rosen, B. (1976). "Geotechnical Oversight Procedures." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.
Sheahan, J. M., and Hunkele, T. F. (1976). "Airport Embankment Utilizes Coal Strip Mine Waste." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.
Stephenson, R. W., and Rockaway, J. D. (1976). "Properties of Coal Mine Floor Shale." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.
Ullrich, C. R. (1976). "Rebound Properties of Remolded Clay Shales." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.
Wood, L. E., et al. (1976). "Guidelines for Compacted Shale Embankments." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

ORVSS VIII – Earth Dams and Embankments: Design and Construction

Castro, G. (1977). "Comments on Seismic Stability Evaluation of Embankment Dams." Ohio River Valley Soils Seminar No. 8, Louisville, Kentucky.
Couch Jr., F. B. (1977). "Foundations Seepage Problems at Wolf Creek Dam." Ohio River Valley Soils Seminar No. 8, Louisville, Kentucky.
Hanson, W. E., and Daniels, D. E. (1977). "Small Dams - Particular Problems and Considerations." Ohio River Valley Soils Seminar No. 8, Louisville, Kentucky.
Moore, L. H. (1977). "Design and Construction of Highway Embankments in New York State." Ohio River Valley Soils Seminar No. 8, Louisville, Kentucky.
Nieto, A. S. (1977). "Significant Engineering-Geology Features at Damsites in Flat-Lying Sedimentary Rocks." Ohio River Valley Soils Seminar No. 8, Louisville, Kentucky.
Palmer, E. C., et al. (1977). "Movements of a Natural Slope and an Embankment - Two Case Histories." Ohio River Valley Soils Seminar No. 8, Louisville, Kentucky.

ORVSS IX – Deep Foundations

Cutter, W. A., and Warder, D. L. (1978). "Friction Piles in Sand - A Review of Static Design Procedures." Ohio River Valley Soils Seminar No. 9, Fort Mitchell, Kentucky.
Durbin, W. L., et al. (1978). "Load Transfer Measurements in Concrete Piles." Ohio River Valley Soils Seminar No. 9, Fort Mitchell, Kentucky.
Fellenius, B. H. (1978). "Interpretation and Analysis of Pile Load Tests." Ohio River Valley Soils Seminar No. 9, Fort Mitchell, Kentucky.
Friels, D. R. (1978). "Axial Compression and Uplift Resistance of Steel H-Piles." Ohio River Valley Soils Seminar No. 9, Fort Mitchell, Kentucky.
Goble, G. G. (1978). "A Pile Design and Installation Specifications Based on the Load Factor Concept. " Ohio River Valley Soils Seminar No. 9, Fort Mitchell, Kentucky.
Lennertz, C. R. (1978). "Contracting for Deep Foundations - Legal Aspects." Ohio River Valley Soils Seminar No. 9, Fort Mitchell, Kentucky.
Reese, L. C. (1978). "Tests to Obtain Behavior of Drilled Shafts under Axial Load." Ohio River Valley Soils Seminar No. 9, Fort Mitchell, Kentucky.

ORVSS X – Geotechnics of Mining

Charlie, W. A., et al. (1979). "Seepage and Stability Analysis for an Inundated Mill Tailing Impoundment, A Case Study." Ohio River Valley Soils Seminar No. 10, Lexington, Kentucky.
Cowherd, D. C. (1979). "The Necessity for Scrutinizing Government Mining Regulations." Ohio River Valley Soils Seminar No. 10, Lexington, Kentucky.
Darnell, K. E., et al. (1979). "Geotechnical Considerations for Deadheading a Marion 5761 Shovel." Ohio River Valley Soils Seminar No. 10, Lexington, Kentucky.
Hale, B., and Lovell, C. W. (1979). "Point Load Strength Testing of Coal Spoil." Ohio River Valley Soils Seminar No. 10, Lexington, Kentucky.
Nieto, A. S. (1979). "Evaluation of Damage Potential to Earth Dams by Subsurface Coal Mining at Rend Lake, Illinois." Ohio River Valley Soils Seminar No. 10, Lexington, Kentucky.
O'Rourke, T. D., and Turner, S. M. (1979). "A Critical Evaluation of Coal Mining Subsidence Patterns." Ohio River Valley Soils Seminar No. 10, Lexington, Kentucky.
Surendra, M., and Lovell, C. W. (1979). "Chemical Additives to Change the Durability of Shales." Ohio River Valley Soils Seminar No. 10, Lexington, Kentucky.
Thacker, B. K., and Cowherd, D. D. (1979). "Disposal of Coal Processing Wastes at Sites of Limited Size." Ohio River Valley Soils Seminar No. 10, Lexington, Kentucky.
Vandre, B. C. (1979). "The Review and Regulation of Slope Stability." Ohio River Valley Soils Seminar No. 10, Lexington, Kentucky.

ORVSS XI – Earth Pressures and Retaining Structures

Kerr, J. J. (1980). "Practical Underpinning Operations." Ohio River Valley Soils Seminar No. 11, Clarksville, Indiana.
Kinner, E. B., et al. (1980). "Design, Construction and Performance of a Cellular Cofferdam in Deep Water." Ohio River Valley Soils Seminar No. 11, Clarksville, Indiana.
Lacroix, Y., and Almatzidis, D. K. (1980). "Design, Construction, and Performance of Anchored Bulkheads." Ohio River Valley Soils Seminar No. 11, Clarksville, Indiana.
O'Rourke, T. D. (1980). "Ground Movements Associated with Deep Braced Cut Excavations." Ohio River Valley Soils Seminar No. 11, Clarksville, Indiana.
Riggs, C. O. (1980). "Tie-Back Membrane Walls in Venezuela." Ohio River Valley Soils Seminar No. 11, Clarksville, Indiana.
Selig, E. T. (1980). "Large Buried Metal Culvert Design and Construction." Ohio River Valley Soils Seminar No. 11, Clarksville, Indiana.

ORVSS XII – Groundwater: Monitoring, Evaluation, and Control

Alizadeh, M. M. (1981). "Design, Installation, and Operation of Dewatering System for Pumping Station Approach Channel." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.
Anderson, R. D., et al. (1981). "Dewatering for Soil Improvement." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.
Bailey, B., and Cutter, W. A. (1981). "Multi-Use Well System for High-Rise Office Building in Indianapolis." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.
Bishop, C. S., and Munson, W. E. (1981). "Horizontal Drains: Predicting Effectiveness in Advance of Installation." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.
Cox, G. C. (1981). "Dewatering of a Construction Site and an Existing Structure Utilizing Deep Wells." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.
Fetzer, C. A., and Plummer, P. M. (1981). "Installation of Adequately Scaled Piezometers." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.
Gleason, T. A., and Kaufmann, R. F. (1981). "Hydrogeologic and Water Quality Assessment for an Existing Class I (Hazardous) Waste Disposal Site." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.
Hagerty, D. J. (1981). "Fundamental Aspects of Groundwater Flow." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.
Huyakorn, P. S., and Dougherty, D. E. (1981). "Application of Computer Model to Groundwater Flow and Contaminant Transport Investigation." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.

O'Rourke, J. E., and O'Connor, K. (1981). "Dewatering Surface Mines in the Interior Coal Province." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.
Sullivan, P. J. (1981). "Synthetic Fabrics (Geotextiles) in Drainage Applications." Ohio River Valley Soils Seminar No. 12, Fort Mitchell, Kentucky.

ORVSS XIII – Recent Advances in Geotechnical Engineering

Cho, Y. Y. (1982). "Design Considerations using SPTs." Ohio River Valley Soils Seminar No. 13, Lexington, Kentucky.
Cutter, W. A., and Bailey, B. (1982). "Loose Sand Pipes in Glacial Outwash: How Did They Develop and Are They Significant?" Ohio River Valley Soils Seminar No. 13, Lexington, Kentucky.
Deo, P., and Nona, D. (1982). "Use of Cylinder Pile Retaining Wall to Stabilize Excavation Sides and Protect Existing Structures." Ohio River Valley Soils Seminar No. 13, Lexington, Kentucky.
Groves, C. B., and Kleber, B. (1982). "Instrumentation of Sheet Pile Cofferdam at Lock and Dam 26 Replacement Site." Ohio River Valley Soils Seminar No. 13, Lexington, Kentucky.
Harr, M. E. (1982). "Reliability in Geotechnical Engineering." Ohio River Valley Soils Seminar No. 13, Lexington, Kentucky.
Lovell, C. W. (1982). "Three-Dimensional Slope Stability." Ohio River Valley Soils Seminar No. 13, Lexington, Kentucky.
Paris, J. E. (1982). "Problems Associated with Construction of a Rockfill Embankment." Ohio River Valley Soils Seminar No. 13, Lexington, Kentucky.
Preber, T. (1982). "Sampling and Testing of the Maquoketa Shale in Northwestern Illinois." Ohio River Valley Soils Seminar No. 13, Lexington, Kentucky.
Welsh, J. P., and Snyder, R. (1982). "Chemical Grouting Utilized for Underpinning and Water Control for a Pit Installation." Ohio River Valley Soils Seminar No. 13, Lexington, Kentucky.

ORVSS XIV – Foundation Instrumentation and Geophysical

Bodocsi, A., and Lockwood, M. (1983). "Field Measurements of Tie-Back Bar Performances in an Excavation Bracing." Ohio River Valley Soils Seminar No. 14, Clarksville, Indiana.
Drnevich, V. P., and Hall Jr., J. R. (1983). "Use of Spectral Analysis Techniques for Structural and Foundation Vibrations Analyses." Ohio River Valley Soils Seminar No. 14, Clarksville, Indiana.
Fowler, J. (1983). "Ground Penetrating Radar: A New Tool in Geotechnical Engineering." Ohio River Valley Soils Seminar No. 14, Clarksville, Indiana.
Hannigan, P. J. (1983). "Performance Monitoring of Pile Foundation Installations." Ohio River Valley Soils Seminar No. 14, Clarksville, Indiana.
Madej, G. P. (1983). "A Case Study of a Tied-Back Soldier Pile and Lagging Retaining Wall." Ohio River Valley Soils Seminar No. 14, Clarksville, Indiana.
Nazarian, S., et al. (1983). "Use of Spectral Analysis of Surface Waves Method for Determination of Moduli and Thicknesses of Pavement Systems." Ohio River Valley Soils Seminar No. 14, Clarksville, Indiana.
Riggs, C. O. (1983). "The SPT - A Summary of Some Recent Studies." Ohio River Valley Soils Seminar No. 14, Clarksville, Indiana.
Thacker, B. K., and Schad, J. A. (1983). "Rapid Construction of a Combined Coal Refuse Embankment." Ohio River Valley Soils Seminar No. 14, Clarksville, Indiana.
White, R. (1983). "Borehole Geophysics as Applied to a Foundation Investigation in Limestone: A Case History." Ohio River Valley Soils Seminar No. 14, Clarksville, Indiana.

ORVSS XV – Practical Application of Drainage in Geotechnical Engineering

Alvi, P. M. (1984). "Drainage Requirements, Design Concepts, Drainage Analysis Calculations, Typical Designs, Potential Problems, and Related Solutions." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.
Anderson, T. C. (1984). "Drainage and Frost Protection for Tiedback Walls." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.
Bird, D. W. (1984). "Slope Stabilization through Groundwater Drainage." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.
Camp, G., et al. (1984). "Measurements and Modeling of Two-Dimensional Subsurface Water Movement." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.
Charles, R. D. (1984). "Performance of Vertical Wick Drains in Soft Soils." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.
Cutter, W. A., and Waterman, R. C. (1984). "Riprap Design for the Ohio River: A Change in Philosophy from Big Stone to Positive Bank Drainage." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.
Dewey, R. L. (1984). "Riverbank Stabilization with Radial Drains from a Shaft at Grand Coulee Dam." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.
Lessley, J. C., and Barksdale, R. D. (1984). "A Microcomputer Program for the Design of Site Dewatering Systems." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.
Loughney, R. W. (1984). "A Practical Approach to Construction Dewatering." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.
van Wijk, A. J., and Lovell, C. W. (1984). "Importance of Drainage to Rigid Pavement Performance." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.

Warder, D. L., et al. (1984). "Design and Construction of a Permanent Dewatering System for a High-Technology Facility." Ohio River Valley Soils Seminar No. 15, Fort Mitchell, Kentucky.

ORVSS XVI – Applied Soil Dynamics

Amato, V. E. (1985). "Large Scale Laboratory Testing for Liquefaction Potential of Saturated Sands." Ohio River Valley Soils Seminar No. 16, Lexington, Kentucky.

Balbis, R. E. (1985). "Applying Dynamic Precompression Treatment (DPI) in Built-Up or Downtown Areas in South Florida." Ohio River Valley Soils Seminar No. 16, Lexington, Kentucky.

Hagerty, D. J. (1985). "Effects of Blasting on Residential Structures." Ohio River Valley Soils Seminar No. 16, Lexington, Kentucky.

Harris, S. A. (1985). "Amplification of Earthquake Motions at Maysville, Kentucky." Ohio River Valley Soils Seminar No. 16, Lexington, Kentucky.

Knuppel, L. A. (1985). "Barkley Dam Seismic Stability Study." Ohio River Valley Soils Seminar No. 16, Lexington, Kentucky.

Reil, J. W. (1985). "How Geology Affects Ground Vibrations." Ohio River Valley Soils Seminar No. 16, Lexington, Kentucky.

Richardson, G. N. (1985). "Comparison of Theoretical and Field Performance of Machine Foundations on Ohio River Alluvial Deposits." Ohio River Valley Soils Seminar No. 16, Lexington, Kentucky.

Shwenk, J. L. (1985). "Foundation Installation for 6 1/2-Gateway Dam at Lock and Dam 26 (replacement)." Ohio River Valley Soils Seminar No. 16, Lexington, Kentucky.

Sommers, S. A. (1985). "Earthquake-Induced Responses of Model Retaining Walls." Ohio River Valley Soils Seminar No. 16, Lexington, Kentucky.

ORVSS XVII – Natural Slope Stability and Instrumentation

Bishop, C. S., et al. (1986). "Design of Highway Embankments on Unstable Natural Slopes." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

Boudra, L. H., and Vandevelde, G. T. (1986). "Red Mountain Landslide Susceptibility Study." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

Bump, V., and Bang, S. (1986). "Investigation of Forest City Landslides in South Dakota." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

Camp, G. M., and Veith, J. (1986). "Soil Retention and Soil Stabilization with Geotextile Fabric." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

Duncan, J. M. (1986). "Methods of Analyzing the Stability of Natural Slopes." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

Dunnicliff, J. (1986). "Instrumentation of Cut and Natural Slopes in Soil and Rock." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

Hall, G. A. (1986). "Landslide Recognition and Constructive Prevention in the Appalachian Area." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

Holbrook, R. M., et al. (1986). "Repair of Smokey Landslide Using a Tied-Back Wall." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

Sites, M. A., and Hagerty, D. J. (1986). "A Case Study of Slope Stability in New Providence Shale." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

Weber, L. C., and Wilson, L. E. (1986). "Landslides in the Colluvial Soils of Southwestern Davidson County and Northern Williamson County, Tennessee." Ohio River Valley Soils Seminar No. 17, Clarksville, Indiana.

ORVSS XVIII – Liability Issues in Geotechnical Engineering and Construction

Ashar, M. L (1987). "Effective Use of Expert Witnesses. The Baking a Cake Approach." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Baker Jr., C. N. (1986). "Earth Retention Design, Ground Movement Monitoring and Liability. A Case History." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Budinger, F. C. (1987). "Engineering in the Courtroom." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Cheeks, J. R. (1987). "Professionalism and Quality: Foundations for the New Road." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Kastman, K. H., and Hendron, D. M. (1987). "Soil/Acid Immersion Test as Focus of Court Testimony." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Lennertz, C. R. (1987). "The Engineers' Standard of Care." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Myers, R. W. (1987). "Impact of the Differing Site Conditions." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Payne, J. L. (1987). "Managing Liability in a Consulting Firm." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Petrie, B. J. (1987). "Legal Issues for Geotechnical Engineers and Contractors." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Shane, R. A. (1987). "How to Control Disputes, Claims and Litigation." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

ORVSS XIX – Chemical and Mechanical Stabilization of Soil Subgrades

Forssblad, L. (1988). "Roller-Mounted Compaction Meters - Principles, Field Tests, and Practical Experiences." Ohio River Valley Soils Seminar No. 19, Lexington, Kentucky.

Gnaedinger, J. P. (1988). "Utilization of Incinerator Ash." Ohio River Valley Soils Seminar No. 19, Lexington, Kentucky.

Hopkins, T. C., et al. (1988). "Highway Field Trials of Chemically Stabilized Soil Subgrades." Ohio River Valley Soils Seminar No. 19, Lexington, Kentucky.
Rose, J. G., and Huang, J. H. (1988). "Hot-Mix Asphalt Stabilized Railroad Track Beds." Ohio River Valley Soils Seminar No. 19, Lexington, Kentucky.
Storm, J. W., and Hagerty, D. J. (1988). "Improvement of Subgrade Support with Blasted Rock." Ohio River Valley Soils Seminar No. 19, Lexington, Kentucky.
Thompson, M. R. (1988). "Admixture Stabilization of Subgrades." Ohio River Valley Soils Seminar No. 19, Lexington, Kentucky.
Voor, B. H. (1988). "Case History of a Cement Stabilized Coal Transfer Yard." Ohio River Valley Soils Seminar No. 19, Lexington, Kentucky.
Williams, N., and Beech, J. (1988). "Highway Application of Geosynthetics." Ohio River Valley Soils Seminar No. 19, Lexington, Kentucky.
Zimmerman, J. R. (1988). "Chemical and Mechanical Stabilization of Railroad Subgrades." Ohio River Valley Soils Seminar No. 19, Lexington, Kentucky.

ORVSS XX – Construction In and On Rock

Belgeri, J. J., and Shin, C. J. (1989). "Subsurface Conditions in and Foundation Construction on Pinnacled Carbonate Bedrock." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.
Brill, G. T., and Wells, B. (1989). "Stabilization of a Coal Dumping Highwall Using a Tied-Back Structure." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.
Bruce, D. A. (1989). "An Overview of Current U. S. Practice in Dam Stabilization Using Prestressed Rock Anchors." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.
Cowherd, D. C., and Perlea, V. G. (1989). "Rockfill Dams on Rock Foundations - Case Histories." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.
Hornbeck, S. T. (1989). "Rock Testing for the Gallipolis Replacement." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.
Leary, R. M., and Sullivan, W. R. (1989). "The Cumberland Gap Pilot Bore." Ohio River Valley Soils Seminar No. 20. Louisville, Kentucky.
Longelin, R., et al. (1989). "Mechanical Pre-Cutting as a Tunneling Technique." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.
Matheson. G. M., and Mason. J. E. (1989). "Evaluation of In Situ Rock Mass Modulus by Use of Borehole Pressure Cell at Two Dam Sites." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.
Rose, J. P., and Ilsley, R. C. (1989). "Pre-Grouting of the North Shore Tunnel, Milwaukee, Wisconsin." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.
Salami, M. R., and Hamoush. S. A. (1989). "Computer Analysis of Long-Term Stability of a Salt Dome in Relation to CAES Cavern Development." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.
Smith, J. D., and Crowl, T. (1989). "Testing Rock-Socketed Drilled Piers Using the Osterberg Load Cell." Ohio River Valley Soils Seminar No. 20, Louisville, Kentucky.

ORVSS XXI – Environmental Aspects of Geotechnical Engineering

Bowders, J. J., and McClelland, S. W. (1990). "Effects of Freeze/Thaw on the Hydraulic Conductivity of Compacted Soils." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Cluxton, P. R., et al. (1990). "Computer Aided Assessment of Contaminated Sites." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Heydinger, A. G. (1990). "Dynamics of Unsaturated Flow: An Examination of Environmental/Geotechnical Problems." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Hurst, D. J., and Weber, L. C. (1990). "Geotechnical and Environmental Considerations for Highway Construction in Mountainous Terrain with Acid-Producing Bedrock." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Jennings, A. A., and Ravi, V. (1990). "Mechanisms, Impacts, and Modeling of Chemically-Induced Changes in Saturated Soil Conductivity." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Kee, T. C., et al. (1990). "Environmental Effects of Bottom Ash as a Geotechnical Material." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Lane, D. J. (1990). "Geotechnical Considerations at the Lake Sandy Jo Superfund Site." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Mundell, J. A., and Boos, T. A. (1990). "Interpretation of Field Permeability Test Results on Full Scale Liner Systems." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Murdoch, L. C., et al. (1990). "Increased Permeability of Soils by Hydraulic Fracturing: A Field Test." Ohio River Valley Soils Seminar No. 21. Cincinnati, Ohio.
Randolph, B. W. (1990). "The Permeability Test in Environmental Geotechnology." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Spencce, R. C., et al. (1990). "State of Stress and Hydraulic Fracturing Potential in Soil/Bentonite Cut-Off Walls." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.
Wells, R. C. (1990). "Environmental Drilling: The Critical Phase for Geoenvironmental Consultants." Ohio River Valley Soils Seminar No. 21, Cincinnati, Ohio.

ORVSS XXII – Design and Construction with Geosynthetics

Armour, D. W., and Avery, C. M. (1991). "Design, Construction and Performance of a Test Embankment on Hydraulically Placed Ash." Ohio River Valley Soils Seminar No. 22. Lexington, Kentucky.
Bono, B. (1991). "Settlement Analysis for Landfill Geomembrane Covers." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.

Carroll, R. G., and Rodencal, J. (1991). "Evaluation of and Performance Guidelines for Turf Reinforcement Mats, and Erosion Control and Revegetation Mats." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Chewnig, R. J., et al. (1991). "Evaluation of Geogrid to Wall Facing Connections for Modular Block Earth Retaining Wall Systems." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Elsharief. A. M., and Lovell, C. W. (1991). "The Effect of Geometrical Properties of Non-Woven Geotextiles on Their Filtration Behavior." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Gill, S. A., and Bushell, T. D. (1991). "Geogrid Reinforced Soil-Cement Embankment." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Mandal, J. N., et al. (1991). "Bearing Capacity of Geotextile Reinforced Clay by Experimental and Finite Element Methods." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Mandal, J. N., and Jamble, K. S. (1991). "Finite Element Analysis of Geosynthetic Reinforced Soil Wall." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Mandal, J. N., and Sah, H. S. (1991). "Bearing Capacity of Strip Footing with Geosynthetic at Sand-Clay Interface." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Martin, J. S. (1991). "The Repair of the Ohio State Road 541 Slide using Polyester Geogrid Reinforcement." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Masada, T., et al. (1991). "Characteristics of Geotextile/HDPE Geomembrane Interface under Direct Shear Conditions." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Raschke, S. A., and Randolph, B. W. (1991). "Altered Friction Properties of Degraded Geomembranes." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Spang, W., et al. (1991). "Construction of a Geogrid Reinforced Wall over Soft Alluvial Clay." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Sprague, C. J. (1991). "Geotextiles in Road Structures." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Wetzel, R., et a1. (1991). "Pullout Testing for Modular Concrete Retaining Walls Reinforced with Geogrid." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.
Williams, N. D. (1991). "Design Considerations for the Closure of Wastewater Treatment Sludge Landfills." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.

ORVSS XXIII – In Situ Soil Modification

Brill, G. T., and Hussin, J. D. (1992). "The Use of Compaction Grouting to Remediate a Railroad Embankment in a Karst Environment." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.
Bruce, D. A. (1992). "New Horizons in Ground Anchorages, Pinpiles, and Cement Grouting." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.
Burke, G. K., and Brill, G. T. (1992). "Foundation Stabilization System Using Jet Grouting Techniques." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.
Jent, J. P. (1992). "Foundation Stabilization for Salt River Bridge, Fort Knox. Kentucky." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.
Lambrechts, J. R., et al. (1993). "Micro-Tunneling to Replace Boston's St. James Avenue Sewers." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.
Manley, D. V., and Pengelly, A. D. (1992). "Application of Lime-Fly Ash Injection in Runway Rehabilitation." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.
McNulty, E. G. (1992). "Sand Drain Induced Consolidation of a Peat." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.
Pasternack, S. C., and Longo, D. G. (1992). "Construction of an Apartment Complex over a Reclaimed Quarry: Arbors of Watermark, Columbus, Ohio." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.
Ryan, C. R., and Walker, A. D. (1992). "Soil Mixing for Soil Improvement - Two Case Studies." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.
Welsh, J. P. (1992). "In-Situ Ground Modification for New and Remedial Construction." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.

ORVSS XXIV – Geotechnical Aspects of Infrastructure Reconstruction

Ahmed, I., et al. (1993). "Rehabilitation of Roads Across Soft Soils." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.
Bodocsi, A., et al. (1993). "Impact of Utility Cuts on Performance of Street Pavements." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.
Bowers, M. T. (1993). "Soil-Structure Interaction Relationship for Buried Conduits: Modulus of Soil Reaction." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.
Bowers, D. A. (1993). "Contemporary Practice in the Stabilization of Concrete Dams by Post-Tensioned Rock Anchors." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.
Gruner, P. G., and Tober, D. (1993). "Geotechnical Impact on the Design of an Urban Highway Relocation." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.
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Geotechnical Evaluation of the Elsinore Water Tunnel Cincinnati, Ohio

By
Fred W. Erdmann
H.C. Nutting Company

ABSTRACT

Most engineers like to apply the latest technology to new projects, but sometimes things seem to work in reverse. Cincinnati Water Works asked the H.C. Nutting Company to look at a brick-lined tunnel built during the Civil War Era and provide an opinion as to whether it might last another hundred years. Replacing old water pipes in the existing tunnel would save quite a bit of money, but the project would only be viable if the tunnel itself was structurally sound. Elsinore Tunnel was constructed in the 1860's and was probably driven with hand tools, or perhaps with the aid of black powder (dynamite wasn't invented until 1866 and was patented a year later). The tunnel was lined with brick that appeared to be mortared with Portland cement, which just became generally available about that time. This paper describes how H.C. Nutting Company personnel inspected the tunnel, and followed through with vibration and crack monitoring as the water main replacement construction work proceeded. The project was an interesting challenge because it offered the opportunity to work with Civil War-age technology, and extend the useful life of the tunnel into the next century.

INTRODUCTION

The Elsinore Tunnel in Cincinnati, Ohio was constructed in the 1860's and connects the Eden Park Reservoir to water mains that feed the City of Cincinnati, Ohio. Entrance to the Elsinore Tunnel is currently through a limestone tower dated 1883, located at the northeast corner of Elsinore Avenue and Gilbert Avenue. At the time of the study, the Elsinore Tunnel was occupied by two cast-iron water pipes of similar age. A 35-inch-diameter, cast-iron water main installed in the 1860's occupied the north side of the tunnel. The 36-inch-diameter, cast-iron pipe installed in 1874 occupied the south side of the tunnel. Both of these pipes are fed by gravity from the reservoir at Eden Park. A pumping station was constructed in 1965, which was connected to the two old cast iron pipes through a concrete bulkhead.

The Elsinore Water Tunnel itself is a brick-lined structure, and is approximately 9.5 feet high and about 12 feet wide. The two cast iron water pipes were sitting on saddles that were sometimes staggered, and sometimes ran together as a continuous structure. The saddles were constructed of brick and mortar. A thin layer of silt and a small stream of running water covered the floor of the tunnel. No drawings were available to show the exact nature of the construction method used to drive or line this tunnel.

PHYSICAL INSPECTION

Cincinnati Water Works retained the H.C. Nutting Company to perform an integrity inspection of the tunnel. Depending on the outcome of this inspection, a decision would be made as to whether the two old cast iron pipes should be replaced with a single 54-inch diameter concrete water pipe, or to select an alternate route for the new water main. The question boiled down to whether the tunnel, which had been in service for the past 140 years, could be relied upon to remain in service for at least another 100 years.

The initial study consisted of following three tasks:

- Performing a physical inspection of the brick tunnel lining.
- Performing an analysis of historical boring logs and other job files retained in the HCN archives to interpret subsurface conditions along the tunnel alignment.
- Performing a drilling study to assess subsurface conditions directly below the floor of the Elsinore Water Tunnel.

Our initial inspection of the tunnel revealed that it was constructed by the "cut-and-cover" method from Station 0+00 (at the "Elsinore Tower") to Station 1+53. Between these stations, the sidewalls of the Elsinore Tunnel were constructed of gray limestone block, which appeared to be the same type of stone as used to construct the entrance tower (Figure 1). Beyond Station 1+53, the Elsinore Tunnel appeared to be driven in shale and limestone bedrock of the Kope Formation (Figure 2). Brick was used to line the tunnel in a semi-circular section, from the transition point to the eastern terminus at Station 12+50. Overall, the condition of the brick was good, except for the last 20 feet, where most of the brick was soft and spalled. It was found that a five-foot long piece of rebar was the easiest way to demonstrate the integrity of the brick, much in the same way that concrete pavement or rock in a mine roof is "tested" when struck with a metal bar. Hard, sound brick produces a firm ring when struck, but a dull, hollow sound when spalled or deteriorated. In total, 80 deteriorated bricks were identified and recommended for replacement, mostly near the roof at the eastern end of the tunnel. None of the mortar joints appeared to be deteriorated. Deteriorated brick in the last 20 feet could have been a concern, except for the decision to excavate a temporary access shaft and completely remove that section of the tunnel.

Physical inspection revealed bricks of different color above and below the spring line, possibly indicating that brick from different sources was used. Below the spring line, the color varied from yellowish-brown, to red, to brown, depending on the section of the tunnel. Above the spring line, only red brick was observed.

It was evident from the physical inspection that the motared brick lining was readily susseptible to groundwater seepage. The tunnel started out dry at the access tower; however, "soda straws" and other speleothems began to appear on the ceiling of the "cut and cover" section starting at about Station 1+20. Seepage increased significantly as the transition between the "cut and cover" and rock tunnel section was approached. At this point, the walls of the tunnel were wet, and thick deposits of calcium carbonate "flowstone" were encountered, particularly below the spring line of the tunnel. Groundwater seepage started to diminish after Station 2+00, and was virtually dry beyond Station 3+60 where the tunnel was driven in sound rock.

Groundwater seepage entering the tunnel appeared to pose no threat to the integrity of the structure. The groundwater was mineralized enough to cause staining and to create speleothems inside the tunnel wall, but there was no evidence of degradation to either the brick or the mortar. Since seepage entering the tunnel floor was being managed through a storm drain at the western end, it was not viewed as detrimental.

Published geologic maps of Cincinnati indicated that most of the Elsinore Tunnel was driven in shale and limestone strata of the Kope Formation. This was verified using numerous logs of historical borings from H.C. Nutting Company's archive files of the Eden Park area. While details of the tunnel construction technique remain unknown, core borings performed by the H.C. Nutting Company's Special Drill Crew inside of the Elsinore Tunnel indicated that the nominal thickness of the tunnel lining was four courses of brick, or a thickness of about 15 inches. However, there were no bricks in the floor of the cut-and-cover section, only deteriorated shale.

Borings, and later observations during construction of the access shaft at the eastern side, demonstrated that the brick and stone rubble was used extensively to fill overbreak in the underground section, and to widen the base of the cut and cover section. However, the structural arch remained constant at four bricks thick throughout the tunnel wherever the brick and mortar lining was examined.

Based on all of the inspection and testing work conducted by H.C. Nutting Company personnel, the tunnel was judged to be in good condition and likely able to survive for the next hundred years, except for 20 feet at the eastern end, which would be removed to create the access shaft. Cincinnati Water Works agreed with this conclusion and proceeded with retaining a contractor to replace the two old cast iron water pipes with a single 54-inch-diameter concrete water main. Upgrades recommended by H.C. Nutting Company personnel were to install a concrete floor in the cut-and-cover section, replace the deteriorated bricks, and inspect the tunnel twice a year.

UGH – OH! TROUBLE!

H.C. Nutting Company personnel initially mapped the tunnel, noting its construction materials, seepage features and physical condition. However, at the time the renovation work began, a linear crack was discovered on the south wall of the cut-and-cover section of the tunnel, just above the spring line. H.C. Nutting Company personnel were again called upon to determine the cause of the crack, and to assess long-term stability of the tunnel.

A superintendent from Bowen Engineering Company (the renovation contractor) first discovered the horizontal crack in the tunnel in October 2003. H.C. Nutting personnel examined the tunnel and found that the crack was much longer than what Bowen had observed and that it was on both the north and south walls. Photographs and videotapes were examined to see if the crack might have been missed during the preliminary survey six months earlier. The crack was found to run through paint marks applied during the initial survey and did not appear on any of the photographs, proving that it developed only a short time before Bowen discovered it. Now the questions arose, what caused the crack to form and what should be done about it?

H.C. Nutting Company personnel immediately mobilized to map horizontal cracks in the tunnel wall, and to establish crack monitors so that new movement could be observed. Both of the cracks were approximately 80 feet long and were located entirely within the cut and cover section, immediately below the construction entrance to a new building. The distance between the crown of the tunnel and the surface of the overlying driveway was less than two feet, so it was apparent that movement of trucks and other heavy equipment over the tunnel might have caused the damage.

The H.C. Nutting Company Special Drill Crew was again dispatched to core directly through selected portions of the cracks, so that their shape and depth could be examined. The coring equipment consisted of a concrete core drill, specially adapted for underground use. In virtually every case, the crack was widest at the tunnel wall and completely diminished within two or three courses of brick. From this examination, it was evident that the crack was caused by lateral pressure applied to the exterior of the tunnel lining, resulting in tensile failure of the brick on the interior wall. While it is unlikely that the cracks were caused by any single event, it was concluded that heavily loaded construction traffic crossing over the tunnel throughout the summer months may have compacted soil on both sides of the tunnel wall, increasing lateral pressures on the brick lining to the point that it caused a tensile failure. Generally, the horizontal cracks were positioned at about 2 O'clock and 10 O'clock when viewed looking down the tunnel. The cracks were apparently in response to lateral strains, which exceeded the elastic limit of the brittle masonry liner.

The construction company working above the tunnel was asked to limit traffic and to avoid parking heavily loaded vehicles directly above the tunnel. They were also asked to construct a heavily reinforced concrete slab over the tunnel to avoid the application of repeated heavy loads over the tunnel and surrounding soil. The construction company agreed to cooperate. H.C. Nutting Company personnel observed the crack monitors for about 9 months after they were first installed. No additional movements were recorded in any of the crack monitors during the period of observation that followed.

VIBRATION MONITORING

Because of concerns over construction vibrations at the western end of the tunnel, and construction of the access shaft at the eastern end, H.C. Nutting Company installed seismographs at each end of the tunnel. Geophones were attached to the tunnel ceiling using anchor bolts at Stations 0+70 and 12+20. The seismograph recording units were set in secure locations outside of the tunnel and continuously recorded vibrations. The triggers were set to record time histories when vibration levels exceeded a peak particle velocity of 0.5 inches per second, which is the threshold for damage appropriate to unreinforced masonry structures.

Early in the monitoring program, a number of false or "artifact" readings were recorded, some of which appeared to be related to the daily startup and shutdown of commercial radio transmitters in the area. Based on recommendations from the supplier, the geophones were switched out and all of the underground connections were waterproofed using silicone grease and waterproof housings on the cables. This virtually eliminated the artifact signals and increased reliability of the data. H. C. Nutting Company personnel downloaded the seismograph readings weekly, or whenever the crack monitors were observed.

During the tunnel rehabilitation project, no excessive vibrations were recorded on the seismograph at the eastern end of the tunnel. Two minor exceedances were recorded on the western side of the tunnel, that were related to construction traffic for the adjacent building. No seismic events occurred during the observation period that might have caused structural damage to the tunnel liner.

CONCLUSION

The Elsinore Tunnel has been successfully rehabilitated with a new 54-inch water main, which is expected to remain in service for at least 100 years. The tunnel itself appears to be structurally sound; however, horizontal cracks still remain in the first 100-foot portion of the cut and cover section. H.C.Nutting Company engineers have recommended biannual inspections of the tunnel lining and continued observation of the crack monitors by Cincinnati Water Works personnel.

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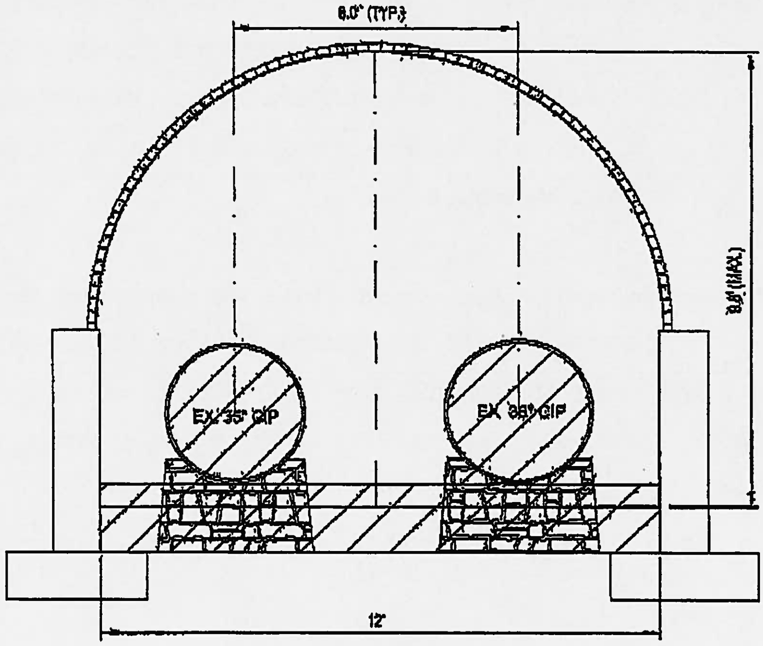

FIGURE 1
TYPICAL TUNNEL SECTION
STA. 0+06 THROUGH 1+53
TUNNEL THROUGH OVERBURDEN.

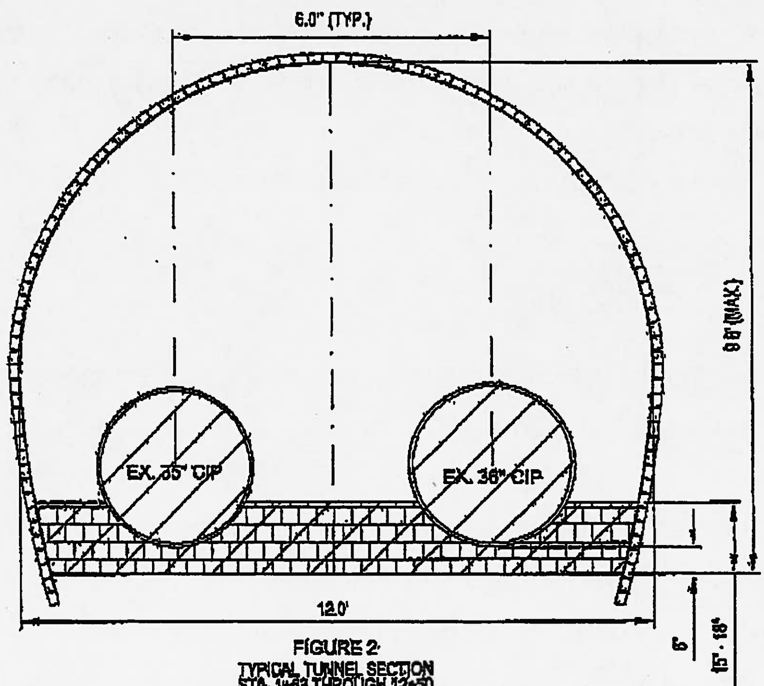

FIGURE 2
TYPICAL TUNNEL SECTION
STA. 1+53 THROUGH 12+50
TUNNEL THROUGH ROCK

H. C. NUTTING COMPANY

BUILDING A TUNNEL TO LEAK

By Peggy H. Duffy[1], Kay Ball[2], Todd Tharpe[3] and D.J. Hagerty[4]

In an innovative project in Louisville, Kentucky, a hardrock tunnel is being built to leak. Water flow will occur, not through discontinuities in the rock, but through wells that will intercept the crown of the tunnel. The water will come from a thick aquifer that lies along and under the Ohio River, and from the river itself. The proposed system will be an application of riverbank filtration.

RIVERBANK FILTRATION—WHAT AND WHY?

Riverbank Filtration (RBF) has been a widely accepted water treatment process in Europe for many years. An aquifer hydraulically connected to a lake or stream is used as the water source instead of the lake or stream. Wells located near the water body typically draw water through sand and gravels, and many contaminants in the surface water are removed. The mechanics of the removal is considered to be similar to what happens in a slow sand filter where physical filtration combines with microbial degradation of organics.

Riverbank filtration is being considered a prime candidate to counter the threat to public health of protozoans such as *Cryptosporidium parvum* and *Giardia lamblia* and other pathogenic microorganisms. National attention was focused on the health effects of these organisms by an outbreak of infection in Milwaukee in 1993 in which 400,000 people were affected, and by outbreaks of related illnesses in ten states. *Cryptosporidium parvum* and *Giardia lamblia* produce oocysts and cysts, respectively. The *Cryptosporidium* oocysts, 3-6 microns in dimension, and the *Giardia lamblia* cysts, 6-8 by 8-14 microns in size, are very resistant to conventional disinfection techniques using chlorine. The oocysts and cysts are produced in the large intestines of affected animals and pass in feces and by other routes into surface waters. Effects on humans include severe diarrhea, stomach cramps, nausea and vomiting lasting for long periods of time. Very severe complications can occur in people with impaired immune systems, and in very young children and the elderly. Because of these effects, increasingly stringent regulations are being proposed and are anticipated to go into effect in the next ten years.

Personnel of the Louisville Water Company have performed substantial research on the effectiveness of RBF for removal of organic material and pathogens, and as a means of satisfying the requirements for removal of *Cryptosporidium* of the proposed EPA Long-Term 2 Surface Water Treatment Rule. Much of the research done thus far worldwide has been directed at the processes by which contaminants are removed at the bank interface in these RBF systems. Little attention has been given to the problem of plugging of the interface when particles and contaminants are removed. The single European study (Schubert 2001) has been supplemented by studies by Hubbs and others (2004) in Louisville, on the details of filtering and microbial interaction at the bank face.

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[1]Pres., Hagerty Engineering, Inc., Jeffersonville, IN [2]Project Engr, Louisville Water Company, Louisville, KY
[3] Project Geol., Jordan, Jones & Goulding, Norcross, GA [4]Prof., CEE Dept., U. Louisville, Louisville, KY

Although the primary focus recently in considering RBF for water supply systems in the United States has been concern about the health effects of protozoans and their cysts, other factors favor RBF. An obvious advantage of RBF is that sediments and contaminants are left in the source surface water body rather than being brought into a treatment facility. In conventional water treatment systems, sediments are removed in grit chambers and sedimentation tanks. Coagulating agents often are added to water to promote flocculation of very small sediment particles and other contaminants that are difficult to remove by sedimentation. Thus, the expense of coagulating agents, the cost of building flocculation tanks and sedimentation basins, and the cost of sludge disposal are eliminated through use of RBF systems. Additionally, drawing water through the aquifer instead of taking it directly from the river will moderate fluctuations in water temperature, and eliminate concerns about infestation of surface water intakes by zebra mussels and Asiatic clams, two common water treatment system "pests."

The Louisville Water Company considered a number of alternative systems for enhancing their supply system and reducing risks associated with pathogenic microorganisms, and they also considered the need to remove disinfection by-products, pesticides and herbicides from the water taken from the Ohio River. The alternative systems evaluated by LWC included use of ultraviolet radiation, enhanced coagulation, use of ozone, adsorption on granulated activated carbon, biological treatment and membrane filtration processes. Factors considered in the evaluation included: removal efficiencies for *Cryptosporidium*, disinfection by-products and synthetic organic compounds; broad-spectrum disinfection capabilities; impacts on the distribution system; and impacts on the nearby community. Some treatments addressed only certain risks (e.g., granulated carbon had no effect on microbial populations), other treatments had detrimental effects on the distribution system (e.g., ozone treatment breaks down organics to feed stocks for microbes) and other treatments were difficult to control (e.g., biological treatment). For a variety of reasons, RBF received the most favorable evaluation as a way to meet future water treatment needs.

PREVIOUS LWC EXPERIENCE

Reliability of an economical potable water supply is the primary concern of the Louisville Water Company, so attention has been directed throughout the history of this project to the issue of reliability of an RBF system. Predicting long-term yield from an underground water source usually begins with site evaluations in which underlying strata are identified and quantified, and samples of subsurface materials are characterized. Monitoring wells typically are installed in borings where subsurface exploration is done. Then, pumping tests are run to estimate aquifer properties such as hydraulic conductivity, transmissivity and specific yield. In riverbank filtration systems, a parameter of prime importance is the leakance or yield of water through the river bed and banks, as a function of the head loss generated to pull water from the river. Computer models based on finite-difference or finite-elements methods of analysis are used to simulate the aquifer, and aquifer parameters are adjusted until predicted values of head match (to some desired level of agreement) heads observed in monitoring wells. Such work has been done at the B.E. Payne plant site of the Louisville Water Company near Prospect, Kentucky.

The design of an RBF system depends on the physical characteristics of the aquifer and the adjacent surface water body from which water will be drawn. The site variables that dominate design include the entrance conditions (the hydraulic characteristics of the bed and banks of the river), the transfer conditions between the river and the well system (the depth and transmissivity of the aquifer itself), and the hydraulics of the flow into the zone around the well itself (well pack and wellscreen dynamics). For a small capacity system (1 to 2 million gallons per day) near a large stream, the flow in most cases would be constrained by the available head loss which depends on the depth of the aquifer, and the head loss through the well pack and wellscreen. For a larger capacity system (five to 20 million gallons per day), flow velocities increase around the well and head losses near the well become large. To reduce such velocities and to increase average gradient, horizontal collectors can be used, and, if possible are positioned in layers of high hydraulic conductivity. A Ranney collector well was installed at the B.E. Payne plant to take advantage of the high potential capacity of a horizontal collector system. For such horizontal collector systems, aquifer transmissity and leakance govern performance, and the Louisville Water Company constructed a Ranney collector well at the Payne plant and began field trials in 1999 to determine reliable estimates of long-term aquifer performance.

Experience with Ranney well—A Ranney collector well consists of a large-diameter vertical shaft from which radial horizontal collectors are constructed through ports in the wall of the vertical shaft at one or more levels. At the Payne plant site, the Ranney well shaft is 16 feet in inside diameter and extends down to rock at a depth of about 107 feet. Seven radial collectors were installed by jacking from the vertical shaft at a depth of 92 feet; one collector extends 200 feet back perpendicular to the river bank and two other collectors, 200 feet long, extend upstream and downstream parallel to the river bank. Four other collectors, 240 feet long, extend under the river bed and are spaced at approximately equal angles (36 degrees) around an arc from upstream to downstream directions, as shown in Figure 1. The Ranney well system was designed for a capacity between 15 and 20 million gallons per day. A wealth of information was collected during a monitoring period from August 1999 through May 2000.

Most, but not all of the monitoring information confirmed the excellent potential of RBF as a water treatment technique. Turbidity in the Ohio River varies seasonally between about 3 Nephlometric Turbidity Units (NTU) at the end of long periods of slow flow to over 700 NTU during floods; during the monitoring program, NTU values ranged from 0.07 to 0.4 in water drawn from horizontal collector #4 under the river. Total coliform levels in the Ohio River exceed 5,000 in plate counts for much of the year and can reach levels of 25,000 or more; during the monitoring period, total coliform removal was virtually 100 percent in the RBF system. During the monitoring period, atrazine levels in the river reached as high as 3 micrograms per liter but atrazine levels in the RBF system water were below the detection limit of 0.1 micrograms per liter. 2-methylisoborneol (MIB) levels in the river fluctuated between ten and 120 nanograms per liter, but were approximately five nanograms per liter in the RBF outflow. MIB is an indicator used to identify the presence of cyanobacteria. The only downside to the performance results was an apparent decrease in leakance shown by decreased yield at constant drawdown.

Decrease in leakance— Because leakance through the river bed and bank appeared to decrease with time, field testing was extended through 2002. Six long-term pumping tests were done at intervals during the monitoring period from September 1999 through June 2002. For a constant drawdown in the pumped well, the yield decreased to about

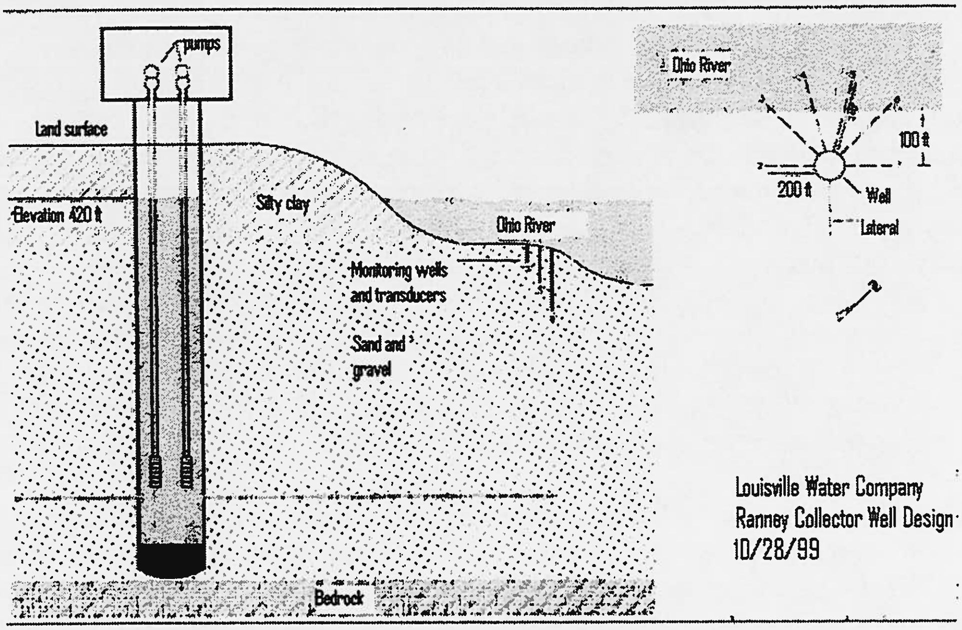

Figure 1. The Ranney collector well, Phase I, LWC Payne Plant RBF System.

85 percent of the original yield between September 1999 and March 2000, and dropped to about 70 percent of the original yield in September 2000 and March 2001. LWC began annual surveys of the piezometric head in the aquifer, in conjunction with the United States Geological Survey, in the vicinity of the collector well, beginning with the start of pumping from the collector well in 1999. Data from those investigations are consistent with a decrease in leakance through the river bed and bank. Piezometer readings and evaluation of the March 2002 pumping test indicated a slight increase in leakance after a bank-full flood in that month, but the latest pumping tests indicated the yield had become relatively constant at about 2/3 of the original yield.

To supplement the pumping test data, pressure-temperature probes were buried under the river bed at depths of two, five and ten feet under the surface, at a location approximately 100 feet from the bank. In August 2002, additional probes were installed at a depth of five feet in the river bed along a line perpendicular to the bank, at distances of 400, 650 and 950 feet from the bank. Piezometric head readings have been obtained from those probes weekly since they were installed. Preliminary investigations indicated the source of the reduction in yield is plugging of the river bed and bank in a relatively shallow zone (less than one foot thick) at the surface of the aquifer. The diver employed to install the probes under the river bed reported that a shallow crust or stiff zone existed at the surface of the river bed and extended as much as 400 feet from the bank; the extent of the stiff zone roughly paralleled the intersection of the piezometric surface with the river bed. This crust resists penetration by a diver's knife, but tends to disintegrate when samples are brought out of the river. When the bed was disturbed by the diver, a cloud of sediment formed in the water but was soon pulled down into the hole where the bed sample had been removed. Drillers who installed monitoring wells in the same area where the divers worked also reported higher penetration resistance in a shallow zone near the river bed surface. Similar phenomena have been noted in the Rhine River bed at Dusseldorf after dredging near the RBF system there (Schubert, 2001).

Several hypotheses have been put forward to account for the decrease in leakance across the river bed:

1. The change in leakance is a result of compaction of the bed materials under the head difference caused by the depth of water above the bed and the lowered water pressure caused by pumping in the aquifer, with a resultant decrease in hydraulic conductivity;

2. The action of drawing water into the bed has caused fine sediment particles to penetrate into interstices between sands in the river bed, decreasing the hydraulic conductivity in the shallow zone of fines penetration;

3. Microbes have formed a slime zone on the river bed and have caused a sharp decrease in hydraulic conductivity there; and

4. All three of the previously listed mechanisms are operating to some degree.

The authors feel that hypothesis 2 is the most reasonable, but a slight decrease in void ratio as a result of compression could reduce hydraulic conductivity slightly. In any event, as river bed leakance decreases, drawdown required for constant yield increases, and the radius of influence of the well system increases. With increase in zone of influence, approach velocities induced at the river bed decrease. Also, scouring effects of river currents become more important as the mechanisms for plugging decrease in effect with distance from the pumped well. At some point, equilibrium is reached. In some river systems, a plugged zone apparently has formed across the entire width of the river (e.g., the lower Hudson River, the Rhine River at Cologne, and intermittently the LLobregat River at Barcelona (Hubbs 2004)). Detailed investigations are being pursued currently to identify the mechanism(s) by which plugging is occurring and to evaluate the relative contributions of scour and plugging mechanisms to the equilibrium state. Questions remaining to be answered include determination of conditions within and just under the stiff zone (fines content, microbial activity, degree of saturation, etc.)

Expansion of the System—Because of the favorable results of the field trials of RBF at the Payne plant site, a decision was made to expand the capacity of the system. The Payne plant has a capacity of 60 million gallons per day (mgd). The intended capacity of the Ranney well was 20 mgd; a decision was made to increase the capacity of the RBF system to 45 million gallons per day. Design of the expanded system included consideration of several alternatives, including a soft-ground tunnel equipped with horizontal collectors pushed out under the river bed, a hard-rock tunnel connecting three widely spaced Ranney collector wells (including the original well), and a hard-rock tunnel into which a large number of vertical collector wells would feed water. The final design features the third alternative. That choice was influenced by the sequence and character of the geological materials at the site, as well as considerations of well hydraulics and project economics.

CONDITIONS ON THE SITE

The topography at the site includes an upper area on which the Payne plant is situated, at an elevation of about 460 feet above Mean Sea Level, and a lower area adjacent to the Ohio River, at an elevation of about 435 feet. The existing Ranney well and raw water intake structures were built on the lower area. The level of the river is controlled to facilitate navigation and is maintained at 420 feet or higher.

Area geologic setting--Sedimentary rock strata of Silurian and Ordovician age dominate the structural geology of the Payne plant site. During the Ordovician Period, limestones were deposited in moderately deep seas throughout most of Kentucky. The marine environment changed in Late Ordovician times, and shales and shaley limestones were formed, often containing abundant fossils. In Silurian times, thick limestones were deposited over the shales and shaley limestones of the Ordovician. Fossiliferous limestones and thick shale beds were deposited during the Devonian age, but the shales and all later rock layers have been removed by erosion at the site. The strata in which the leaky tunnel will be built include the Saluda Dolomite, the Bardstown Member and the Rowland Member, all parts of the Drakes Formation of Ordovician age. Access shafts to the tunnel will be built through younger Ordovician strata and through overlying limestones of Silurian age.

During the Pleistocene Epoch, streams along the front of the ice sheets were incised deeply because of the drop in sea level. Several cycles of glacial advance and retreat occurred. When the ice sheets melted, incised streams were filled with thick layers of alluvium by melt-water flows much larger in volume than present-day discharges. The Ohio River at the Payne plant flows in a bedrock valley that is about 30 meters deep. The valley was filled and eroded sequentially, in at least eight stages, with the result that three terraces are present in the valley-fill materials at the plant site. The third, or highest terrace, consists of sediments deposited across the full width of the bedrock valley that was cut in the first stage. The third terrace was cut by renewed erosion, and the bedrock valley was deepened. Sand and gravel was deposited over the third terrace, with fine sediments in the narrow channel of the contemporary river. The sand and gravel form a second terrace, eroded but not penetrated by subsequent erosion. The first terrace sediments were deposited in the narrow channel that was formed during that erosion. The first terrace deposits were subsequently eroded, forming the terrace. Construction of the navigation dams on the river permanently flooded the first terrace in the 1930s.

Bedrock geology-- Most of the formations of interest to this project do not outcrop near the Payne plant site, but appear in highway cuts and in a few quarries to the east and southeast of the plant site. In general, the bedrock layers consist of thick sequences of fossiliferous shaley limestone and calcareous shale, with thin inclusions of limestone and shale. The rock layers are nearly horizontal, with dips that vary but average about five degrees to the west. Infrequent high-angle secondary discontinuities cut across the flat-lying strata, although tight and healed joints are present in all the layers. Stratification and indicator fossil beds allowed approximate correlation between information obtained during exploration at the Payne plant site, and data obtained by examination of outcrops in quarries and highway cuts, and information contained in published literature. The nearly horizontal strata are defined by tight or closed bedding planes. Some bedding planes are filled with fine-grained products of shale decomposition. The Ordovician strata below the plant site generally are tight and yield little groundwater. However, the Silurian strata that form the valley walls around the aquifer are cut by solution cavities, especially where stress relief caused joints to widen.

Subsurface exploration--The preliminary design for the RBF system included a horizontal tunnel equipped with 200-foot long lateral collectors installed at a spacing of 60 feet, under the bed of the river. The tunnel alignment followed the riverbank closely. Construction was to begin at the upstream end of the tunnel, which was to be approximately 1,500 feet long, with a recovery shaft at the downstream end. Previous exploration on the site and published information indicated a general sequence of alluvial strata over glaciofluvial outwash layers. Upper layers, beginning at an elevation of about 435 ft MSL consist of thinly bedded silty sands, silts and silty clays. At a depth of about 35 to 40 feet, the recent alluvial layers are underlain by a deep sequence of coarse sediments that range from medium sand to large gravel, with occasional lenses of cobbles and some boulders.

To investigate the feasibility of a soft-ground tunnel, auger borings were made in April 2000 at eight locations spaced evenly along the tunnel alignment. Two core borings were made along the tunnel alignment to characterize the alluvium and to investigate bedrock conditions. Standard Penetration Tests were done in the two core borings, from about 63 feet below ground surface to the top of rock, and disturbed samples from one of those borings were used for grain size analysis. Large-diameter bucket augers were used by Reynolds Environmental Drilling Company to advance the eight borings to the bedrock surface. A 36 inch-diameter auger was used to allow collection of small boulders and cobbles. Sieve analyses were done on samples obtained from the auger borings, for the range in size from silt (passing the No. 200 sieve) to boulders more than 12 inches (30 cm) in dimension. The five-foot sample intervals extended from about 60 feet below the ground surface to the top of rock. Gravel, cobbles and boulders were sieved and weighed in the field; sand and fines were sieved in a laboratory.

SOFT-GROUND TUNNEL

The soft-ground tunnel was proposed to be made with invert elevations of 339 ft at the northern end of the tunnel and 336 feet at the southern end (to facilitate drainage during tunnel maintenance). The tunnel would be 14 feet in outer diameter, with a finished inner diameter of 12 feet. Two 48-inch diameter carrier pipes would be installed in the bottom of the tunnel. Lateral or well maintenance was a major consideration in tunnel design. Screened wells and laterals will plug as a result of buildup of mineral deposits. In Ranney wells, laterals have been cleaned chemically by taking the entire system out of service. A rotating hydrojet is used to inject water at high pressure into a lateral; the jet cuts encrustations off the screen and breaks up the sand pack outside the screen. Pipe elbows and valves allow access to each lateral. A similar operation could be done in the soft-ground tunnel for the Payne plant, with either the entire tunnel taken out of service, or only half the tunnel cleaned at one time (requiring additional bulkheads, etc.). Alternately, a wet-dry system could be used, with pipes carrying the water inside a tunnel to which access would be obtained by a separate vertical shaft within the pump station shaft. This latter option was considered most practicable for the soft-ground tunnel.

The tunnel would be bored mainly through sand and gravel, but the tunnel crown would encounter cobbles over much of the length of the tunnel. Tunnel excavation would require hydrostatic balance to prevent sands from flowing into the tunnel. Either a slurry shield machine (SSM) or an earth-pressure balance machine (EPBM) would be required. A closed-face cutterhead with drag bits or picks would be used. The SSM would have to be fitted with a jaw crusher and/or disk cutters, and disk cutters would likely be required on the EPBM. Precast concrete segments would be used as ground support and final lining, in a one-pass construction sequence. The segments would be bolted and equipped with gaskets to minimize leaking through joints. The segments also would provide the reaction against which the tunneling machine would be propelled forward. Ports would be precast into the lining segments to allow lateral installation (alternately, a contractor could elect to drill through the lining). The TBM would be launched through a shaft approximately 32 feet in diameter, drilled at the north end of the tunnel. The large diameter would be required later when the shaft would function as the wet well for the system a the pump station, which could be expanded to a capacity of 120 mgd. The recovery shaft at the south end of the tunnel would be only 19 feet in diameter. The shafts would be drilled to the top of rock and sealed with concrete, and then pumped dry prior to tunnel excavation. Laterals would be jacked by means of a frame assembled in the tunnel. After a casing was installed for the full 200-ft length of the lateral, a slotted well screen would be installed inside the casing and the casing pipe would be withdrawn.

Evaluation of the results of the first stage of exploration indicated that a soft-ground tunnel would be constrained by the presence of zones of cobbles in the alluvium. Figure 2 shows the grain size analysis results from the auger boring samples.

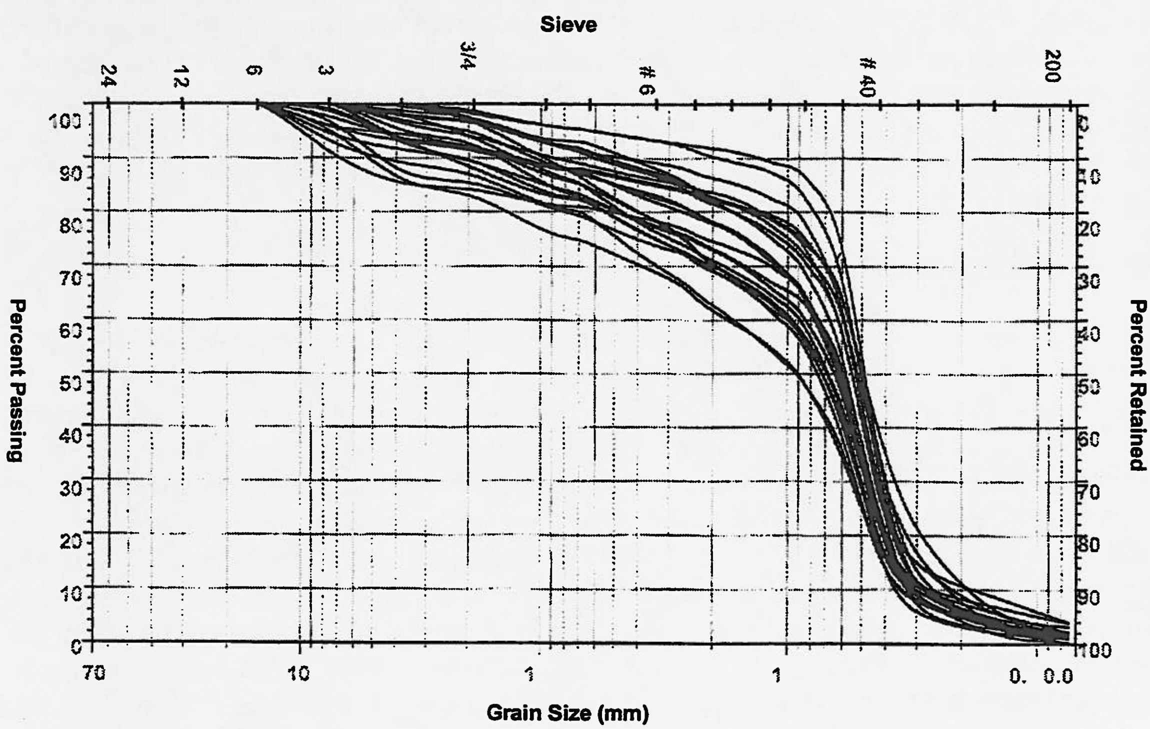

Figure 2. Grain Size Analysis results, auger boring samples, Payne plant site.

BEDROCK TUNNEL 1

In 2000, a preliminary design for a bedrock tunnel also was developed. A pump station would be built, with a raw water transmission line aligned between two sludge lagoons on the second terrace, and the tunnel, parallel to the river bank, would connect the pump station to two Ranney wells at the ends of the tunnel. The tunnel was proposed to be 800 feet long, so that the three Ranney wells at the plant site would be spaced about 800 feet from each other. Lateral collectors would be installed in the Ranney wells as had been done in the original Ranney well. Computer models were used to predict safe yields from the aquifer and to optimize placement of the laterals. Ten or eleven laterals were recommended for each Ranney well. Preliminary tunnel design was based on the results of the two rock-core borings mentioned previously.

Rock core borings—In the April 2000 borings, bedrock cores were taken using a CME-95 drill rig. Coring was done using triple tube methods. Augers were extended only to top of bedrock (about 110 ft below surface). Rock Quality Designation (RQD) was determined as the cores were obtained and noted in the logs. Complete core logs were prepared after all cores were collected and boxed. Core recovery ranged from 97 percent to 101 percent (cored material expanded slightly upon exposure). RQD values ranged from 74 percent to 100 percent in Core Boring 1 and from 64 percent to 100 percent in Core Boring 2. The lowest values of RQD were obtained in alternating shaley limestones and limey shales between depths of about 116 feet and about 125 feet. Between depths of about 125 feet and 165 feet, RQD values were high, between 96 percent and 100 percent. Several clay shale layers deteriorated severely during the coring process, particularly between depths of about 118 feet and 125 feet in Core Boring 1. Evaluation of bedrock cores indicated the rock units on the whole are competent with infrequent fractures, lineaments, or weathering.

Other experience—A thin, coquinoidal limestone encountered in the borings contains small voids that could hold water, but overlying shales and shaley limestones are very low in hydraulic conductivity. An underground ramp was constructed near an existing quarry in eastern Jefferson County by blasting through the same Ordovician rock units that are present under the Payne plant site: the Saluda Dolomite, the Bardstown Member and the Rowland Member (of the Drakes Formation). No significant seepage was experienced in the unlined ramp opening during construction even though water was ponded tens of feet deep in surface workings above the ramp construction level, and the ramp was advanced by blasting. Moreover, no significant instability occurred in the roof or walls of the ramp opening, even though the ramp was constructed at a slope of about 19 percent. The rock layers are almost horizontal at the quarry location as at the Payne plant site, so the ramp excavation would have left leading edges of rock strata in place in the roof of the opening as the ramp was advanced. Only widely spaced, relatively short rock bolts were required to provide a stable roof for the ramp. Excavation was rapidly done, and it was not difficult to break the rock to relatively smooth faces in the thinly bedded calcareous shales and shaley limestones of the Drakes Formation.

Evaluation of conditions relative to tunneling--The sedimentary rock layers encountered in the two core borings at the Payne plant appeared to have very low effective porosity (the volume of connected pores in a given rock element divided by the total volume of that element). Examination of the rock cores was supplemented by observation of numerous outcrops where the cored rock strata are exposed. The rock layers in which the tunnel would be most likely to be built consist of limey shales, shaley limestones and fossiliferous limestones. In neither core nor in numerous outcrops were solution features observed in these rock units. The rock layers in the members of the Drakes Formation are relatively thin, and the presence of shale or shaley limestone above and below relatively thin layers of fossiliferous limestone apparently has prevented the formation of solution cavities in the fossiliferous limestones by limiting groundwater flow through the rock masses. Joints were not detected in the rock cores taken at the Payne plant. In outcrops of the Rowland Member and in the upper half of the Saluda Dolomite, joints were relatively widely spaced (one meter or more between joints) and narrow (joint openings of one centimeter or less); apparent joint width certainly was influenced by rock disturbance during blasting of the rock masses where outcrops were examined.

Bedrock tunnel design—The invert of the bedrock tunnel was set at elevation 290 feet, based primarily on the quality of the rock. The pump station shaft would have to be 150 feet deep. Two excavation methods were evaluated: drilling and blasting; or a tunnel boring machine. If drilling and blasting were used, excavation would proceed from the pump station shaft toward the ends of the tunnel to the Ranney wells. The tunnel was to have a horseshoe shape, with a minimum excavated diameter for a circular area of eight feet. Excavated materials would be removed via the central pump station shaft. Drilling and blasting is the most common excavation method in the limestones and shales found in the area of Jefferson County, and a number of large surface and underground quarries have been mined successfully using drilling and blasting. However, citizen resistance to drilling and blasting is strong, and has forced closure of at least one underground quarry. Alternatively, the tunnel would be excavated using a TBM, beginning at one of the Ranney well locations and excavation would extend through the central pump station shaft to the other end of the tunnel. A circular tunnel eight feet in diameter was planned. If drilling and blasting was used, shotcrete would be applied to the walls of the tunnel as excavation occurred. Shotcrete typically is not applied in conjunction with use of a TBM, but such a lining could be required for hydraulic purposes in this tunnel. It was not expected that a cast-in-place concrete lining would be required.

Shaft design—If the tunnel were built by drilling and blasting, the pump station shaft was proposed to be 24 feet in diameter in rock, but was to be 32 feet in diameter in soil to provide for future expansion of the facility to a capacity of 120 mgd. The two Ranney wells also would be 19 feet in diameter if drill-and-blast methods were used for the excavation. Laterals would then be spaced at 4 feet, a minimum spacing considered required for structural integrity of the shaft. A four foot-diameter shaft would connect the Ranney wells, excavated down to rock, to the underlying tunnel. If a TBM were used for excavation, one Ranney well would be 32 feet in diameter and extend down into the rock to allow launch of the TBM. The tunnel would pass through the central pump station shaft and on to the recovery shaft, which would be 19 feet in diameter. The pump station shaft would be 32 feet in diameter in the soil, and 24 feet in diameter in the rock.

Laterals—Ten or eleven laterals would be installed in each Ranney well, with six laterals installed toward the river, two laterals parallel to the river bank, and two or three laterals installed away from the river. Laterals would consist of 12 inch-diameter slotted well screen pipe installed through precast holes in the shafts. If drilling and blasting were done, lateral installation would be delayed until after all rock removal; if a TBM were used, laterals would be installed before the bottoms of the shafts were excavated down to the tunnel. Laterals would be maintained by taking an entire Ranney well out of service and treating all the laterals in that well. More complex and costly arrangements would be required for isolating a Ranney well if a TBM were used, because of the large-diameter connection between each shaft and the tunnel.

COMPARISON OF ALTERNATIVES

When the preliminary designs for the soft-ground tunnel and the bedrock tunnel were prepared, one of the objectives was to develop designs and methods that could be used in a larger RBF project near the Zorn Avenue raw water pumping station for the Crescent Hill Plant of LWC. Two primary questions had to be answered: Could a tunnel be excavated in the alluvium along the Ohio River, and could laterals be installed from such a tunnel safely? For a bedrock tunnel, the quality of the rock relative to tunnel construction was the primary issue.

No boulders or zones of cobbles, or bedrock ridges or pinnacles were encountered in the exploration. Cobbles and boulders in the lower portions of the alluvium, in zones of irregular thickness and elevation, would be very difficult to accommodate with a tunnel boring machine. An unexpected bedrock ridge would be very problematic for a soft-ground TBM. Laterals have not been installed from a tunnel, and the effect of jacking forces on a tunnel lining, especially the gaskets between segments of the lining, was an issue of concern. However, contractors with experience in building Ranney wells indicated that laterals could be installed safely from a tunnel, but that the more difficult working conditions would make the installation cost about 50 percent higher than costs for lateral installation in a Ranney well. Experience gained during construction of a soft-ground tunnel at the Payne plant site could be used to reduce contingency cost allowances if a larger tunnel were to be built at the Zorn Avenue site, if geological conditions there are similar.

With regard to a tunnel in bedrock, the two core borings and examinations of outcrops, plus experience in excavations in the same rock units, indicated that the rock conditions were eminently suitable. Both drill-and-blast methods and use of a TBM are proven technologies in similar rock conditions throughout the world. Moreover, installation of laterals from within a Ranney well is proven technology. Construction of a bedrock tunnel at the Zorn Avenue location also would be feasible, if the rock quality there was equivalent to that at the Payne plant site, but a TBM would be more economical than drilling and blasting for a long tunnel.

Costs estimates showed that the cost for a soft-ground tunnel, with a predicted safe yield of 44 to 61 mgd, was about $ 24 million, almost double the $ 12.5 million estimated cost for a tunnel in bedrock, with a safe yield of 37 to 51 mgd from several Ranney wells along the length of the tunnel. Most of the difference in estimated cost was due to the anticipated high cost of obtaining a soft-ground TBM, the high cost of precast concrete lining, and the higher cost of lateral installation (in a tunnel rather than in Ranney wells). Lateral maintenance could be done one lateral at a time with the soft-ground tunnel, whereas the bedrock tunnel system would require that an entire Ranney well be taken out of service when any laterals were serviced. Because of the large difference in cost estimates, a third option was investigated: constructing several Ranney wells, each with its own above-grade pump station. This option would be likely to meet considerable opposition from the local community who had been very concerned with the aesthetic effects of the original Ranney collector well on the scenic quality of the area. Based on the cost of building the original Ranney well in 1997, the cost for two more wells, with capacities of 30 mgd each, was estimated at about $ 11.6 million. However, two pump stations would be required in the last system, with associated higher operation and maintenance costs than for either of the tunnel options.

A thorough consideration of the factors listed in the preceding paragraph led to a recommendation that a bedrock tunnel be designed and built. Tunnel excavation was recommended to be done by drilling and blasting, because drilling and blasting would not require mobilization of TBM equipment and the Ranney well shafts would not extend into rock as would be required with a TBM. Additionally, with drilling and blasting, only a four foot-diameter shaft would connect the Ranney wells with the tunnel, leaving an area at the bottoms of the shafts adequate for lateral maintenance activities.

LWC personnel chose use of a TBM to avoid community impacts from drilling and blasting activities and commissioned further studies to improve the capacity and effectiveness of the RBF system. Additional pumping tests were approved, as was additional design work and subsurface exploration. The alignment of the tunnel was changed, and Ranney wells were abandoned in favor of vertical collector wells.

BEDROCK TUNNEL WITH VERTICAL COLLECTOR WELLS

After a decision was made to pursue a tunnel in bedrock, much more site evaluation was done. That evaluation included pumping tests in 2002, limited subsurface exploration on the second terrace, and much more extensive subsurface exploration in the third terrace and in the Silurian rock strata that underlie the third terrace. The alignment of the tunnel was changed significantly, as was the profile. The original bedrock tunnel was to be located on a straight line parallel to the river bank, with an invert elevation of 290 feet, and a length of 800 feet. The revised tunnel alignment will follow the river bank downstream but will extend well beyond the boundaries of the LWC property at the Payne plant, almost to Harrods Creek. The tunnel will be 8,415 feet long and ten feet in finished diameter, with an excavated diameter of about 12 feet. For design and construction purposes, the tunnel has been assigned stations, with the downstream end near Station 1 + 50 and the upstream end near Station 86 + 50. Near the upstream end of the tunnel, the alignment turns to the southeast, away from the river.

Subsurface exploration and testing—In June of 2002, two borings were drilled with 4.25 inch-diameter hollow stem augers to auger refusal for the purpose of investigating the alluvial deposits of the third terrace. Soil samples were collected by means of a split spoon from the ground surface to the phreatic surface in one boring; below the phreatic surface, a bailer was used to collect samples. That boring was located about 1,800 feet southeast of the existing Payne water treatment plant. Water was encountered at a depth of 31 feet, or at about elevation 430 feet. From the ground surface down to a depth of about 30 feet, the soils consisted of layers of fine-grained sand and silt. The bailer recovered only sands and gravels below the phreatic surface down to the top of rock at a depth of about 52 feet, or an elevation of about 408 feet. The rock surface there was about 90 feet higher than the rock surface near the river bank. The second boring was located about 600 feet north of the water treatment plant, near the edge of the third terrace. Groundwater was not encountered in that boring, so only a split spoon was used to obtain soil samples. Silt and fine sand layers were found to a depth of about 26 feet where some gravel was found with silty sand, extending down to auger refusal at a depth of about 37 feet, or at an elevation of 421 feet, about 100 feet higher than the elevation of bedrock under the river bank.

Three borings were drilled in June 2002 along the proposed new tunnel alignment to be used as production wells in pumping tests. One boring was advanced near the downstream end of the proposed alignment, a second boring was conducted near Station 21 + 40, and the third boring was performed near Station 42 + 00 but at a distance of about 600 feet east-southeast from the alignment. Temporary wells were constructed in these borings. Drill cuttings were visually examined and field classifications were logged. The alluvial soils found in these borings were very similar to those found in earlier exploration: clayey silts and silty clays near the ground surface, underlain by silts and fine sands at depths of about 35 feet, underlain by sands and gravelly sands to auger refusal at depths between 92 and 103 feet below ground surface.

Seven borings were drilled with 4.25 inch-diameter hollow stem augers to provide access for installation of piezometers around the three locations of pump wells listed in the preceding paragraph. Four of the piezometers were installed in June of 2002, and the other devices were installed in October and November of 2002. Soil cuttings were visually examined and logged during drilling, and Standard Penetration Test values were obtained between the ground surface and the phreatic surface. Typically, SPT values averaged about 20 blows per foot in the upper ten feet of depth, but decreased to an average of about 5 blows per foot at a depth of about 15 feet. Below a depth of about 25 feet, SPT values increased to values in the mid-twenties when clean coarse sands or gravelly sands were encountered. The piezometers were constructed with two inch-diameter PVC casing, with well screens 80 feet long. Bentonite was used to seal the annular spaces in the borings from the top of the well screens to the ground surface.

When redesign efforts indicated that the pump station for the revised tunnel system would be located at the north end of the tunnel near the water treatment plant, six additional borings were drilled within the footprint of the station and one boring was drilled in the roadway between sludge lagoons, in May of 2003. In these borings, soil samples were secured from the ground surface to full depth with a split spoon inside 4.25 inch-diameter hollow stem augers. Conditions found in these borings were very similar to those found in the seven borings done in June through November of 2002, and in earlier borings. Refusal was encountered at a depth of 117 feet in the boring done in the road between sludge lagoons, on the second terrace, and at depths of 89 and 97 feet in two other borings carried to refusal near the pump station location on the third terrace. The other four borings were done to a depth of 50 feet.

Design changes made it necessary to relocate the pump station, and four borings were drilled at the new location, in November of 2003. Three of the borings were done within the footprint of the relocated station, and the fourth boring was done near the proposed location of a tunnel construction shaft. All borings were drilled with 4.25 inch-diameter hollow stem augers, to auger refusal. Soil was sampled continuously to refusal depth with a split spoon, samples were examined, and field classifications were logged. Standard Penetration Tests were done in both sets of borings for design of the pump station foundations. Typically, blow counts varied from a low value of 4 blows per foot in one boring to more than 50 blows per foot in gravelly sands in a number of the borings.

Pumping tests— Three pumping tests were conducted on July 10, 2002, July 12, 2002 and November 6, 2002, to estimate further the potential yield of the alluvial aquifer. The tests were done for 24 hours, and aquifer recovery data were collected until groundwater levels had recovered to within 10 percent of the levels recorded prior to pumping. Pressure transducers, connected to a data logger, were installed in each pumping well and piezometer. Background water pressure (level) data were collected every hour for 24 hours prior to the start of pumping. Then, the production wells were pumped at a constant rate of about 1,600 gallons per minute for 24 hours. Calculated transmissivity ranged from 267,000 gallons per day per foot in November to 281,000 gallons per day per foot on July 12 to 309,000 gallons per day per foot on July 10. Three difference locations were pumped in these tests. Storage coefficient was estimated to range from 4.3E-4 to 5.4E-4, for the three locations.

Bedrock exploration—Cores were obtained in the bedrock at seven locations along the tunnel alignment, in addition to the cores that were obtained during the first phase of the design study. Six more cores were obtained in borings done within the footprints of the two proposed locations for the new pump station. Two borings were drilled on the south side of Harrods Creek, to investigate the feasibility of extending the RBF system downstream. The first boring encountered natural gas under pressure, and the second boring was done to obtain additional data on that gas.

Coring was done using a Longyear HQ triple-tube system that produces a 3.78 inch-diameter hole and a 2.5 inch-diameter core. The triple-tube system is intended to reduce core disturbance during retrieval and handling. To the extent practical, each piece of core was marked for depth and uphole direction. Cores were logged preliminarily and described during field operations. All cores then were compared and evaluated in comparison to conditions exposed in outcrops at other locations. Rock description, core recovery and RQD were obtained for each core run. Typical values of recovery and RQD are shown in Table 1, for some of the core borings, including the original two core borings, B1 and B2, and Table 2 shows a summary of RQD data.

Packer hydraulic conductivity tests were done in four of the core borings made along the tunnel alignment. A single packer was set in the borehole several feet below the bottom of the casing. Nitrogen was used to inflate the packer to seal off the tested

Table 1. Recovery and RQD Values

Brg	Top	Bottm	Recy %	RQD, %	Brg	Top	Bottm	Recy %	RQD, %
B1	110.0	111.1	100	100	S1	97.5	102.5	24	0
	111.1	116.1	99	85		109.1	119.1	100	82
	116.1	126.4	100	72		119.1	129.1	100	96
	126.4	136.4	100	100		129.1	139.1	100	78
	136.4	146.6	100	96		139.1	149.1	100	93
	146.6	156.5	100	98		149.1	159.1	100	89
	156.5	166.4	100	100		159.1	169.1	100	83
	166.4	172.0	92	84		169.1	179.1	100	70
B2	110.0	116.6	97	83		179.1	189.1	100	85
	116.6	126.5	98	65		189.1	195.1	100	90
	126.5	136.6	100	99	F3	73.3	83.3	80	32
	136.6	146.6	100	96	F4	40.5	50.1	50	13
	146.6	156.6	99	99		50.1	60.0	91	44
	156.6	166.6	100	100		60.0	70.0	36	14
	166.6	176.6	100	100		70.0	80.0	100	60
B8	100.0	106.0	100	50		80.0	90.0	100	100
	106.0	116.0	100	97		90.0	100.0	100	97
	116.0	120.0	98	37		100.0	110.0	100	100
	120.0	126.0	98	61		110.0	120.0	100	100
	126.0	135.4	100	85		120.0	130.0	100	100
	135.4	145.7	100	75		130.0	140.0	100	100
	145.7	155.7	100	61		140.0	150.0	100	100
	155.7	165.9	95	88		150.0	160.0	100	97
	165.9	176.0	97	87		160.0	170.0	100	100
	176.0	186.0	100	90		170.0	180.0	100	100
	186.0	196.0	100	98					

Table 2. Summary of RQD Values

RQD	Counts	Percentage
>90 – 100	41	46.1
>75 – 90	19	21.3
>50 – 75	11	12.4
>25 – 50	4	4.5
0 – 25	0	0

interval of rock from the rest of the borehole. Data collected during the test include the volume of water per unit time injected into the rock under constant pressure. Values of hydraulic conductivity obtained in the tests are shown in Table 3. Values for Borings 3 and 4 are averages of two tests. In Boring 6, the second packer test was abandoned because of the presence of natural gas under high pressure in the boring.

Table 3. Packer Test Data

Boring	Top, tested interval, ft bgs	Bottom, tested interval, ft bgs	Hydraulic Conductivity, cm/s
B3	134.4	194.2	1.3×10^{-7}
B4	149.4	194.2	1.8×10^{-6}
B5	134.4	194.7	7.1×10^{-6}
B5	144.4	194.7	2.5×10^{-6}
B6	134.4	204.5	0.0

Gas under pressure was discovered in a boring advanced south of the mouth of Harrods Creek, at an elevation of about 320 feet, about 115 feet below ground surface. At first encounter, the gas was hardly noticed, but as the coring was continued to the desired depth of 204.5 feet, the pressure gradually increased. The gas was thought to be in a small pocket, and the boring was vented for three days; however, when the boring was closed after that time, the gas pressure quickly built back to its original level. That hole was completely grouted, and a second boring was done about 40 feet from the first boring, to determine more accurately the extent of the gas. Gas was encountered in the second boring during coring at a depth of 170 feet; rock coring was terminated at a depth of 193 feet. A hook-wall mechanical packer was set in the hole at a depth of about 120 feet below ground surface, and connected to AQ rods that extended about ground surface. The annulus between the rods and the casing was grouted with neat cement. The initial gas pressure was 112 psia. During four days of free venting through an orifice 1/8 inch in diameter, the flow rate varied but averaged about 19.5 thousand cubic feet per day. When the well head valve was closed, the pressure built back to over 100 psia within six minutes, and then gradually increased to a steady state at 125 psia after 470 minutes (0.3 days). The steady state pressure is equivalent to a water head greater than the depth to the producing zone; the sustained flow and high pressure suggested that a fracture in the rock is connected to a deep reservoir. A chemical analysis of gas samples showed primarily methane, with trace amounts of ethane, propane, pentane and butane.

Borehole geophysical logging was done in two borings along the south end of the tunnel alignment, and in the second borings on the left bank of Harrods Creek, downstream from the end of the tunnel. A Digital Acoustic Televiewer (DATV) was used to obtain an oriented representation of the borehole using high-resolution sound waves. Amplitude and travel time for an emitted acoustic signal are recorded. This information can be used to measure the strike, dip, depth and width of each joint intercepted by the borehole. No clear borehole fluid is needed because the DATV uses high-resolution sound waves. At each sample interval of 0.02 feet, 256 measurements are made around the borehole perimeter. From the collected data, the orientation of planar features in the rock can be determined. A Multi-Parameter electrical resistivity tool was used to investigate the local physical condition of the rock in a sample interval, and the hydraulic characteristics of the rock. Natural gamma radiation, spontaneous resistivity, fluid resistivity, 16-inch short-normal resistivity, 64-inch long-normal resistivity, lateral resistivity and temperature are obtained.

Rock core samples were selected from four of the borings made along the tunnel alignment and sent to the Earth Mechanics Institute of the Colorado School of Mines for testing. Uniaxial compressive strength (UCS), Brazilian tensile strength (BTS) and Cerchar Abrasivity Index (CAI) were determined for those samples, with the results shown in Table 4 and Table 5. The CAI is a combined measure of rock abrasivity and strength, and has been correlated to cutter wear and expected linear feet of cutter travel to allow projections of cutter costs per rock volume or linear foot of tunnel. A value of 1 is low for CAI, while a value of 6 is extremely abrasive.

Table 4. Rock Core Sample Test Results

Sample	UCS		Comments
B3-169-SH	6,381 psi	44 MPa	Non-structural failure
B3-136-LS	13,477 psi	93 MPa	Non-structural failure
B3-143-LSu	9,122 psi	63 MPa	Non-structural failure
B3-143-LSl	17,597 psi	121 MPa	Non-structural failure
B3-147-LSu1	5,214 psi	35 MPa	Non-structural failure
B3-147-LSu2	17,272 psi	119 MPa	Non-structural failure
B3-151-LSu1	12,352 psi	85 MPa	Non-structural failure
B4-143-LSu1	11,496 psi	79 MPa	Non-structural failure
B4-143-LSu2	15,805 psi	109 MPa	Non-structural failure
B4-149-LS	9,101 psi	63 MPa	Non-structural failure
B5-147-LS	9,151 psi	63 MPa	Non-structural failure
B5-163-LSu1	15,126 psi	104 MPa	Non-structural failure
B5-163-LSu2	13,139 psi	91 MPa	Non-structural failure
B6-145-LS	11,454 psi	79 Mpa	Non-structural failure
B6-159-LSu1	10,340 psi	71 MPa	Non-structural failure
B6-159-LSu2	14,888 psi	103 Mpa	Non-structural failure
Sample	**BTS**		**CAI**
B3-169-SHb	884 psi	6.1 MPa	--
B4-147-SHb	516 psi	3.6 MPa	--
B3-143-LSb	1,339 psi	9.2 MPa	--
B3-147-LSb	1,344 psi	9.3 MPa	--
B4-149-LSb	869 psi	6.0 MPa	0.9
B5-163-LSb	1,056 psi	7.3 MPa	1.0
B6-145-LSb	1,323 psi	9.1 MPa	--
B6-159-LSb	870 psi	6.0 MPa	0.9
B5-147-LSc	--	--	1.1

Table 5. Summary of Laboratory Tests on Rock Cores

Percentage of Tunnel Rock	Percentile	UCS, psi	BTS, psi
Weakest 10 percent	10th	7,600	700
Average	50th	12,900	1,150
Strongest 10 percent	90th	17,300	1,590
Strongest 0.01 percent	99.99th	27,500	2,440

CURRENT DESIGN

In late July, 2004, the bedrock tunnel was proposed to slope downward from an invert elevation of 291 feet at Station 1 + 60 at the south end of the tunnel to an invert elevation of 282.5 feet at the construction shaft at Station 86 + 17. Excavation will start at Station 86 +17 and proceed downstream to completion, and then the TBM will be moved back through the tunnel and out of the construction shaft. The excavated diameter of the tunnel will be about 12 feet, so the tunnel will be mined in the rock between elevation 294 feet and elevation 282 feet, approximately. That zone in the rock was selected to minimize difficulties in tunnel excavation and to provide ample head to drive water into the recovery system. All of the tunnel will be excavated by a TBM with a diameter of at least 11 feet 8 inches, within the Ordovician rock strata that have been explored and evaluated comprehensively. Those strata will consist of shaley limestones from the top of bedrock to below the tunnel invert, in general; layers and lenses of limestone, shale and calcareous shale, and thin layers of medium stiff clay occur in that zone also.

Concern that the long-term leakance from the river bed was less than what had been anticipated led designers to propose a much longer tunnel. Also, extending the tunnel along the river downstream would facilitate further extension to the Zorn Avenue site more easily if such extension were deemed necessary in the future. Extending the tunnel to the southeast relocated the construction shaft and the pump station on the third terrace, near the existing water treatment plant, and minimized impact on the community. Placing the construction shaft and pump station on the third terrace also reduced flood hazards during construction (and permanently for the pump station). Vertical wells were chosen rather than Ranney wells with laterals because of better overall geometry with respect to riverbank filtration and specific yield, and for ease of maintenance compared to laterals.

For core samples taken from the tunnel zone, only one RQD value was less than 70 percent, and the majority of values were above 90 percent. Most of the discontinuities in the cored rock consisted of bedding planes along shale beds; such openings could define slabs in the tunnel crown that would require support. High-angle joints were only rarely found in cores, but are known in outcrops of the strata from the tunnel zone. Where RQD is equal to or greater than 75 percent (expected to be along 80 percent of the tunnel length), friction dowels should provide adequate roof support, supplemented with mesh to control movement of small, thin slabs. For about twenty percent of the tunnel length (about 1,600 feet), more frequent joints will be encountered, and/or more frequent dowels and mine straps will be required, with mesh to control slab movement as necessary. Full-circle steel rings spaced four feet apart longitudinally, with welder wire fabric and steel lagging as needed, are expected to be needed for only about 100 feet of the tunnel, where crushed rock or badly oriented clay seams thicker than six inches occur. If such support is required, liner plate could be used instead of rings to avoid problems with location of vertical wells.

Because of the gassy conditions found in some borings, the equipment used to construct the tunnel must be operable in an explosive atmosphere. The tunnel will be a Class I environment under provisions of 29 CFR 1926.449. Continuous probing in two holes ahead of the tunnel face will be done to search for gas under pressure and for solution cavities. Sustained flow of gas and/or water can be expected if a gas-bearing zone or water-bearing solution cavity is encountered. Gas zones will require grouting with microfine cement under a pressure of at least 120 psig. Type III Portland cement can be used under a pressure of at least 70 psig to grout solution cavities. If gas or solution cavities are encountered, six additional holes will be drilled around the face after grouting is done to verify that the flow feature has been grouted. If more gas flow or solution features are found, an additional cycle of grouting and probing will be done. Gas is likely to occur, if found, in horizontal bedding plane openings, and will be difficult to discover in horizontal probe holes.

Two access shafts will be driven, on the tunnel centerline at Station 1 + 60 and at a point about 12 feet left of centerline at Station 67 + 00. These shafts will be excavated before the tunnel reaches those stations. Each shaft will be lined with a 54 inch-diameter steel casing that will be grouted into place in the boreholes, which will have a minimum diameter of 68 inches. A small chamber will be blasted out of the rock at the side of the tunnel, at Station 67 + 00, to connect the shaft to the tunnel.

The construction shaft will be driven through about 40 feet of alluvium (the shaft will be dug on the third terrace) and then through about 150 feet of bedrock. The construction shaft will have an excavated diameter of about 35 feet, governed by contractor requirements for the TBM, and will be lined with cast-in-place concrete. A pump station will be built along the tunnel between about Station 83 + 70 and Station 84 + 70, and will consist of five shafts at least 68 inches in diameter drilled at intervals of 20 feet along the tunnel length. Each shaft will contain a 54 inch-diameter steel pipe that will house a single submersible pump. The pipes will be about 172 feet long, extending from one foot above ground to a point two feet above the excavated tunnel crown. The boundary between the third terrace and the (lower) second terrace occurs at about Station 82, so that pump station shafts also will penetrate only about 40 feet of soil and then about 150 feet of rock. A number of shafts were chosen for the pumps rather than a single large shaft for reasons of economy in excavation. Shaft excavation in bedrock is expected to be done by drilling and blasting.

When the locations of the construction shaft and the pump station shafts were established, a new concern arose in connection with solution cavities in the Silurian rock strata under the third terrace. Solution cavities are not expected to occur below about elevation 320 feet, about ten to 20 feet below the top of rock under the third terrace. The solution channels are expected to consist of vertical features (widened joints) less than one foot wide and small horizontal cavities (widened bedding planes) less than one foot in diameter. However, large amounts of groundwater inflow can be expected from the saturated sand and gravel and through solution cavities in the Silurian strata.

Thirty-one vertical wells, consisting of a surface casing, a casing and screen assembly 16 inches in diameter, a foot casing 12 inches in diameter, and a drop pipe, will be drilled through the alluvium into the bedrock. The wells will be screened over the lower portion of the sand and gravel aquifer, and the foot casing, cone reducer and part of the lower casing will be grouted into the bedrock. The vertical wells can be drilled, installed, developed, tested, and disinfected concurrently with tunnel excavation. After the tunnel is completed, it will be filled with water and 11.75 inch-diameter holes will be drilled through the bottoms of the wells down 30 to 35 feet into the tunnel crown. The drop casing will be about 10.75 inches in diameter and will hang from the cone reducer down to the tunnel crown. Water will be pumped from the pump station, causing a reduction in head in the tunnel which will draw water from the vertical wells. The heads of the vertical wells will be sealed, and no pumps or plumbing devices will be present in any well.

REMAINING UNCERTAINTIES

Fundamental uncertainty remains about the long-term productivity of the RBF system at the Payne plant site because of uncertainty about the extent, severity and permanence of plugging of the bank/bed and consequent reduction in leakance into the pumped aquifer. In autumn 2004, finite-element method computer models are being developed to simulate the aquifer under the Payne plant. Those models will include consideration of the change in water temperature with seasons in the Ohio River and the effects of heat exchange between the moving water and the aquifer materials. Also, the University of Louisville and the Louisville Water Company in conjunction with the U.S. Geological Survey (USGS) are doing field work to provide additional measures of head, conductance, and characterization of the riverbed system, under sponsorship of the American Water Works Association Research Foundation. That research is intended to develop field measurements for characterizing RBF systems and to show how such parameter values could be used to predict long-term sustainable yield in RBF systems of varying aquifer geology and stream hydraulics. The research includes examination of site conditions and system performance for

- The Ohio River at Louisville
- The Rhine River at Dusseldorf
- The Great Miami River in Cincinnati
- The Missouri River at Kansas City
- The Russian River at Sonoma County, California
- The Platte River at Lincoln, Nebraska
- The Raccoon River in Des Moines, Iowa
- The Hudson River in New York
- The Llobregat River in Spain
- The Drava River in Slovenia, and
- The Danube River in Hungary

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EVOLUTION OF ROCK ANCHOR PRACTICE OVER THREE DECADES

Dr. Donald A. Bruce[1]

Abstract

In the absence of a formal national standard, the Recommendations of the Post Tensioning Institute (1996) have served the dam industry well regarding the design, construction, testing, and performance of rock anchors. These Recommendations are being updated again for re-issue in 2004, with special emphasis being devoted to the results of recent researches into issues related to epoxy protected strand. Recently, the author has been involved in the performance assessment of high capacity anchors installed in a dam in the Pacific Northwest in 1975. Very comprehensive contemporary records exist from the project, involving multiwire button head tendons. These records clearly illustrate the "State of Practice" as it was for anchors at the time. This paper compares and contrasts these historical data with contemporary practices, illustrating the evolution of certain aspects of dam anchor practice in the U.S. over three decades.

1. Introduction

The history of prestressed rock anchors as a remedial tool to stabilize existing concrete dams in the U.S. extends back to the early 1970s (Bruce, 1989, 1993). In recent years, it is estimated (ADSC, 2002-2003) that between 10 and 15 dams are remediated in this fashion annually, with about one third using epoxy coated strand as the tendon material. This represents an intensity of effort unmatched in any other country, both for quantity and duration: it is more typical to find that a certain country experiences a relatively short-lived phase of dam anchoring on groups of dams, determined by geography, age, and/or design. In this regard, the attention now being focused on the dams operated by Hydro Tasmania is a typical example.

Practice on U.S. dams has understandably evolved over the last 30 years, in response to changes in equipment, materials, and design concepts and philosophies, primarily those relating to corrosion and corrosion protection. In addition, the trend towards specifying progressively higher capacity tendons (a current project in British Columbia has tendons comprising 93 number 0.6-inch diameter strands) has further caused contractors to revise their methodologies in order to satisfy the considerable logistical challenges the handling, installation, grouting, and stressing of such massive tendons pose. The author was recently involved in the reevaluation of 142 anchors installed in 1975 in a major dam in the Pacific Northwest. This study involved not only a physical inspection of the heads of four of the anchors, but afforded the opportunity to appraise the specification, and study the comprehensive as-built construction records. The observations contained in this paper reflect upon various aspects of 1970s rock anchor practice as generalized by Littlejohn and Bruce (1977) and typified by this project, and current practice, which tends to follow closely the existing Recommendations of the Post Tensioning Institute (1996), which will be further enhanced in the upcoming 2004 edition. As is described below, the rate and extent of progress over the last 30 years has not been uniform in all aspects of the technology.

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[1] President, Geosystems, L.P., P.O. Box 237, Venetia, PA 15367, U.S.A., Phone: 724-942-0570, Fax: 724-942-1911, dabruce@geosystemsbruce.com

2. Scope of 1975 Anchor Project

A total of 142 vertical anchors were installed to resist sliding with total lengths ranging from 55 to 168 feet. "Rock embedment" ranged from 27 to 82 feet with first stage grouting lengths (i.e., bond lengths) ranging from 11 to 61 feet. The "effective prestressing force" per anchor varied from 205 to 1490 kips, each anchor comprising multiwire tendons of 240-ksi wire of 11.78 kips per wire Guaranteed Ultimate Tensile Strength. In four anchors, "minitendons" were formed by encapsulating the free lengths of four adjacent wires in a debonding sleeve to allow periodic lift off testing of these wires during service life. This, in itself, was a novel and thoughtful feature.

3. Geotechnical Design Aspects

Then

In terms of the overall prestressing load requirement to resist sliding, the design was classified recently as "exceedingly conservative", given the anticipated loading conditions, the parameters selected for the rock/concrete contact, and the magnitude of the factors of safety employed. Indeed, the recent reappraisal indicated that the number of anchors actually required using current FERC guidelines would now be considerably less. The key issue of rock-ground bond typified contemporary practice: bond distribution was considered to be uniform and the working bond magnitude (in generally hard volcanics) was conservative (20 to 35 kips/ft or about 100 to 130 psi). Bond lengths and diameters (4 to 8¼ inches) were called out in the anchor schedule, based on these average bond values. Values of 150 to 200 psi for working bond were common in U.S. practice at the time (Littlejohn and Bruce, 1977) for such rock types. Although it is not specifically described, the question of overall stability design is interesting to speculate upon. It is known that the state of practice was to assume that a volume of rock would be engaged by each anchor, being a cone of included angle 60° to 90° and with the apex located at some point along the bond zone, usually the mid point. The uplift capacity was thus the submerged weight of rock in this cone (or wedge of overlapping cones for closely spaced anchors).

Now

Overall anchor load requirements are calculated in accordance with appropriate statutory requirements and using contemporary computational methods and aids. However, given the "consequences of failure" facing a dam remediation program, it is typical that designs remain conservative, if not as excessively so in former years. Regarding rock-grout bond, a plethora of data (Barley and Windsor, 2000) confirms that bond is not evenly distributed and that designs so based provide inefficient load transfer conditions. Nevertheless, despite (or perhaps because of) the great mass of field-generated data, the "uniform load" assumption remains standard practice, and typical bond values, as listed in PTI (1996) remain of the same order of magnitude as those used in 1975 (i.e., <u>ultimate</u> values of 250 to 450 psi). Likewise, the very simple and simplistic "weight of rock in a cone/wedge" remains the typical basis for overall stability design: very few designers consider shear strength contributions, and fewer still employ the more sophisticated equations developed in the 1970s by Czech and German rock mechanics engineers.

4. Construction

Then

<u>Drilling</u>. Diamond drilling was specified through the concrete, whereas rotary or rotary percussive techniques were permitted in the rock. Certain anchor holes were to be fully cored and then

reamed out to final diameter. Hole alignment was to be monitored to prevent total deviation being larger than 1 in 100. Pressure grouting was permitted to combat unstable ground conditions. A "sump", 18 to 24 inches deep, was to be allowed for at the bottom of each hole to collect any debris which could not otherwise be evacuated by the drill flush. Full and accurate geologic drill logs were to be maintained of all the major lithological and structural variations in the rock mass.

Water Pressure Testing. The full length of every hole in rock was to be tested, the acceptance criterion being a loss of 0.5-gpm at 60 psi. Failure would result in pressure grouting, redrilling, and retesting. More typical of U.S. practice at the time was a criterion of 0.001 gal/inch diameter/ft/min at an excess pressure of 5 psi, highlighting the very conservative nature of the project's specifications.

Grouting. A proprietary, presumably non-shrink, high strength pre-blended grout was specified for the first stage, and a Type II grout with non-shrink additive for the second stage. Stressing could commence when the grout reached a strength in excess of 3000 psi. The water/cement ratio was limited to 0.40 to 0.45 with the proviso of "minimal shrinkage". Pre-construction testing of grout mixes was to be conducted to demonstrate the grouts' acceptability. The mixer had to be able to measure (accurately) the grout volume, while it was also specified as having to be a high speed, high shear mixer, coupled to a paddle agitated storage tank. Pumping was to be via a Moyno pump. Good grout return at the top of the anchor hole during secondary grouting (with the tremie method) was also required to ensure full and thorough secondary grouting.

Tendon. The contractor was permitted to select the tendon type, in the case of the higher capacity anchors, the choice being strand or wire. The wire was specified to ASTM contemporary standards, and had to perform within reasonable, safe, stressing levels (i.e., at no time being loaded above 80% Guaranteed Ultimate Tensile Strength). No provisions were made for corrosion protection directly placed on or around the tendon, other than the grout itself.

The Specification reflected both the open mindedness of its drafters (in allowing choice of tendon type) and the typical contemporary standards with respect to attitudes towards corrosion protection. In effect, it was tacitly assumed that an efficiently water proofed and grouted anchor in a rock or concrete mass which is naturally (or is rendered) virtually impermeable, without aggressive ground water, will not experience significant corrosion in the hole. This was generally supported by the FIP study (1986) which showed, inter al., that under such advantageous conditions, corrosion was a possibility only at, or under, the head. In the case of fully bonded tendons, as here, this is not a threat to the anchor's long term performance after final lock off.

Now

Drilling. Except in rare cases where there is a risk of significant embedded steel in the concrete, or where the dam's "hearting" could be extremely sensitive and voided, coring techniques are not employed as a regular construction tool. Rotary percussion (by down-the-hole (DTH) hammer) is most common, being selected for its cost, speed, and deviation control advantages. Studies have indicated that the vibrations induced by rotary percussion have minimal impact on the structure being penetrated (Bianchi and Bruce, 1993). It is also known that coring rock produces a smooth borehole surface which can inhibit subsequent rock-grout bond development. Contemporary DTH drilling permits holes of up to 15-inch diameter to be drilled to over 300 feet deep with deviations of less than 1 in 150 or better with relatively standard equipment. Sumps are typically 3 to 5 feet deep, and MWD (Measurement While Drilling) recording is common as an indirect guide to rock quality conditions (Bruce, 2003).

Water Pressure Testing. Current practices reflect the knowledge that the permissible water loss calculation should be independent of hole diameter or length, since the critical fissure ($\geq 160\ \mu$ wide) can exist anywhere along the hole and flow is not diameter-driven. The 1975 project-specific criterion was very severe, but reflected the fact that grout was the only level of corrosion protection on the steel tendon. The current criterion of 10 gallons in 10 minutes at an excess pressure of 5 psi is much less onerous, but, for anchors longer than about 80 feet, remains the most conservative of all the international standards (Figure 1). This talks of higher quality in U.S. practice than elsewhere.

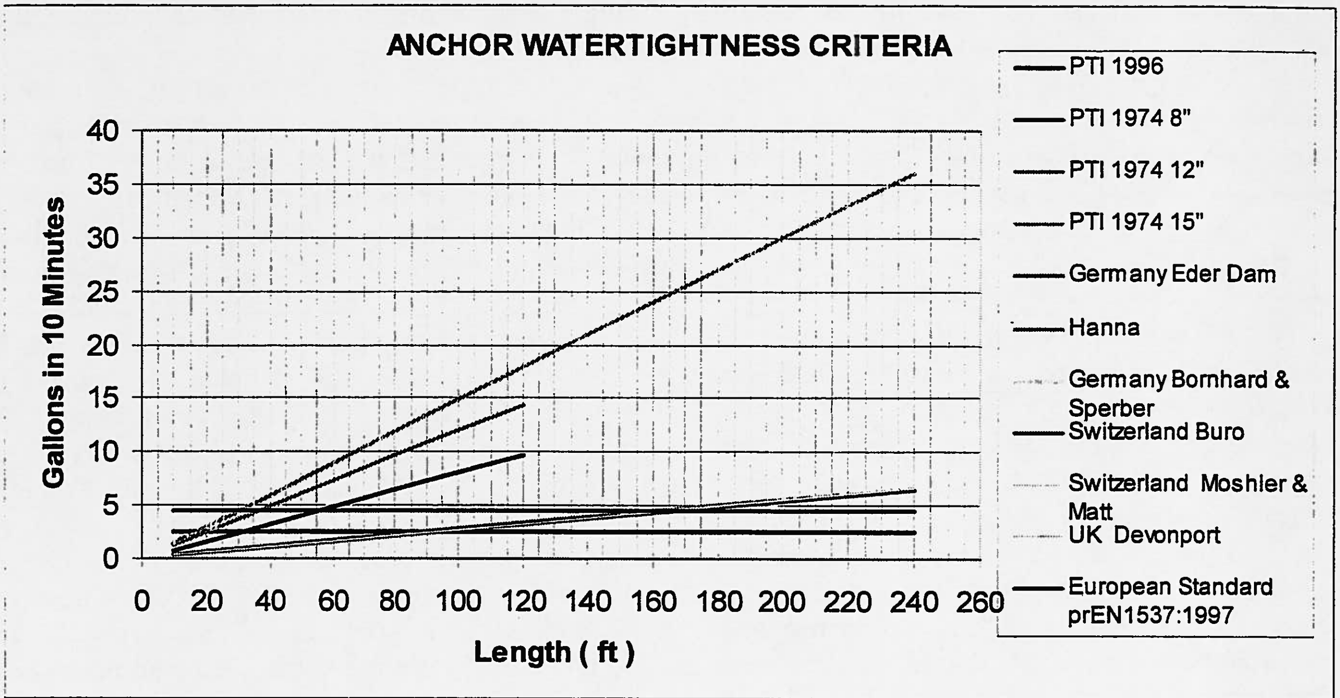

Figure 1. International criteria for water tightness of anchors.

Grouting. Again, in all critical aspects the 1975 specification is excellent and is surprisingly close to current practice. For example, the use of high shear mixers to mix grouts of low water/cement ratio would ensure homogeneous, pumpable, stable grout of high strength and low permeability and so superior durability in situ. The requirement for pre-construction site mix testing is also illustrative both of good practice, and common sense. Furthermore, the decision to use two stage grouting would automatically a) permit the bond zone to be load tested in isolation and b) provide bond to the tendon in the free length (after stressing) thus ensuring that load could not subsequently be lost due to mechanical failure of the top anchorage (i.e., in this case a button head system).

Today, proprietary premixed, preblends are neither necessary nor cost effective. Our advanced knowledge of cement grout rheology, as impacted by admixtures, provides adequate solutions to the problems of stability, pumpability, and durability. Use of Type III cements permits stressing within 3 to 4 days, while ultimate grout strengths in excess of 6000 psi are typical of low water:cement ratio mixes. This can permit higher average grout/rock bond values to be assumed in the design process.

Tendon. The tendon composition (invariably 7-wire strand to appropriate ASTM standards) is specified and the Contractor must provide details of the tendon geometry (i.e., spacers, centralizers, grout tubes, etc.), as well as his plans for shipping, handling, and installing the tendon to minimize damage. Wire tendons are not used while only low capacity anchors (say ≤ 100 kips) of moderate length (say ≤ 50 feet) can be logistically or economically satisfied with bar tendons. The most significant developments in tendon design, however, relate to corrosion protection, and contemporary protection levels for permanent dam anchors are Class 1, as summarized in PTI (1996) Table 5.1.

This superior level of corrosion protection is justified based on service life, consequences of failure, and incremental in place costs, especially. Practice in this regard has improved dramatically in the last 10 years or so.

CLASS	PROTECTION REQUIREMENTS		
	ANCHORAGE	UNBONDED LENGTH	TENDON BOND LENGTH
I ENCAPSULATED TENDON	1. TRUMPET 2. COVER IF EXPOSED	1. GREASE-FILLED SHEATH, OR 2. GROUT-FILLED SHEATH, OR 3. EPOXY FOR FULLY BONDED ANCHORS	1. GROUT-FILLED ENCAPSULATION, OR 2. EPOXY
II GROUT PROTECTED TENDON	1. TRUMPET 2. COVER IF EXPOSED	1. GREASE-FILLED SHEATH, OR 2. HEAT SHRINK SLEEVE	GROUT

Table 5.1 Corrosion protection requirements (PTI, 1996).

5. Stressing and Testing

Then

The stressing was to be conducted in progressive steps corresponding to 40, 60, 80 and 100% of the "maximum jacking force" (which would, however, be less than or equal to 80% of the tendon Guaranteed Ultimate Tensile Strength). Strict criteria were placed on elastic extension (within 5% of the theoretical) and permanent movement (1 inch allowed). The tendon was to be locked off at the "desired effective prestressed load" or 70% GUTS, whichever was greater. A lift-off test would then be conducted (prior to secondary grouting) no earlier than 2 days after successful initial stressing, with a 5% load variance criterion. In general, the stressing provisions are, by contemporary standards, the weakest part of the specifications in terms of the steps to be taken in reaching the maximum load. However, relatively strict acceptance criteria were applied to the total movement (it would have been impossible to arithmetically separate permanent from elastic movement components without progressive cyclic loading, equivalent to PTI's "Performance Test" sequence), and on the transfer load retention efficiency after 48 hours. Given the range of tendon types permitted (reflecting the input of the various post tensioning companies who had likely contributed to the specification's content), the stressing criteria were probably a reasonable consensus of the different methods. In any case, the stressing data, though quite rudimentary, would upon analysis provide a quite sensitive and accurate picture of each anchor's performance.

Long term performance was to be gauged by "minitendons" within 4 of the anchors, subjected to periodic lift off testing (which would, of course to a certain degree upset the top anchor corrosion protection system).

Now

The 1996 Recommendations corrected certain misconceptions that had worked their way through previous successive editions. In particular, clear distinction was drawn about the measurement and analysis of elastic movements, as opposed to total movements. This is possible if progressive cyclic loading is used, as in the Performance Test (Figures 8.1a and b), which is conducted on pre-production ("disposable") anchors and/or on a limited number of production anchors. The use of strand tendons, and a multi-part wedge system of load retention, facilitates such multiphase stressing, which would have been very awkward with the wire tendons and their "button head" top anchor system. Likewise, the hydraulic jacking systems now in use greatly speed the execution of such tests and improve the accuracy of load application. Long term performance is now monitored (if debonded free lengths are provided) via load cell and/or total head lift off testing provided access can be maintained to the head without compromising corrosion protection.

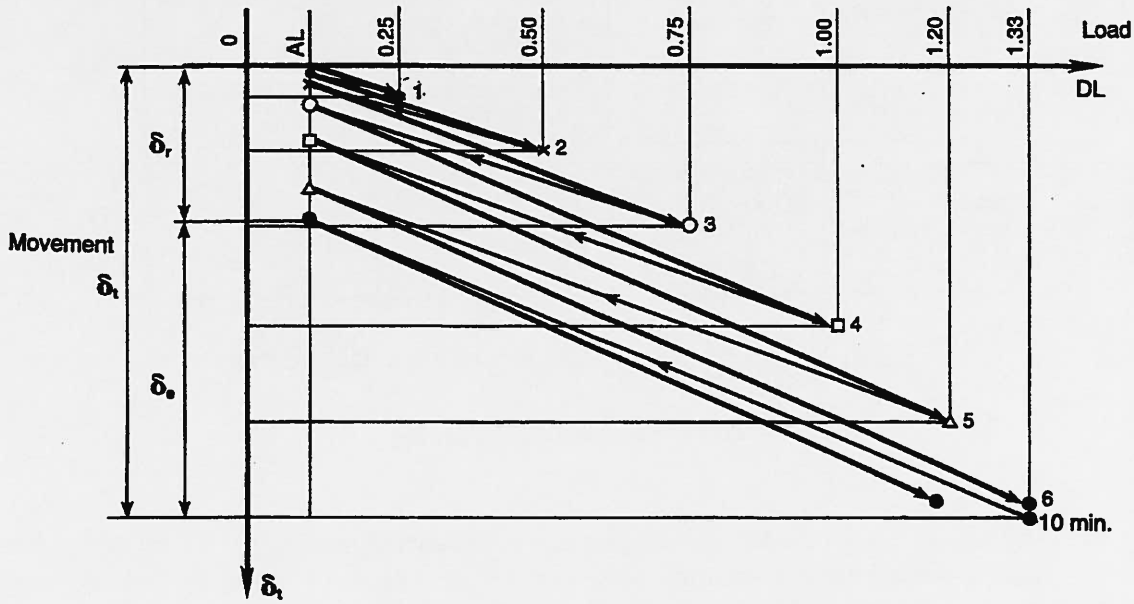

Figure 8.1a. Plotting of Performance Test data.

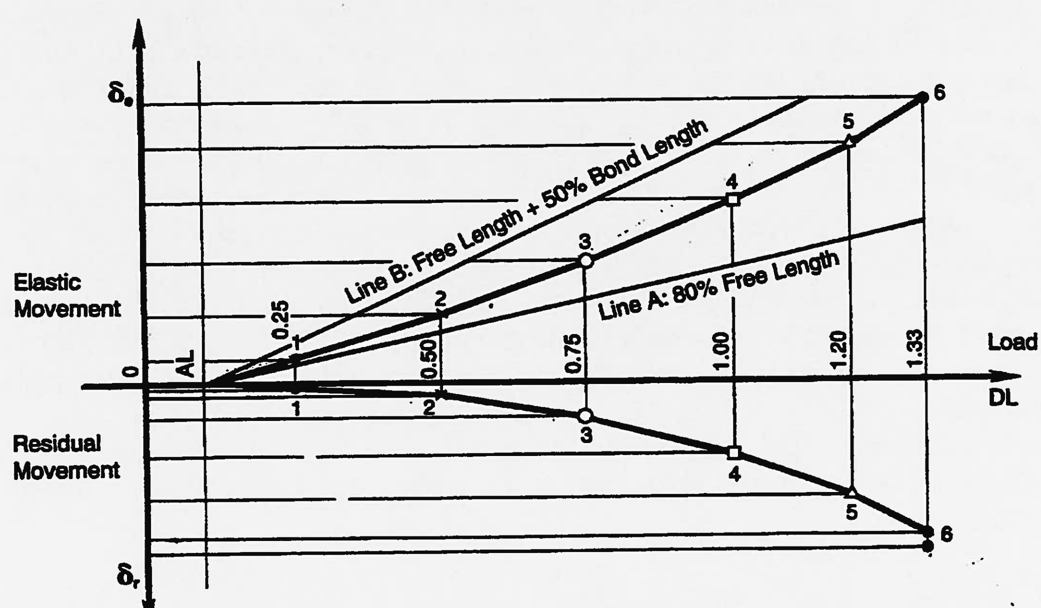

Figure 8.1b. Graphical analysis of Performance Test data.

6. As-Built Anchor Records

Then

Two separate sets of drawings summarized the salient construction data for every anchor, including

- Geological log (for each anchor position) as based on the core results.
- Hole inclination, depth, and diameter.
- Elevations of grout placed, and number of bags injected.
- Water pressure test data.
- Grout rate of gain of strength (Primary and Secondary stages), and similar concrete data.
- Load-extension curve, plus lift-off data.
- Dates for each construction step.

It may be observed that every hole met the drill deviation tolerance and that every stage in rock was water pressure tested to and passed at the 60 psi excess pressure, with the packer placed in the concrete, near its base.

The records are very comprehensive and show that the work was conducted in strict accordance with the Specification. In particular, every hole, prior to tendon insertion and grouting, had a very low permeability (typically less than 0.3 Lugeon) and no hole appeared to show unusual grout takes, also indicative of tight rock conditions. Every tendon provided an extremely linear load-movement performance, indicative of minimal debonding, and so proved considerable bond capacity in excess of that actually mobilized during testing. No anomalies were noted during lift-off to suggest any tendency for creep to have occurred (indeed, the nature of the rock mass would also argue against even the possibility of creep being allowed to occur).

Now

The PTI document requires neat, legible and "suitable for reproduction" records to be provided comprising

- As-built drawings.
- Materials certifications.
- Drilling and grouting records, water testing, grout mix design, laboratory tests on grout cubes.
- Anchor test and monitoring results and corresponding graphs.

These are now computer generated and maintained, supplemented by digital progress photographs. However, it is clear the contents of such reports have not changed over 30 years (except perhaps for details regarding corrosion protection).

7. Overview

This brief review highlights that while progress in the technology has most certainly been made over the last 30 years, it has not been at a constant rate across all the various aspects of the technique. Developments in equipment, technique, and materials have permitted engineers to design increasingly higher capacity, longer anchors. However, the basis for the key elements of their designs remains largely unaltered, with particular respect to overall stability and bond stress magnitude and distribution philosophies. Better understanding of rock and concrete properties, and of the nature of the loads imposed on structures has – at best – allowed designs to refine from “exceedingly conservative” to merely “overconservative”.

In contrast, the skills of equipment and materials manufacturers and of contractors have been honed on the stones of experience, expedience, and competition, to the extent that the industry can satisfy economically the logistical challenges posed by designers. Of particular relevance are advances in drilling capabilities, understanding of rock mass permeability issues, and developments in the assembly, handling, installation, protection, and grouting of multistrand tendons. In particular, major and significant philosophical changes in attitudes to corrosion protection have been enacted, particularly within the last decade.

A similar picture can be painted of the stressing and testing aspect of the technology wherein the development of high capacity, long extension hydraulic jacks complements the development of multiwire strand systems, and enables sophisticated loading and analysis routines to be conducted, thereby enhancing quality and reliability. Quality assurance is also reflected in the routine maintenance of thorough construction records. The quality of the (manual) records for the 1975 project described herein was astounding: it may be speculated, however, that it was exceptional, not standard, and reflected great credit on all the parties concerned.

Such historical perspectives can have extreme value as well as interest, since it is vital to understand the path of technological evolution in order to predict the course of future needs and development. Throughout the country in dusty boxes in abandoned store rooms there remain the records of dozens of anchor projects covering 30 years of anchor construction. Contemporary MIS techniques assure that it has never been easier or quicker to permanently, electronically archive such data, before they are thrown out, lost, or otherwise destroyed. This is a national need and an unfulfilled initiative. It requires dedicated, funded resources to accomplish it. It must be done before the generation that built the projects is lost to their future service. There is a wealth of data to be collected, archived, and analyzed. The task would benefit future technology and create justified international recognition for the contributions of U.S. engineers to its evolution for over three decades.

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REHABILTATION OF BIG SAVAGE TUNNEL

Paul J. Lewis, P.E.[1], Mitchell Weber, P.G.[2],
James Eppley, P.E.[3], Peter Bowman[4]

ABSTRACT: Construction on the 3300 foot long Big Savage Railroad Tunnel began in October of 1909. After progressing approximately 640 feet from the west heading, tunneling crews encountered a rapid inflow of fine water-bearing sand. In February of 1910, an air lock was installed and tunneling progressed with considerable difficulty using compressed air methods until the heading again encountered competent rock approximately 900 feet from the north portal. Since completion of construction in 1911, the Big Savage Railroad Tunnel has undergone numerous events of structural rehabilitation to correct failures caused by freeze thaw, significant water pressure, and inflows causing collapse of the crown and walls at several locations. Western Maryland Railway abandoned the tunnel in 1976 and a massive roof collapse blocked the entrance to the north portal over the winter of 1995/1996.

The tunnel was rehabilitated to serve as a vital link in the Great Allegheny Passage, a recreational trail from Pittsburgh, PA to Washington D.C. This paper presents the geologic setting, data from the original construction, and details of the innovative techniques utilized to stabilize and rehabilitate the tunnel. Stabilization techniques utilized included the installation of over 6000 swellex anchors, lightweight (cellufoam) grouting, a geocomposite drainage layer, an extensive underdrain system, and a steel fiber reinforced shotcrete liner as a combined solution working with the original liner systems to improve the overall condition of the tunnel.

I. Introduction

The Big Savage Tunnel is located in Somerset County, PA, between Somerset, Pennsylvania and Frostburg, Maryland, near the town of Mount Savage, MD. The tunnel is about 3300 feet in length and is an abandoned, horse-shoe shaped railroad tunnel. The dimensions of the tunnel were quite variable, but in general were approximately 17 feet wide and 23 feet high. Contract drawings were prepared for the project owners / sponsors including the Pennsylvania Department of Conservation and Natural Resources, the Pennsylvania Department of General Services, the Allegheny Trail Alliance, and Somerset County.

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[1]Senior Geotechnical Engineer, Gannett Fleming, Inc.., Harrisburg, PA
[2]Senior Engineering Geologist, Gannett Fleming, Inc., Columbus, OH
[3]Chief Engineer, Pennsylvania Department of Conservation and Natural Resources, Harrisburg, PA
[4]Project Manager, Advanced Construction Techniques, Maple, Ontario

However, during performance of initial site preparation activities, unknown and unforeseen site conditions were discovered that prohibited the safe execution of the original design plans. These unknown conditions would have resulted in substantial work item quantity and cost overruns. The contractor, Advanced Construction Techniques, Ltd. (ACT) retained Gannett Fleming, Inc. (GF) to evaluate the condition of the tunnel. Working together with the Project Owners / Stakeholders, the ACT/GF team developed a remediation strategy utilizing innovative materials and techniques to stabilize and improve the tunnel condition to meet the project requirements, and to eliminate unnecessary cost overruns.

II. Original Construction

History of the Tunnel

Western Maryland Railway desired to gain access to the high traffic district in and around Pittsburgh, PA. Construction of the Connellsville Connector would provide a freight route from Pittsburgh to Baltimore.

Surveying for construction began in 1909 with track construction starting in July 1910. Maximum grade on the line was 1.75% ascending the eastern face of the Allegheny Mountains. The descent down the western flanks was 0.8%, creating the flattest rail grade across the Allegheny Mountains. Advancing from Connellsville, PA, the route encountered Big Savage Mountain at Mile 20. Big Savage Mountain, at 2340 MSL, was the highest point along the route, so a 3300-foot long, single-track tunnel was planned to cross the mountain.

Construction Methods

The *Engineering Record* (December 31, 1910) reported the tunnel dimensions as "17 ft. wide in the clear, with a full semi-circle arch on a radius of 8 ft. 6 in., with the springline at 15 ft. 3in. above the subgrade". A total of 8,000,000 cubic yards of material was estimated for the tunnel excavation.

The tunnel was advanced from both portals. Only the arch section above the springline was to be through-constructed before the bench below the springline was excavated.

Excavation was advanced by pattern drilling the rock face and blasting with dynamite and black power. Each heading was advanced 5 to 7 feet per shot. Progress for both headings was about 18 feet/day for two 8-hour shifts. The December 23, 1910 *Engineering Record* reported that "material encountered is hard sandstone, coal, and fire clay, requiring considerable timbering."

Trouble For The Western Heading

On December 19, 1910 the western heading had progressed about 640 feet into the tunnel. The crews were working in rock so hard that "practically a full shift was required to drill a round of holes". At the completion of the drilling, the holes were loaded and detonated. Immediately, a rapid inflow of saturated soil and rock burst into the western arch heading filling the excavation to a point about 200 feet into the tunnel. The material was described as a "mixture of fine water-bearing sand" and disintegrated rock fragments.

Assuming a small soft pocket had been encountered, the crews began to muck out the material and drive a small drift along the crest of the crown. The excavation had to be advanced with full box timbering and breast boarding. Straw was used to pack between the boards to minimize infiltration of the saturated soil. After one month, the pilot heading had only advanced 15 feet.

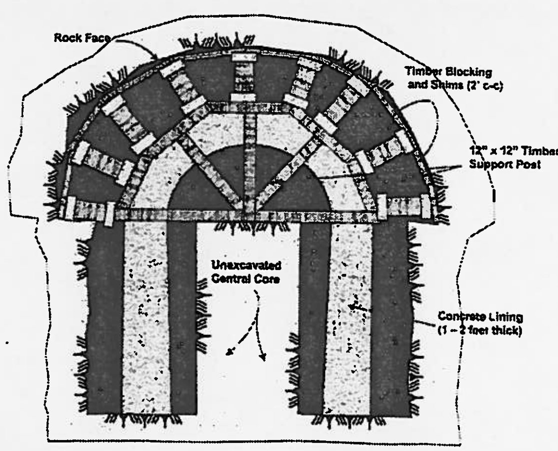
Introduction of Compressed Air Construction Methods.

By February 1, 1911, the ground began to thaw and a 20 x 20 x 16 ft. sinkhole developed over the 'trouble zone' 200 feet above the crown. New material flowed into the crown heading as fast as the workers could muck it out. The Carter Construction Company of Chicago had been doing the work. John Carter, the company President, was aware of the compressed air excavation being performed for the North River Tunnels in New York City and he obtained one of the O'Rouke air locks used for that project.

After the air lock arrived at the Big Savage tunnel site, it was installed in a bulkhead that had been constructed in the heading about 40-feet away from the point of the blow out. The air lock was 7 feet in diameter, 25 feet long, and equipped with muck cars on rails It was determined that a pressure between 20 and 35 pound per square inch (psi) was required to keep the inflow of material to a minimum. Working in the compressed air environment was very difficult and 'local' experienced laborers were not available. Once again, Mr. Carter went to New York City to recruit experienced compressed air work force. Seventy-five men working across three shifts labored 3 hours on, 3 hours off until competent rock was once again encountered.

Excavation

Even with the compressed air, the excavation had to be braced every step of the way. Short crown sections were advanced; then 12-inch square, oak timber posts and blocking were installed. With the crown thus stabilized, the tedious process of excavating the side walls began. The side wall excavations were strutted on 2-foot centers top to bottom to provide excavation support. Once the floor elevation was reached, 12-inch square vertical support timbers were installed and the roof load was transferred to the vertical posts. With the timber post support system carrying



the loads, the central core was excavated and the side walls were formed and concrete placed; thereby, encasing the vertical timber post in concrete. This method of advancement continued for about 40 feet when the "bottom fell out".

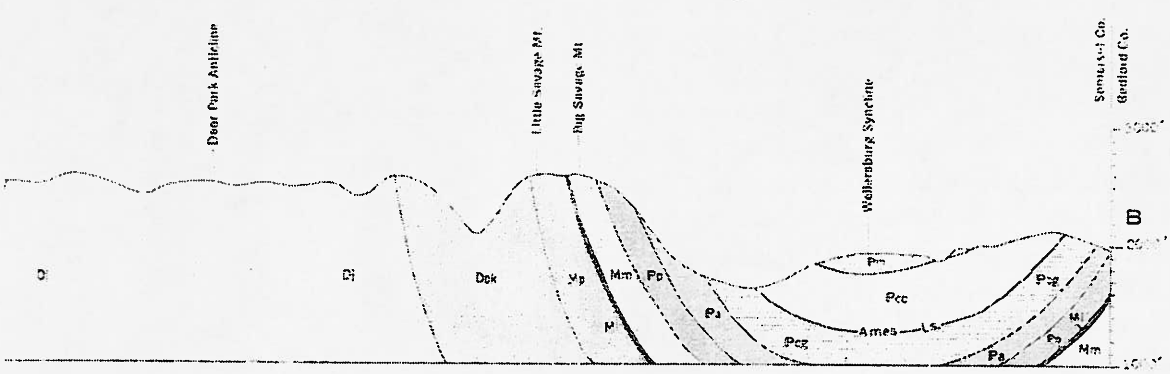
The dip of the rock strata was in the direction of the heading, therefore, as the tunnel progressed, the depth to the firm stratum continued to drop. At about 40 feet into the soft strata, the tunnel floor elevation transitioned from firm sandstone to the soft strata that caused the initial roof run in. At this point the entire tunnel was in the soft material. The sidewalls were deepened to a depth 16.5 feet below the floor elevation and a concrete footing was poured. The contractor now had to install extra lengths of the 12 x 12-inch timber posts to act as support piling from the footing to the concrete floor slab. This method of advancement continued for about 150 feet where the next firm stratum was encountered. Thinking the conditions had been stabilized, the pressure was released but the floor in front of the completed concrete invert heaved and the pressure was reapplied for another 60 feet.

By November 1911, the pressure tunneling had been completed and the heading advanced without incident. The tunnel served the Western Maryland Railway until 1976 when it was abandoned. The tunnel deteriorated due to neglect and severe freeze-thaw affects. In 1995/96 the northern portal collapsed, effectively blocking passage through the tunnel.

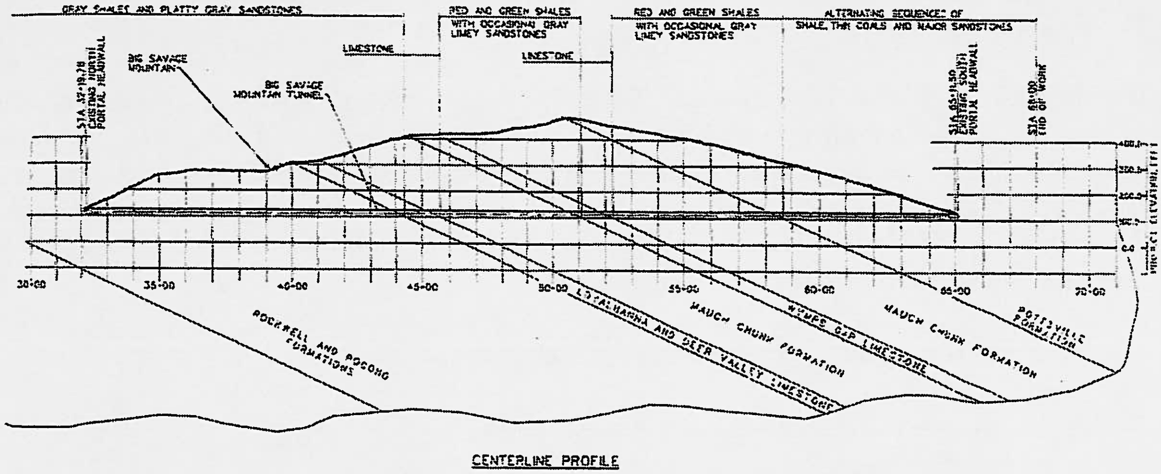
The tunnel and railroad right-of-way was acquired by the Allegheny Trail Alliance to complete the Great Allegheny Passage, a recreational trail extending from Pittsburgh, PA to Washington, D.C..

III. Geologic Setting

Big Savage Tunnel cuts through sedimentary rock strata of the northern limb of the Wellerburg Syncline in southern Somerset County, PA. Along its northwest to southeast trend, the tunnel penetrates the Pocono, Loyalhanna, Mauch Chunk, and Pottsville formations. The strata dip at about 27 ° SE with a strike that nearly parallels the tunnel heading.



A portion of the geologic cross-section B-B' from the PaGS publication *C56A, Geology and Mineral Resources of Southern Somerset, County PA.*



Tunnel Geology (modified after Shaulis, 1995).

The simplified cross-section of the geology along the tunnel trend shows the north to south (left to right) progression of strata from the upper Mississippian Rockwell, Pocono, Loyalhanna, and Mauch Chunk formations to the Pennsylvanian Pottsville formation. The Rockwell and Pocono Formations are comprised of 70% sandstone, 30% shale and siltstone. Neither rock units are calcareous. The Mauch Chunk is one of the principal 'red bed' units in the Mississippian consisting of 56% shale, 36% sandstone, 7% siltstone, and 1% limestone.

The unit responsible for the run-in and compressed air excavation is the Loyalhanna Limestone. The "limestone" designation is a misnomer in this unit since petrographic analysis shows that fresh specimens are 60% silica sand and 40% calcite (Flint, 1981). During the rehabilitation work, samples of white, fine-grained sand nodules were collected from artesian location in the tunnel. These samples had no reaction to HCl and readily crumbled with slight finger pressure. It appears that the calcite cement that bonded the silica clasts had been completely dissolved. The total loss of cement created a highly permeable 'drain' in the stratigraphic column, which was saturated to an elevation at least 195 feet above the tunnel crown. The *Engineering Record, (*Vol. 64, No. 96*)* reported that springs were flowing in the vicinity of the sinkhole before the run-in occurred. After the collapse, the springs never re-appeared.

The geological conditions that combined to challenge the advancement of the Big Savage Tunnel can be summarized as follows:

- Calcite leaching in the Loyalhanna Formation produced a nearly 100-foot thick, highly permeable, sand layer.
- The Loyalhanna was hydrogeologically confined between the Pocono and Mauch Chunk formations resulting in a very high elevation head (± 200 feet).
- The structural orientation of the bedding combined with the tunnel heading presented the contractor with an 'unforeseen' condition that resulted in the months of arduous labor producing minimal tunnel advancement.

IV. Rehabilitation

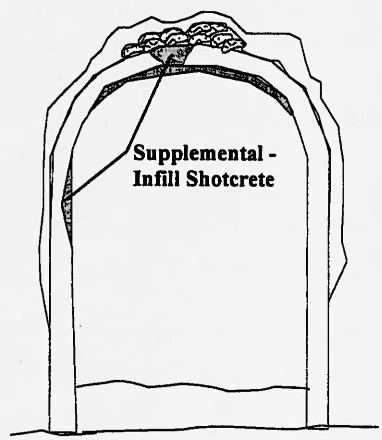
Rehabilitation of the Big Savage Tunnel began with a thorough inspection and assessment of the existing tunnel conditions by the contractor, Advanced Construction Techniques, Ltd. and Gannett Fleming, Inc.. Together, Gannett Fleming, Inc. and Advanced Construction Techniques, working with the Project Owners / Sponsors, developed a rehabilitation strategy using innovative techniques to improve the stability of the existing tunnel liner system.

The rehabilitation strategy involved the following steps:

- Shotcrete Infill / Structural Shotcrete
- Swellex Anchors
- Cellufoam Grouting
- Drilled Drainage Holes
- Geocomposite Wall Drain
- Subdrain Collection System
- Final Shotcrete Liner
- Final Trail Surface
- Portal Reconstruction

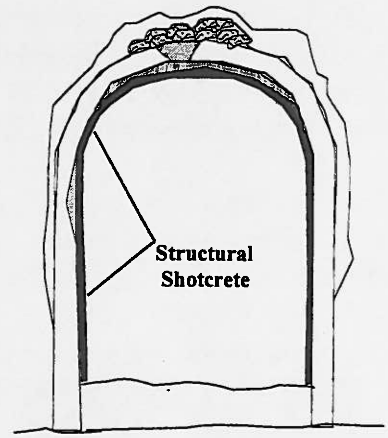
<u>Shotcrete Infill / Structural Shotcrete</u>

A field survey and layout of repair areas was conducted to identify locations of voids in the tunnel crown and walls that required infilling. Other areas throughout the tunnel were identified as structurally deficient where extensive cracking or displacements of the existing concrete tunnel liner system were observed. Void areas were filled with 5,000 psi shotcrete infill. Structurally deficient areas were treated with a 4-inch thick (minimum) layer of 7,000 psi, steel fiber reinforced shotcrete. The structural shotcrete was generally applied around the full perimeter of the horse shoe to supplement the supporting capacity of the existing liner and provide a shell for later installation of the swellex anchors.



The 5000 psi material was installed using the dry-mix shotcrete process. Raw materials were proportioned using a volumetric concrete truck (i.e. mobile truck) which traveled inside the tunnel to the repair areas. The shotcrete material was then placed using a conventional dry shotcrete rotary gun.

The 7,000 psi, steel fiber reinforced, structural shotcrete was installed using the wet-mix shotcrete process. Shotcrete materials were delivered to the jobsite in ready-mix concrete trucks. Upon arrival at the jobsite, steel fibers were added to the truck at a dosage rate of 50 p/cy during mixing. Mixing would continue for a minimum duration of 5 minutes to ensure that the fibers were uniformly distributed into the mix. After mixing, the concrete trucks traveled inside the tunnel to the repair areas. The shotcrete material was then placed using a high volume trailer mounted concrete pump.



The following mix designs were utilized for the 5,000 psi shotcrete infill and the 7,000 psi steel fiber reinforced structural shotcrete:

5,000 psi Shotcrete Infill:	
Cement (lbs)	638
Aggregate	2994
W/C Ratio	0.40

7,000 psi Structural Shotcrete:	
Cement (lbs)	638
Grancem (lbs)	212
Sand (lbs)	1891
Stone (lbs)	800
Water (lbs)	340
W/C	0.40
Polyheed-N oz./yd.	60
Steel Fiber (lbs)	50

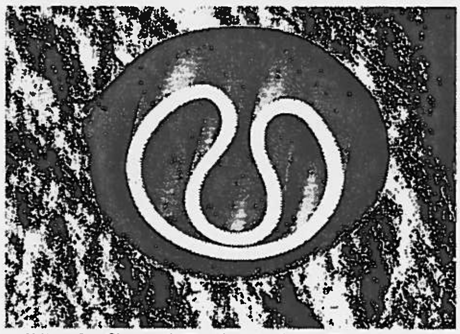
Steel fibers were Dramix ZP305 as manufactured by Bekaert Corporation. The ZP305 fiber is 1 3/16 inch long and 0.022 inch diameter cold-drawn wire with hooked end.

Swellex Anchors

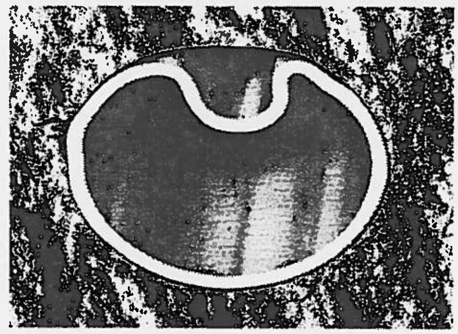
The contract plans called for installation of post-tensioned rock bolts to support the existing tunnel lining. Design called for using Grade 60, #8 epoxy coated all-thread bars anchored with resin epoxy cartridges, post-tensioned to a working load of 23 kips. Installation of the cartridges proved difficult during field testing. The dip of the rock strata, void space behind the concrete tunnel lining, and the backfilling of the tunnel lining with stowed rock caused rock fragments to fall into the boreholes; effectively blocking the installation of the cartridges. Several of the bolts that were installed into the Rockwell and Mauch Chunk Formations failed the 130% fy' pull test due to bond failure between the resin and rock. The mudstone units were subject to rapid water slaking, which created a clay lubrication along the interior of the bolt holes. Drilling with air only did not alleviate the problem because the elevated groundwater table immediately filled the holes. Bolts installed in the massive sandstone and siltstone units performed well.

The design team evaluated the use of mechanical anchors to overcome the slaking and blockage problems. Atlas Copco Construction and Mining, Canada demonstrated the use of their Swellex® rockbolts at the project site. The test section utilizing the Standard Swellex® rock bolts proved the product was well-suited for this application. The team chose the Swellex rock bolts because its advantages mitigated the installation problem that occurred when trying to install the resin bolts.

The Swellex bolt is unique. The bolts are made from circular steel tubes, which have been folded to reduce the diameter. The following figures are cross-sections of an un-inflated and inflated bolt in a borehole.

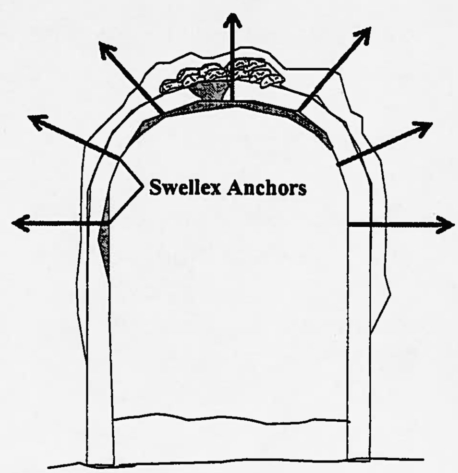


Un-inflated Bolt(Atlas Copco).



Inflated bolt. (Atlas Copco)

Bolt holes were drilled using a fully mechanized Boltec tunnel bolting machine as manufactured by Atlas Copco Inc. The unique design of the equipment permitted hole drilling and bolt installation at one location without repositioning the drill mast. The equipment featured two large booms; the first containing the drillers platform and rock bolt storage; the second containing the rock drill assembly.



Immediately upon the completion of drilling, the bolt is placed into drilled hole and expanded with water using a high pressure pump (4,350 psi). The bolt conforms to fit rock irregularities and can also compact material adjacent to the hole. The advantages of the Swellex system that appealed to the design team follow:

- The load of the rock is transferred to the bolt directly without the use of mechanical locking devices or grouting agents.
- The bolt conforms to the borehole asperities increasing pull-out resistance.
- The installation immediately follows the drilling with the same equipment and personnel.
- Installation is very fast and flexible.
- Presence of water or debris in hole did not impair installation.
- Bolts could be immediately tested since no set time is required.
- The installation and material cost were very effective.

Rock Bolt Design Considerations

Observed damage and distress in the tunnel lining indicated the need to install rock reinforcement before the construction activities in the tunnel began. Based on the physical inspection and the original construction records, the tunnel was categorized into three design reaches.

Reach 1: Stations 33+40 to 47+00

The reach included areas with large crown and wall voids, and well-developed stress cracks in the crown and wall lining. This section included areas of prior crown and wall lining rehabilitation efforts and the compressed excavated section. Additionally, this reach had significant seepage infiltration through cracks in the lining and several plugged drain holes. Based on tributary area calculations and a working capacity of 11 tons/bolt, a 4-foot center-center crown rock reinforcement pattern was designed. The radial pattern extended from two-feet below the springline through the crown to two-feet below the springline on the opposite wall. Nine (9) bolts were installed into each radial section.

Reach 2: Stations 47+00 to 52+00

This reach included area with small crown and wall voids, well-developed stress cracks in the crown and wall lining, and areas of previous crown and wall rehabilitation efforts. This reach experienced limited wall and springline deformation. Few areas of seepage infiltration were observed in this reach. The rock bolt pattern in this reach had bolts on 4-foot centers; confined to the tunnel crown section only.

Reach 3: Stations 52+00 to 64+84

This reach contained few crown and wall voids, well-developed stress cracks in the crown, sporadic cracking in walls, prior crown rehabilitation efforts, and minor areas of seepage infiltration. Two bolt patterns were used in this reach; a 4-foot center-center rock reinforcement bolt pattern within the significantly distressed sections and a 4-foot x 8-foot rock reinforcement bolt pattern in less distressed section. Seven (7) Swellex bolts were used in each radial section.

Compressed Air Section

During drilling and installation of swellex anchors in the area of the tunnel originally construction by compressed air methods, inflows of water and fine sand were encountered requiring a change in the rehabilitation approach. The design / construction team re-evaluated options for rehabilitation in this approximately 230 foot long area and developed a scheme consisting of a 6-inch thick heavily reinforced shotcrete liner. The new shotcrete liner was doweled (mechanically connected) to the existing concrete liner in this area to create a composite section.

Cellufoam Grouting of Voids

Observed tunnel conditions indicated the presence of voids and rubble filled zones greater than 2 feet in height above the tunnel crown. Grouting was necessary to reduce the potential for damage to the concrete liner due to rockfall. In addition, grouting the void space and rubble zones, together with the swellex anchors further reinforced the arch above the tunnel crown, thereby increasing the overall stability of the liner system.

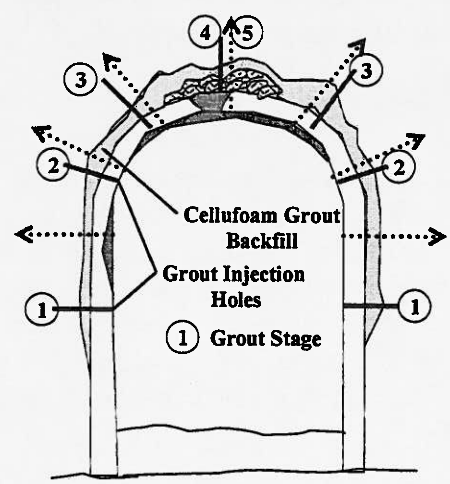
Void space above the tunnel crown was grouted with light weight cellufoam grout. Light weight grout was selected to minimize the load imposed on the liner system during installation and curing. The cellufoam grout consisted of cement, water, and a pre-generated aqueous foam. The following mix design was utilized to provide a yield of 1 cubic yard of cellufoam grout in place:

Type 1 Portland Cement (ASTM C150)	752 lbs. (8 bags)
WF-60 Foaming Agent	29 ounces
(as produced by Cellufoam Concrete Systems)	
Water	44.1 gallons
Wet Density	40 pcf +/- 2 pcf
Compressive Strength (28 day)	200 psi (minimum)

Records of the drilling and installation of the Swellex anchors were maintained and analyzed to estimate the void / rubble fill limits. These logs were then utilized for estimation of grout quantities and comparison to actual grout takes to give reasonable assurance of the effectiveness of the grouting program.

Drilling of grout injection / relief holes was performed with an Atlas Copco Boltec hydraulic drill rig. The hole pattern consisted of 7 radial, 3-inch diameter holes installed on an 8-foot center along the tunnel. Holes were located 8 feet above existing grade and 1 foot above springline (typical both sides) and at 10:00, 12:00 and 2:00 radial through the crown. Each hole was logged during drilling and carried to a depth beyond the void / rubble fill.

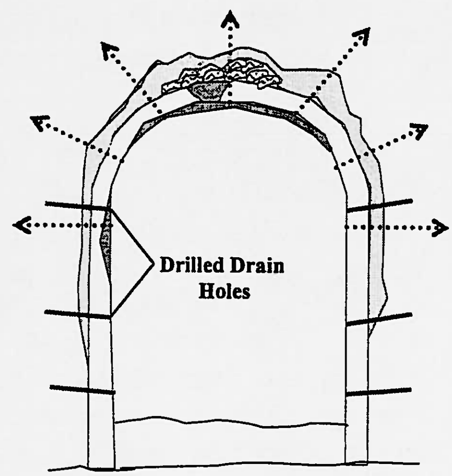
Grouting of the void / rubble fill was completed in a series of 5 stages as shown on the accompanying figure. Stage 1 began with holes located on the walls approximately 8 feet above grade. Stage 2 holes were located 1 foot above spring line. Stage 3 holes were located at 10:00 and 2:00 in the tunnel crown and Stage 4 holes (closure) were located at the 12:00 position. Grout injection was performed through expandable mechanical packers. Grouting continued in each stage until a maximum pressure of 10 psi was achieved at the connection point. Adjacent holes showing evidence of connection via continuous voids



(presence of grout flowing from the hole) were plugged using prefabricated hardwood plugs to contain the grout. Stage 5 holes (verification) were drilled and subsequently grouted at the 12:00 position at 8-foot centers longitudinally between the previously grouted Stage 4 holes to verify that the crown void was filled at the highest point. Over 6000 cubic yards of lightweight cellufoam grout were installed to fill voids, consolidate existing rubble fill and reinforce the tunnel crown.

Drilled Drainage Holes

Drainage holes were drilled to provide drainage paths through the existing liner system to minimize the potential for build up of excess hydrostatic pressure and convey seepage to the geocomposite wall drain to be installed on the tunnel liner.



The drain hole pattern consisted of 3 rows on 8-foot centers, located 1 foot above and, 6 and 10 feet below the spring line as shown on the accompanying figure. Two-inch diameter drainage holes were drilled to a depth of 4 feet to penetrate beyond the existing tunnel liner system. The holes were drilled at a 15 degree up angle from horizontal to provide a positive drainage slope.

Massive Water / Debris Inflow

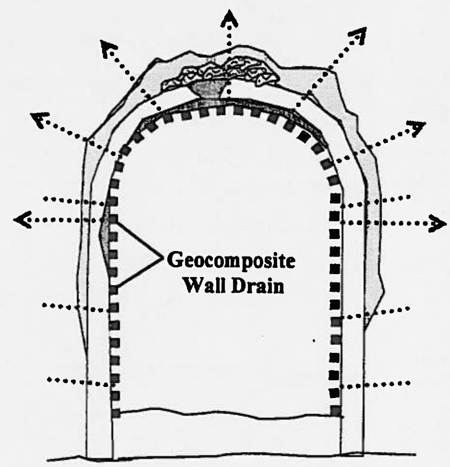
Following a rapid spring thaw and period of heavy rain, massive inflows of water, sand and rock fragments were encountered in the area of the tunnel in close proximity to the section originally constructed by compressed air methods. The inflows occurred through the drilled drain holes and as many as 5 to 6 holes discharging 50 to 100 gpm were observed. Remediation to prevent the inflow of solid material, while allowing water to pass through, thus relieving hydrostatic pressures was required. Shotform, being used on site as backing for shotcrete applications, was installed over the drilled drain holes to "screen" the inflowing material. Observations of continued flow indicated that coarse material was being trapped behind the screen with subsequently finer particles collecting behind the coarse particles. The shotform "screen" in fact allowed the material to build up as a naturally forming filter. Flows through the "screen" gradually became clear, transporting a minimal amount of fines into the tunnel, but allowing a significant amount of water to pass. The reduction in flow however did increase hydrostatic pressures and more drilled drain holes began to produce water with fine sand and rock fragments. The process of "screening" the drilled drain holes continued over a period of several days until all drain holes in the area transmitted water carrying little to no fines into the tunnel.

Geocomposite Wall Drain

A geocomposite drainage system consisting of high-flow dimpled drainage core bonded to a non-woven geotextile fabric was attached to the existing liner system to convey seepage to the subdrain collector system to be installed in the tunnel floor. The geocomposite selected was J-Drain 700 as manufactured by JDR Enterprises, Inc. The characteristics of the geocomposite are as follows:

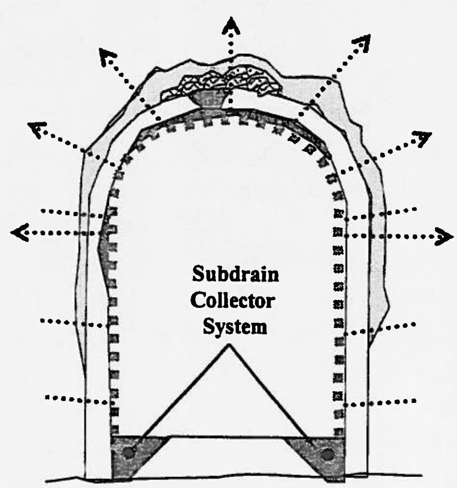
Thickness of Geocomposite	0.40 inches
Compressive Strength of Geocomposite	18,000 psf
Core Flow Capacity	18 gpm / foot of width
Fabric Flow	110 gpm / sq. ft.
Roll Width	4 ft.
Roll Length	50 ft.

The gecomposite drainage layer was affixed to the existing liner system concurrent with a backmesh for subsequent application of the final shotcrete liner. The steel backmesh "Shotform" and geocomposite were affixed to the existing concrete liner system with powder actuated fasters. The geocomposite was installed horizontally in 50-foot lengths starting at the groundline with each subsequent layer "shingled" with the top layer behind the next lowest layer to provide a positive drainage path.



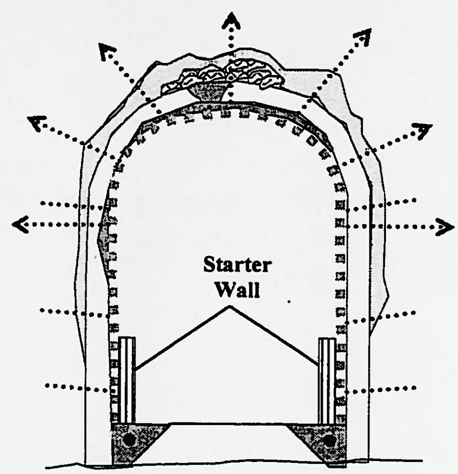
Subdrain Collection System

A dual subdrain collection system consisting of smooth-interior, perforated HDPE pipes was installed parallel and adjacent to the existing tunnel walls to convey seepage to the tunnel portals for discharge into drainage channels located outside the portals. Survey elevations of the tunnel floor and test pits conducted along the walls indicated a high point in the northern third of the tunnel and provided a top of rock line that would limit the depth and profile of drainage system. The piping was installed at or above the top of rock to avoid excavation that could compromise the existing tunnel wall foundations. The dual piping system collects seepage discharged from the geocomposite wall drain as well as seepage observed to be emanating from boils in the tunnel floor and under the tunnel walls.



Observations indicated that the majority of the tunnel seepage originated in the northern third portion of the tunnel. Therefore, dual 15-inch diameter pipes were selected for the northern third, while dual 10-inch diameter pipes were selected for the southern two thirds of the tunnel alignment. Smooth-interior HDPE pipes (ADS N-12 as manufactured by Advanced Drainage Systems) were installed in shallow excavations. The pipes were enveloped in AASHTO No. 57 aggregate surrounded by a woven geotextile fabric (Nicolon FW 402) with a percent open area of 10% and apparent opening size of 40 mm. Tunnel grades were very flat, therefore, the slopes on the drainage system were limited to 0.35 percent from the high point to the north and 0.50 percent from the high point to the south. Due to the limited slopes, the pipes were oversized such that the furnished capacity would allow for potential seasonal fluctuations and sediment build-up in the piping system. In addition, cleanout locations were provided on approximately 300-foot intervals and cross-over interconnections were provided to distribute flows across the dual piping systems. A final cover thickness of 2.5 feet was achieved by placing additional fill inside the tunnel.

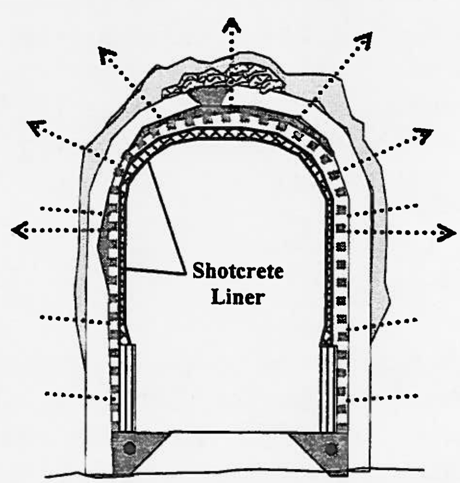
<u>Final Shotcrete Liner</u>



Construction of the final shotcrete liner began with a "starter wall". The starter wall was formed using conventional forming techniques, and cast-in-place using a 5000 psi, wet shotcrete mix. The starter wall was 4-inches thick and provided grade control and support for the application of the 2.5-inch thick shotcrete liner.

The following mix design was implemented for the 5,000 psi shotcrete for starter wall construction:

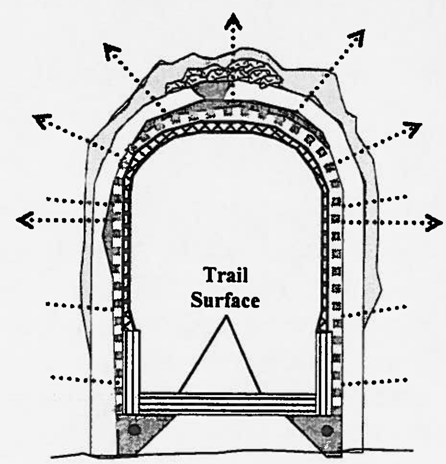
5,000 psi Shotcrete (Starter Wall)	
Cement (lbs)	478
Grancem (lbs)	257
Sand (lbs)	2055
Stone (lbs)	850
Water (lbs)	294
W/C	0.40
Polyheed-N oz./yd.	51



Upon completion of the starter wall, construction of the final lining began. Bulk materials (cement and aggregate) were delivered to the jobsite and stored outside the tunnel in weatherproof enclosures. A semi-automated mobile shotcrete batch plant was mobilized inside the tunnel and was utilized to proportion, batch, and convey the high-performance shotcrete material. Cement was loaded into steel delivery bins and transported to the batch plant by forklift. The aggregate blend was delivered to the batch plant using a conventional wheeled loader. Steel fibers were added at a dosage rate of 50 pcy using automated fiber dosing equipment.

The following mix design was implemented for the 7,000 psi steel fiber reinforced shotcrete for the final lining: Steel fiber reinforcement was added to control / minimize shrinkage cracking.

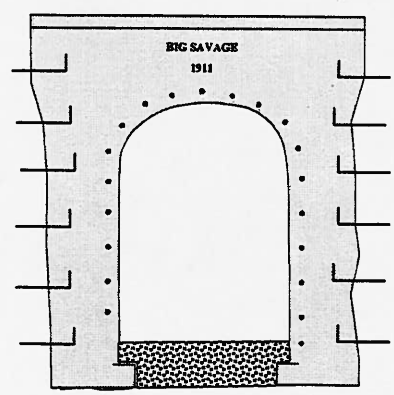
<u>7,000 psi Shotcrete (Final Lining)</u>	
Cement Type SF (Silica Fume)	638 pcy
Aggregate (ACI Gradation No. 2)	2994 pcy
W/C Ratio	0.40
Steel Fiber (Dramix ZP305)	50 pcy



<u>Final Trail Surface</u>

The final trail surface was placed and compacted to provide a walking / biking surface for intended trail users. The final trail surface consisted of 2-inches of AASHTO No. 10 crushed stone over a well graded structural aggregate.

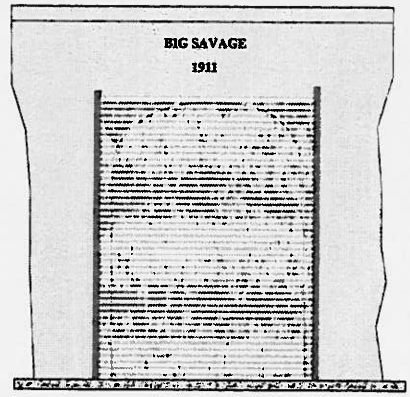
<u>Portal Reconstruction</u>



The portals were designed and constructed to match the original concrete portals (constructed in 1911) to preserve the historical appearance of the structure. Dimensions were taken from remnants of the original portals to replicate the details of the structure. A mold of the lettering was also obtained so that the lettering above the portal would match that of the original structure. Roof drains were provided and cast into the portal walls to convey drainage to the subdrain collector system.

<u>Portal Closure</u>

Observations and review of previous reports indicate that a significant portion of the tunnel's deterioration within the first 100 feet from the portals can be attributed to freeze-thaw. Recognizing the severe winter temperatures and the damaging effects on the tunnel, an insulation layer was included in the final liner system over the first 100 feet from each portal. In addition, a removable "stop-log" closure system was provided that could easily be removed and replaced by maintenance crews during winter closing of the tunnel trail. The "stop-log" closure system consisted of steel beams (W6x9) bolted to the face of each portal. Pressure treated 4"x4" lumber was installed between the flanges of the steel beams and effectively isolated the tunnel from the winter elements. The pressure treated 4x4's are easily installed and removed by hand, are readily available and can be replaced when necessary without significant cost. The bolted on steel beams can also be easily removed during the in-season trail usage period if desired.



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SECOND AVENUE CSO TUNNEL
NASHVILLE, TENNESSEE

Daniel J. Hurst, P.G., Geotechnical Unit Manager, AMEC Earth & Environmental, Inc. Nashville, TN.
James Wm. Martin, P.E. VP, AMEC Earth & Environmental, Inc. Nashville, TN.

ABSTRACT

This paper discusses the geotechnical and environmental considerations for a 3500± LF, 102-inch (I.D.), CSO tunnel that parallels and connects to the Cumberland River in Nashville, TN. Topics include the subsurface exploration and testing programs completed to provide information to both tunnel designers and contractors regarding the site geotechnical and environmental characteristics. A primary focus is the concept of, and benefits derived from, utilizing a Geotechnical Data Report and Geotechnical Baseline Report.

The tunnel excavation was advanced by a fully shielded TBM and most of the alignment was through thin-bedded limestone. However, an approximately 500-foot long zone was significantly degraded by faulting and karst activity such that open and soil-filled caves, detached blocks and zones of running ground were present. Compaction grouting was employed to stabilize the ground prior to TBM passage, and steel ribs and lagging were used as primary lining. At one end of the alignment, a braced excavation 60 feet in depth (50 feet of fill, 10 feet of bedrock) within a buried stream valley was required in order to construct the Kerrigan Diversion Structure and to tie in with a 100 year old, 16.5-foot diameter, brick tunnel; a series of caves were discovered at depths of 15 to 30 feet below the bottom of the structure. In addition, contaminated soils, high ground water and a near-by superfund site were considerations.

The Geotechnical Baseline Report (GBR) used for this project provided subsurface data and contractual baselines that formed the basis of contractor bids, including detailed descriptions of anticipated ground behavior and construction considerations. After completion of the tunnel, both the Owner and Contractor acknowledged that the information in the GBR was instrumental in communicating the difficulty of the project, prior to bidding, such that the project could be executed within-budget, on-schedule, and without undue safety concerns.

PROJECT DESCRIPTION AND BACKGROUND

The recently completed Second Avenue Tunnel project was one of over 100 projects in the Metropolitan Nashville-Davidson County Overflow Abatement Program, being implemented to respond to a 1990 Commissioner's Order from the Tennessee Department of Environment and Conservation (TDEC) to abate Combined Sewer Overflows (CSOs) in Metropolitan Nashville.

The concept of the Second Avenue Tunnel project was to minimize overflows in an area north of downtown by conveying the combined sewage from the existing Kerrigan Outfall to the Central Wastewater Treatment Plant.

The project required construction of the Van Buren Junction Chamber near the Central Pumping Station at the north end of the project, as well as a tie-in to the existing storm water system via the Kerrigan Diversion Structure at the south end.

The Second Avenue Tunnel sewer pipe is 102 inches in diameter (inside diameter) installed in a machine excavated, 11-foot diameter tunnel with a total length of 3,437 feet.

As shown in Figure 1, the tunnel alignment generally follows the right-of-way of Second Avenue North. Planned tunnel invert elevations range from 341.66 feet (MSL), at the Central Pumping Station wetwell to 368.50 feet MSL, at the proposed Kerrigan Diversion Structure. Depth of the tunnel crown below the existing ground surface ranges from 42 to 58 feet between the Kerrigan Diversion Structure and the Van Buren Junction Chamber and from 58 to 71 feet between the Van Buren Junction Chamber and the Central Pumping Station.

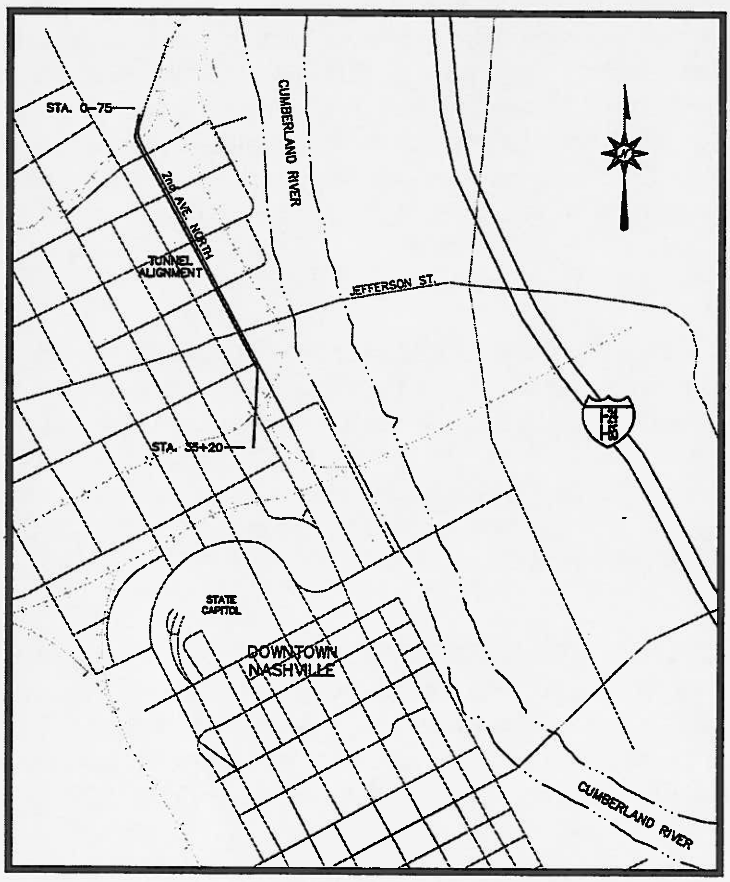

FIGURE 1 – PROJECT LOCATION

Geotechnical Baseline Reports

Because of the difficulty in accurately defining subsurface conditions and ground behavior, underground construction projects have often been associated with construction difficulties, delays and costly litigation. However, beginning in the mid 1970's, tunneling practioners and the National Research Council's U.S. National Committee on Tunneling Technology, and later, the Underground Technology Research Council (UTRC), began to address these issues. After review of a large number of underground projects, one of their main conclusions was that the higher the investment in defining and clearly communicating the subsurface conditions, the lower were the final construction costs of the project. Thereafter a series of publications provided guidance for interpretive geotechnical reports and details of issues that should be covered. One of the more recent publications, which attempted to distill the previous reports and the feedback received from the industry, is: "*Geotechnical Baseline Reports for Underground Construction*"(ASCE, 1997). That guideline document recommended that the interpretive report included in the contract documents be called a Geotechnical Baseline Report (GBR).

Following the guidelines established by the UTRC, a Geotechnical Baseline Report was prepared for the Second Avenue Tunnel project. The GBR prepared for the Second Avenue Tunnel project was the first use of such an approach for a major tunnel in the State of Tennessee.

As opposed to the typical approach taken by many designers and Owners whereby geotechnical data is provided to bidders, along with disclaimers about the accuracy and the Contractor's ability to rely on the geotechnical data, the primary purpose of the GBR for the project was to establish a binding contractual statement of the geotechnical conditions to be encountered during underground construction. This contractual statement is referred to as the "baseline." Risks associated with conditions consistent with the baseline are allocated to the Contractor, while those more adverse than the baseline are the Owner's. The GBR also presents the geotechnical and construction considerations that form the basis of design.

The baseline assessment, the risk allocations, and construction considerations included within the GBR represent a significant shift in philosophy for why the report is prepared, and how it is intended to be used, as compared with a typical geotechnical report. The application of this philosophy as to types of studies and methods of reporting the geotechnical conditions for the Second Avenue Tunnel project resulted in significant improvements in terms of understanding of geologic conditions and the risks involved, selection of the type of equipment required, and allocation of risks between parties. In addition, bidders could remove contingencies from bids as budgetary allowances were established for various subsurface conditions that might affect progress. Finally, the GBR resulted in substantial cost savings for the project, allowed the contractor to meet schedule, and enhanced project safety.

The primary value of a GBR to the engineering profession is that it provides a means of minimizing and resolving disputes in underground construction while allowing for rational design solutions to anticipated field problems. Underground projects present unique risks that must be assumed by either the Owner or the Contractor. By far, the greatest risks are those associated with the subsurface materials encountered along with their behavior during excavation and installation of support. Definition and allocation of these risks are the primary focus of the GBR. Adequate subsurface investigation, professional interpretation of results, and clear communication of baseline conditions in Contract Documents reduces the risk to the Owner and Contractor, allows for more competitive bids, reduces disputes, and provides for objective criteria to be used in determining differing site conditions. Such an approach elevates the stature of the engineering profession as true "team" players and problem solvers.

For the 2nd Ave project, the GBR contained the following statements; *"This document is a summary of geotechnical conditions to be assumed for purposes of Contractors' baseline pricing for the proposed Second Avenue Tunnel and its related vertical structures. The purpose of this Geotechnical Baseline Report (GBR) is to describe assumed geotechnical conditions and their influences on the project design and construction. This GBR is intended to assist the Contractor in bidding and planning the work, and assist the Engineer and Owner in reviewing the Contractor's submittals and operations. This document has been developed solely to provide a level playing field for all bidders. This GBR is part of the construction contract documents and the Baseline statements herein represent contractual assumptions of ground conditions to be encountered during the project. The geotechnical data for this project is contained within a geological data report (GDR). The conditions discussed in the GBR involve interpolation and extrapolation beyond the data presented in the GDR. Therefore, the Baseline assumptions may or may not be solely based upon the information in the GDR. Since Baseline statements do represent contractual assumptions, this GBR is the only interpretative source of subsurface information from which the Contractor can use in preparation of his bid and project related submittals."*

SECOND AVE TUNNEL

Tunneling operations in Middle Tennessee have utilized hard rock tunnel boring machines for over four decades. However, no previous tunnel projects have encountered the extraordinary ground conditions identified along the alignment of the Second Avenue Tunnel project. The authors firm analyzed the subsurface data generated by others and interpreted this data to form the baseline conditions for bidding purposes. After completion of the tunnel, both the Owner and Contractor acknowledged that the information in the GBR was instrumental in communicating the difficulty of the project, prior to bidding, such that the project could be executed within-budget, on-schedule, and without undue safety concerns. The analysis summarized in the GBR indicated that, for the first time ever in Middle Tennessee, a fully shielded Tunnel Boring Machine (TBM) would be required to successfully complete the tunneling. The GBR correctly identified the location, length, and characteristics of "bad ground" zones along the tunnel alignment, and made recommendations for how to proceed with construction within bad ground. Such specialized recommendations added initial cost to the project, but allowed the Contractor to mobilize the proper equipment and materials necessary for advancement of the tunnel with minimal delays for installation of temporary ground support required for worker protection.

The complexity of clearly communicating the expected ground conditions was made all the more difficult by the extraordinary subsurface conditions encountered including:

- **Faulted Zone** – A faulted zone was identified along the 2nd Avenue alignment between Taylor and Monroe Streets (see Figure 2). Weathering along fractures had been extensive, such that the Faulted Zone existed in a chaotic and predominantly decomposed state. The primary problem was to estimate the response of the faulted zone to tunneling operations and to quantify the remedial measures required.
- **Collapsed Zone** – A 120 feet long zone was characterized as a region of collapsed bedrock, consisting of caves, mud, and distorted bedrock. The Contractor was required to stabilize the ground prior to tunneling operations.
- **Cave System** – Project exploration at the south end of the project, in the vicinity of the Kerrigan Diversion Structure, revealed significant karst weathering. At several locations, weathering was so extensive that large open cavities, greater than 50 feet in vertical extent, had developed. The discovery of the extensive network of karst led, ultimately, to the shifting of the Kerrigan Diversion Structure about 40 feet west. Detailed recommendations concerning foundations and excavation above the cave system were also included in the GBR.

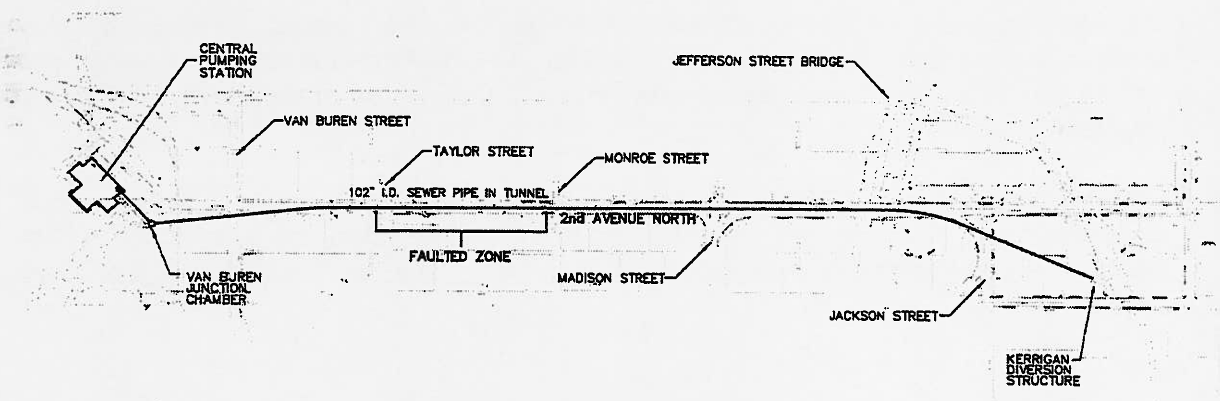

FIGURE 2 – PROJECT LAYOUT

GENERAL SITE GEOLOGY

The Central Basin of Tennessee is a large elliptically shaped topographic depression, centered in Rutherford County and encompassing all of Nashville. The dominant geologic structure is a broad dome. Superimposed on the dome are many minor folds. In general, the regional dip of bedrock is less than 25 feet per mile, with maximum localized dip generally less than 5 feet vertical per 100 feet horizontal. Faults with displacement of a few feet to tens of feet occur in Nashville, but most have not been mapped.

Published literature indicates the surficial geologic unit across the site is alluvium, deposited within the ancient flood plain of the Cumberland River. Published geologic literature also indicates, and the results of the subsurface studies support, that Ordovician age bedrock units are present at fairly shallow depths. A combination of alluvial deposits, residual soil and man-made fill overlie the bedrock units.

Soil

Overburden consists of man-made fill, alluvial deposits (alluvium) and residual soils (residuum). The thickness of the overburden is highly variable and ranges in thickness from two feet thick towards the north end of the project to 70 feet thick within the Faulted Zone, to 54 feet thick at the project's south end.

The alluvial deposits are from the Cumberland River. They are general y interbedded layers of silt and/or fine sand, clay and gravel. The thickness of the deposit is variable, ranging from 0 to 42 feet thick. The thicker deposits are present in the vicinity of the Kerrigan Diversion Structure.

The residuum consists of highly plastic clays, silty clays and clayey silts with variable amounts of sand. Where present, the residuum ranges from less than one-foot thick to 37-feet thick. Maximum thickness of overburden including fill, alluvial and residual soils was about 70 feet.

Bedrock

Rock formations within the Nashville area are generally carbonates susceptible to development of karst features. Karst features form by dissolution; e.g. removal of calcium carbonate in limestone by chemical weathering. Common karst features are evidenced by sinkholes, widened soil filled or open fractures/cutters, caves, pinnacled bedrock, and underground drainage.

The Hermitage Limestone is the host formation for the tunnel. Much of the formation is a limy, sandy siltstone with disseminated clay. It is thin-bedded with beds a few inches thick and thin shale interbeds. In some areas, weathering along fractures has been extensive, so that the Hermitage Formation exists in a predominantly decomposed state. In areas along the alignment where the fracture density is relatively slight and where they are generally tight, the Hermitage is a relatively competent unit.

The Carters Limestone is beneath the Hermitage. The upper portion of the formation is about 10-feet thick and has thin shale partings between beds, typically 6-inches or less thick. A significant bentonite bed (T-3) up to 12-inches thick separates the upper member from the lower member. The lower member is dense- to fine-grained with small cherty nodules. The Carters is prone to aggressive dissolution along secondary openings in the bedrock mass. Weathering occurs unevenly along widened, near-vertical joints and along bedding planes. Open sinkholes, depressions, underground streams and cave systems are common. In some areas, the cave advancement has ascended into the overlying Hermitage Formation.

EXPLORATION AND TESTING

The following subsections summarize the project's site exploration and testing programs. The level of study completed for this project is a example of the type of detailed investigation required to provide the specific data needed for successful underground design and construction.

Environmental Assessments

A Phase 1 Environmental Assessment was conducted along the proposed sewer tunnel alignment to assess sites with the potential for creating adverse environmental impacts to the project. The data collected during the Phase 1 Environmental Assessment did not reveal a record of known contamination or substantive violations within the study area. The study designated two properties that suggest that the potential exists for contamination. As a result of the Phase 1 findings, a Phase 2 Assessment was conducted whereby selected soil and ground water samples were submitted to a laboratory for analytical testing.

Analytical Testing Program

During drilling operations along the tunnel alignment and around the Kerrigan Diversion Structure, soil samples from selected boreholes were screened using an Organic Vapor Meter (OVM). The sample with the highest OVM reading was submitted to an environmental laboratory for analysis of the following contaminants: total organic halogens, oil and grease, total petroleum hydrocarbons (TPH-IR), gasoline range organics (GRO), diesel range organics (DRO); the eight TCLP metals, plus copper, nickel, thallium and zinc; and six constituents of the pyrene family. Ground water samples from the four monitoring wells were submitted to the laboratory for the following analysis: TPH-IR, GRO, DRO; eight TCLP metals, plus copper, nickel, thallium and zinc.

Site Exploration

For the GDR, 90 test borings (geotechnical and/or environmental) and 55 air percussion drill holes were drilled. Of these, 40 test borings were drilled along the proposed tunnel alignment, including 2 angle borings. Two borings were drilled near the Van Buren Diversion Structure No. 2. Twenty-nine test borings, 2 angle borings and 55 air percussion drill holes, were drilled around the proposed Kerrigan Diversion Structure. Six other borings were drilled as monitoring wells in nearby areas.

Observation wells were installed in 18 locations to monitor ground water levels, conduct hydrologic tests and to provide ground water samples.

Seventy-seven of the test borings drilled for the project were extended through the overburden into the underlying limestone bedrock. Individual logs and photographs of the recovered core were provided to contractors. The coring termination depths ranged from 14.7 to 126.6 feet below ground surface. The initial borings were terminated at predetermined depths below the existing ground surface to provide a general soil and rock profile along the proposed tunnel alignment. Once bedrock strata were profiled and concerns such as clay seams, highly fractured or weathered zones and open cavities were identified, additional borings and air percussion drill holes were performed to further define the bedrock profile and to provide an estimate of the relative size and extent of bedrock anomalies.

Geotechnical Testing Program

The geotechnical field testing program included Standard Penetration Test (SPT), undisturbed samples used for laboratory testing, pressure (packer) test, slug test, pump test, measurement of static water levels and geophysical testing (temperature, caliper, density and neutron density).

Core samples of the bedrock were recovered to select intervals for physical laboratory tests and to describe the bedrock lithology, i.e. megascopic physical characteristics of the formation, such as color, texture, extent and relative degree of weathering, presence of fractures and bedding planes. In addition, the coring operations reveal general, yet important, geotechnical engineering aspects of the bedrock as expressed in terms of Rock Quality Designation (RQD) and percent core recovery.

Simple standpipe piezometers and/or monitoring wells were installed in 20 borings to collect information regarding ground water levels and hydrologic properties. Static water levels from 13 of the wells and river stage levels were recorded approximately weekly. The piezometers and wells provided a means to conduct a variety of tests to evaluate the hydrologic properties of the natural soils and bedrock. The different tests included slug test, packer test and pump test.

Extensive laboratory testing on soil and rock samples was completed, and the type and number of tests are summarized below.

Test Type	Number of Tests Completed	
	Soils (1)	Bedrock (2)
Natural Moisture (ASTM D 2216)	149	NA
Wet Unit Weight (pcf)	8	0
Dry Unit Weight (pcf)	8	0
Specific Gravity (ASTM D 854/C 127)	4	0
Atterbergs Limits (ASTM D 4318)	22	NA
Grain Size - 200 Wash	9	NA
Vertical Permeability (ASTM D 5084)	4	0
Triaxial (Unconsolidated, Undrained ASTM D 4767)	4	0
Corrosivity pH and resistivity	4	0
Unconfined Compressive Strength (ASTM D 2166)	0	14
Splitting Tensile Strength	NA	14
Hardness (Schmidt Rebound)	NA	8
Cherchar Abrasivity Index	NA	11
Punch Penetration (average, peak and areal)	NA	10
Loss on ignition	0	10
Chemical Composition	0	10

GROUND CHARACTERIZATION

Within the GBR, the physical and environmental characteristics of the overburden and bedrock were presented, but specific values for Baseline assumptions were also provided. In addition, the GBR discussed the expected ground behavior and how that might influence the selection of construction equipment or methods. The baseline assumptions were contractually binding values as to expected conditions, both geotechnical and environmental; if conditions were as the baseline assumed, the contractor had no basis for any extra cost claims. In addition, the GBR language was coordinated with the contract documents to set-up allowances for items that could not be fully quantified in advance. Some specific examples from the GBR that address the more difficult areas such as the faulted area (where only minor ground movements occurred, but the fault weakness allowed accelerated karst weathering) and the Kerrigan Diversion Structure area, where a cave system existed at shallow depths below the structure, include:

Faulted Zone

Soil Conditions

"Baseline assumptions of soil conditions within the Faulted Zone are not based on the findings presented in the GDR. The GDR identifies this faulted zone between Stations 11+00 and 15+00. An alternative interpretation of the Logs (GDR, Appendix B) has been developed

to describe ground conditions believed likely to be encountered through this section of the project. The GDR estimates that the overburden extends from the ground surface to the depth of auger refusal suggesting a overburden thickness ranging from 8.5 to 44.5 feet. For the purposes of this GBR and Baseline considerations the thickness of the overburden was extended to a depth where the bedrock quality is judged (based on RQD's) relatively competent. The alternative interpretation is presented because it provides the Contractor with a greater sense of the excavability of the earth materials likely within the Faulted Zone. Within this GBR the faulted zone exists from Station 9+50 to Station 15+00.

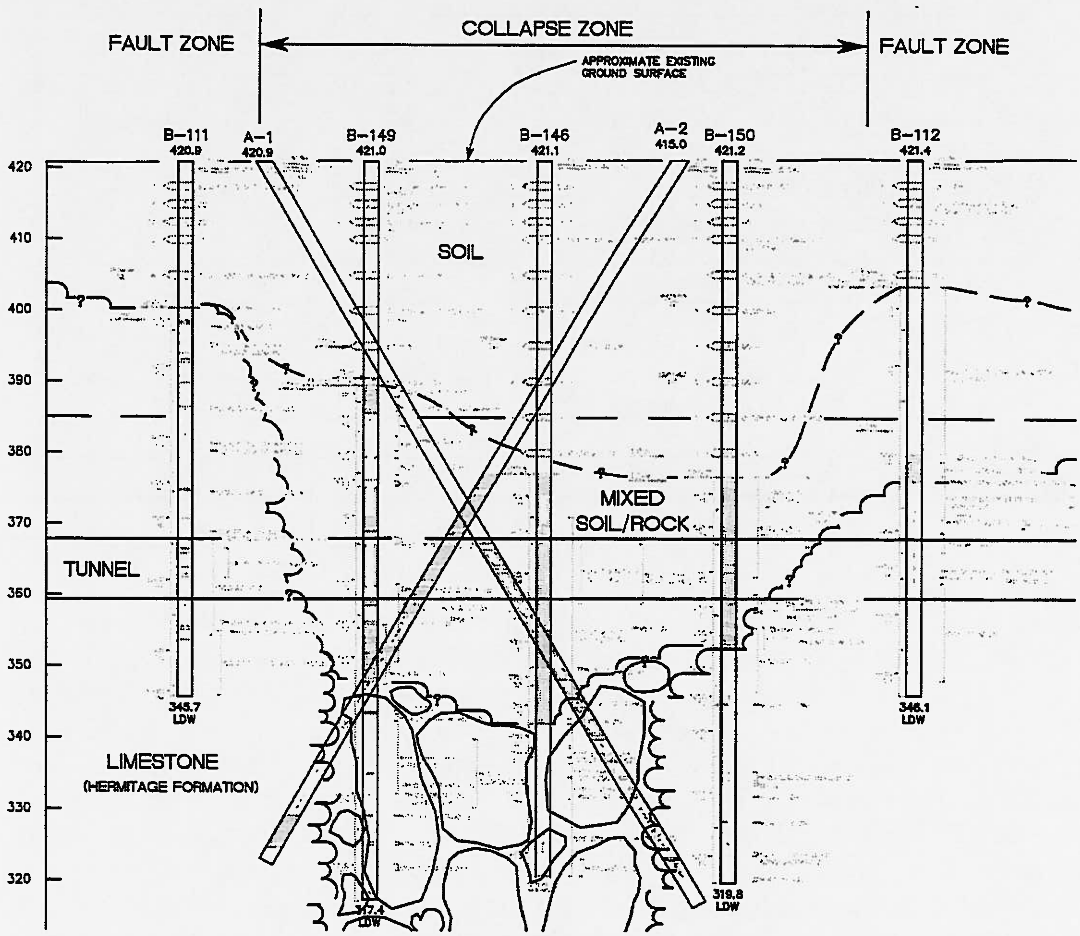

FIGURE 3 – FAULTED ZONE

"The thickest soils within the Faulted Zone (greater than the average thickness) exist between Stations 12+30 and 13+50. This segment of the Faulted Zone is hereafter referred to as the Collapsed Interval.

"Within the Faulted Zone and Collapsed Interval the following Baseline values for soil conditions are established.

- *For Baseline conditions, regardless of origin, no material distinction is made between fill and residuum. The overburden thickness between stations 9+00 and 12+30 and between 13+50 and 15+00 is 25 feet; the overburden thickness within the Collapsed Interval is 80 feet.*
- *Overburden soils mixed with bedrock blocks will be intercepted by the tunnel excavation through the Collapsed Interval beginning at Station 12+30. The tunnel excavation will exit mixed overburden/bedrock blocks at Station 13+50.*
- *Soft, very soft soil and running ground conditions will be encountered within the Collapsed Interval. For field identification soft soils are easily penetrated several inches by a workman's thumb, very soft soils are easily penetrated several inches by a workman's fist and running ground is considered as ground that is not capable of supporting its mass and flows into the tunnel excavation, either in a wet or dry state.*
- *Float blocks and boulders will be encountered through the Collapsed Interval. For field identification boulders are between 12 and 48 inches in the longest dimension and float blocks are greater than 48 inches in the longest dimension. Through the Collapsed Interval of the Faulted Zone the excavation will encounter up to 50 boulders and up to 10 blocks of float."*

<u>Bedrock Conditions</u>

"Between the referenced stations, the rock core descriptions indicate the extent to which the bedrock has decomposed. Bedrock is described as stained. Open and mud filled cavities and zones of intense fracturing are common. The inclined bedding planes (some as much as 60°) indicate numerous large, disconnected bedrock blocks along this segment of the project.

"Between stations 9+50 and 15+00, at the sewer pipe invert and crown elevations, RQD values range from 0 to 100, with a range average of 45 to 76. This range of values indicates that the overall bedrock quality varies from poor to fair rock. This correlates with the General Tunneler's Classification as shattered, very blocky and seamy to blocky and seamy.

"Within the Collapsed Interval (between stations 12+30 and 13+50) ground degradation intensifies to a greater extent and the rock quality is very poor to poor, with the RQD range averaging between 9 and 46 (crushed to shattered, very blocky and seamy).

"Within the Faulted Zone the following Baseline values are established. Since most of the earth material within the Collapsed Interval is deemed soil and disconnected rock float within a soil matrix the following Baseline values exclude Baseline assumptions for the Collapsed Interval in Section.

- *The rock type to be excavated is slightly weathered to decomposed shaley limestone. Where slightly weathered the nominal shale bed thickness is 2 inches. Where severely weathered to decomposed, the shale has been altered to silty, sandy clay.*
- *The quality of the bedrock to be excavated is poor; shattered, very blocky and seamy.*

- *Baseline values for the bedrock's engineering and construction properties are presented in Table 11.0.*
- *The tunnel excavation will exit rock conditions and enter mixed soil/rock conditions at Station 12+30 and re-enter rock conditions at Station 13+50."*

Water Inflows

"The Faulted Zone portion of the tunnel is expected to contain significant potential for ground water inflows. The volume of inflows can be reduced by the Contractor's efforts to improve ground conditions, by sealing open joints and fractures prior to tunneling. If no efforts are made to mitigate the inflow, potential inflows will be very large, greater than 5,000 GPM. Since that volume of water cannot easily be handled, it is necessary that the Contractor's Work Plan include measures to isolate the tunnel from the surrounding water bearing zones, such that less than 500 GPM is experienced. The inflows from this portion of the work are NOT included in the baseline inflows and any work required to reduce inflows to 500 GPM shall be incidental to the work. To avoid damage to surrounding structures bearing on shallow foundations, and to minimize the risk of drawing contaminated water into the excavation, the water control plan must avoid any significant draw-down in the surface of the water table."

Construction Considerations

"Based on the amount and extent of significant weathering and bedrock weaknesses noted within the bedrock cores, excavation and progression of the tunnel through this portion of the project will be difficult. ***Regardless of the method of excavation chosen by the Contractor, the work will be slowed.*** *Conditions worsen between 12+30 and 13+50 (the Collapsed Interval).*

"Bedrock weathering has resulted in deep, wide zones of open or filled fractures and cavities. Where filled they may contain soft or running soils and contain significant quantities of ground water. There is the potential for encountering dislodged rock slabs and/or boulders, mudflows and inadequate structural integrity of the remaining rock.

"Primary support will be required from Station 9+50 to 15+00. Other localized zones may require watertight /mud-tight support systems.

"The method of construction the Contractor chooses to use through this section must provide worker safety as well as sufficient structural support. The support system must be installed contemporaneously with the excavation. If a TBM is used to mine through the Faulted Zone, then, through this section at least, it must be fully shielded.

"Within the Collapsed Interval, water-tight/mud-tight construction techniques will be necessary. Also, in its present condition, the ground is judged incapable of supporting a TBM. Prior to tunneling through the Collapsed Interval, the Contractor must stabilize the ground. The ways and means to accomplish ground stabilization shall be the responsibility of

the Contractor. Prior to performing the required stabilization, the Contractor must submit to the Engineer, for review and comment, his Work Plan for the ground stabilization.

"The design of all temporary ground support systems (including shoring and bracing) will be the responsibility of the Contractor. In particular, the Contractor is to evaluate the Faulted Zone, Collapsed Interval and all open-cut portions and design applicable methods for stabilizing the ground, progressing the work through these zones and appropriate ground support systems. The methods selected shall be those that will meet all performance and schedule requirements listed within the project documents. ***The Contractor's attention is directed to the requirement that ground stabilization through the Collapsed Interval (Station 12+30 to Station 13+50) be completed, by the Contractor as part of his base bid, prior to his excavation through this area."***

In order to facilitate tunneling through the Fault Zone and Collapsed Interval with a Tunnel Boring Machine (TBM), the geotechnical designers and the tunneling contractor elected to perform ground modification consisting of compaction grouting. Use of compaction grouting to provide stabilized ground for tunneling in karst bedrock was a fairly unique ground modification program. The compaction grouting, completed in two phases, resulted in injection of over 1000 cubic yards of grout over a 120-foot length of the alignment. Based upon the grout volume injected and the area involved, that grout quantity equates to replacement or displacement of about 50% of the ground volume involved. The contractor indicated that, had the GBR not identified that zone in advance, delays to change equipment and methods to deal with this 120-foot long zone could have easily resulted in an extra cost to the project of over $1 million, and delays of 3 to 6 months.

Kerrigan Structure

Water Inflows

"For the most part inflows of water into the tunnel leading to the Kerrigan Structure are expected to be relatively minor, on the order of 10 GPM to 100 GPM; however, inflows from discrete fractures of several hundred GPM can be experienced. For Baseline assumptions maximum sustained inflows through this section of the tunnel will be 1,500 GPM. The ***Contractor must be prepared to handle a total of 3,000 GPM by traditional methods through this zone, but handling volumes above 1500 GPM will be compensated through an allowance."***

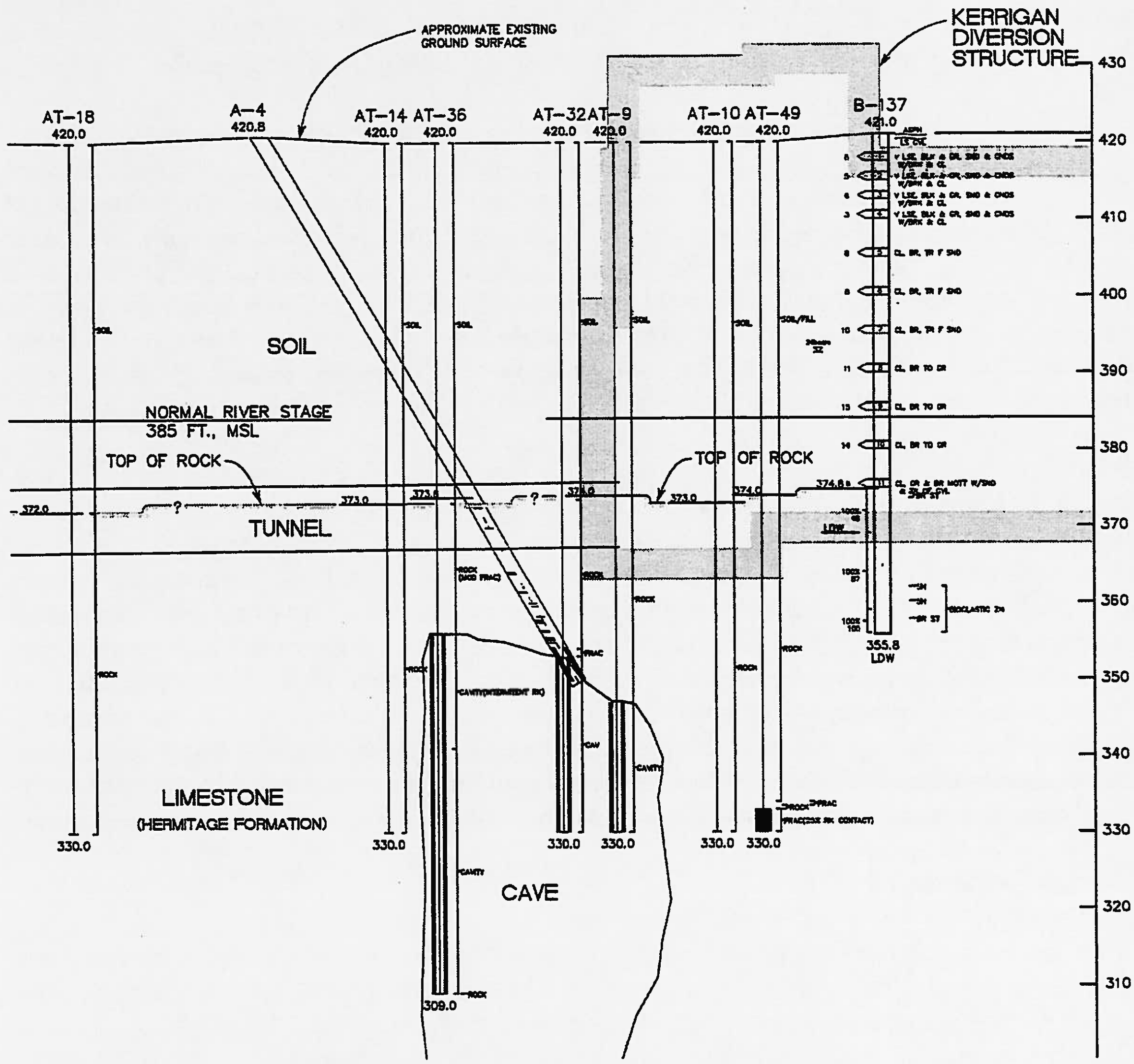

FIGURE 4 – KERRIGAN STRUCTURE

<u>Soil Conditions (Environmental)</u>

"Based on the analytical results it is anticipated that excavated materials will not be subject to special handling or disposal requirements because of their environmental characteristics. One exception is within the Kerrigan Structure where some of the excavated materials will need to be disposed of as special waste or construction debris. ***A separate bid item will be included for disposal of those materials.*** *Also, it is important to note that no excavated materials can remain on-site. It is also important for the Contractor to realize that he becomes the owner of all excavated materials and is wholly responsible for proper disposal of all excavated materials.* ***To ensure proper disposition of the excavated materials, the Contractor must implement a Sampling and Analysis program meeting the minimum requirements stipulated by the Specifications.*** *Even though the analytical sampling*

conducted to date indicates that there is only a limited potential for soil to be contaminated, there will be a major impact to costs and schedule if no provisions are made for dealing with unexpected contaminated soils. There is also significant potential liability associated with improper disposal of contaminated soil, even if it was not known at the time of disposal that the soil contained contaminants. ***Therefore, in the event additional materials beyond those included within the appropriate bid item are encountered, the Contract Documents provide an allowance for handling and disposal for soil characterized as a regulated waste."***

<u>Ground Water Conditions (Environmental)</u>

"For Baseline assumptions, the chemical quality of water entering and pumped from the excavations will be deemed acceptable for discharging into the City sewer system. ***An allowance is contained within the bid documents to compensate the Contractor for his actual efforts in disposing of the water in the event that the Contractor's Sampling and Analysis program indicates that the waters chemical quality is deemed unacceptable for discharge into the sewer system.*** *The Sampling and Analysis program must consider not only routine sampling, but also the program must include for increased sampling frequency if there is an indication of olfactory or visual contamination. Further, the Sampling and Analysis program must provide for monitoring of gaseous vapors within the water holding vessel (silt tank) required for solids control prior to discharge. This monitoring is to be accomplished by use of a lower explosive limit (LEL) metering device measuring the atmosphere within the holding vessel. No discharge to the City sewer system can begin or continue if the LEL alarm is activated. Additionally, prior to discharge into the City sewer system the water must meet standards for suspended solids. The Contractor shall be responsible for ensuring that prior to discharge the water meets this and all applicable discharge standards as described within the Specifications."*

<u>Construction Considerations</u>

"At approximate Station 33+50 the thickness of rock cover decreases to about one half the assumed tunnel diameter of 12 feet. South of Station 33+75 the tunnel excavation will encounter relatively difficult advancement, due to degraded rock quality, mixed conditions, increased water inflow, more and larger joints and soil filled seams. These conditions will necessitate construction techniques that include primary ground support and face control during mining. Furthermore, dewatering, water control and water quality monitoring will be essential aspects of tunnel construction.

"During the driving of sheetpiles (if utilized) as well as when the existing sewer is cut open to allow construction, the Contractor must take appropriate precautions to avoid damaging the brick-lined sewer. If the sewer is inadvertently damaged, or during the removal of the section required to construct the Kerrigan Diversion Structure, the brick sewer may collapse if not supported. The exterior of the brick sewer must be protected from damage.

"If the Contractor elects to open-cut the sewer pipe excavation (south of Station 33+75) then the Contractor shall design appropriate shoring and bracing for the Open-Cut Segment of the project excavation. The excavation shoring and bracing design shall meet or exceed the

latest applicable OSHA requirements. To assist the Contractor in his design, attention is called to Table 24.0 (Liner Loading) and Figure 3.0 (Braced Retaining Wall Loading Diagram) of this GBR.

"The contract documents include a budget allowance from which to compensate the Contractor for efforts to reduce specific inflows or to increase pumping capacity above 1500 GPM within the Laminated Rock tunnel portion of the project. If the anticipated inflow limits stated herein will create difficulties for the Contractor in conducting the work, the specifications permit him to reduce any or all inflows at his discretion without additional compensation.

"In and around the Kerrigan Diversion Structure, numerous open, water-filled voids (caves) were encountered. For the most part, there appears to be 10 to 15 feet of rock cover above the cave openings. If the cap rock above the cave(s) is fractured by blasting operations a massive inflow of ground water will occur, and support for the structure will be lost. ***Because of the severe consequences that will result if the bedrock cover above the caves is breached, blasting will not be allowed for bedrock removal within the Kerrigan Diversion Structure excavation, or for installation of the 50 linear feet of 102 inch sewer pipe that connects to the structure. The use of pneumatic rams or, an expansive chemical breaker will be the likely method of excavation selected by the Contractor."***

CONCLUSION

Underground construction is typically fraught with uncertainties and, therefore, major cost overruns and schedule delays are common. The Second Avenue Tunnel project was part of a multi-project, approximately $700 million program where cost control and schedule adherence was critical. With the aid of the GBR, the project was completed on-time, within budget, and exceeded the Owner's expectation. The GBR provided meaningful information and accurately predicted ground conditions and behavior; the bid documents adequately described the subsurface conditions and the Contractor was prepared to address the geotechnical challenges of the tunnel construction. Ultimately, the total cost of the project was reduced by information within the GBR; the Owner spent less money for the constructed product versus total costs had a GBR not been utilized. Equally important to the Owner, budget overruns and claims for extras were eliminated.

The success of the project was directly attributable to the investment made in collecting and presenting the subsurface data and allocating the potential risks to the appropriate parties. Some of the techniques used in successfully dealing with the complex geologic and contractual issues encountered on this project are applicable for most projects involving underground construction. Key issues for controlling unanticipated costs and avoiding disputes include:

Cost Control - In order to control the potential for large cost overruns in underground construction, critical contracting issues have to be identified during the study and design phases, and those issues must be clearly addressed in the contract documents. Various construction methods and contract models may need to be examined and the advantages and drawbacks of each must be clearly recognized by the designers AND the Owner. It is especially necessary to identify issues of risk sharing, how to define/measure acceptable quality of various constructed items, the effect of the ground conditions on various potential means and methods the contractor may consider for pursuing the work, and the assumptions used during design that may affect construction methods, cost and schedule.

Construction Issues - In addition to the typical design and payment issues that must be addressed by the contact documents, they must also address non-design issues that may affect the contactor. Those are related to subsurface and geotechnical issues, utilities and buried structures, third party approvals and permits, differing site conditions, contractors' contingencies, risk sharing and risk management.

Geotechnical Risks - Control of costs will specifically be influenced by **geotechnical risks** related to difficult ground conditions, contaminated soil and ground water, boulders and buried objects, extent of geotechnical investigation, as well as interpretation of geotechnical data and ground behavior. The critical examination of geotechnical risks well in advance of finalizing design plans and bid items is especially important. Furthermore, the practice of requiring the contractor to perform his own exploratory work during the bidding, or requiring the contractor to provide the instrumentation and interpretation of instruments during construction, may result in poor quality or disparities in information, leading to high claims during construction. The Owner and design team should be responsible for procuring all geotechnical data and performance monitoring of ground behavior during construction.

Geotechnical Disclosure - Geotechnical issues and the full disclosure of those issues and how the contractor should account for them are of paramount importance in regard to cost control. There are numerous standard reports that should be fully disclosed to the parties interested in the project. Such reports include a Geotechnical Data Report and Geotechnical Baseline Report.

Risk Management / Sharing - Underground infrastructure design and construction needs not only risk management but also fair risk sharing. It is critical that the design team and Owner understand the risks involved, and fairly decide which party should bear the risks. Differing site conditions should be addressed in a differing site condition clause. Value Engineering should be used in order to allow the contractor to offer his experiences and new technologies to the client on the basis of acceptable partnering conditions. Unit prices in a fair combination of lump sum costs will help to avoid disputes. Contingency bid items for work that may be required but for which definitive quantities or efforts required could not be established in advance should be used as part of the contract. Incentives for early completion of the project, meeting high safety standards and producing good quality can and should be included in the contract.

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Rock Tunneling between the Great Lakes and the Ohio River

Edward J. Cording

1. Introduction:

Understanding the geologic environment through which a tunnel will be driven is one of the early considerations in evaluating uncertainty and risk on a tunnel project. Although the geology is variable and conditions at each site are unique; the regional geology and previous experience in the area serves as a guide for the site specific exploration. Understanding how the ground conditions will impact the design and construction – the behavior -- is also essential in evaluating uncertainty and risk Previous experience in similar ground and with similar construction methods provides a reference for evaluating behavior.

The focus in this paper is on the geologic environment in the Midwest, between the Great Lakes and the Ohio River where the rock is largely hidden, because most of the region is covered by glacial deposits. The rock is limestone, shale and sandstone derived from sediments laid down in shallow seas that covered the Midwest during the Paleozoic era, and it is a stable mid-continent region that has not been subjected to extensive folding and faulting from major tectonic activity. However, certain of the depositional environments, in combination with subsequent weathering and solutioning, have produced ground conditions that are difficult to define and whose behavior has a major impact on tunnel progress. Case histories are presented for some of the rock tunnel projects for subways, interceptor sewers and retention tunnels built in the past 30 years in the Midwest.

2. Index properties, classification, the geologic environment and tunnel ground behavior.

Development of index properties for rock core

Forty years ago, the only quantitative information in rock core logs was the percentage of core recovery. The logs described mineralogy and geologic structure but did not summarize or provide any quantitative information on features of engineering significance. In one case, a contractor claimed a differing site condition because improved coring techniques had resulted in recovery of the poor quality rock, whereas he had assumed that the high core recovery meant that the quality of the rock was high.

In 1964, in the process of siting a large underground cavern in granite at the Nevada Test Site, the rock mass was cored and a detailed descriptive geologic log prepared by the USGS, but there was no quantitative information that would allow the rock to be indexed so that the best quality rock for locating the cavern could be determined. In order to recommend a location for the cavern, Don U. Deere, who was serving as a consultant to the designer, developed the RQD method and logged the core using the method. He described the RQD as a modified core recovery method in which only the percentage of sound core greater than 4 in. in length for a given core run was counted. In this way, the relative amounts of fracturing and weathering along the length of the core and the higher rock qualities preferred for the cavern could be located.

The RQD has become a standard in all engineering boring logs and has proven to be very useful in assessing rock quality. The RQD is an indicator of several possible rock conditions, and the conditions will be different in different geologic environments. Low rock qualities in a core log should be checked to determine what rock conditions are causing the low values. Table 1 describes the conditions that influence the RQD. Thus, the Informed RQD:

Table 2.1 Informed RQD:

Low RQD values are produced by:

1. Faults or shear zone
2. Fracture zones
3. Weathering
4. Solutioning
5. Closely spaced bedding or foliation plane partings
6. Break up of core along vertical joints
7. Core loss in soil or weak zones
8. Mineral boundaries that are large with respect to the core diameter
9. Voids that are large with respect to the core diameter.
10. Poor drilling techniques
11. Drying of slake-sensitive core.

Some of the causes of low RQD, such as in items 6-10 are the result of disturbance or the scale of the core with respect to the scale of the discontinuities, and thus are not an indicator of the quality of the rock.

In the metamorphic schists and gneisses found in Washington, D. C. RQD values below 50% in a core run in unweathered rock, were indicative of shear zones containing closely fractured rock, slickensided shear planes and seams of clay gouge. RQD values between 50 and 75% were indicative of fracture zones. The impact of the features on the behavior of the tunnels and excavations was not determined directly from the RQD value but by locating and orienting the features with respect to the tunnels or excavations and by considering the mechanics of the behavior. The behavior that controlled stability on these projects was the separation and sliding of large rock blocks formed on a combination of shear zones parallel to foliation and conjugate shears or planar joints dipping in the opposite direction but having the same strike. Stability was most strongly impacted when tunnels or excavation walls were parallel to the foliation, which was the case for the tunnels and station chambers extending to the NW along Connecticut and Wisconsin Avenues.

Bedded deposits having the same RQD values as those obtained for the Washington D.C. schists and gneisses have a different behavior. In the flat-lying bedded limestones and shales of the Midwest, low RQD may be produced by closely spaced bedding plane partings, particularly when shale partings are present in the limestone, or when shale is laminated. Closely spaced bedding planes will limit the unsupported roof span in a bedded deposit. Shale partings are susceptible to swelling and slaking with time, so that a roof that is initially stable may have slabbing and fallouts some time after the heading has advanced. Low RQD values may be the result of solutioning and weathering along joints and bedding in the limestones. Rock quality is also influenced by the orientation of the joints. Vertical borings do not provide a representative sample of the vertical joints common to horizontally bedded deposits. The combination of vertical joints and horizontal bedding produces blocks that separate from the crown or from the side arch of the tunnel creating a corbelled shape.

Classification schemes

In the 1970's, following the development of the RQD, a series of rock classification schemes were developed for describing a rock mass and providing relationships between rock mass classification and support requirements.

Although many of the properties used in the classifications are useful in describing significant characteristics of the rock, there has been a tendency to apply the classifications directly to the design of a rock tunnel project rather than evaluating the geologic structures and their impact on tunnel ground behavior.

A few years ago, in a national conference session on rock mass classification, one speaker presented a rock classification scheme for pioneer road construction, in which numbers from 1 to 4 and letters A through D were used to describe rock properties such as strength, weathering and jointing. He described how the classification scheme was used in the field and noted that on one project, the project engineer, when told the classification was A2C4, asked him what the rock type was. He replied to the project engineer that he didn't need to know the rock type, the classification number was all that was required.

It is not possible to convey all the significant characteristics of the geology at a site in a summary classification. The geologic environment needs to be understood.

One of the published reports summarizing the application of a classification scheme to tunneling was entitled "Tunnel design by rock mass classification." One geotechnical engineer when asked how he was designing a tunnel stated that he did not base the design on a single classification. He considered the design process to consist of a comparison of multiple classification schemes.

The focus should be on the geologic environment rather than classification and the resulting behavior rather than developing a design from a classification or even multiple classifications. Knowing the geologic environment allows an assessment of potential rock conditions and exploration can be planned to locate such features or prove that they are absent. Once behavior is understood, design can be selected to fit the geologic environment and accommodate the behavior that occurs within that environment.

Classification of ground types on a project is most useful when the classification is closely linked to support requirements and it fits the range of ground conditions anticipated on the project. .Commonly three or four ground and initial support types are defined in the geotechnical baseline report and the specifications for TBM tunnels supported with rock bolts and steel ribs.

Key features of engineering geology significance

In a given geologic environment, there are key engineering geology features that control behavior and impact the design and construction. Deere summarized the key engineering geology features for dam foundations in terms of rock mass features (volumetric) and rock joint features (planar). They also apply to tunneling.

Table 2.2: Key Engineering Geology Features.

MASS FEATURES	Fault Zones
	Weathering Profile
	Volcanic Interbeds
	Solution Features
PLANAR FEATURES	Bedding and Foliation Surfaces
	Sheared Surfaces
	Master Joints

In the Midwest, one of the key engineering geology features is solution zones in the limestones. On several projects, gypsum and anhydrite, which are susceptible to solutioning, were deposited in limestones and shales in the late Silurian and early Devonian Periods.

Volcanic interbeds of tuff and lava flows are not found in the Midwest, but in the late Ordovician period, volcanic ash was deposited in the limestones and shales which has subsequently weathered to bentonite. One of these beds is found in Cincinnati.

Environments and tunnel ground behavior

In Table 2.3 is presented a matrix of tunnel ground environments and rock quality related to tunnel ground behavior in bedded limestones, shales and sandstones in the Midwest.

Table 2.3: Midwest Environments and Tunnel Ground Behavior

Environments		Rock Quality			
		Intact	Jointed	Closely Jointed	Soil-like
Self Weight			Blocks bounded by bedding & joints	Weathered and solutioned joints	Solution filled caverns, Fault zones
Shear Stress	Brittle: Stress Slabbing	Stiff, layers and high stresses in crown & invert	Buckling of bedding slabs in crown.		
	Ductile				
Volumetric Stress	Swelling	Shales			
	Slaking	Shale surfaces	Shale partings and joints		
Water		High flows in solutioned zones and at top of rock			
Gas		Methane and gas from hydrocarbons in shale and limestone; H2S coming out of solution with high in flows in limestones			
Thermal					

Logging rock core

Over the past 40 years, a number of index properties have been developed to describe the rock mass and rock joint properties. In exploration, the following information should be developed:

1. major weak zones (faults, weathered rock, solution zones, other soil-like zones),
2. ubiquitous patterns of foliation (planar fabrics formed from metamorphism) and bedding:
3. joint sets: orientation, spacing, planarity, persistence, roughness, aperture, filling.

Some of these properties cannot be obtained from logging rock core alone. Rock exposures, previous tunneling in the region, and regional geology are needed to provide further perspective. Summarized in the following paragraphs is information that should be obtained from boring and coring.

Key Drilling and borehole sampling information: Rod drops, lost circulation, hole collapse, and gas concentrations are some of the items observed during drilling that should be presented in the boring log.

Televiewer log: The quality of information obtained from core borings is substantially enhanced by the televiewer, which provides an oriented 360 degree picture of the fractures on the wall of the borehole. The tool can be used in water filled as well as open holes. Using the televiewer the strike of the joints can be determined, as well as the dip, and the joint aperture. On the Second Avenue Subway project in Manhattan, televiewer data is being used as a standard tool in all borings. The data is being correlated directly with the core logs, and presented in an integrated borehole log along with the core description and core photo.

Core recovery: Low core recoveries can occur when drilling breaks up, grinds up or washes out the rock and soil. Often, core recovery is low in the highly weathered, soil-like zones near the top of rock. Improved coring equipment and techniques have improved core recovery, even in the poor quality ground.

Core recoveries less than 100% may be indicative of solution zones. In one project, possible solution zones were located by highlighting all core recoveries less than 95%. The absence of a gypsum or anhydrite layer at an elevation where other borings showed the layer could be correlated with core recoveries less than 95%. Drill-rod drops of 6 inches, recorded during drilling, provide additional evidence that open solution zones were present.

RQD: The RQD should be obtained, along with the Core Recovery, for each core run.

Separate descriptions of foliation or bedding: Foliation planes and bedding planes form an ubiquitous pattern in the rock mass that should be described. The joints or partings along foliation or bedding can be differentiated from other joint sets, even in borings where the core is not oriented, and their characteristics should be described separately.

Orientation: For vertical boreholes, the dip of joints, bedding, and foliation should be reported. With the televiewer, both strike and dip are determined, allowing the joint sets to be differentiated and described.

Joint alteration, filling, aperture, and joint roughness In addition to qualitative descriptions, a quantitative or defined qualitative summary, such as the joint aperture index , Ja, and Jr used in the Q system is useful. Individual index properties are more valuable than a single classification number when evaluating ground conditions that affect behavior.

Weathering classes: Classes that range from I, unweathered, to V, soil-like are commonly summarized on the boring log.

Packer and PumpTests: Packer tests are an essential part of evaluating permeability in a borehole, but Pump tests provide the opportunity to determine more than the local permeability around the hole and are key to understanding the groundwater flow system.

Identifying solution features: It is important to recognize that major solution features, like boulders, will usually be missed by borings. As noted in the case histories described in this paper, the depositional environments and the presence of easily solutioned materials such as gypsum and anhydrite indicate the potential for solutioning. Regional information is important, but north of the Ohio river, glacial drift over the rock has limited the surface expression of solution features making it more difficult to identify features from the surface. In the core, some evidence of solutioning and weathering, and carbonate deposition may be observed along joint surfaces. Solutionings along bedding may be evidenced by a smooth water-worn surface. Rod drops and core loss may indicate the presence of solution voids.

3. The Geologic Environment in the Midwest

The focus in this paper is on the geologic environment in the Midwest, between the Great Lakes and the Ohio river where the rock is largely hidden, because most of the region is covered by glacial deposits.

Tills deposited from the thick ice sheets that moved south from the Great Lakes are highly overconsolidated and have a high silt-clay content. Thus, much of the soil is cohesive and will stand in an open face and behave as "firm" tunneling ground, or raveling ground if the clays are jointed. However, pockets and lenses and thick deposits of outwash and lacustrine silts, sands and gravels were also deposited and they create running or flowing ground in an open tunnel face.

In urban centers located at the margin of the Great Lakes, including Milwaukee, Chicago, Detroit, and Cleveland, soils deposited by the waning glaciers include soft clay deposited from ice sheets floating in the lakes and layers of organics, clays, silts, sands and gravels deposited in bogs, lakes and outwash deposits. In Chicago, tunneling in the soft Chicago Clay for freight tunnels 100 years ago and for the subway in 1938-1941 was accomplished using compressed air to limit squeezing. Compressed air was also used in tunneling to prevent ground water inflow in the layered sands, silts and clays throughout the region. Compressed air has seldom been used in the past 30 years because of the high cost of working safely under air pressure. The alternative has been to dewater, but layered sands, silts and clays are difficult to dewater, with wells spaced on the order of 100 to 300 ft, and many tunnels driven in the past 30 years have encountered flowing ground and large ground losses using open faced shields without compressed air. In the last ten years, pressurized face machines, either slurry shield or earth pressure balanced machines (EPBM) have become the standard in the U. S, and provide support of the tunnel face and balance water pressures so that once again tunneling can be accomplished in soils below the water table without dewatering.

Beneath the glacial deposits are sedimentary rocks deposited in the early to mid-

Paleozoic (Ancient life) Era (Table 3.1). In the past 30 years, sewer relief, storage and flood control projects in the Midwest have resulted in major tunnel projects sited in the Paleozoic sedimentary rocks. During much of the Paleozoic era limestones, shales, and sandstones were being deposited in shallow seas. The sediments are believed to have been derived from highlands located in the vicinity of the present East Coast, at a time when the North American continent was bumped up against other continental masses. The oldest sedimentary rocks are present on the margins of the Illinois, Michigan and Appalachian basins, including the Cincinnati Arch which extends NE from Cincinnati, while the younger sediments compose the top of rock toward the center of the basins.

In any given geologic period, there will be facies changes across the region so that the rock type encountered in one area will differ from the rock type deposited at the same time at another location. During some periods, continental deposits such as sandstones and red-bed shales were being deposited in the east, near the Appalachian uplift, while marine sediments (limestones and black shales) were being deposited to the west

. **Table 3.1 Paleozoic Era in the Midwest:**

Paleozoic Era	**Depositional Environments**	**Formations**			
Periods:		***In Chicago & Milwaukee***	***In Detroit & Dearborn***	***In Columbus***	***In Buffalo***
Permian (late Palezoic)					
Pennsylvanian	Swamps form coal	(Found in Illinois Basin, S. of Chicago and in Michigan Basin, NW of Detroit)			
Mississippian					
Upper Devonian			Antrim Sh Traverse Fm		
Lower Devonian,	Barrier reefs create closed basins with evaporite deposits, including gypsum	***Milwaukee:*** **Solutioned Thiensville Ls** overlies Niagaran Ls and tunnels on North Shore	**Dundee Ls, Detroit River Group**	***Columbus:*** Delaware Ls. **Columbus Ls**	Onandaga Ls Akron Dolostone
Upper Silurian	Evaporite deposits				**Bertie Dol. Camillus Sh w/ solutioned gypsum**
Silurian	Widespread	Niagaran			

	carbonates beyond barrier reef	Series limestone /dolomite			
Late to Middle Ordovician	Appalachian uplift & volcanoes in east ash deposited globally, weathers to bentonite. Shallow muddy bottom produces shales in Illinois-Ohio region, with limestone to SW.	Maquoketa Formation			***Cincinnati:*** Kope **Point Pleasant** **Lexington Ls** (Millbrig Bentonite) Black River Gr.
Ordovician	Flooding and deposition of massive limestones. Shallow dunes, sandstones	Galena-Plattville limestones St. Peter Sandstone			
Cambrian (early Paleozoic)					

During the late Silurian and early Devonian periods, the shallow seas were in closed basins which produced large deposits of salt and other evaporites, including gypsum and anhydrite. Late Silurian and early Devonian limestones and shales are found in many of the cities of the region, including one of the deep tunnel projects in Milwaukee, in Detroit, Dearborn, Cleveland, Columbus, and Buffalo. The presence of gypsum and anhydrite interbeds in some of the profiles has led to significant solutioning and high permeabilities and high inflows into tunnels. Often the water contains hydrogen sulfide, which is released upon inflow into the tunnel. Fig. 3.1 is a bedrock surface map showing the distribution of early Devonian formations and Fig. 3.2 also includes the formations of deposited in the Ordovician period. The information is summarized from "Fold and Thrust Belts of the United States, 1984.

Summarized in the following sections is experience gained on some of the TBM tunnel projects between the Great Lakes and the Ohio River. Three cases are described in which solutioning affected support requirements and groundwater inflows. Two cases are described in which exploration was focused on determining available rock cover to avoid intersecting deep buried valleys.

4..Buffalo Light Rail Rapid Transit

Project C-11 for Buffalo Light Rail Rapid Transit consisted of two 18.5-ft-diameter tunnels mined using TBMs by Fruin Colnon. Formations at the site were Upper Silurian, a time when evaporites were being deposited. The tunnels were sited in the Bertie Dolomite, just above the contact with the underlying Camillus Shale to avoid this gypsum bearing, solutioned shale (Fig. 4.1). The solutioning impacted the tunneling in two ways: (1) High permeabilities, principally in the Camillus, required deep well dewatering and led to an unanticipated grouting program to seal inflows after the concrete lining was cast and water levels were restored. (2) At two locations in both tunnels, flow to the solution zones in the Camillus shale caused solutioning on joints and bedding seams in the Bertie formation which led to loosening and instability of rock blocks at the cutterhead and over the shield .

A Robbins main beam TBM (with a short shield) was used to drive the tunnel. Tunnel was built in 1980. Cutters were mounted on face and were not recessed.

The underlying Camillus shale was known to contain gypsum and borings and showed evidence of fracturing and solutioning. Pump tests revealed high permeabilities in the range of 10^{-1} cm/sec. Ground water contained hydrogen sulfide. Prior to tunneling, deep wells were extended into the Camillus to dewater the tunnel. When water levels were restored after the concrete lining was installed, leakage through construction joints and shrinkage cracks required an unanticipated and extensive grouting program, in addition to the contact grout placed behind the lining.

Support in the TBM tunnels was anticipated to be rock bolts. Steel ribs were only specified where the tunnel passed beneath an existing rail line, and were to be installed away from the heading (behind the TBM grippers), thus they were not required for initial support.

Rock bolts were placed throughout the tunnel in a 4-bolt pattern per cross section, without straps or mesh. Days to weeks after passing the location, thin slabs (a few inches thick), formed and loosened between the two center bolts, and fell from the crown, as result of horizontal stress concentrations at the crown and slaking (humidity changes) along shaley seams, which caused the delayed reaction. Additional support was placed between the bolts to hold the slabs.

More significant support problems developed in two sections of the tunnel, where the TBM encountered blocky ground in which large blocks loosened and separated at the cutterhead and over the top of the shield. The blocks were formed by a combination of vertical joints and open horizontal bedding planes. Solutioning had created open horizontal gaps along bedding which were approximately1/2 to 2 in. wide, into which a tape measure could be extended as much as 10 ft away from the tunnel wall. As a result, the rock blocks were not interlocked and essentially were already detached from the adjacent rock before the tunnel was excavated. At the cutterhead, blocks jammed in the buckets and between cutters creating high impact forces. Above the TBM shield, the blocks were already loose and detached so that it was not possible to support them with rock bolts. Steel ribs were installed under the finger shield (between the cutterhead and the grippers) to support the blocks in place. Progress was reduced from 80 ft per day to 10 ft per day through the blocky, loosened ground.

The unstable blocks were in two sections of the tunnel which were above a zone in the Camillus shale that had been identified as solutioned, fractured and permeable (Fig 4.1) The impact of this zone on the Bertie had not been anticipated. A series of vertical joints were present in the Bertie formation above the solutioned zones in the Camillus shale, striking at an angle of 30 degrees to the tunnel axis. The vertical joints intersected the permeable zones in the Camillus and thereby served as pathways for vertical flow, which led to solutioning on the horizontal bedding seams in the Bertie formation. The vertical joints and open bedding bounding the rock blocks served to isolate them and allow them to separate and fall around the TBM. `

5. Dearborn Retention Tunnel

The City of Dearborn Retention Tunnel Project was terminated in as a result of groundwater inflows and extensive grouting in the construction shaft. Alternatives for tunneling in rock and EPBM tunneling in soil were investigated in subsequent years.

The tunnel was to be driven in the lower Devonian formation, the Dundee limestone. It was sited approximately 40 ft below the top of rock to avoid the shallow weathered rock zone and was located 30 to 60 ft above the contact between the Dundee limestone and the dolomites of the underlying Detroit River Group. The contact between the two formations was an erosional unconformity on which weathering and solutioning (karst features) had developed before deposition of the overlying Dundee limestone, and it had high permeability (packer tests gave permeabilities in the range of 10^{-3} cm/sec). It was anticipated that vertical joints intersecting the solution zones would feed water to the tunnel. Gypsum interbeds were also present in the underlying Detroit River Group. The presence of hydrogen sulfide in the water required limiting the volume of ground water inflows into the tunnel. Dewatering of the formation would have required high pumping rates and high capacities for treating the water. If grouting were conducted from the tunnel, large volumes of grout were anticipated.

6. Columbus: Upper Scioto West Interceptor.

The Upper Scioto West Interceptor was driven with a 10-ft 9-in.-diameter TBM in the Columbus Limestone of the Upper Devonian Period by KM&M joint venture. Solution features such as cavities, open fractures, and solution channels were known to be present in the Columbus limestone, although most of the features were anticipated to be above the tunnel level and within 30 to 36 ft from the top of rock.

Ground water levels were drawn down by dewatering, and inflows were concentrated in the tunnel invert.

An unanticipated buried valley of till was encountered early in the tunnel drive that required placement of an emergency shaft ahead of the TBM, and hand mining back to the TBM through 165 ft of clay soils (Day, et al, 1997).

A chert zone approximately 2 to 7 ft thick was anticipated near tunnel level and its hardness was expected to affect bit wear, downtime, and penetration rates. The first TBM was designed with large (17-in.-dia.) front loading disk cutters to handle the harder rock.

During tunneling, the chert zone was observed to be bounded by a tan clay layer approximately 1/8 to 1/ 2 in. thick with a black bituminous seam at the base of the clay (Fig. 6.1). A series of solution caverns were found to be concentrated along the chert layer throughout the 23,000 ft length of the tunnel, and many of the cavern floors and roofs were bounded by the clay seams (Fig. 6.2). The solutioning appears to have been channeled along the clay seams. Solution caverns, most of which extended horizontally, were typically 6 to 12 ft wide and bounded by bedding clay seams (Fig.6.3, 6.4, and 6.5) Solution caverns were both open and filled with clay and blocks of limestone. In some cases, gravel stream deposits were found in the caverns. Solution channels several ft wide, either open or filled with clay, were observed to extend upward along steeply dipping joints, sometimes in excess of 10 ft above the tunnel crown.

Additionally, the clay and bituminous seams bounding the chert layer formed a separation surface for blocks and slabs that loosened above the crown, and when the clay seams dropped to the level of the upper side arch of the tunnel, they formed unstable corbel blocks in combination with intersecting near-vertical joints. These conditions required the use of the 120-degree channels with 4 bolts (Category B support) rather than the anticipated 90 degree channels with 2 bolts (Category A support). When the clay seam was in the lower portion of the face, blocks would separate and slide out on the seams at the cutterhead. Blocks loosening from the crown, arch and lower arch in the face placed high impact loads on the front loading cutters used on the TBM, and, after 5000 feet of advance, the bull gear teeth failed. One of the operators described the conditions: "You could tell you were coming into bad ground by the sound the machine would make. It would start to grind, bind, jerk and shake. The best way I could describe the mining with the machine would be if you were driving 60 mph and shifted into second gear. You could hear the motors grinding and grabbing as you tried to progress forward. You knew the ground was taking its toll on the machine as we tried to mine." A rescue shaft was sunk and the bull gear was replaced and the TBM head was replaced with rear loading (less exposure ahead of the face), smaller diameter (14-in.-dia) cutters.

Class 3 support consisted of steel ribs placed 4 ft on center. Because of space limitations, the trailing gear could not be dragged over full circle steel ribs, and so channels and bolts were often used in lieu of the ribs. The channels were placed on very close centers in solution features and areas where blocks were separating and loosening over the shield.. Loose blocks present on top of the shield were supported by setting the channels on 1 ft centers immediately behind the tail of the shield as the shield advanced so that the support for the blocks was transferred from the shield to the channels. The channels were supported by rock bolts which were placed wherever support could be gained around the perimeter in the limestone that was not solutioned.

7. Cleveland

The 20-ft-dia. Mill Creek 2 and 3 tunnels presented challenges because they were tunneled in shales beneath deep valleys filled with outwash and lacustrine deposits incised in the bedrock during the late glacial stages. Two of the construction shafts were frozen in order to be advanced through the permeable sands, gravels and silts.

On the Mill Creek 2 tunnel, rock cover over the tunnel in the buried valley was in excess of 10 ft (Fig 7.1), but on the Mill Creek 3 tunnel, rock cover was approaching 10 ft, as determined from four borings concentrated near the bottom of the 200-ft-deep buried valley, and after the tunnel profile had been set at the lowest possible elevation (Fig 7.2 and 7.3).

During tunneling beneath the valley, probe drilling ahead of and above the tunnel crown would be required to ensure that the buried valley would not be intersected, with flooding of the tunnel. However, if the buried valley was found to intersect the path of the TBM, an extensive program of grouting and freezing would be required, working from behind the TBM, with an anticipated delay of several months. The risk of such an event would be difficult for a contractor to assume in bidding the project.

The construction of Mill Creek 2 before Mill Creek 3 provided the opportunity to specify that an exploratory tunnel be advanced from a nearby shaft using drill and blast procedures in order to locate the contact and carry out remedial measures, if necessary, before the contract was let for the Mill Creeek 3 Project. The work was accomplished and probing showed that the cover over the tunnel was adequate to advance the TBM for the Mill Creek 3 contract without the risk of intersecting the buried valley. Cross-hole seismic tomography was also employed to assess the rock cover.

8. Cincinnati

A current feasibility level investigation is being conducted by the Corps of Engineers for a major large diameter storm and sewer relief tunnel extending down the Mill Creek Valley in Cincinnati. The tunnel will be sited in late Ordovician shales and limestones of the Point Pleasant Formation and Lexington Limestone.

During the late Ordovician, mountain building was taking place to the east, in the Appalachian Orogenic Belt, and winds spread ash from volcanoes, which has subsequently weathered into bentonite beds in the limestone and shales. The preliminary tunnel profile for the proposed Mill Creek project in Cincinnati has been set above the bentonite beds to avoid tunneling in them. A major focus of the exploration is to ensure that this tunnel alignment provides enough rock cover below the bottom of the buried valley and tributaries of Mill Creek, which the tunnel will cross at several locations. Borings and seismic profiles across the valleys are being obtained.

The project includes a series of drop shafts, and several larger diameter construction shafts. The locations of the construction shafts, and the boundaries between construction contracts, are being adjusted to minimize the depth of overburden through which large shafts will have to be advanced.

10. Summary and Conclusions

The ground conditions that control the behavior of a tunnel and influence its design and construction are not determined from a rock classification correlated with published relationships for support requirements. Rock quality indices developed over the past 40 years have been extremely valuable in providing a means of quantifying the information obtained from exploratory boreholes, but can be misleading if used in isolation from an in-depth investigation and understanding of the regional and site geology.

Correspondingly, selecting a tunnel support design based on a correlation with a rock classification is inadequate and misleading when it is used in isolation from an evaluation of how the geology – the ground -- behaves during tunneling. The tunnel can be designed if its behavior in the anticipated ground conditions is understood.

To evaluate risk and carry out the process of exploring, siting, designing, managing, and constructing a tunnel project, the two disciplines of geology and underground civil engineering must be fully engaged, and they must be linked. Risks are related to bidders and bid prices, bids not in line with budget, construction delays, cost overruns, disputes, the ability of TBM to advance, stability of the works, support, groundwater, gas, ground movements and damage, and other impacts on third parties.

The first geotechnical priority is the ground: What are the significant engineering geology features in the given geologic environment? Find them or prove they are absent. Index and baseline the features.

The second priority is ground behavior: How will the geologic materials act under the imposed design and construction? Provide a geotechnical baseline, design, specification, construction plan that fits the anticipated behavior.

In the geologic environment in the Midwest, between the Great Lakes and the Ohio River the rock is largely hidden, because most of the region is covered by glacial deposits. The rock is limestone, shale and sandstone derived from sediments laid down in shallow seas that covered the Midwest during the Paleozoic era, and it is a stable mid-continent region that has not been subjected to extensive folding and faulting from major tectonic activity. The case histories of tunneling in rock between the Great Lakes and the Ohio River highlight the importance of the depositional history and the regional geology on the behavior of the ground. Certain of the depositional environments, in combination with subsequent weathering and solutioning, have produced ground conditions that are difficult to define and whose behavior, in terms of groundwater and gas inflows and tunnel stability, has a major impact on the success of the project. The case histories show that the most critical time in the life of the project is in the early stages when exploration is underway and the vertical elevation of the tunnel is selected. With the continued development and use of pressurized face tunnel boring machines, more options are becoming available for siting tunnels in either soil or rock.

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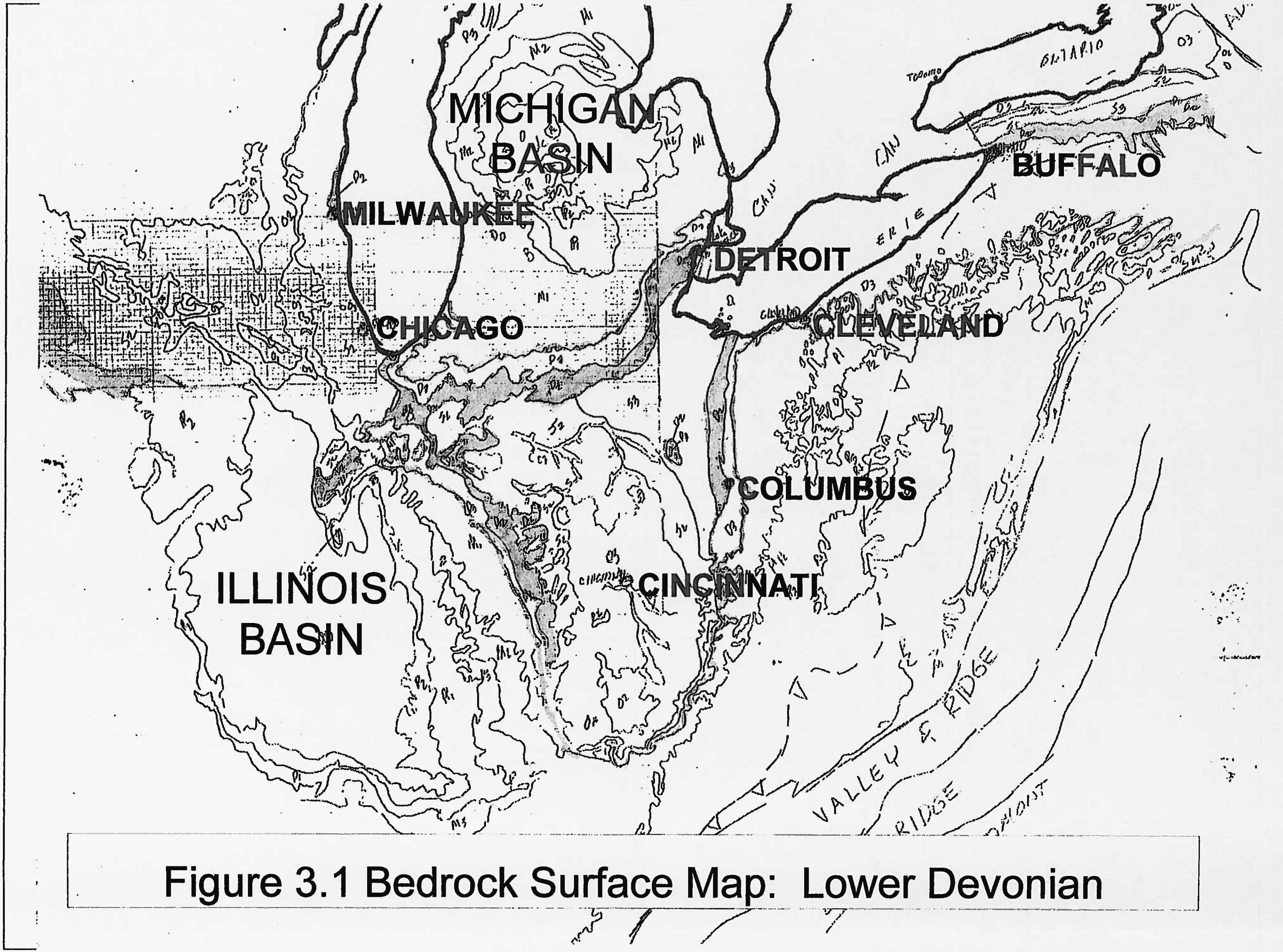

Figure 3.1 Bedrock Surface Map: Lower Devonian

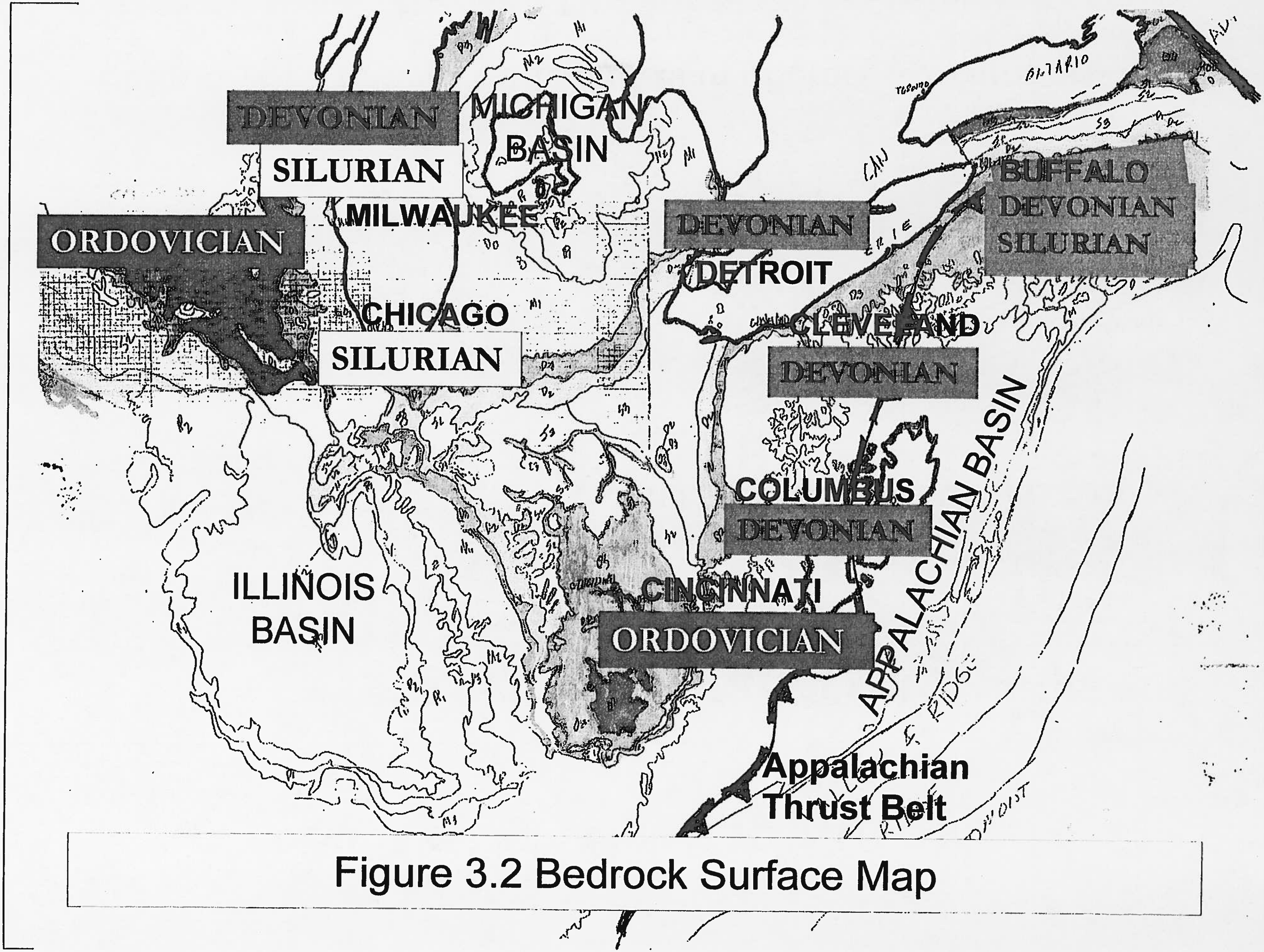

Figure 3.2 Bedrock Surface Map

Buffalo Light Rail Rapid Transit

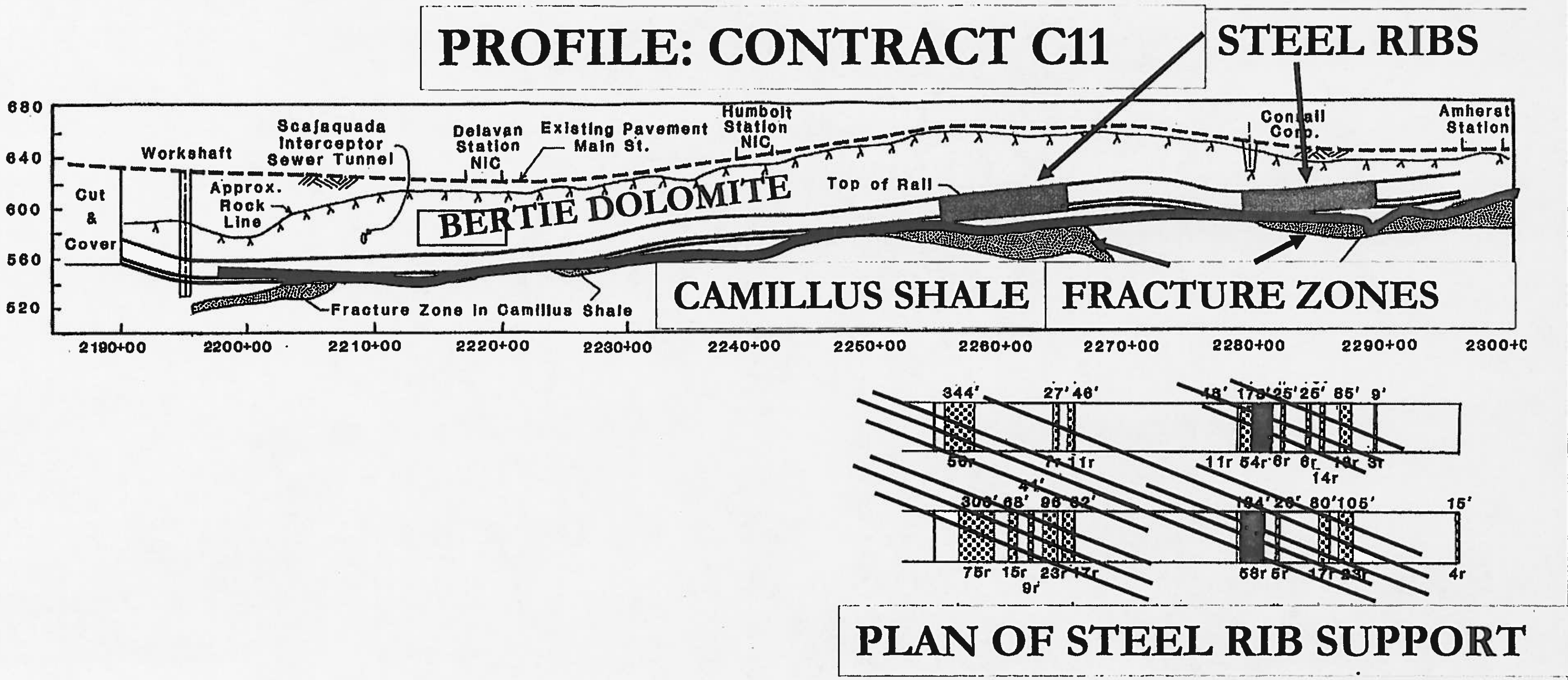

Figure 4.1 Tunnel Profile and Plan of Steel Rib Support

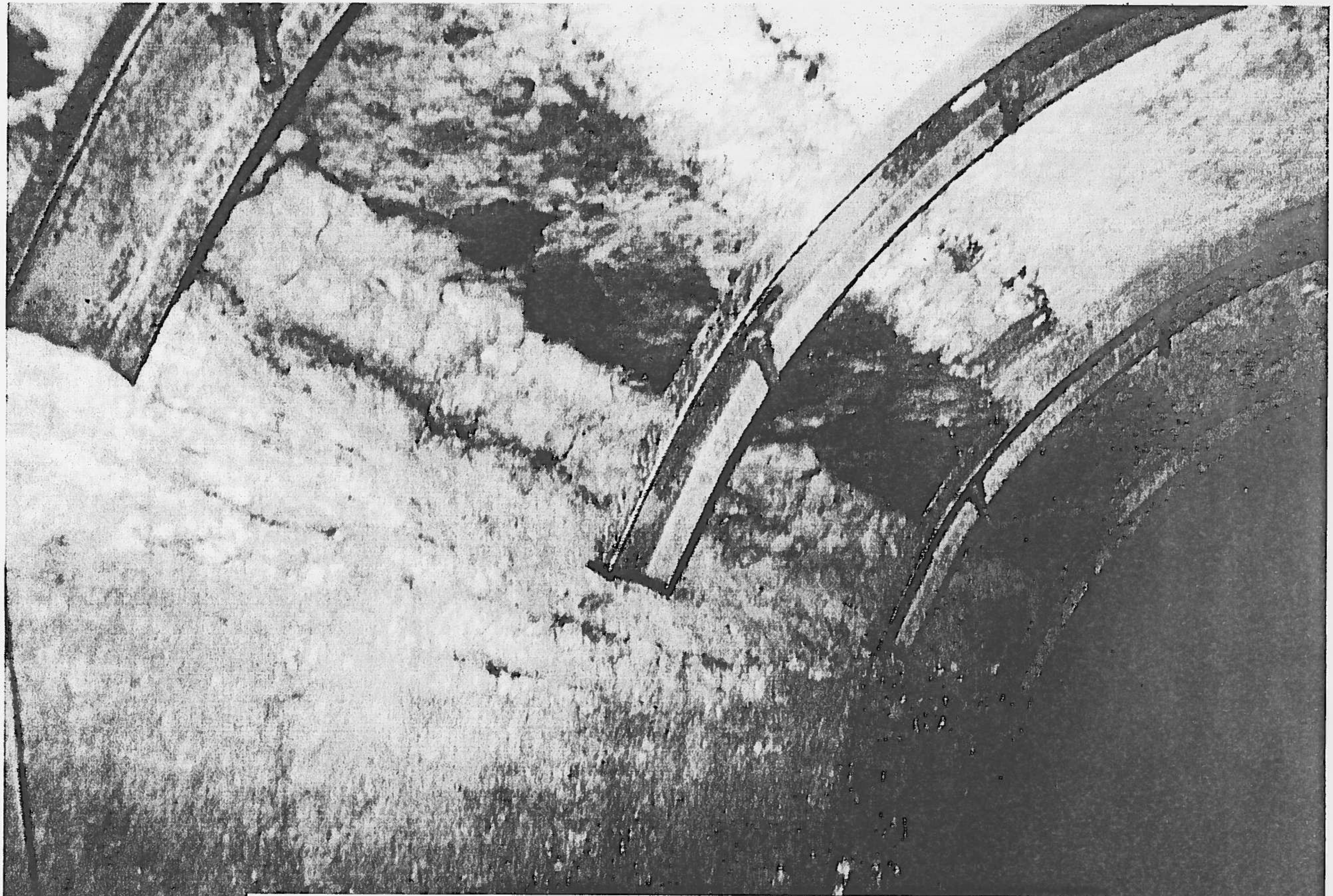

Fig 6.1 Overbreak at clay and bitumen layer
Class B, 4-bolt channel support

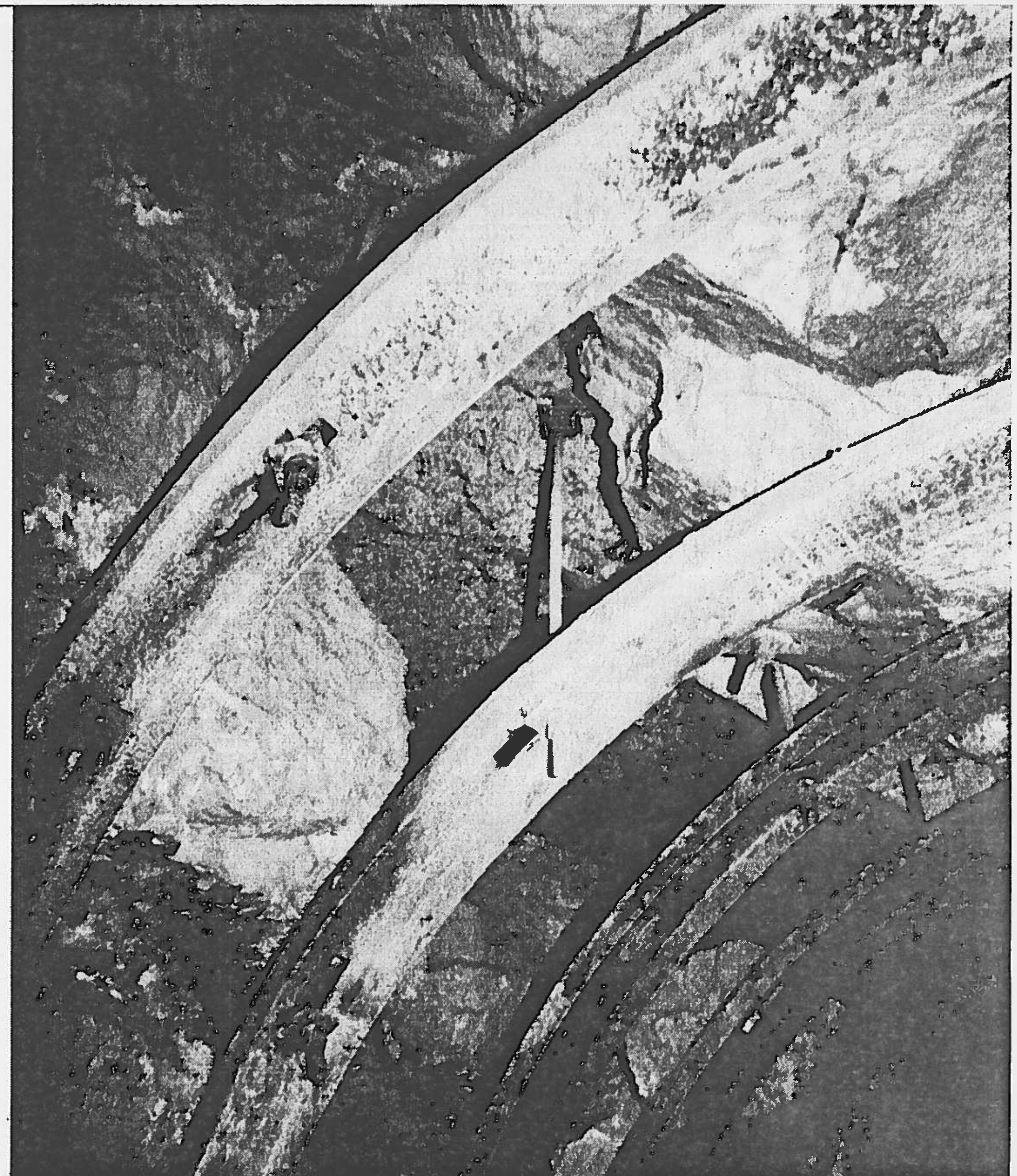

Figure 6.2 Solution cavity; loose blocks supported by bolted channel.

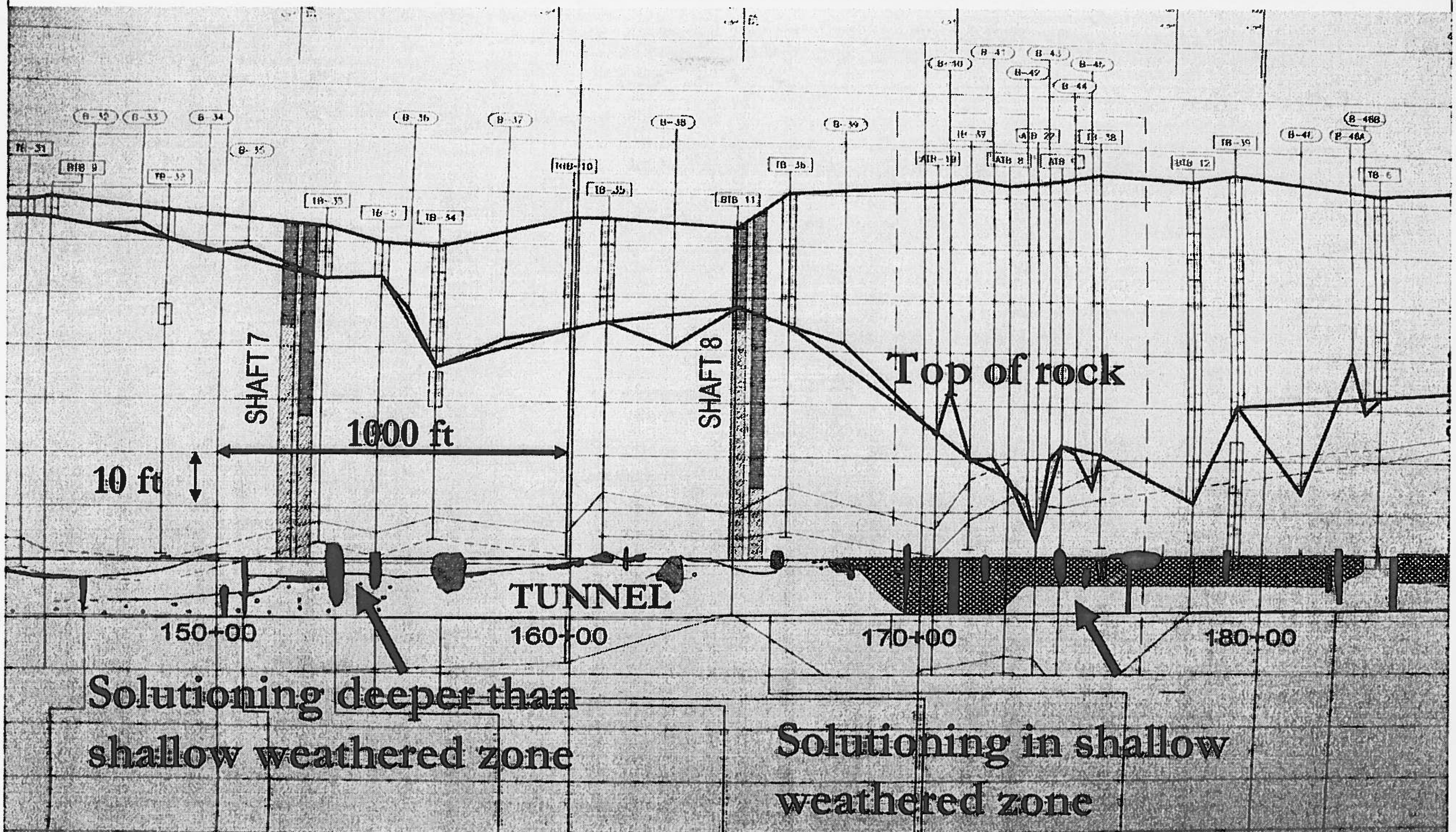

Fig 6.3 Solution Caverns in Columbus Limestone

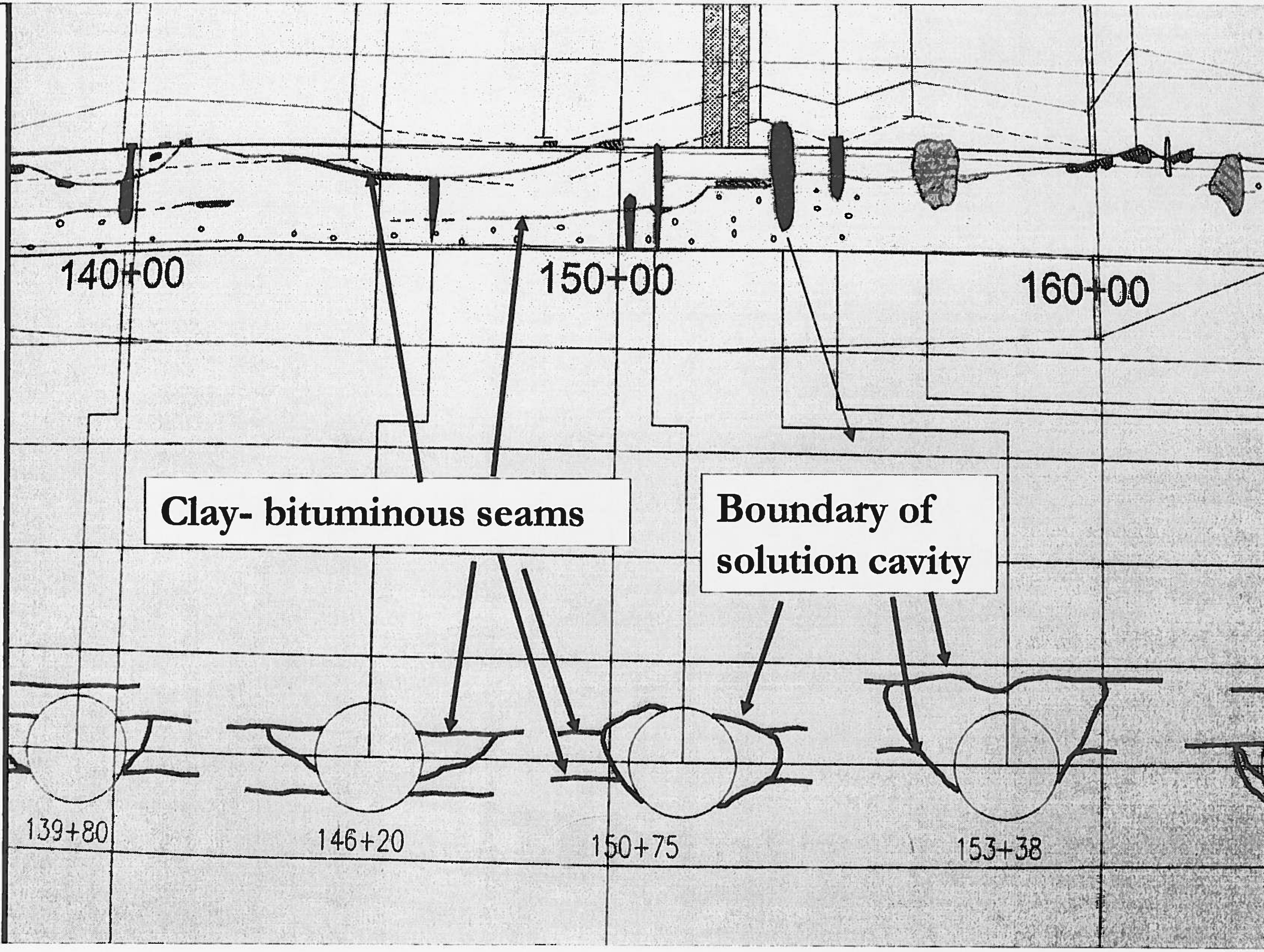

Fig 6.4 Solution cavities concentrated in cherty zones between clay - bituminous seams

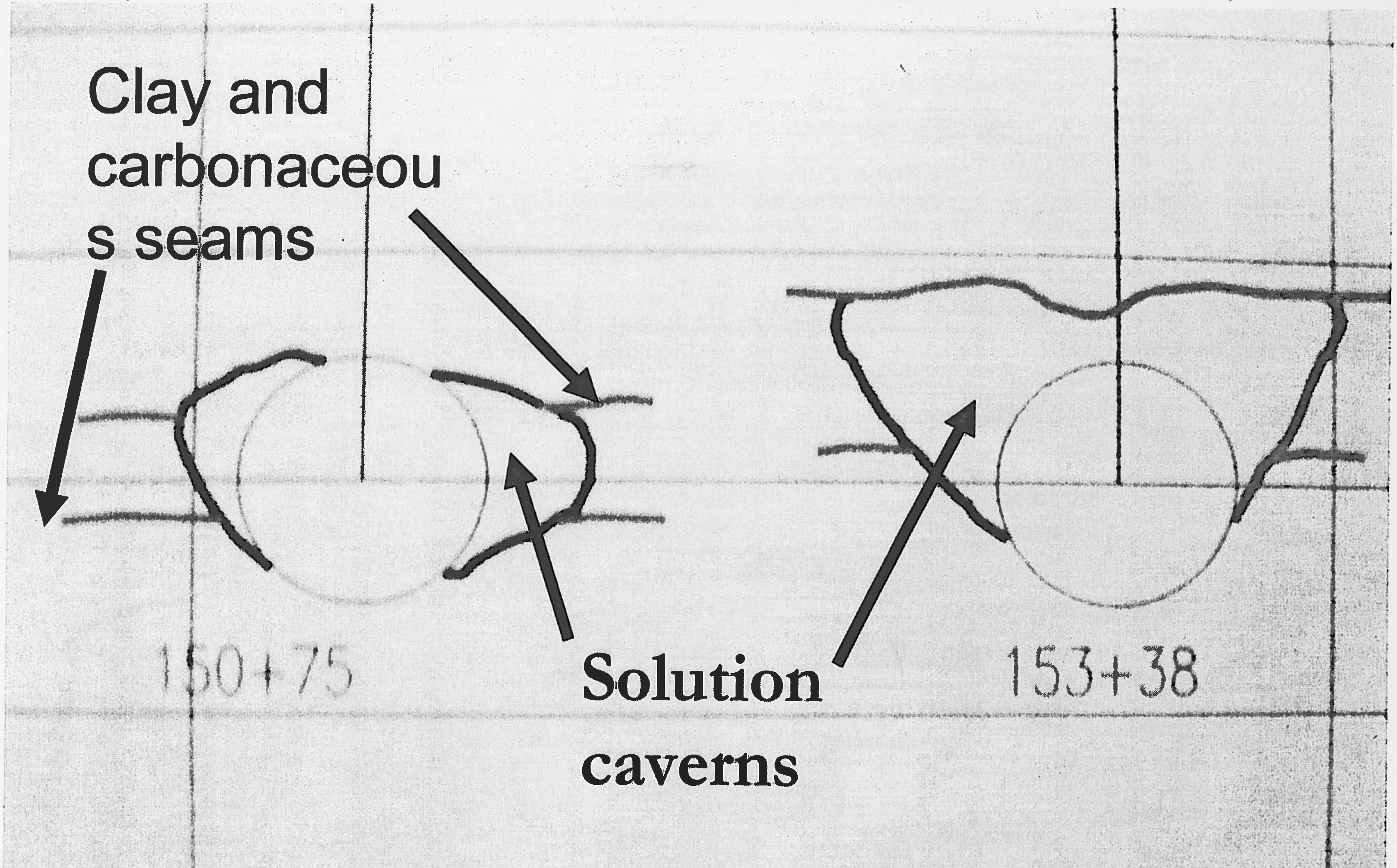

Fig 6.5 Solution caverns between clay seams in Columbus Limestone

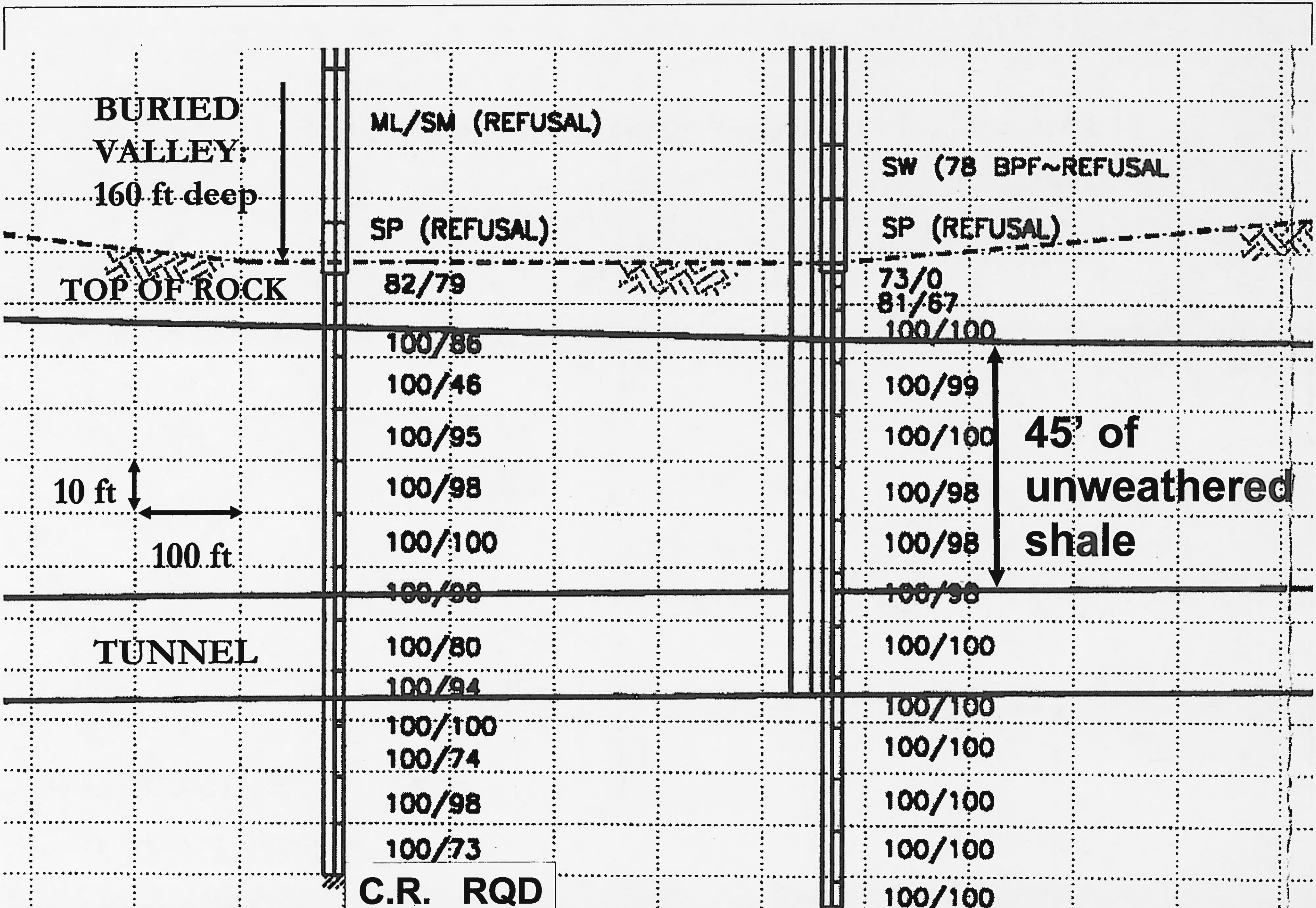

Fig. 7.1 Mill Creek 2: Rock Cover at Buried Valley

MILL CREEK 3: Buried Valley

Sand, silt, gravel, boulders and cobbles over shale

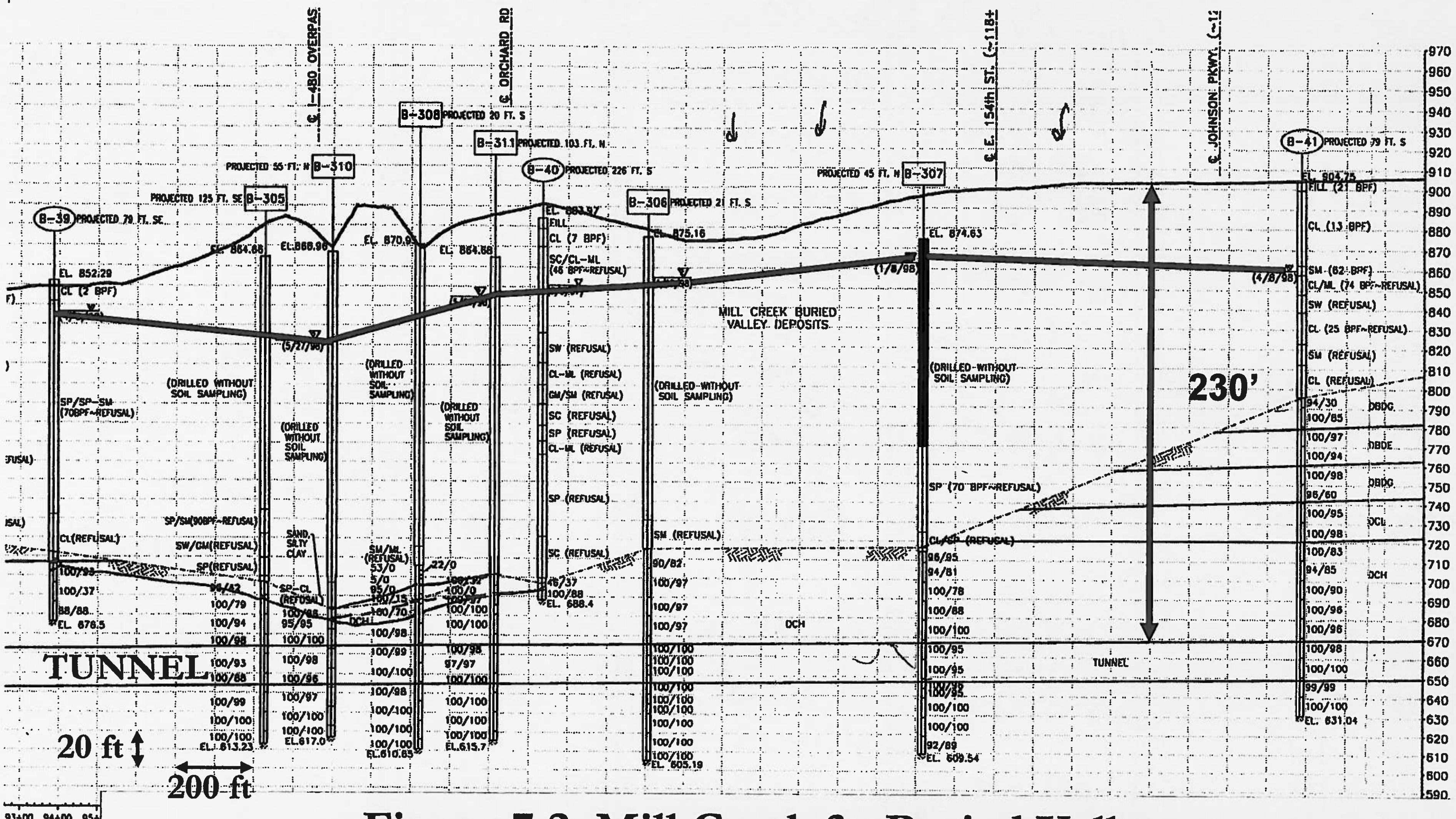

Figure 7.2 Mill Creek 3: Buried Valley.

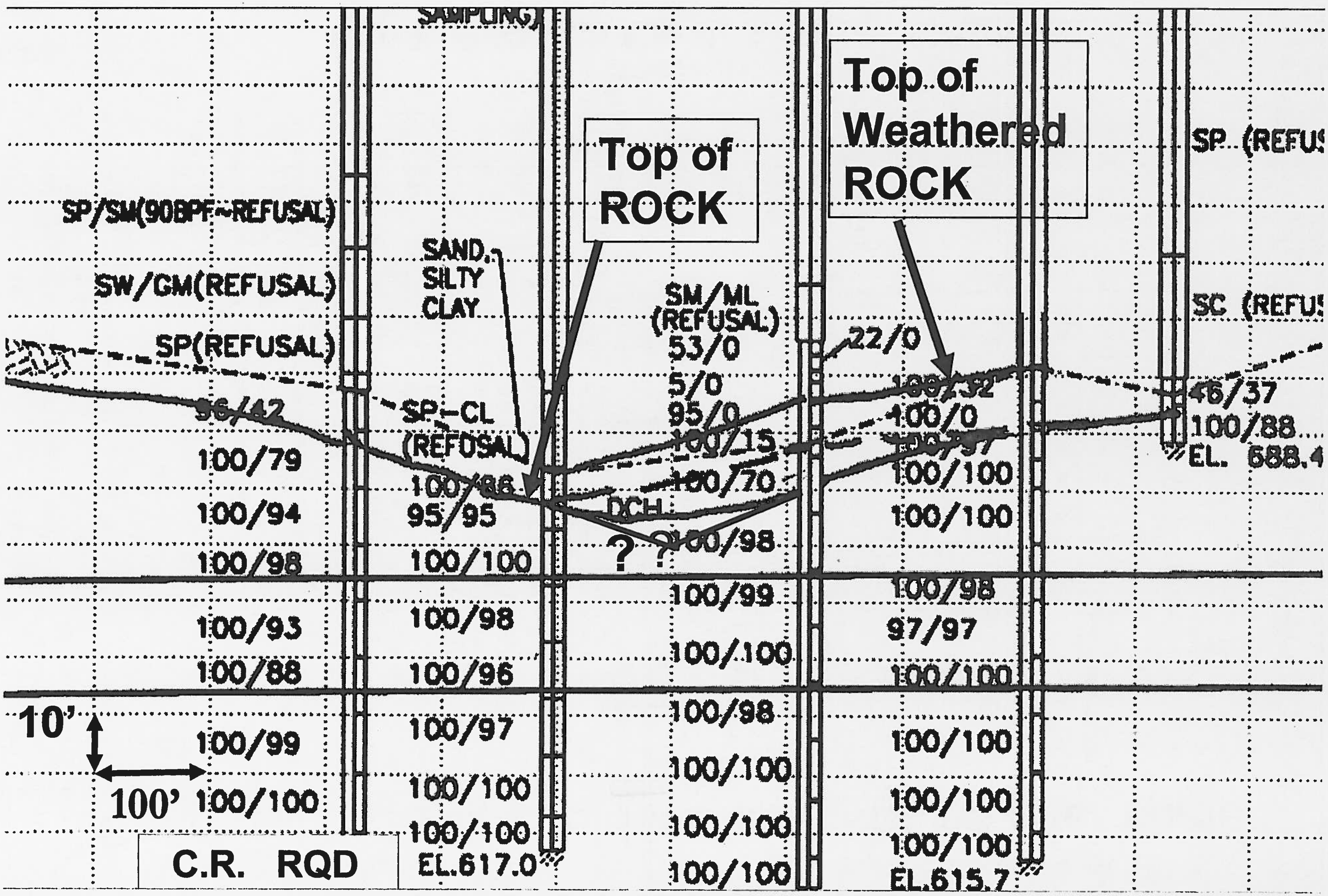

Fig 7.3 Mill Creek 3 Buried Valley, Shallow rock cover over tunnel

A case study on open pit mine rock slope stability

P. H. S. W. Kulatilake, J. Um & B. Morin
Department of Mining & Geological Engineering, University of Arizona, Tucson, AZ 85721, USA

ABSTRACT: Development of a three-dimensional mine visualization model for a section of a mine is addressed first. Discontinuity orientation and location information was taken from this visualization model for use in slope stability analyses. Estimated shear strength properties of discontinuities and mechanical properties of intact rock from the rock mass samples obtained from the mine are discussed next. The paper is then focused on the calculated maximum safe slope angles based on the performed kinematic and block theory analyses using the mapped discontinuities at the mine. Finally, the effects of water that exist in the rock mass, tension cracks, slope face inclination, overall wedge height and double benching on factor of safety of wedge stability are illustrated through limit equilibrium slope stability analyses.

1 INRODUCTION

The Phelps Dodge Sierrita-Esperanza open pit copper mine is located 40 km south–southwest of Tucson, Arizona, on the southeast flank of the Sierrita Mountain Range. Figure 1 shows the investigated area that is on the north side of the Esperanza pit. The area investigated includes eleven benches each about 15 m high and approximately 305 m in length, encompassing two different rock types namely, Esperanza Quartz Monzonite Porphyry (EQMP) and the Triassic Oxframe Andesite (TrOA). This area is very much affected by the Cooper fault (Fig.1), better described as a fault zone. This fault strikes NE-SW and dips towards NW. The main trace of this fault is the apparent contact of the EQMP and TrOA. As material has raveled down the pit slope, the trace of the fault has been obscured, but at depth it is clear that the main trace of the Cooper Fault zone is not the contact between the rock types. The gouge zone of the Cooper Fault is difficult, at best, to ascertain, but it appears to taper down with depth.

The other important component of the structure in this area is the difference in fracture intensities between the TrOA and EQMP. In places, the TrOA was so heavily fractured, it was difficult to perform scanline mapping. This problem was not seen with the EQMP. Nearly all exposures of EQMP were workable with some localized heavy fracture intensity, most probably due to blasting. This paper covers the studies performed to investigate the slope stability of the selected area of the mine.

Figure 1. A photograph of the study area.

2 DISCONTINUITY MAPPING

Discontinuity mapping with scanline surveys were performed on 1020, 1065 and 1125 m benches of Esperenza pit near Cooper fault. A total of 27 scanlines were completed (Um et al., 2000). Out of these, 4 scanlines were in the TrOA rock mass. The rest were in the EQMP rock mass. A total of 1145 discontinuities (883 from EQMP and 262 from TrOA) were mapped from these scanlines. In addition, information on 87 major discontinuities (length greater than about 15 m) at bench levels starting at

1095, 1110 and 1125 m and on 42 major discontinuities at bench levels starting at 1020 and 1035 m were available for this study from previous investigations conducted for the mine. For the discontinuity trace data on scanlines, the boundaries in the vertical direction were set as the top and bottom of the bench face, and the boundaries in the horizontal direction were set according to the length of the planar segments of the bench face. To avoid having curved scanlines, the curve of the cut slope face was approximated with straight sections. The dip direction and dip of the bench face, the global location, elevation, trend and plunge of the scanline, the rock type and the exposure condition of the rock mass (unweathered, weathered or altered) were recorded. For discontinuities that intersect the scanline, the intersection distance, strike, dip, apparent dip, semi trace lengths on each side of the scanline, termination type for each trace and aperture were recorded. Eighteen inches was used as the cut-off length for the traces. All of the data collected in the field were entered on a scanline survey logging form designed at the University of Arizona by the Rock Mass Modeling Research Group headed by Professor Kulatilake.

3 DEVELOPMENT OF A 3D MINE VISUALIZATION MODEL

In order to visualize discontinuities that have been mapped within the mine, a research initiation was made to develop a three-dimensional visualization model. The goals of this model are to act as a repository for all geologic mapping in the mine and as a tool to visually inspect the pit walls for areas of potential instability. For the scope of this project, only discontinuities mapped on the final pit wall in the North side of the Esperanza Pit were used.

The foundation of the visualization model is an aerial survey, which was performed with the resulting map entered into an AutoCAD® drawing. Input data for the Visualization Model came from numerous sources including mine geologic maps, University of Arizona research, and scanlines conducted for this project. Some of the data were digitized into the visualization model through the use of a digitizing tablet. All starting and ending points, of discontinuities, were assigned three-dimensional coordinates with their respective dip direction and dip placed next to the dip symbol in the model. By assigning three-dimensional coordinates to all of the input entities in the model, the model can then be viewed isometrically with the entities retaining their original positions.

The model has four basic input entities namely, faults, joints and joint sets, scanlines and scanline data, and lithology. Other important features of the model are the toe-lines of the benches, crest-lines of the benches and the 3 m (10 feet) elevation contour lines. A snapshot of the visualization model can be seen in Figure 2. Shown discontinuity lines indicate the strike direction with the orientation given by a three-digit dip direction followed by a two-digit dip along with a dip symbol indicating the direction of the dip. When projecting discontinuities onto the upper and lower benches dashed lines were utilized to indicate uncertainty

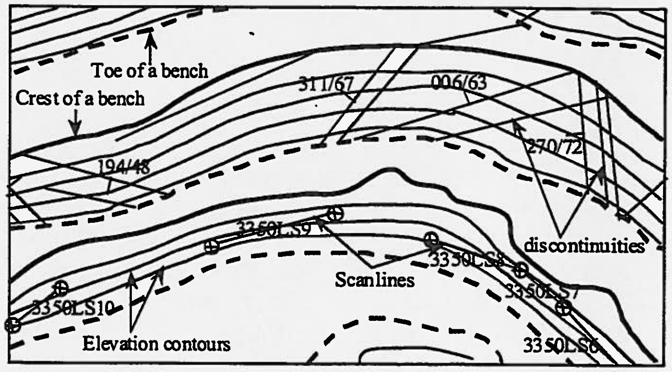

Figure 2. A snapshot of a preliminary mine visualization model.

4 LABORATORY TESTING AND RESULTS

Testing was performed on both the EQMP and TrOA rock types gathered from the mine, via the University of Arizona's Department of Mining and Geological Engineering, Rock Mechanics Laboratory. A series of tests were performed to obtain the intact rock and joint strength properties of both rock types, namely, Brazilian Disk Tension, Uniaxial Compression, and the Large Scale Direct Shear Tests. A total of 9 Brazilian Disk Tension, 10 Uniaxial Compression, and 4 Large Scale Direct Shear tests were performed between the two rock types. A large variability in the quality of TrOA samples was noticed, during testing, with most being heavily fractured and veined, while the EQMP was a competent rock with few fractures and little veining.

4.1 *Intact Rock Test Results*

The Brazilian Disk Tension Test was performed on the EQMP and TrOA rock types to obtain the tensile strength of the intact rock. Brazilian Disk sample testing consisted of cutting 51 mm diameter core to a length/diameter (l/d) ratio of 0.5, and then loading the disk across its diameter until tensile failure occurred. The sample preparation and testing was conducted according to ASTM D3967-81 testing standards. Tensile strength for the EQMP was determined to be 12.5 MPa with a 0.07 coefficient of variation. The mean tensile strength obtained for the TrOA rock type was 18.4 MPa with a 0.06 coefficient of variation. The tensile strengths of the two rock types indicate that the intact material of the two rock types is strong.

All Uniaxial Compression Tests were performed with axial strain measurement following ASTM D2938-79 and D3148-86 testing standards. Samples were tested in a servo-controlled SBEL Loading frame, by applying a vertical load, at a constant rate of 0.00076 mm/sec, until failure occurred. The average compressive strength for the EQMP was 168.9 MPa, with a 0.13 coefficient of variation. An average Young's Modulus of 21 GPa with a 0.09 coefficient of variation was obtained for EQMP. These values indicate that the intact material of the EQMP rock type is strong. The resulting intact rock characteristics of TrOA have an average compressive strength of 171.5 MPa with a 0.19 coefficient of variation. An average Young's Modulus of 19.5 GPa was obtained for TrOA with a 0.04 coefficient of variation. These values indicate that the intact material of the TrOA rock type is strong.

However, because both rock masses are highly fractured, the rock mass strength and deformability of both rock masses would be quite low. There was evidence for this fact even from the test results of a few 51 mm diameter TrOA and EQMP samples that had discontinuities. Note that strength and deformability values of such samples were not taken into account in estimating mechanical property values of EQMP and TrOA intact rock.

4.2 *Discontinuity Test Results*

Each sample was cut from a block of rock collected from the mine site down to a 152 mm cube. Samples prepared for the natural joint testing had a more or less horizontal discontinuity present at the mid height level of the sample while the samples for saw cut joint testing had a saw cut in place of the natural discontinuity. Samples were then placed in a molding box having 305 m by 305 m bottom with a height of 178 mm. Once the sample was centered in the box, a calcium-aluminate and #20 silica sand cement mixture was made. The molding box was then filled with approximately 63.5 mm of compacted cement followed by 51 mm of #30 silica sand and finally filling the molding box with compacted cement. Note that with this arrangement, #30 silica sand exists around the joint. The top of the box was then leveled and cleaned. Curing time for the cement was approximately 18 hours at room temperature, after which the molding box was opened, the sand and two halves of the joint sample removed.

In order to quantify the range of joint properties that can be seen in a rock mass, It is necessary to test different types of joints such as (a) natural rough unfilled rock joints, (b) natural rough rock joints with filling material, (c) slickensided unfilled joints, (d) slickensided joints with filling material and (e) saw cut joints. To find slickensided joints, it may be necessary to obtain samples of many joints. On the other hand, slickensided joints with weak filling material, slickensided unfilled joints or rough rock joints with very thick weak filling material most probably would provide the minimum strength parameters for a rock joint. However, to cut down the cost and time, only two different types of samples (namely, a saw-cut joint and a natural rock joint) were tested for each rock type. The purpose of the saw-cut joint is to obtain values close to the basic friction angle (ϕ_b) of the rock type, while the natural rock joint would be a representative sample of many joints one might see in the rock mass. Testing was performed using a Wykeham Farrance 25502 large-scale direct shear machine at a shear displacement rate of 0.06 mm/min. The tests were performed using five normal stresses, namely, 0.25MPa, 0.50MPa, 0.75MPa, 1.00MPa and 1.25MPa. During testing, the shear displacement, normal stress, and shear stress were recorded, via a data acquisition system, over a total shear displacement of at least 15.2 mm.

For both rock types, natural joints produced higher strengths compared to the saw cut joints due to existence of roughness (Um et al., 2000). Basic friction angles of 30° and 35° were obtained for TrOA and EQMP joints through saw cut samples, respectively. As the worst scenario, it may be possible to have some slickensided rock joints with weak filling material in the considered rock masses. Therefore, to be on the conservative side, a friction angle of 25° was used for discontinuities of both rock types in the rock slope stability analyses conducted in this investigation.

5 KINEMATIC ANALYSES

5.1 *Introduction*

"Kinematic" refers to the motion of bodies without reference to the forces that cause them to move (Goodman, 1989). For the area investigated in this study, kinematic analyses were performed to estimate maximum safe slope angles (MSSA) with respect to the three basic failure modes: plane sliding, wedge sliding and toppling, under only gravitational loading. The basic concepts related to estimation of maximum safe slope angles for the three basic modes of failure are given in (Goodman, 1989).

5.2 *Performed Analyses and Results*

From the discontinuities mapped through scanline surveys, the longest 6% were selected from each of the two rock types EQMP and TrOA. This provided 53 and 16 discontinuities, respectively from EQMP and TrOA rock types. These discontinuities are identified by a five digit number and they are greater than about 10 m for EQMP and 5 m for TrOA. In the five digit number, the first digit represents the bench level (lower benches=1, middle bench=2 and upper benches=3), the second and third digits represent the scanline number and the last two digits identify the particular discontinuity. In addition, there were 129 other discontinuities that were more than 15 m in length obtained from other sources. The orientation and global location of all these discontinuities are known. Discontinuities were sorted into the following 6 groups (Um et al., 2001): (a) lower benches (1020-1050 m) of EQMP rock mass, (b) middle bench (1065-1080 m) of EQMP rock mass, (c) upper benches (1095-1140 m) of EQMP rock mass, (d) lower bench (1035-1050 m) of TrOA rock mass, (e) middle bench (1065-1080 m) of TrOA rock mass and (f) upper benches (1095-1140 m) of TrOA rock mass.

Kinematic analyses for plane sliding and toppling were performed for each of the aforementioned 6 groups of discontinuities to find the MSSA using the computer program KINEM developed by Kulatilake and Chen in 1996 (Um et al., 1996). Different cut slope directions were used to simulate the changing strike direction of the open pit mine in the investigated area. For both EQMP and TrOA rock discontinuities values greater than 30° were obtained for the basic friction angle (Um et al., 2000). As the worst case scenario, it may be possible to have some slickensided rock discontinuities with weak filling material in the considered rock masses. Therefore, to be on the conservative side a friction angle of 25° was used for the kinematic analyses conducted in this investigation. Kinematic analysis for wedge sliding was also performed for each of the aforementioned 6 groups of discontinuities selecting different major discontinuity combinations to calculate the MSSA for each combination. Within each group, each pair of discontinuities within about 30 m distance apart was considered with respect to forming possible wedges.

However, these pairs were picked either only from the scanline data or only from the other major discontinuity data. Different cut slope directions were used to simulate the changing strike direction of the open pit mine in the investigated area. Discontinuity friction angle of 25° was used for the wedge kinematic analysis.

Typical results obtained from kinematic analysis for discontinuity data of the lower benches (1020-1050 m) of EQMP rock mass are shown in Figure 3. Table 1 shows the results of obtained maximum safe slope angle corresponding to the plane sliding and toppling modes for the different rock types. The results show that only 3% of the major discontinuities can give rise to toppling failure under gravitational loading for slope angles greater than 70 degrees in TrOA rock mass. For EQMP rock mass, it changes to 10.6%. If the slope angle is brought down to 50 degrees, 10.6% can be brought down to 4.5%. The results show that about 24.2% of the major discontinuities can give rise to single plane sliding under gravitational loading for slope angles greater than 70 degrees in EQMP rock mass. If the slope angle is brought down to 50 degrees, 24.2% can be brought down to 14.4%. The results show that about 24.6% of the major discontinuities can give rise to single plane sliding under gravitational loading for slope angles greater than 70 degrees in TrOA rock mass. If the slope angle is brought down to 50 degrees, 24.6% can be brought down to 10.8%.

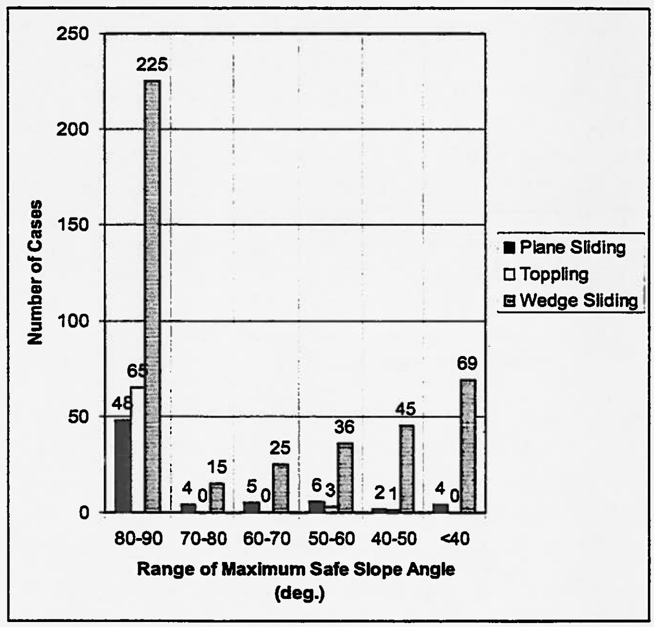

Figure 3. Typical results obtained from kinematic analysis for discontinuity data present in the Lower Benches (1020-1050 m) of EQMP rock mass.

The results given in Tables 1 also show that about 80.9% of the major discontinuities have contributed as main discontinuities in forming wedges in wedge sliding under gravitational loading for slope angles greater than 70 degrees in EQMP rock mass. This 80.9% can be brought down to 69.5% by reducing the slope angle to 50 degrees. The results show that about 50.7% of the major discontinuities have contributed as main discontinuities in forming wedges in wedge sliding under gravitational loading for slope angles greater than 70 degrees in TrOA rock mass. This 50.7% can be brought down to about 35.8% by reducing the slope angle to 50 degrees.

Table 1. Percentages of Major Discontinuities Contributed to Possible Plane, Wedge and Toppling Instabilities in the Two Rock Masses for Different Slope Angles According to Kinematic Analysis.

Rock Type	EQMP		TrOA	
Cut Slope Angle	50°	>70°	50°	>70°
Plane Sliding	14.4%	24.2%	10.8%	24.6%
Toppling	4.5%	10.6%	-	3%
Wedge Sliding	69.5%	80.9%	35.8%	50.7%

6 BLOCK THEORY ANALYSES

6.1 *Introduction*

Figure 4 shows five types of blocks in a surface excavation formed by discontinuities. Although the actual blocks are in three dimensions, to simplify the illustration a two-dimensional figure is used. An infinite block (type V), as shown in Fig. 4(a) is not dangerous as long as it is incapable of internal cracking. Fig. 4 (b) is an example of type IV nonremovable tapered blocks. It is finite, but it cannot come out to free space because of its tapered shape. Finite and removable blocks can be separated into three categories, namely type III, type II, and type I. As shown in Fig. 4(c), a type III block is stable without friction under its gravity alone. A type II block as shown in Fig. 4(d) can remain stable as long as the sliding force on the block is less than its frictional resistance.

Under only gravitational loading, the type II blocks are stable. However, they can come out into the free surface of excavation if there are external forces like water forces, inertia forces etc. that make the total sliding force to be greater than the frictional resistance. Therefore, type II blocks are also called potential key blocks. Finally, a key block, that is denoted by type I and shown in Fig. 4(e), can slide into free space under gravitational loading without any external force unless a proper support system is provided. Therefore, the identification of key blocks and potential key blocks is one of the most important parts in a rock slope stability analysis. The MSSA, for the excavation containing the blocks, can then be determined, through the use of block theory, for each of the key and potential key blocks. Procedures are given in the literature to separate these different block types along with the assumptions used in block theory (Goodman and Shi, 1985).

Kulatilake and Um (see Um and Kulatilake, 1996 and 2001; and Um et al., 2001) have developed computer programs to perform block theory analysis and to calculate maximum safe slope angles corresponding to types I and II blocks.

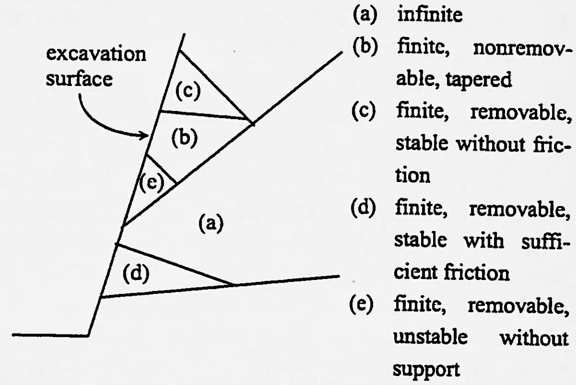

Figure 4. Blocks in a surface cut. (a) infinite, (b) tapered, (c) stable, (d) potential key block, (e)key block.

6.2 *Performed Analyses and Results*

Block theory analysis was performed separately for each of the 6 groups of discontinuities mentioned in the previous Section. Within each group, each combination of 3 to 7 discontinuities located within a distance up to about 30 m was considered to form possible blocks according to block theory. Different cut slope directions were used to simulate the changing strike direction of the open pit mine in the investigated area. Discontinuity friction angle of 25° was used for the block theory analysis. From each combination of discontinuities, key blocks (type I) and potential key blocks (type II blocks) were identified. For each of the two block types, the corresponding sliding mode was determined as either plane sliding or wedge sliding. Through this way, all possible blocks having a number of faces between 4 and 8 producing a type I or type II block having either a plane sliding or wedge sliding mode were determined.

Results obtained for each of the 3 to 7 discontinuity combinations showed that the same sliding mode exists for a number of type I or type II blocks having a particular number of faces or different number of faces (Um et al., 2001). This indicates that several type I blocks have the potential to slide along the same discontinuity plane. Therefore, as far as the instability picture is considered, all these type I blocks having the same sliding direction can be represented by one main discontinuity along which the blocks slide and a number of secondary discontinuities that can contribute through different subsets to form different blocks; for these blocks one MSSA exists corresponding to the strike of the cut slope. Similarly, several type I blocks have the potential for wedge sliding along a line of intersection between two particular discontinuity planes. These type I blocks can be represented by two main discontinuity planes that form the line of intersection and a number of secondary discontinuity planes that contribute through different subsets in forming different blocks; for these blocks one MSSA exists corresponding to the strike of the cut slope. Type II blocks can be represented in a similar manner separately for single plane sliding and wedge sliding. This means for each of the 6 groups mentioned in the previous section, the MSSA can be given under 2 separate tables as follows: (a) Type I blocks-single plane and wedge sliding, and (b) Type II blocks-single plane and wedge sliding. In each of these tables the MSSA were arranged in the decreasing order separately for single plane and wedge sliding (Um et al., 2001). One of these tables is shown in Table 2. In each of these tables, by going from the bottom to the top, one can find the discontinuities that can give rise to slope instability under a particular cut slope angle. For example, for a cut slope angle of 70°, all the discontinuity cases having a MSSA less than 70° can give rise to slope instability under only gravitational loading. Note that each group tells from which benches and what locations the discontinuities are coming from. Each table also provides the type of block and the corresponding sliding mode. This information should be useful in designing and managing pit slopes. This information also can be used to predict the behavior of the pit slope at a certain location when the information about the new discontinuities at that location is known.

Table 3 shows the percentages of major discontinuities contributed to possible plane and wedge instabilities under block types I (key blocks) and II (potential key blocks) in the two rock masses for different slope angles according to block theory analysis. The results given in Table 3 show that about 16.0% of the major discontinuities can contribute as main discontinuities in forming type I blocks and give rise to single plane sliding under gravitational loading for slope angles greater than 70° in EQMP rock mass. If the slope angle is brought down to 50°, the 16.0% can be brought down to 7%. The corresponding percentages associated with type I blocks in TrOA rock mass are respectively 4.5% and 0%. These four percentages are significantly lower than the corresponding percentages obtained for single plane sliding mode under kinematic analysis in EQMP and TrOA rock masses (see Table 1). It is important to note that under kinematic analysis, types I through V blocks can produce single plane sliding. In reality, types IV and V blocks cannot cause instability situations. In addition, types II and III blocks would be stable under gravitational

Table 2. Maximum safe slope angles for type I blocks having either single plane or double plane sliding modes formed by the discontinuities present in the middle bench (1065-1080 m) of TrOA rock mass.

(a) Single Plane Sliding

Main Discontinuity	ID #s of Supporting Discontinuities that Contribute to Forming Blocks													Sliding Direction		Cut Slope	Max. Safe
	1	2	3	4	5	6	7	8	9	10	11	12	13	Dip (deg.)	DipDir. (deg.)	DipDir. (deg.)	Slope Angle(deg.)
20334	20165	10310	20331	20354	20356	20363	20201	20243	20310	20434				88	296	242.5	90
20165	20334	20354	20356	20363	20401	20201	20202	20204	20205	20213	20243	20310	20331	44	278	242.5	53

(b) Double Plane Sliding (Wedge Sliding)

ID #s of Main Discontinuities		ID #s of Supporting Discontinuities that Contribute to Forming Blocks													Sliding Direction		Cut Slope	Max. Safe
1	2	1	2	3	4	5	6	7	8	9	10	11	12	13	Dip (deg.)	DipDir. (deg.)	DipDir. (deg.)	Slope Angle(deg.)
20334	20243	20165	20310	20331	20354	20356									88	287	242.5	89
20201	20334	20354	20356	20363											86	240	242.5	86
20354	20243	20331	20334	20356											81	287	242.5	83
20201	20354	20334	20356	20363											77	283	242.5	80
20363	20201	20334	20354	20356											45	293	242.5	57
20363	20243	20356	20401												44	287	242.5	53
20165	20354	20310	20331	20334	20356	20363	20401								44	276	242.5	49
20363	20354	20165	20334	20356	20401	20201									42	276	242.5	47

Table 3. Percentages of major discontinuities contributed to possible plane and wedge instabilities under block types I (key blocks) and II (potential key blocks) in the two rock masses for different slope angles according to block theory analysis.

Rock Type		EQMP		TrOA	
Cut Slope Angle		50°	>70°	50°	>70°
Type I	Single Plane Sliding	7%	16%	0%	4.5%
	Wedge Sliding	27.5%	46.6%	10.4%	29.8%
Type II	Single Plane Sliding	0%	0%	0%	0%
	Wedge Sliding	54.2%	61.8%	26.9%	28.4%

loading. Therefore, the number of single plane sliding situations given by kinematic analysis would be always higher than the reality. Single plane sliding situations given by block theory would be much closer to the reality.

The results given in aforementioned tables show that 46.6% of the major discontinuities can contribute as main discontinuities in forming type I blocks and give rise to wedge sliding under gravitational loading for slope angles greater than 70° in EQMP rock mass. If the slope angle is brought down to 50°, the 46.6% can be brought down to 27.5%. The corresponding percentages associated with type I blocks giving rise to wedge sliding in TrOA rock mass are respectively 29.8% and 10.4%. These four percentages are significantly lower than the corresponding percentages obtained for wedge sliding mode under kinematic analysis in EQMP and TrOA rock masses (see Table 1). This difference is due to the same reasons as mentioned in the previous paragraph under single plane sliding. Therefore, the number of wedge sliding situations given by kinematic analysis would be always higher than the reality. Wedge sliding situations given by block theory would be much closer to the reality.

It is important to note that not a single type II block exists that has potential to fail under single plane sliding in the presence of gravitational plus external forces in both EQMP and TrOA rock masses. To form a type II block under single plane sliding, it is necessary for the dip angle of the main discontinuity to be less than the friction angle of the discontinuity. From all the selected major discontinuity data, only four discontinuities (two for each rock type) have dip angles less than the friction angle. Most probably even these 4 discontinuities might have contributed to forming only type IV or type V blocks.

For type II blocks under wedge sliding mode and gravitational loading, irrespective of the plunge angle of the line of intersection, the sliding force due to the weight of the block is less than the frictional resistance of the discontinuities. Therefore, under only gravitational loading, each type II block is stable and the MSSA corresponding to each type II block is 90°. However, in this study, the MSSA for each type II block was calculated assuming that the resultant force of the weight of the block and possible external forces acting on the block exceeds the frictional resistance. This means the slope angle that makes a type II block becoming a type V block is considered as the MSSA value for each type II block in this study. It is important to note that this assumption produces the worst possible result with respect to instability for type II blocks. The results given in Table III show that 61.8% of the major discontinuities can contribute as main discontinuities in forming type II blocks and provide possible wedge sliding under gravitational plus external loading for slope angles greater than 70° in EQMP rock mass. If the slope angle is brought down to 50°, the 61.8% can be brought down to 54.2%. The corresponding percentages associated with type II blocks in TrOA rock mass are respectively, 28.4% and 26.9%. According to the procedure used in the kinematic analysis, the lines of intersections having a plunge angle less than the friction angle of the discontinuities produce maximum safe slope angles of 90°. Therefore, for type II blocks it is not possible to make a comparison between the results obtained through kinematic and block theory analyses.

The primary advantage of block theory over traditional kinematic analysis is that it gives the ability to identify the key blocks that require immediate attention. It separates the most important and dangerous blocks from the less critical ones. However, without the ability to look into toppling failure, kinematic analysis should be performed in conjunction with block theory. Both methods contain assumptions that lead to both conservative and non-conservative results. The assumption that all discontinuities are of infinite extent leads to conservative results. Leaving many discontinuities out and not considering repeating joint sets, on the other hand, leads to non-conservative results.

7 LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSES

7.1 *Scope of the Investigation*

The purpose of this section is to show the effect of water force, a tension crack, lower slope face inclination, overall wedge height and double benching on factor of safety of wedge stability. All these are illustrated by performing limit equilibrium slope stability analyses on a single tetrahedral wedge belonging to type II block category that exist in the investigated area in the mine. All the limit equilibrium slope stability analyses were conducted using a computer program developed in 1987 (Kulatilake and Fuenkajorn, 1987).

7.2 *Wedge Geometry and Material Properties used in the Wedge Stability Analyses*

The upper slope face of the wedge was considered as horizontal like the top face of a bench at the mine. The two discontinuity planes that form the type II block along with the excavation face were used to form the required wedge. The orientation of the line of intersection for the chosen type II block has trend=218.5° and plunge = 35.1°. For the wedge with a tension crack, it was considered that the strike of the tension crack is parallel to the lower slope face and the dip of the tension crack face is 90°. The cohesion of the two discontinuity planes was considered to be zero. The other input parameter values used for the wedge stability analyses are given in Table 4. For both EQMP and TrOA rock discontinuities, values greater than 30° were obtained for the friction angle (Um et al., 2000). As the worst case scenario, it may be possible to have some slickensided rock discontinuities with weak filling material in the considered rock masses. Therefore, to be on the conservative side a friction angle of 25° was used for the stability analysis conducted in this investigation. Note that the dip direction of the lower slope face is 214°. Therefore, the apparent plunge of the line of intersection along the dip direction of the lower slope face is 35.2°. This means in the wedge stability analyses, lower slope face angles greater than 35.2° should be used. The minimum value used for the lower slope face angle in this investigation was 40°.

Table 4. Input parameter values for Limit Equilibrium Wedge Slope Stability Analyses.

Dip direction of lower slope face	214.0 degrees
Dip angle of upper slope face	0.0 degrees
Dip direction of tension crack face	214.0 degrees
Dip angle of tension crack face	90.0 degrees
Unit weight of rock	25.1 KN/m^3
Unit weight of water	9.8 KN/m^3
Dip direction of plane A	132.0 degrees
Dip direction of plane B	292.0 degrees
Dip angle of plane A	85.0 degrees
Dip angle of plane B	68.0 degrees
Angle of friction on plane A	25.0 degrees
Angle of friction on plane B	25.0 degrees

7.3 *Different Analyses Performed and the Results*

Wedge stability analysis was first performed for the dry wedge without the tension crack assuming a bench height of 15 m. A similar analysis was then performed assuming a bench height of 30 m. Then for the dry wedge with the tension crack, wedge stability analysis was performed for bench heights of 15 and 30 m. For all the above four cases, irrespective of the lower slope angle and the distance of the tension crack from the top edge of the wedge, a factor of safety of 2.28 was obtained. Note that this is correct because the cohesion was assumed to be zero for both discontinuity planes. Note that this factor of safety indicates a stable type II tetrahedral block under dry conditions.

The same type II block was then considered with the tension crack and water pressure. It was assumed that the tension crack is filled up with water and there is unrestricted hydraulic connection between the base of the tension crack and the two discontinuity planes. Also it was assumed that the intact rock is impermeable and hence all of the water around the wedge is transmitted along the two discontinuity planes and the face of the tension crack. According to these assumptions, the water pressure increases linearly with depth from zero at the top edge of the tension crack to a maximum value at the base of the tension crack where the tension crack meets the line of intersection of the two discontinuity planes. On the two discontinuity planes, the maximum water pressure occurs at this same location. Along the line of intersection between the two discontinuity planes, the water pressure decreases linearly from a maximum value at this point to zero value at the location the line of intersection appears on the lower slope face. Water pressure is also zero on the lines of intersections between each of the two discontinuities and the upper and lower slope faces. On the discontinuity planes, the water pressure increases linearly from zero at the upper or lower slope faces to the maximum value at the location where the tension crack meets the line of intersection of the two discontinuity planes. This water pressure distribution is believed to be representative of the extreme conditions that could occur during very heavy rain at the mine.

The factor of safety for wedge stability was calculated for lower slope face angles between 40° and 90° at an increment of 10° assuming the distance of the tension crack from the top edge of the wedge as 6 m and the overall height of the wedge as 30 m. These calculations were also repeated assuming the overall height of the wedge as 15 and 60 m. The obtained results are shown in Figure 5. Note that the factor of safety value is not given for the slope angle=40° in Fig. 5 for the case with overall height of the wedge = 15 m. For this particular geometry, the line of intersection of the two discontinuity planes does not intersect the tension crack that is placed 6 m from the top edge of the wedge. Therefore the factor of safety value cannot be calculated for this particular geometry of the wedge. From Fig. 5, it is clear how a stable slope under dry condition can fail after a heavy rainfall. The figure also shows how a stable slope under a flatter lower slope face angle can fail as the inclination of lower slope face increases. From Fig. 5, it is also clear how a stable slope under single benching (overall wedge height= 15 m) can fail under double benching (overall wedge height= 30 m). For the chosen type II block, to achieve a factor of safety of 1.5, it is necessary to pick decreasing lower slope angles as height of the wedge increases from 15 m to 60 m (see Fig. 5). This information basically provides the concept related to designing slope angles for single benches (15 m), double benches (30 m), inter-ramps (greater than 30 m) and overall pit slopes (greater than 60 m) in mines.

As the last set of sensitivity analyses, the factor of safety for wedge stability was calculated for the wedge with the tension crack and water pressure changing the distance of the tension crack from the top edge of the wedge from 3 m to 15 m at an increment of 3 m. These calculations were done for overall wedge heights of 15, 30 and 60 m using a lower slope face angle of 80°. The results are shown in Figure 6. This figure shows how a stable slope can become an unstable slope as the distance of the tension crack from the top edge of the wedge decreases. The figure also shows the effect of the location of the tension crack on double benching and overall slope height.

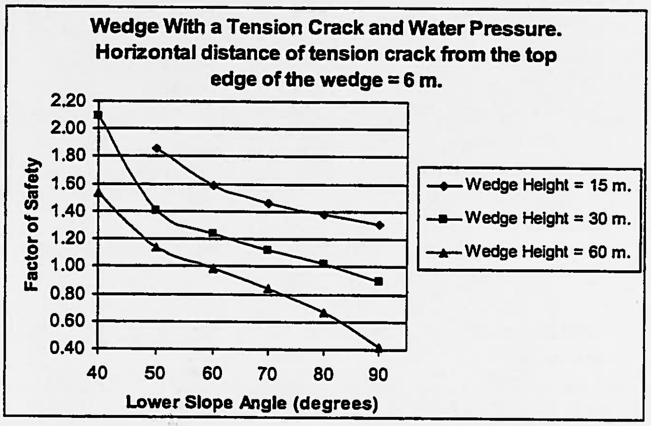

Figure 5. Effect of lower slope angle and wedge height on factor of safety.

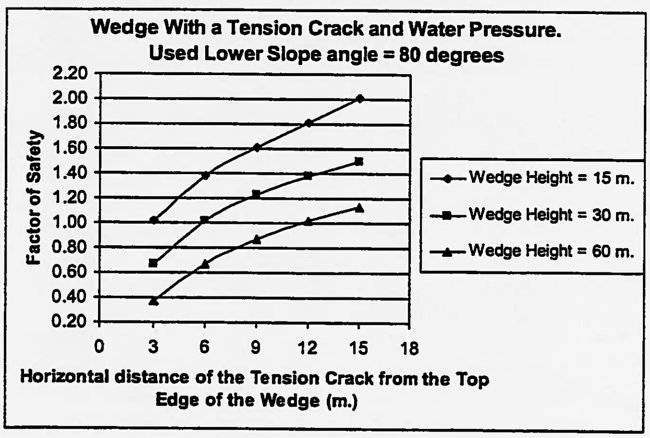

Figure 6. Effect of location of tension crack and wedge height on factor of safety.

8 CONCLUSIONS

The developed preliminary version of the mine visualization model expected to be a useful tool in selecting discontinuity data for rock slope stability analyses. The model can be improved by showing the discontinuity planes and the possible blocks that can be formed by the discontinuity planes in three dimensions. Such information will be extremely useful in selecting the rock blocks that should be subjected to rock slope stability analyses. The blocks that are found to be unstable during stability analyses then can be marked on the visualization model. This will be extremely useful in identifying the instability areas of the mine.

Intact material of the two rock types was found to be strong based on the mechanical property test results of intact rock. However, because both rock masses are highly fractured, the rock mass strength and deformability of both rock masses seem to be quite low. For both rock types, natural joints produced higher strengths compared to the saw cut joints due to existence of roughness. Basic friction angles of 30° and 35° were obtained for TrOA and EQMP joints, respectively. As the worst scenario, it may be possible to have some slickensided rock joints with weak filling material in the considered rock masses. Therefore, to be on the conservative side, a discontinuity friction angle of 25° was used in performing rock slope stability analyses.

For single plane and wedge sliding modes, the block theory provides results closer to the reality compared to the kinematic analysis. However, for toppling mode, one has to rely on the results coming from kinematic analysis. For both EQMP and TrOA rock masses, the percentages of major discontinuities that can give rise to plane, wedge and toppling failure under gravitational loading for different slope angles are given in the paper. These results indicate that toppling failures would be low in the investigated region. Under only gravitational loading, type II blocks that can be formed in both EQMP and TrOA rock masses are stable under both single plane and wedge sliding modes. Therefore, the instability block conditions in the investigated region of the mine under only gravitational loading depend on the possibility of forming type I blocks. The obtained results clearly indicate the possibility of finding more wedge instabilities compared to the plane instabilities through type I blocks in the investigated region.

When the gravitational load is combined with the external loading such as water forces or dynamic forces, in addition to all type I blocks, some type II blocks can give rise to failure under single or wedge sliding. The number of failures due to type II blocks increases as the magnitude of the external loading at the site increases. Only two percent of the selected major discontinuities have a dip angle greater than the friction angle of the discontinuities. Therefore, chance of single plane sliding arising from a type II block in the investigated area is negligible. The obtained results indicate that the chance of wedge sliding taking place through a type II block under a combined gravitational and external loading is quite high in the investigated area. The orientations and locations of the main discontinuities that can contribute to forming type I or type II blocks and give rise to possible plane and wedge failures under a selected slope angle can be found from the results obtained (Um et al., 2001). Slope instability taking place due to type II blocks can be reduced by reducing the magnitude of the external loading at the site. Therefore, slope dewatering should be done regularly to keep the water levels down to the bear minimum in the slopes in the investigated region.

The results obtained through limit equilibrium slope stability analyses conducted on a single tetrahedral wedge belonging to type II block category that exist in the investigated area in the mine show the following: (a) how a stable slope under dry condition (factor of safety = 2.28) can fail after a heavy rainfall, (b) how a stable slope under a flatter lower slope face angle can fail as the inclination of lower slope face increases, (c) how a stable slope under single benching (overall wedge height = 15 m) can fail under double benching (overall wedge height = 30 m), (d) how a stable slope can become an unstable slope as the distance of a tension crack from the top edge of the wedge decreases and (e) the effect of the location of the tension crack on double benching and overall slope height. For the chosen type II block, to achieve a factor of safety of 1.5, it is necessary to pick decreasing lower slope face angles as height of the wedge increases from 15 m to 60 m (see Fig. 5). This information basically provides the concept related to designing slope angles for single benches (15 m), double benches (30 m), inter-ramps (greater than 30 m) and overall pit slopes (greater than 60 m) in mines.

The intact strength and deformability properties of the two rock types indicate that both rock types are strong as intact material. However, because both rock masses are highly fractured, the rock mass strength and deformability of both rock masses seem to be quite low. In addition to that since the mean block size for both rock masses is small, there is more tendency for instability of the rock masses through movements occurring through interactions between many small rock blocks compared to separation of a few large rock blocks under plane or wedge failures. Such possibilities can be investigated by performing stress and deformation analysis of the rock mass incorporating a discontinuum numerical technique.

ACKNOWLEDGEMENT

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Presenters Introduction Form

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Ohio River Valley Soils Seminar XXXV
Rock Engineering and Tunneling
October 20, 2004

Office Use Only
Session Time:
Moderator:

PAPER TITLE: **A Case Study on Open Pit Mine Slope Stability**

AUDIO-VISUAL NEEDS:
(A laptop computer and screen will be provided. Additional equipment should be noted below.)

PRESENTER: (please note Mr., Mrs., Ms., Dr.)
(If applicable, please provide a phonetic spelling of the presenter's name to assist the moderator during introductions)

NAME: **Dr. Pinnaduwa H.S.W. Kulatilake**

TITLE: **Professor**

COMPANY: **Dept. of Mining & Geological Engineering, University of Arizona**

PRESENTER BIO: **Pinnaduwa Kulatilake is a Professor of Geotechnical Engineering at the University of Arizona. He has over 25 years of experience in rock mechanics, geotechnical engineering, and application of probabilistic and numerical methods to geotechnical engineering. He has written over 125 papers and is a member of several technical committees. He has been serving over 18 years as an examiner for the geological engineering professional exam conducted by the Arizona State Board of Technical Registration. He has delivered 10 keynote lectures and 32 other invited lectures throughout the world on topics related to fracture network modeling, probabilistic geotechnics, mechanical properties of joints, rock slope stability and mechanical and hydraulic behaviour of rock masses. He is a research paper reviewer for 15 technical Journals and an editorial member for Int. Jour. of Rock Mechanics & Mining Sciences and Int. Jour. of Geotechnical and Geological Engineering. Due to the contributions that he made on teaching, research, consulting and service activities, he was elected to the Fellow Rank of the American Society of Civil Engineers at the relatively young age of 45. In 2002, he received Distinguished Alumnus Award from the College of Engineering, Ohio State University and Outstanding Asian American Faculty Award from the University of Arizona in recognition of his distinguished achievements and eminent contributions made to the advancement of his profession.**

Planning and Design Considerations – Drumanard Highway Tunnel

Randall J. Essex, P.E.
Director of Tunneling, Hatch Mott MacDonald

David P. Field, P.E.
Senior Tunnel Engineer, Hatch Mott MacDonald

ABSTRACT

The $2.2 Billion Louisville-Southern Indiana Oho River Bridges Project that will link Louisville and southern Indiana with two bridges across the Ohio River includes a multi-bore tunnel system required to carry six lanes of traffic beneath U.S. 42 and an historic property, the Drumanard Estate, located about 7 miles northeast of Lousiville. This paper describes some of the features of the tunnels system, including geotechnical, environmental, and operational issues. Another similar highway tunnel project completed in Colorado in the 1990s, the Hanging Lake Tunnel, is also discussed.

1.0 INTRODUCTION

The East End Approach is one of six design sections of the $2.2 Billion Louisville-Southern Indiana Ohio River Bridges Project that will link Kentucky and Indiana with two bridges across the Ohio River near Louisville. The six sections consist of two bridge sections and four approach sections to the two bridges. Of the four approach sections, the East End Approach is unique in that it includes the excavation of multiple tunnels. The tunnels are required to carry six lanes of traffic beneath US Route 42 and the historic Drumanard Estate in Jefferson County, Kentucky.

Hatch Mott MacDonald (HMM) is responsible for managing the geotechnical engineering for the tunnel project, and is primary sub-consultant to H. W. Lochner, Inc of Lexington for tunnel and systems design. A number of other sub-consultants are included on the design team to address aspects such as aerial mapping and photography, landscape architecture, public outreach, and visualization technologies.

This paper describes some of the features of the tunnels on the project, and addresses a number of geotechnical challenges that will need to be overcome in planning, designing, and constructing the Drumanard Tunnels. Additionally it discusses similarities between this project and the twin-bore Hanging Lake Tunnel in Colorado.

At the time of this writing, preliminary design for the project is just underway. As such, the items addressed in this paper are based on the authors' conceptual understanding and are likely to change as feedback from the public is obtained, and planning and design decisions are made.

2.0 PROJECT OVERVIEW

The region of the twin bridges program is shown in Figure 1. Design and construction of the two approach sections in Kentucky will be administered by the Kentucky Transportation Cabinet (KYTC); the two approach sections in Indiana will be administered by the Indiana

Department of Transportation; and design of the twin bridges across the Ohio River will be administered by a Bi-State Commission which includes representatives of the Federal Highway Administration (FHWA).

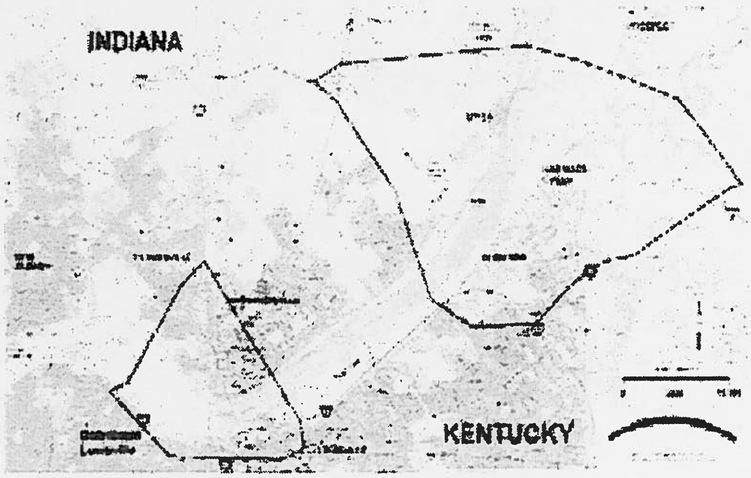


Figure 1 – Region of Twin Bridges Program

The East End Section, shown in Figure 2, is part of the southern approach to the eastern bridge across the Ohio River. The Drumanard Estate is located on a topographic ridge northwest of the intersection of US Route 42 and KY State Route 841, approximately 7

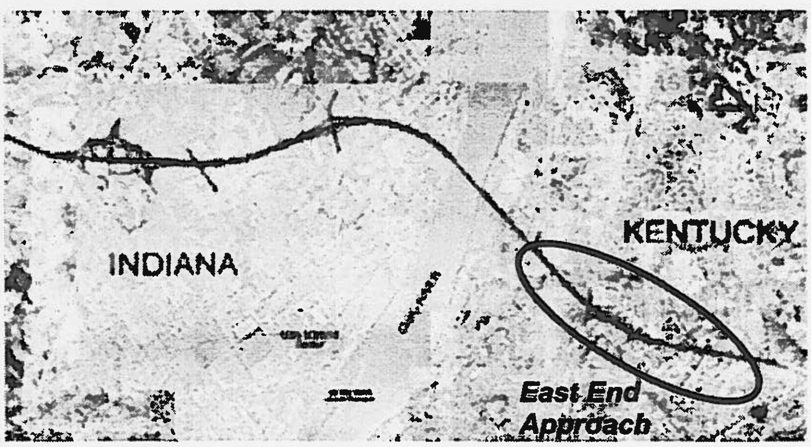


Figure 2 – East Bridge Crossing and East End Appraoch

miles northeast of downtown Louisville. The meandering Harrods Creek, identified on Figure 1 and shown relative to the tunnels on Figure 3, is located immediately northwest of the ridge through which the tunnels will pass. In order to avoid environmental impacts to the creek, bridges will be founded in the vicinity of the north portal, and will carry traffic north of the tunnels across the creek, to viaducts that lead to the eastern bridge.

Design, construction, aesthetic, and operational criteria have not been confirmed as yet. However, in addition to passing beneath the Drumanard Estate, the project extends through an environmentally sensitive area. As such, the project will be designed and implemented in accordance with Context Sensitive Design standards established by the KYTC.

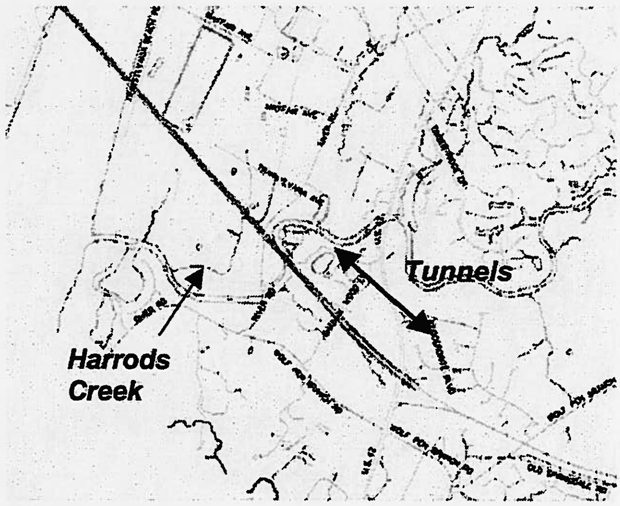


Figure 3 – Drumanard Tunnel and Harrods Creek

3.0 GEOLOGIC CONSIDERATIONS

A number of geologic issues are expected to drive tunnel layout and geometries. The project site setting is the bluffs overlooking the Ohio River Valley northeast of Louisville. Ground elevations along the bluff tops range from El. 580 ft. to about El. 630 ft. The bluffs are crossed at a near right angle by an unnamed stream valley that cuts the bluff at El. 550 ft. The Ohio River and river valley are located north of the site at about El. 420 ft.

3.1 Geologic Conditions

The rocks in the area are more or less flat lying with a very gentle dip to the WNW. Joints shown on available maps are near vertical and strike in a variety of directions. It is expected that the tunnels will encounter the following formations/materials:

- Overburden
- Sellesburg and Jeffersonville Limestones
- Louisville Limestone (portal and tunnel)
- Waldron Shale (portal and tunnel)
- Laurel Dolomite (portal and tunnel)

The following paragraphs describe the formations listed above:

Overburden – The overburden is expected to consist of a 30-inch thick veneer of loess (windblown silt) overlying clay-rich soils weathered from the carbonate rocks below. Overburden is anticipated to be less than 15 feet thick in the portal areas.

Sellersburg and Jeffersonville Limestones – This formation serves as the "cap rock" of the bluffs and consists of limestone to dolomitic limestone. The entire thickness of this formation, about 27 feet, is exposed in a slope immediately north of the project site. At the intersection of Interstates I-71 and I-264 located about 3 miles to the south, 30- to 50-foot high vertical wall cuts of sound Jeffersonville Limestone display little or no evidence of joints, spalling, or other degradation.

Louisville Limestone – The tunnels will likely be excavated in the Louisville Limestone. This formation is difficult to visually distinguish from the Sellersburg and Jeffersonville Limestones, except with a hydrochloric acid bottle - the Sellersburg/Jeffersonville formations are primarily limestone, whereas the Louisville is predominately dolomite. In the Louisville area, the formation is 40 to 80 feet thick and prominent bench-forming massive beds have been noted at 35 and 60 feet above the formation bottom. One such bench is present immediately northwest of the highway intersection. On the topographic maps, this bench is "pockmarked" with depressions reflective of sinkholes that are known to develop on the uplands.

This limestone is present in a number of local quarries, and has been observed to contain a higher frequency of discontinuities than overlying shale formations. The discontinuities are frequently weathered, open, and have calcite infillings, and are thought to be oriented from 0 to 50 degrees.

Waldron Shale - This formation consists of 8 to 15 feet of clay shale with a basal 1-foot thick resistant dolomitic bed. Local references note that the over-steepened banks in the Waldron Shale are subject to failure by sliding, and that cuts require proper drainage and shoring. The slake durability of the shale may be variable, and will need to be quantified across the tunnel alignment.

Laurel Dolomite – The northern end of the project alignment may be located in the Laurel Dolomite, which consists of about 20 feet of thinly bedded greenish gray microcrystalline dolomite overlying 30 feet of massive porous mottled dolomite in two bedding sets separated by a dolomitic clay shale layer.

Groundwater and Natural Gas – The depth of the groundwater is not well known at this time, but is anticipated to be locally perched upon the less permeable Waldron Shale. Local experience suggests that naturally occurring gas is not present in the formations in the project area. Constant gas monitoring will occur during drilling of boreholes advanced as part of planned subsurface investigations.

To the maximum extent possible, the profile of the tunnels will be located so as to maximize the benefits of the more competent limestone layers, and minimize the impacts of the less competent shale layers. Because the tunnels will need to be positioned to accommodate other roadway grade constraints, there is a practical limit to the freedom with which the tunnel profile will be selected. Eventually, tunnel support measures, as well as tunnel excavation sequences, will need to be developed so as to cope with the geologic conditions. Because the tunnels will grade upward to the south, and at a grade steeper than the sub-horizontal bedding of the sedimentary formations, variable rock mass conditions are expected along the tunnel alignment.

3.2 Geotechnical Evaluations

Because of the sensitive nature of the project environment, including the proximity of the tunnels to nearby residences, the minimization of drill-and-blast related vibrations and overpressures will be an important consideration. The strongest limestones may have unconfined compressive strengths on the order of 22,000 to 25,000 psi, which, up until recently, would have been considered beyond the practical limit of mining equipment such as road headers. However, a number of tunneling projects in North America, including rail/transit projects in Montreal and New York City, have involved the utilization of specially designed high-capacity road headers to excavate rocks with strengths comparable to those anticipated for this project. Thus, the geotechnical program will include the evaluation of high-capacity roadheaders as an alternative or enhancement to the use of drill and blast excavation methods.

Typical of a number of complex highway tunnel projects in the U.S. over the last 20 years, including the Cumberland Gap Tunnels in southern Kentucky, an exploratory tunnel will be considered as a key element of the site exploration effort, in order to better identify the nature of the subsurface conditions. This would be carried out following an initial drilling exploration program and prior to final design of the main tunnels. One consideration would be to include a roadheader demonstration program as part of the overall exploratory tunnel project. Demonstrating the technical feasibility on-site would provide a compelling basis to plan the excavation of the main tunnels around the use of roadheaders. These issues will be developed during preliminary design evaluations.

4.0 SIMILARITIES TO HANGING LAKE TUNNEL

As part of the upgrading of Interstate I-70 through Glenwood Canyon, Colorado, the Hanging Lake Tunnel Project conveys four lanes of traffic through twin drill and blast excavated, 42-ft diameter, and 3,800 ft-long bores (see Figure 4). Constructed in the early 1990s, there were a number of roadway design-related, river-related, and environmental constraints that are similar to the challenges to be faced by the Drumanard Tunnels. These similarities are as follows:

- The west portals of Hanging Lake Tunnel include abutments on which twin bridges are founded. The bridges carry traffic across the Colorado River. The east portals are in close proximity to another set of bridges to the east that also carry traffic across the Colorado. Depending on the location of the Drumanard Tunnel's north portals, the bridges across Harrods Creek could be similar to the east portal/bridge configuration of the Hanging Lake Tunnel.
- Due to overall roadway design criteria that demanded design continuity through the 12-mile long Glenwood Canyon alignment, the highway alignment dictated the portal locations. A similar constraint has been established for Drumanard, principally because of the sensitive Drumanard Estate over the tunnel.
- The Hanging Lake Tunnels site is situated within the White River National Forest. As a result, there were stringent environmental constraints imposed on portal layouts, limits of disturbance, final configurations, and restoration vegetation. Similar constraints are anticipated for Drumanard.

It is anticipated that planning and design for Drumanard will draw from a number of successes pioneered on the award-winning Hanging Lake Highway Tunnel project in Colorado.

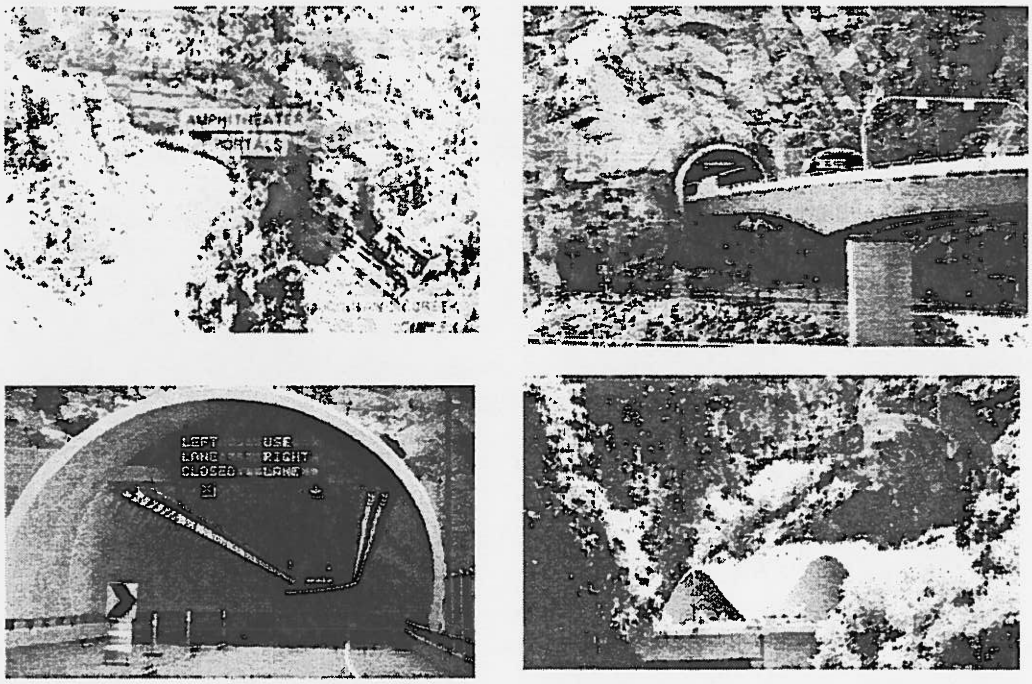


Figure 4 – Hanging Lake Highway Tunnel

5.0 DESIGN CONSIDERATIONS

5.1 Two vs. Three Bores

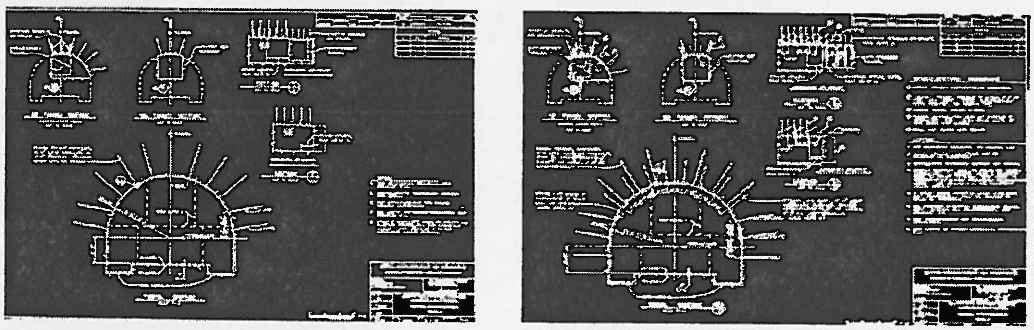
The project requires the provision of six lanes of traffic. One alternative would be to provide twin bores with three lanes per bore. A second alternative would be to provide three bores with two lanes per bore. Both alternatives will be evaluated during the initial stages of design. Considerations will include optimized traffic flow, enhanced operation and maintenance activities, geometric layout relative to the geologic conditions, excavation quantities, and enhanced security/emergency response. The two-bore option would allow for three lanes of flow per tunnel, where as the three-bore option would allow for two lanes of flow per tunnel.

One geotechnical consideration in evaluating the two alternatives relates to the relatively steep downward grade toward the north portal from the south. Because the rock cover is a critical criterion in locating the vertical location of a tunnel portal, the smaller cross-sectional height of the two-lane tunnels would allow the northern portals to be raised in elevation while maintaining the same amount of rock cover. By raising the north portal, a flatter tunnel grade could be achieved. This would result in reduced exhaust caused by uphill, southbound vehicles, and reduced noise generated by (jake) braking trucks traveling downhill, in the northbound direction.

One consideration that favors the twin three-lane tunnel alternative is this system would likely occupy a narrower "out-to-out" footprint. Also, a single rock pillar is required for the twin tunnels as opposed to two rock pillars for the three-tunnel option. The actual geometrical differences will depend on the tunnel configurations selected (lane and shoulder width requirements) and pillars, which could be different for the two schemes.

5.2 Excavation and Stabilization

As discussed in Section 3, the geologic conditions are expected to be variable along the tunnel alignment, including voids associated with karstic limestone formations and weak shale layers and interbeds. It is anticipated that the range of anticipated ground conditions will be divided into several categories, with ground support and excavation sequences designed to stabilize each category. By observing the ground during the excavation process, decisions regarding ground support will be decided on an advance-by-advance basis, thereby matching the ground support measures to the ground conditions observed. This program of different ground classifications and ground support measures has been used on many highway tunnel projects in the past, including the Hanging Lake Tunnel discussed in Section 4. Cross-sections of the different ground stabilization methods used for Hanging Lake Tunnel are shown on Figure 5.



Ground support – good ground **Ground Support – weak ground**

Figure 5 – Typical Ground Support Measures

5.3 Final Lining

A final lining will be sought that provides the most economic means of providing a suitable structural tunnel lining that meets the service life criteria with least maintenance. Two options, a cast-in-place concrete lining or a finished in-place shotcrete lining will be considered. Factors to be evaluated will include surface reflectivity (tiles, paint, or hanging panels), and the degree to which a waterproofing system such as an impermeable membrane is required.

5.4 Ventilation

Several possible ventilation scenarios exist for the proposed tunnel alternatives, each considering the vehicle fire loads within the tunnel and the possibility of reverse direction traffic flow or bi-directional flow in each tunnel. This will permit the maximum flexibility in the operation of the tunnel. Because of their economical operational costs and their superiority in controlling smoke during a fire, longitudinal jet fan ventilation will be investigated.

Two separate independent power supplies will be required for the emergency tunnel ventilation. The most economical solution will be to provide power at both ends of the tunnel. This is because the cost of running high temperature power cables from one end of the tunnel is high due to the voltage drop that will occur over these cable lengths and the cost of cable protection.

5.5 Fire/Life Safety

Fire/life safety issues will be reviewed with the KYTC and the local Fire Marshal to identify their specific requirements. The governing standard, NFPA 502, does not specifically address this issue, and is subject to local interpretation.

A radio communication system to permit two-way radio use within the tunnel will be investigated. The design for emergency telephones will also be considered, as well as environmental controls to monitor air quality within the tunnels. The design will also provide fire detection systems within the tunnel and may include manual pull stations, as well as a fire detection cable running the length of the tunnel for each travel lane.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Excavation Methods

The manner in which the tunnel portal approaches and tunnels could be excavated is as much an environmental concern as it is technical. The design team will assess the range of possible methods of excavation that could be applied in excavating the tunnels, including drill and blast methods, tunnel boring machines, roadheaders, and mobile miners. Emphasis will be placed on assessing those excavation methods that could mitigate vibrations associated with drill and blast methods. For example, there are a number of projects in North America where high-capacity roadheaders are being utilized to avoid the use of drill and blast methods. Recent case histories demonstrate that high capacity roadheaders are capable of cutting rock with compressive strengths in excess of 30,000 psi under certain circumstances. The economical usage of these machines will be dependent on the rock characteristics such as compressive strength, tensile strength, and abrasivity. The ability to cut the rock while limiting pick consumption to a manageable level will be the key.

Where drill and blast excavation may be unavoidable, elements will be incorporated into the design that serve to control blast vibrations and overpressures to environmentally acceptable levels, as well as preserve the remaining rock mass by minimizing the propagation of blast gasses back into the rock mass and preventing unnecessary rock mass fracturing.

For the three-bore option, the potential for using a TBM to excavate the three parallel bores will be investigated. The use of TBMs to mine highway tunnels and large rail tunnels has gained increasing use around the world, including HMM's Storm Water and Road Tunnel (SMART) in Kuala Lumpur (see Figure 6). The use of a TBM could result in reduced environmental disruption, enhanced worker safety, superior groundwater control (see next section) and narrower pillar widths between the bores. Whether this method would present cost and/or schedule savings would be borne out by the evaluations.

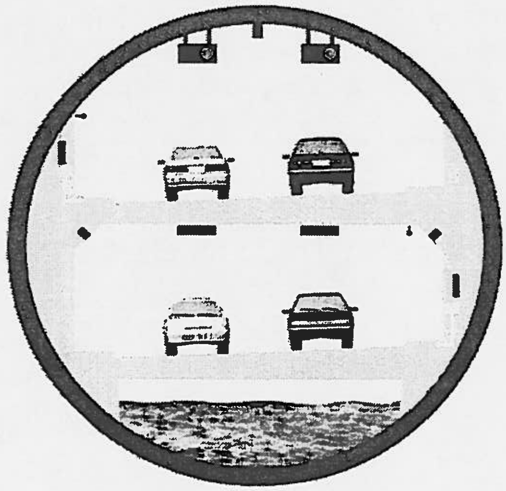


Figure 6 – SMART Tunnel – 44-ft diameter TBM Tunnel

6.2 Groundwater Control

Whether or not there is a regional or perched groundwater table at the site needs to be confirmed. The extent of interconnection between karsts and other voids in the limestones will be a factor in evaluating groundwater control concerns and potential impacts during construction and long-term operation. The design will consider groundwater drainage as a separate system from the roadway drainage. There will be a requirement to treat all roadway runoff to acceptable standards prior to being discharged to a local waterway. In contrast, if a groundwater drainage system is selected for the tunnels, the intent is to collect and divert all groundwater inflows for discharge directly into the closest waterway. Roadway and groundwater drainage systems will be developed that meet all of the discharge issues and requirements.

6.3 Risk Management

A number of measures will be considered to help manage the risks associated with an underground construction project the scale of the Drumanard Tunnel. An exploratory tunnel will be considered as a means of increasing the amount and quality of subsurface information prior to bidding the main tunnel construction. This information will be included in the construction contract as a Geotechnical Data Report. In addition, an interpretive Geotechnical Baseline Report (GBR) will be incorporated into the construction contract. A GBR describes the anticipated ground conditions to be encountered and discusses the adverse behaviors and construction risks attendant to those subsurface conditions. The main focus of the GBR will be to provide a common understanding of the work to be considered by all prospective bidders. By having all bidders "on the same page", more responsive and cost competitive bids are likely to be received. By knowing the risks to be carried by the contractor and owner, bids will be more responsive, and misunderstandings and resulting finger pointing between the parties will be reduced.

The GBR is a fundamental risk-sharing tool of the contract. Other contract provisions, including Prequalification, Disputes Review Board, and Escrow Bid Documentation, will also be proposed. Incorporation of these provisions will help focus the parties' attention on the construction rather than claims engineering, and if there are disputes, will provide an expedient off-line means of resolving them during the project.

7.0 ACKNOWLEDGEMENTS

The authors would like to thank the representatives of the Kentucky Transportation Cabinet and our team partner, HW Lochner, Inc., for the opportunity to present this paper at such an early stage in the project.

Modeling Uplift Pressures and Drain Flow
and
Rock Anchor Field Study

Greg Yankey[1], Rick Deschamps[2], and David J. Bentler[3]

Introduction

Bluestone Dam is located on the New River in the Kanawha River Basin, at Hinton, West Virginia. The dam is a concrete gravity structure approximately 170 feet high and 2,000 feet long that began operation as a flood control structure in 1949. It is owned and operated by the Army Corps of Engineers Huntington District, and controls a watershed area of roughly 4,600 square miles.

The dam has a hydrologic deficiency under the revised project Probable Maximum Flood (PMF), which will overtop the structure by approximately 7 feet. Stability analyses of the structure and foundation have shown that there is danger of a deep-seated sliding failure occurring at pool levels approaching the top of the dam. Consequently, a significant improvement in the stability of the structure is required to bring it into compliance with federal guidelines. This will be accomplished by a two-phase Dam Safety Assurance (DSA) construction effort.

Phase I of the DSA project is currently underway, and includes the construction of a mass concrete thrust block at the toe of the non-overflow portion of the dam. The six penstocks will be extended through this thrust block and controlled by sacrificial bulkheads that can be opened in the event of a severe storm. Phase II of the project will include new and modified training walls, a gate closure for the adjacent state highway, a parapet wall to prevent overtopping, and numerous large rock anchors to apply stabilizing forces to the dam.

This paper describes numerical modeling and a field anchor study performed by Fuller, Mossbarger, Scott, and May Engineers Inc. (FMSM) for the U.S. Army Corps of Engineers, Huntington District to investigate issues relating to the DSA design and construction.

Numerical Modeling

A numerical model of the rock foundation and structure interaction at Bluestone Dam was developed using the distinct element method. The primary objective of the modeling effort was to estimate the uplift pressure distribution at the base of the dam beyond the range of historical pool elevations for the project. Two key components in the modeling efforts were the ability to account for the nonlinear uplift distributions below the dam that result from variable size joint openings in the foundation, and the development of an algorithm to model foundation drains. A comparison of modeling results with measured data illustrate the ability of the approach to provide a realistic representation of a complex system, including drain flow, and provide a basis for estimating uplift pressures at pool elevations beyond the range of historical data.

Foundation Conditions

The foundation beneath Bluestone Dam consists of orthoquartzite interbedded with shale. Multiple near-horizontal bedding surfaces lined with carbonaceous shale occur in the foundation.

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[1] Associate, Fuller, Mossbarger, Scott, and May Engineers, Lexington, KY
[2] Head of Engineering, Nicholson Construction Company, Cuddy, PA
[3] Senior Geotechnical Engineer, Fuller, Mossbarger, Scott, and May Engineers, Lexington, KY

These bedding surfaces are typically undulating and were partially filled with grout during the initial grouting (consolidation grouting) phase of construction. High pressures were used and very high grout takes were observed in many locations along the alignment. Post grouting rock cores contained grout in the bedding joints between the orthoquartzite and the carbonaceous shale. Given the very low tensile strength of these bedding planes and the high grout pressures used, it is believed that the grouting program led to fracturing along these bedding planes.

Historical Uplift Data

During construction of Bluestone Dam, several piezometers were installed along seven cross-sections below the base of the dam. After dam construction, the uplift cells have been read regularly and during several high pool events. Many of the cells showed little response to elevated pools immediately after installation, likely because they were installed in a zone that was isolated from the transient changes in water pressure below the dam. The number of functional cells has continued to decrease since installation, and currently very few cells are able to provide reliable uplift pressure measurements. Accordingly, the most accurate and complete uplift pressure measurements were made shortly after construction. An example of the available data depicting the distribution of uplift head below Monoliths 14 and 44 during the 1955, 1957, 1958 and 1960 storm events is illustrated in Figure 1. Monolith 14 is a non-overflow section, and Monolith 44 is a spillway section. The tailwater is higher for Monolith 44 than 14 because of the presence of a stilling basin downstream of the spillway sections of the dam.

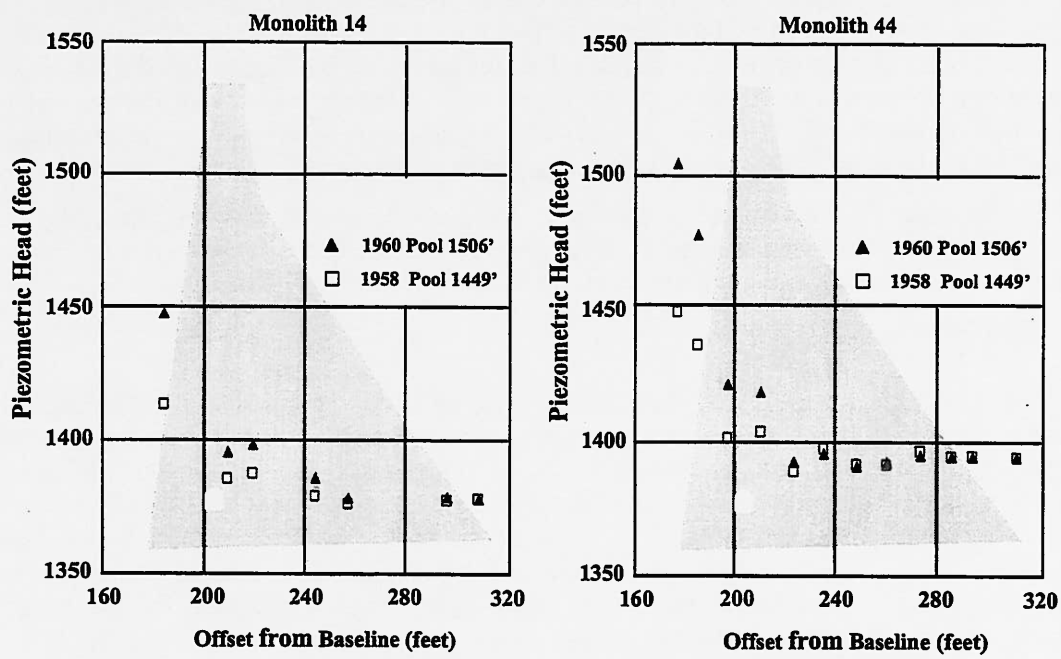
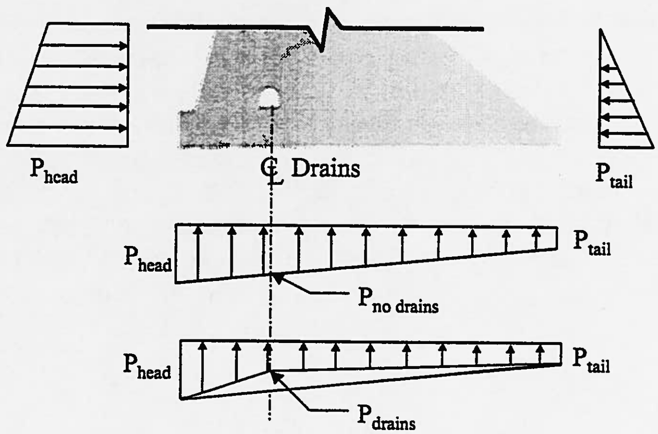


Figure 1. Measured Uplift at Monoliths 14 and 44

Current Design Methods and Potential Limitations

The current design approach used by the U.S. Army Corps of Engineers (EM 1110-2-2200) addresses situations with and without drains. When drains are not present, or are not considered functional, a linear distribution in uplift head is assumed below the dam. Using the base of the dam as the datum, the uplift head varies from the pool head below the heal of the dam to the tail head below the toe (See Figure 2).

Available data and judgment are used to select the design drain efficiency when drains are considered in the analysis. Drain efficiency is a measure of the reduction in uplift head resulting from the drains. The design approach assumes a bi-linear relationship for this case. The definition of drain efficiency is illustrated in Figure 2.



$$\text{Drain Efficiency} = \frac{P_{drains} - P_{tail}}{P_{no\ drains} - P_{tail}}$$

Figure 2. Current Design Assumptions and Definition of Drain Efficiency

Another consideration in selecting the design uplift head is the potential for non-compression to develop below the heal of the dam due to the tendency to tip at elevated pools. The conventional design approach assumes that the full pool head acts along the length of the non-compression zone followed by a linear reduction in head towards the toe for conditions without drains, or linear reduction to the line of drains for conditions with drains. If the non-compression zone extends to, or beyond, the line of drains, the drains are assumed to be overwhelmed by flow and no longer considered in the analysis.

The limitations of the conventional design approach relates to the fact that the uplift distribution below the dam is only linear if the headloss is constant per unit length. Using the theory of flow through parallel plates (Witherspoon et. al., 1980)) as an analogy to flow through rock joints, this requires that the open joints be of constant size across the foundation. A variable aperture size within the bedrock joints leads to highly nonlinear headloss and therefore uplift head. These concepts are discussed in detail by Barton et. al., 1985, and Pace and Ebeling, 1998). Since the stress field changes substantially below in the foundation as the pool water rises it can be expected that the sizes of the joint openings will also vary, thus leading to different distribution of headloss and uplift head at different pool levels. The amount the joint openings change with changes in the stress field depends on the normal "stiffness" of the rock joints (See Barton et. al., 1985; and Huang et. al., 1993).

The decision to use numerical modeling to gain insight into stability at Bluestone dam was made because of the significant uncertainties associated with the uplift head distribution in the jointed foundation, the poor understanding of the drain efficiency before and near failure, and the limited uplift head data available at elevated pools.

Numerical Model Selection

Prior to beginning the numerical modeling, a study was performed to evaluate which numerical method to use. The finite element method and the discrete element method were determined to be good candidates. After a thorough review of the strengths and weaknesses of each method and a survey of the commercially available programs, the distinct element program UDEC (Universal Distinct Element Code) was selected for the modeling effort.

UDEC was developed by Dr. Peter Cundall and coworkers in the 1970's and is now maintained and distributed by the Itasca Consulting Group, Inc.[4]. Several of UDEC's features make it well suited to this project. Some of the key features are: 1) it is a distinct element model that is suited to modeling discontinuous rock masses and large displacements along joints; 2) the distinct blocks can be deformable with specified failure criteria; 3) structural elements are available to simulate cables, beams, and grouted anchors; and 4) the program is capable of performing dynamic analyses. In addition, it is the only commercially available program that can simulate a fully coupled mechanical-hydraulic flow analysis for an intersecting joint system. Finally, UDEC includes an embedded programming language that allowed the development of an algorithm to model drain flow.

Methodology

Figure 3 illustrates the idealized geologic cross-section used in the numerical model. This section represents conditions near Monolith 12, a non-overflow section used in the stability calculations, and includes a toe block fracture and dipping lithology. The bedding planes rise approximately 1° downstream. Transverse joints were not considered in this analysis.

Drains were drilled from the gallery, are spaced every ten feet at Monolith 12 and are typically 45 feet deep, with some drains 85 feet deep. It was assumed that only every other bedding plane joint intersected the drains. Flow from the remaining bedding plane joints could still enter the drains by connecting flow through the vertical joints. The drains were modeled for a depth of 45 feet below the base of the dam. Measurements of foundation drain flow recorded shortly after the dam was constructed indicate that certain "active" drains flowed at about 2.5 gal/min during low pool levels. Using this information, two different cases where developed and analyzed for comparison purposes. The two cases produce approximately 2.5 gal/min at low pools. The term aperture is used synonymously with joint in the following discussion: Two additional terms are defined here for clarity of discussion, physical aperture, and hydraulic aperture. Physical apertures are the bedding plane joints that were used to discritize the bedrock system as illustrated in Figure 3. Hydraulic apertures are bedding plane joints in which water flows. For modeling purposes it was sometimes assumed that there were more hydraulic apertures than physical apertures because it was impractical to discretize the physical system into closely spaced open bedding planes. For Case 1 it was assumed that the hydraulic apertures were the physical apertures, while for Case 2 it was assumed that there were more hydraulic apertures than physical apertures with an hydraulic aperture spacing of five inches. For Cases 2, where the number of hydraulic apertures is greater than the physical apertures, the flow from the hydraulic apertures was lumped at the physical apertures before entering the drain. The same physical apertures are connected to the drains in both cases.

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[4] 708 South Third Street, Suite 310 Minneapolis, MN 55415 USA

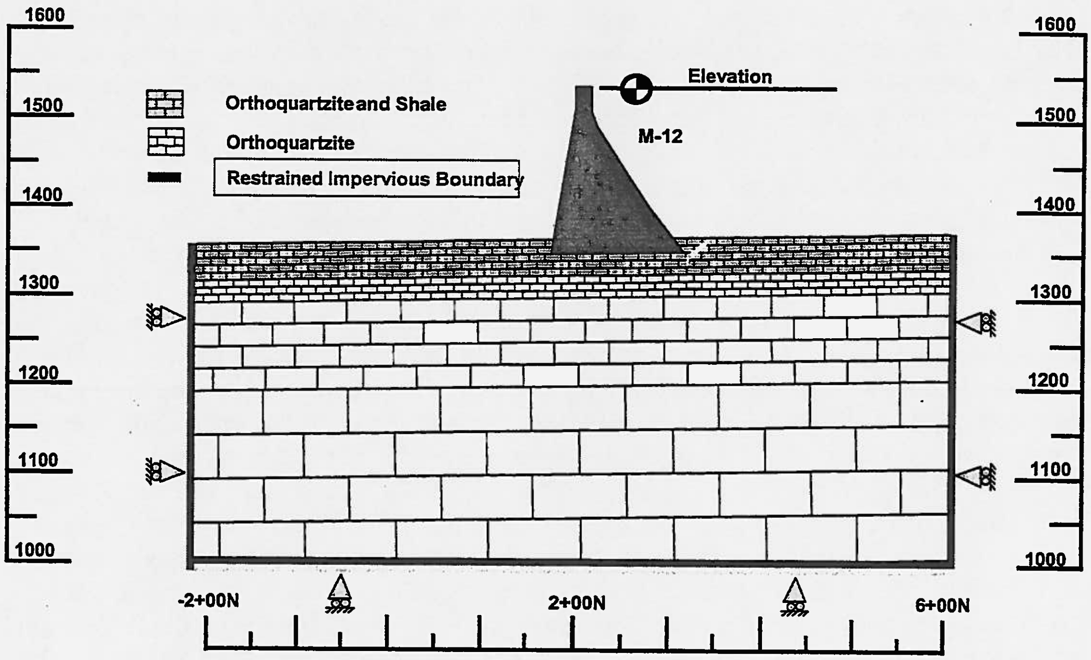

Figure 3. Foundation Model Utilized for Model Calibration

At the time of the modeling, there was insufficient subsurface information to select appropriate site-specific parameters. Parameters were selected based on the limited data for the site and values published in the literature (Goodman, 1974; Kulhawy, 1975; Rosso, 1976). A lower joint normal stiffness was used for Case 2 than Case 1 to illustrate how this parameter has an important influence on the uplift pressure distribution and quantity of flow that reaches the drains. The following illustration shows the case designations.

	Aperture Normal Stiffness	Number of Hydraulic Apertures
Case 1	Stiff (10,000 ksf/ft)	Same as Physical (Fig. 5)
Case 2	Less-Stiff (2,500 ksf/ft)	5-inch vertical spacing

Modeling the Drains

Development of a realistic model for the drains was a formidable challenge. Although there have been some analytical solutions that have been developed to model drain response [Amadei, 1989], to the writers' knowledge, there has not been a rational approach developed for modeling drains that has been incorporated into a numerical model.

In general, the two boundary conditions that are available in numerical models are a constant head boundary and a constant flow boundary. With the constant head boundary, the head at the boundary is fixed, and flow entering or exiting the system is dictated by the relative magnitude of the fixed head and the head in the adjacent element. The flow rate can vary over a large range depending on the head differential. With the constant flow boundary the flow entering or exiting the system is fixed, and the head at the boundary varies relative to the head of the adjacent cell.

Clearly, neither a constant head nor a constant flow boundary condition simulates drain flow because both the flow rate and the head vary at the drain boundary due to changing pool levels and head losses in the system. The head loss is related to the flow rate and the flow rate is related to the joint opening size, number of open joints and pool level.

An algorithm was developed and incorporated into UDEC to realistically model drain flow. The drain model was incorporated into the program by writing a subroutine using FISH, the embedded programming language within UDEC. The basis for the algorithm is fluid flow through a pipe in which there are two basic forms of head loss: entrance losses and friction losses. The entrance loss is related to the velocity of flow and an empirical factor that depends on the geometry of the entrance condition. The friction loss depends on the velocity of flow, the length of the flow path, and an empirical factor that is related to the roughness of the flow channel and the Reynolds number. Once estimates of the empirical parameters have been made, a relationship can be developed between the flow rate and head loss for a given length of flow. The boundary head can be adjusted as a function of the flow rate to achieve the appropriate head loss within the system using this relationship. An iterative algorithm is necessary, but works well within the explicit solution scheme used by UDEC. With the pipe flow model employed, the required variables are the entrance loss coefficient applicable for flow entering the drain from the open joint (assumed to be 0.5 – which is applicable to a sharp entrance and was used by Amadei [1989]). The friction loss coefficient that is a function of drain roughness and the Reynolds number (assumed to be 0.0284, based on a pipe roughness of 0.004 feet and flow velocities encountered), the diameter of the drain (0.25 ft), and the length of flow between apertures (depends on subsurface characterization). It should be noted that the boundary condition for the top of the drain is dependent on the gallery floor elevation (1,375 feet) and the tailwater elevation. The head on the top of the drain is assumed to be 1,375 feet when the tailwater elevation is less than 1,375 feet. When the tailwater is greater than 1,375 feet, the head acting on the top of the drain is assumed to be equal to the tailwater elevation.

In a two dimensional analysis, the applied boundary condition is inherently assumed to apply continuously along the longitudinal length of the model, in effect, the drain is modeled as a longitudinal slot. However, the drains have a spacing of approximately 10 feet at Monolith 12. Therefore, the flow rate generated by the model per foot of dam is multiplied by a factor of ten to account for this spacing because it is idealized that a single drain will intercept all flow within this 10-foot wide region. The implications are that the model cannot capture the rise in piezometric head along the line of drains between drains. Accordingly, uplift cells near the drain alignment, but offset laterally from the drain, are expected to have higher pressure readings than at the drain. The analytical work of Amadei [1989] could be used to approximate this condition in future analyses. It is believed that the approach developed to simulate the drains produces a very realistic response from low pools up to where the dam approaches failure. Figure 4 presents a schematic of the drain formulation algorithm. Flow from the open joints (q_i) enters the drain such that some or all of the physical apertures locations can represent hydraulic apertures. In addition, if a large number of hydraulic apertures are to be modeled, such that it is impractical to represent all apertures by physical apertures, several hydraulic apertures can be lumped at the physical apertures. The multiple arrows entering the single arrow illustrate this.

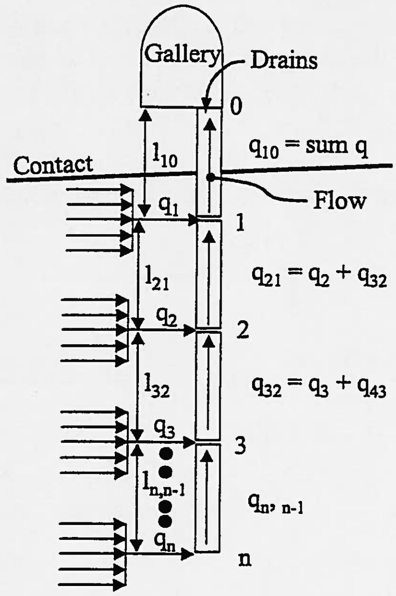

Figure 4. Schematic of Drain Formulation

Results

The uplift head distributions generated with drains from the numerical modeling of the three cases investigated are shown in Figure 5. The results for Case 1 illustrate that the drains do not become overwhelmed at elevated pool levels, while the drains for Case 2 are less effective at elevated pool levels.

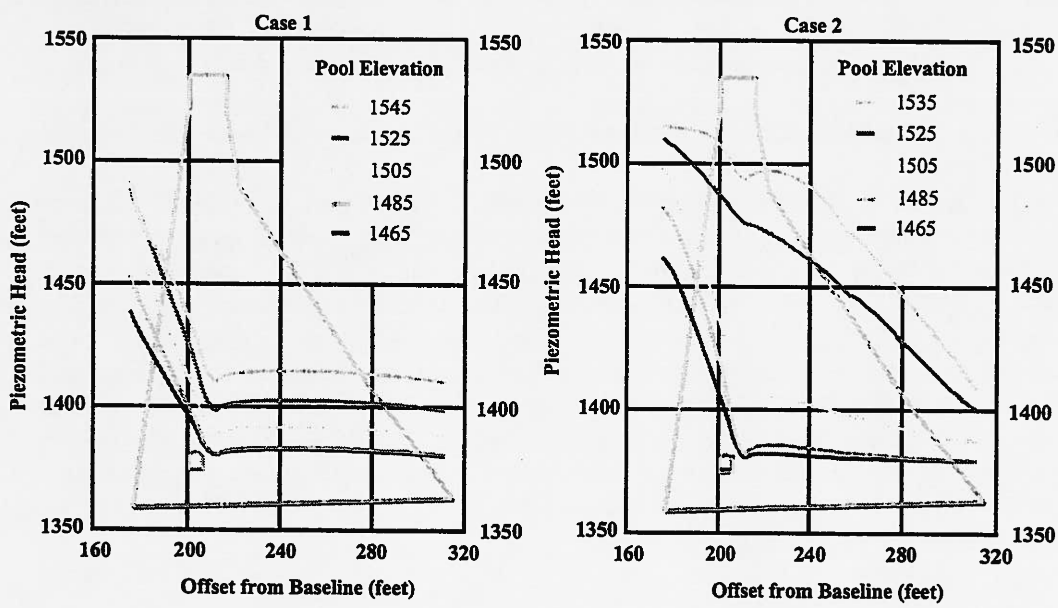

Figure 5. Result of Numerical Modeling with Drains for Various Pool Elevations

The uplift pressure results obtained from modeling Cases 1 and 2 were compared to the current design values (USACE, EM 1110-2-2200) at pool elevations of 1,445, 1515, and 1,535 feet. Conditions at a pool elevation of 1,445 feet are illustrated in Figure 6. The design distributions for no drains and 50 percent drain efficiency are compared to modeling results with and without drains. For Case 1, the design uplift without drains exceeds the model results due to head loss that occurs above the heel. The case with drains is well below the 50 percent drain efficiency design curve. For Case 3, the no-drain case is highly non-linear. This is primarily due to the lower joint normal stiffness used in this case permitting greater joint closure at the dam heel. Both the with, and without drain conditions, are below the design curve for a drain efficiency of 50 percent for Case 2.

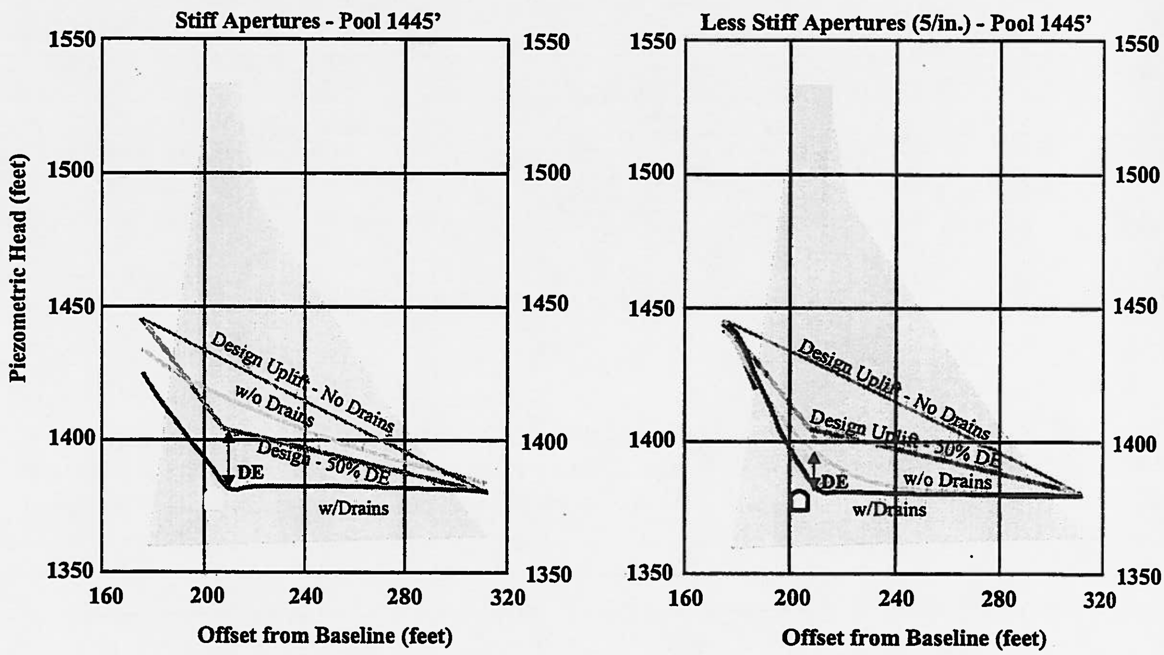

Figure 6. Cases 1 through 3 at a Pool Elevation of 1,445 feet

Cases 1 and 2 at a pool level of 1,515 feet are illustrated in Figure 7. The modeling results show higher uplift pressures for conditions without drains than the design uplift distribution due to the head loss that occurs beyond the toe of the dam, and therefore the uplift pressure is greater than current design standards. The head loss increases at higher pool levels because the lateral load on the dam from the rising pool tends to close vertical joints in the vicinity of the dam toe. As the normal stiffness of the joints decreases a large change in joint opening occurs as the pool rises. This tends to open joints below the heal and close joints below the toe. Accordingly, greater headloss occurs below the toe leading to a nonlinear uplift distribution that is more critical than the conventional design assumption. The uplift distribution for Case 1 with drains is lower than the design curve at 50 percent drain efficiency. Case 2 with drains is comparable to the design curve.

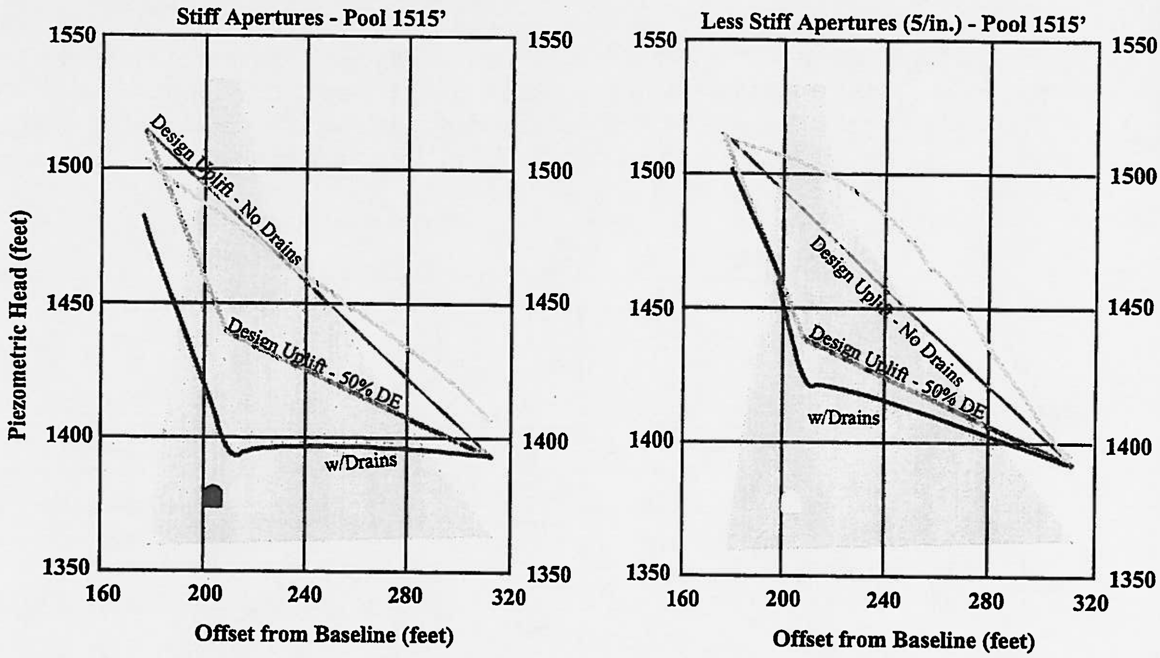

Figure 7. Cases 1 through 3 at Pool Elevation of 1,515 feet

The uplift distributions for Cases 1 and 2 with drains at a pool level of 1,535 feet are illustrated in Figure 8. Case 2 is actually beyond the stable condition and is "failing." Although Case 1 has an uplift distribution that is less than the design assumption at 50 percent drain efficiency, Case 2 has much greater uplift pressure. The likelihood that the drains may become overwhelmed by the quantity of flow increases with the number of open joints. However, there is an interrelationship with the joint normal stiffness.

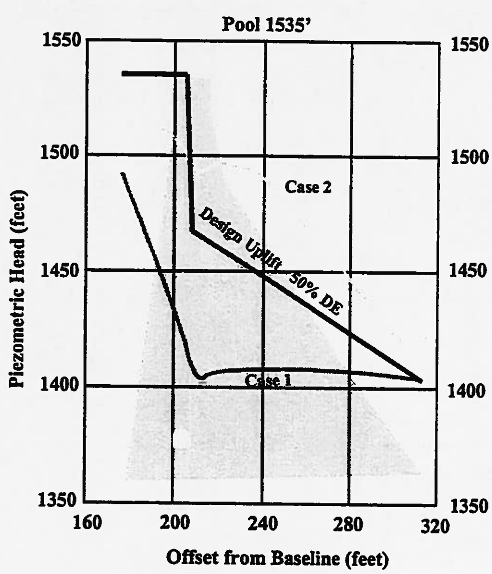

Figure 8. Cases 1 and 2 with Drains at Pool Elevation 1,535 feet

Historic uplift pressure data from Monoliths 14 and 44 are compared to model results in Figure 9 and 10. These figures illustrate the measured uplift pressure distribution across the dam and model results for the two cases. Readings from early historic pool elevations of 1449 and 1506 feet are compared to modeling results at 1445 and 1505 feet, respectively. A consistent pattern is noted between model results and measured data for Cases 1 and 2.

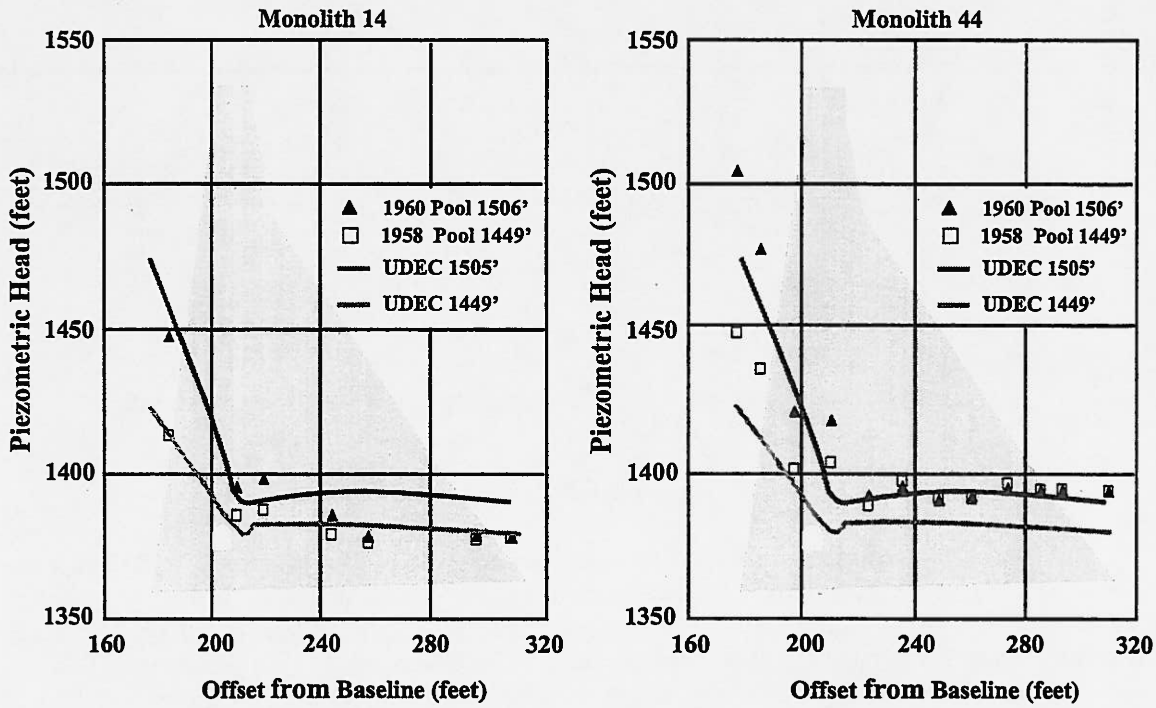

Figure 9. Case 1 Superimposed on Historical Uplift Pressures

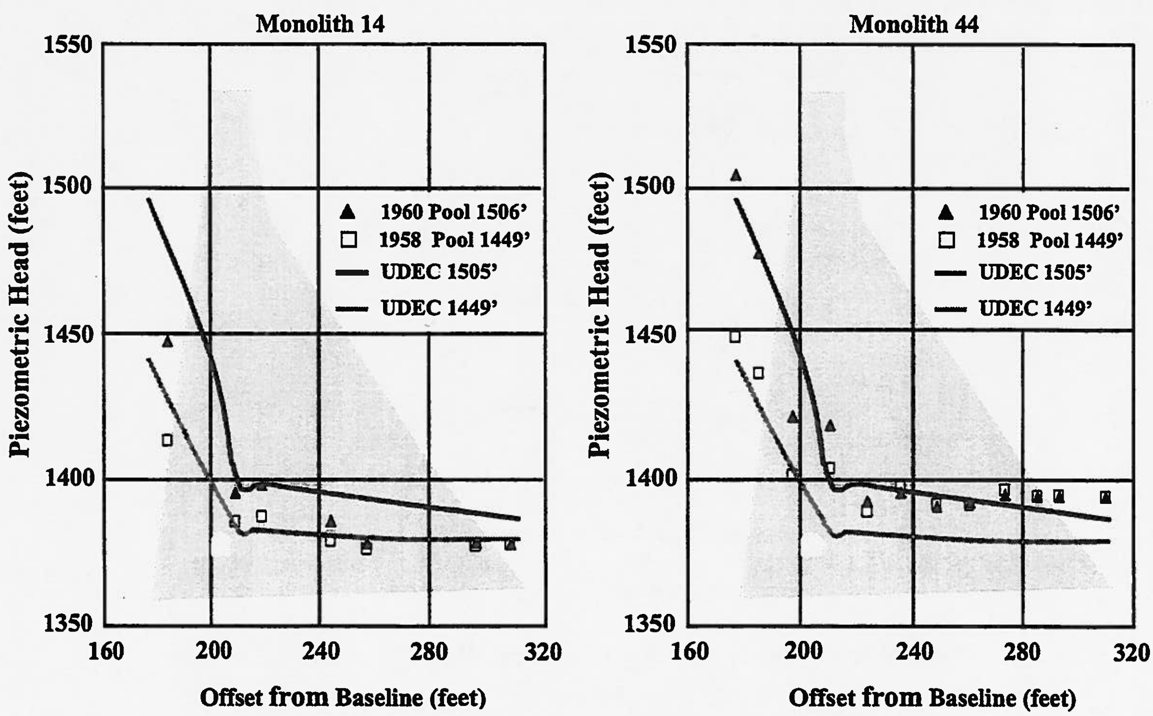

Figure 10. Case 2 Superimposed on Historical Uplift Pressures

Numerical Modeling Summary and Conclusions

The importance of numerous input parameters was identified during the modeling of uplift pressures and associated drain model development. A summary of the important concepts is as follows:

- Joint normal stiffness, joint spacing, and the total number of open joints have an important influence on the distribution of uplift pressure below the dam, and on the changes in magnitude and distribution of uplift pressure that occurs with rising pools.
- The current design uplift distribution (i.e. linear) used in conventional stability analyses when drains are not present may be inappropriate for some foundation conditions.
- The effectiveness of the drains has a very important influence on the uplift pressure distribution, and therefore, continued functioning of the drains is critical to stability.
- For the two cases presented in which drains were functional, the modeling results indicated the uplift pressure distribution can be much greater than, or much less than the design curve at a drain efficiency of 50 percent depending on the input parameters used.
- Consistent results were obtained between a preliminary interpretation of existing data at Monoliths 14 and 44 and the modeled Cases 1 & 2 in which flow rates at low pools were comparable to historic records. This is a positive indication that the numerical modeling can provide a reliable representation of dam response at pool elevations above the pool of record when a better representation of input parameters is obtained.

Finally, the modeling effort shows that foundation drains can be modeled in a realistic manner with the algorithm described in this paper.

Field Anchor Study

The Rock Anchor Field Study consisted of a bond stress test program and a test anchor program. The objectives of the bond stress test program were to compare rock to grout bond strengths from field anchors loaded to failure to bond strengths derived from laboratory anchor pull-out tests and verify design bond strengths. The objectives of the test anchor program were to identify construction issues, investigate anchor/rock load transfer, assess drilling techniques, and determine whether 61-strand anchors could be constructed successfully.

Bond Stress Test Program

Eight 18-strand rock anchors were installed in the four weakest of the six rock lithologies into which rock anchors will be installed during Phase II DSA construction. Laboratory anchor pull-out tests were performed on rock core from each bond stress anchor borehole. Unconfined compression and elastic modulus and Poisson's Ratio tests were performed on rock and anchor grout samples. FMSM cored and sampled the eight anchor boreholes with PQ-size rock core tools, reamed the holes to 5.125 inches in diameter, and performed water-tightness testing prior to anchor installation. FMSM subcontracted the installation of the anchor tendons and performance testing of the bond stress anchors to Nicholson Construction Company (NCC).

Test Anchor Program

Four 61-strand rock anchors were installed in Monolith 46 of the dam. The design of the four anchors was consistent with the Phase II design. The four anchors include two 235-foot long anchors installed from the crest of the monolith at an inclination of 8-degrees from vertical and two 175-foot long anchors installed from the downstream face of the monolith at an inclination of 45-degrees. Figure 11 shows the test anchor layout. The design length of the bond zone for all four anchors was 40 feet. Three anchors were installed using two-stage grouting of the tendon. The remaining anchor was installed using single-stage grouting. The two-stage method involves grouting the bond zone of the anchor and then stressing the anchor before grouting the free length, while the entire tendon is grouted prior to anchor stressing in the single-stage method.

The alignment of each anchor borehole was surveyed to assess the ability to avoid internal features of the dam such as pipes, sluices, and galleries using drilling techniques familiar to anchor contractors. The distribution of load in the bond zone of the anchor was studied with electronic and fiber-optic strain gage instrumentation. The effects of single and two-stage grouting procedures on the load distribution and load-deflection behavior of the anchor were studied. The distribution of the tendon load among the different strands was investigated by single-strand lift-off testing. Issues that could complicate the construction process were investigated, and the rate at which construction took place was tracked.

Nicholson Construction Company performed the installation of the test anchors under subcontract to FMSM, the Huntington District U.S. Army Corps of Engineer's contractor. FMSM installed and monitored the anchor instrumentation and interpreted all test program results.

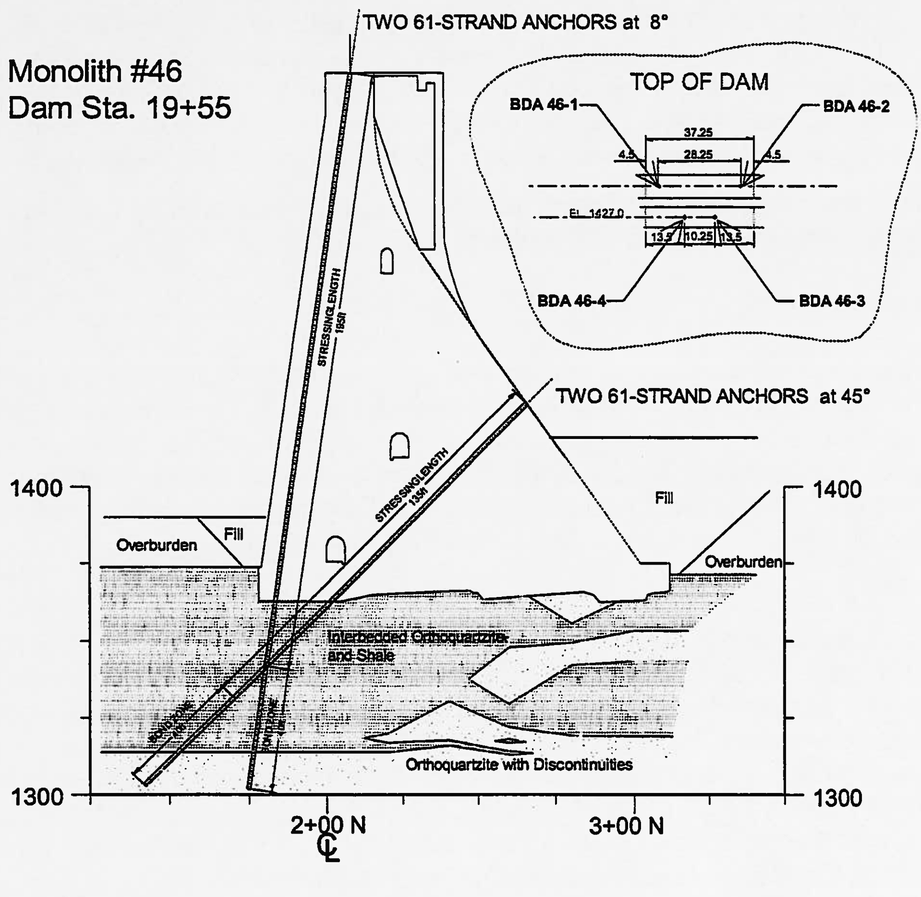

Figure 11. Anchor Geometry for Test Anchor Program.

Bond Stress Test Program

Design

The intent of the field bond stress tests was to determine the ultimate average bond stress at the grout-rock interface. This was a difficult goal to achieve in practice. The average bond stress for a given anchor load is maximized by minimizing the hole size, maximizing the number of strands in a given hole, and minimizing the bond zone length. The minimum hole size was constrained by the PQ-size coring required for laboratory tests. The final hole diameter was about five and an eighth inches. Conventional anchor practice recommends limiting the steel to grout area ratio to 15%, or 12 strands for a 5-inch diameter hole. Eighteen strands were used, which corresponds to an area ratio of 18%. Special spacers were fabricated to allow the installation of an 18-strand anchor into the five-inch diameter hole. Each anchor was fabricated using 0.6-inch diameter seven-wire low-relaxation steel PC (pre-stressed and post-tensioned concrete) strands with a guaranteed ultimate tensile strength (GUTS) of 58.6 kips per strand. The design bond zone length was 10 feet, which is the minimum length required to achieve adequate bond between the tendon and the grout based on industry experience. Anchor working load is normally 60% GUTS, and the maximum testing load is typically 80% GUTS. Therefore, the maximum anchor load was about 845 kips and the corresponding average bond stress was approximately 450 psi. This average stress value of 450 psi is about nine times the working average bond stresses used in preliminary Phase II design for the lithologies tested (see Table 1).

Installation

Eight borings were drilled, one for each of the eight bond stress tests. The project designation of the eight boreholes was C-02-52 through C-02-59. The anchors installed in the eight boreholes are designated A-52 through A-59. The borings were first drilled with a PQ wire-line core barrel that produced a 3.345-inch diameter sample and 4.827-inch diameter hole. Next, the holes were reamed to 5.125-inch using a roller bit, which also roughened the borehole surface.

After drilling, flushing, and water-tightness testing each borehole, an 18-strand anchor tendon was lowered to the target elevation and grouted into place. Neat cement grout was placed by tremie to form the bond zone. The as-built bond zone lengths differed from the 10-foot design length due to difficulty accurately computing and placing the required grout volume. The boreholes were left open above the bond zone.

Testing

Load was applied to the anchors by a center-hole hydraulic jack supported on a reaction beam. The elastic lengthening of the stressing length exceeded the jack's twelve-inch stroke length for some of the anchors. It was necessary to reset the jack (i.e. re-grip the strands) during the test on these anchors. The reaction beam was fitted with a lock-off head to maintain anchor load while the jack was reset. This set-up is shown in Figure 12.

Displacement was measured by a mechanical height gage reading relative movement between the reaction beam and the anchor head. A reference beam was used to measure change in elevation of the reaction beam. A Bourdon-tube gauge on the hydraulic line between the jack and the pump measured the jack pressure, and applied load was determined from a calibration between hydraulic line pressure and load. A Geokon 3000 electrical resistance load cell between the jack and anchor head was used to verify the magnitude of the applied load.

Each anchor was performance tested using a procedure that followed the Post Tensioning Institute's (PTI) recommendations for rock anchors[5]. The performance test begins by loading the anchor to a small alignment load, typically 10% GUTS. The alignment load removes slack from the strands. Then the anchor is repeatedly loaded and unloaded while monitoring load and total deflection. The peak load for each load cycle is progressively increased until the maximum test load, typically 80% GUTS, is reached. Permanent deflection for each peak load is determined by recording the total deflection at the alignment load before and after each load cycle. A creep test is performed at the maximum test load by holding the load constant for 10 to 60 minutes while monitoring deflection. One anchor was subjected to a PTI extended creep test, during which the peak load for each cycle is held for a period of time that increases with each cycle. The maximum hold time was five hours at 80% GUTS.

Laboratory anchor pull-out tests were performed on rock core samples retrieved from the coring of the bond stress test anchor holes. The tests were conducted following the procedure outlined by Lienhart and Stransky[4]. The test's purpose is to determine the bond strength between the grout and the rock. A 1.4-inch diameter hole is cored through the middle of a six-inch long portion of PQ-size rock core specimen. A one-half inch diameter threaded rod is grouted into the hole. Four nuts are threaded on to the rod at equal intervals to prevent failure along the grout-rod interface. All dimensions are measured and grout cubes are produced to determine the strength of the grout. After the grout has reached the desired strength, the rod is pulled until failure occurs (i.e. the grout column is pulled from the rock core). The average bond stress at failure is then computed.

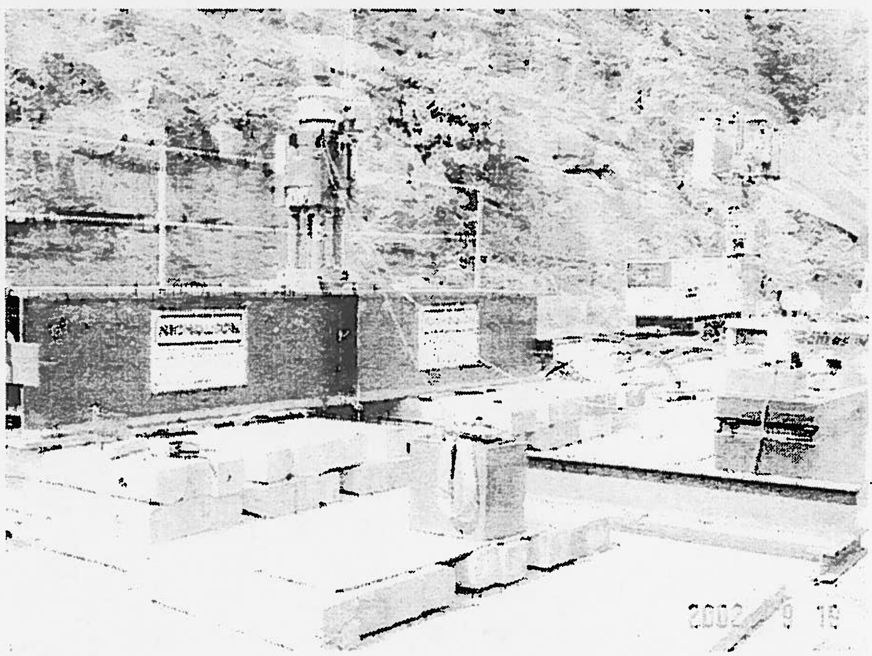

Figure 12. Load Testing Set-Up for Bond Stress Test Anchor.

Results

Table 1 summarizes the results of the bond stress test program. None of the field anchor tests reached the ultimate anchor capacity. However, all of the bond stress tests exceed the PTI criterion for acceptable creep deflection at the maximum test load. This may have occurred because the bond strengths of all bedrock lithologies are comparable, or because the tendon to grout bond controlled anchor performance rather than the grout to rock bond. The minimum bond length of 10 feet suggested by industry is based on anchors with steel to grout area ratios less than 15%. The area ratio in the eight bond stress anchors was 18%, so it is possible that there was insufficient bond length. Accordingly, it is not clear whether the creep failure was due to relative displacements at the grout to rock interface or the grout to steel interface. The maximum average bond stress values from each anchor test are well in excess of the design working average bond stress values, and typically slightly larger than the values from the lab pull-out tests.

Table 1. Results of Bond Stress Test Program

Material	Design Working Bond Stress (psi)	As-Built Bond Lengths (ft.)	Average Lab Bond Stress (psi)	Maximum Field Bond Stress (psi)
Maroon Claystone	50	10, 13, 16	393	370
Gray Siltstone	55	9, 12	294	433
Maroon Siltstone	45	11, 12	369	385
Carbonaceous Shale	50	10	341	452
Interbedded Orthoquartzite with Shale	130	–	–	–
Orthoquartzite	200	–	–	–

Note: All bond stress values presented in Table 1 are calculated average shear stresses over the entire bond length.

Lessons Learned

Measuring Loads with the Jack and the Load Cell

It is widely accepted in the anchor construction industry that a properly calibrated jack provides more consistent data than a load cell, because load cells are sensitive to environmental influences such as temperature, humidity, and boundary conditions. During interpretation of the bond stress load tests it became apparent that the load cell and jack did not provide consistent load measurements. It was later learned that a third party incorrectly calibrated the jack. Re-interpretation with the new jack calibration and the load cell data showed that several anchors were inadvertently loaded to almost 90% GUTS. This experience shows that it is prudent to have two independent methods of monitoring anchor load.

Maximum Achievable Bond Stress

The bond stress test data indicate that creep failure occurred at approximately the same load in all lithologies tested. This fact may indicate that failure was controlled by tendon to grout interface instead of the rock to grout interface. The steel to grout area ratio within the borehole was 18%, slightly larger than the maximum of 15% suggested by industry, in an attempt to maximize applied bond stresses. Following the 15% criterion would have limited the maximum average bond stress to 350 psi.

Field Bond Stress Tests

Single-strand lift-off tests should be conducted on all bond stress anchors following performance testing to identify strands that have debonded from the grout.

Test Anchor Program

Design

Figure 1 shows the location and geometry of the four test anchors installed in monolith 46. Two anchors were installed in the crest at an inclination of eight degrees. Two anchors were installed from the downstream face at an inclination of forty-five degrees.

The anchor head design included 38-inch diameter recess pits 36-inches deep. The pits are capped with manhole covers. Each anchor head is seated over the 13-inch center hole of a 34.75-inch diameter 5-inch thick A572-42 steel bearing plate. The bearing plate rests on a 36-inch diameter ¾-inch thick A36 steel sub-bearing plate seated on the recess pit base. The anchor design provided PTI Class I - encapsulation corrosion protection by encapsulating the tendons in 10-inch diameter corrugated high-density polyethylene (HDPE) pipe. The tendon strands are Polystrand™ seven-wire low-relaxation PC steel strands manufactured by Lang Tendons, Inc. The free length portion of Polystrand™ is sheathed with grease filled polyethylene tubing.

Installation

The installation of a rock anchor with PTI Class I - encapsulation corrosion protection on a concrete dam consists of five general steps. These steps are:

1) Drilling of the anchor borehole.
2) Water tightness testing and pre-grouting of the anchor borehole.
3) Installing the plastic encapsulation pipe.
4) Installing and stressing the anchor.
5) Completing the anchor head protection.

The test anchor boreholes were drilled using a Casagrande C-12A hydraulic crawler drill rig with a Sandvik® SD-12 down-the-hole hammer drill.

The desired alignment tolerance for the four test anchors was 1:150. This is a stringent criterion in light of conventional U.S. rock anchor practice. NCC went to great effort to meet the criteria using conventional anchor drilling equipment. The position of each pilot hole was precisely surveyed. Next, a guide sleeve centered over the surveyed hole location was fastened to the dam. The drill was positioned over the guide sleeve and surveyed using a theodolite and SmartLEVEL® to align the drill head and mast. An 8-foot deep, 18-inch diameter pilot hole was then drilled. Next, an eccentric down-the-hole hammer drill was used to drill the recess pits. NCC used a four-foot long steel trumpet, or guide pipe, to restrain the drill to a specific alignment during the initial stage of drilling the anchor borehole. The trumpet is attached to a sub-bearing plate that rests on the bottom of the recess pit. Set screws in the sub-bearing plate and trumpet wall allowed the trumpet to be positioned before being grouted into place with non-shrink grout. The trumpet has an outer diameter of 16 inches and an inside diameter of 15.25, or slightly larger than the 15-inch drill bit used for the anchor borehole. After grouting the sub-bearing plate and trumpet, a trumpet extension was bolted to the sub-bearing plate. The trumpet extension was removed after the drilling borehole. Two 10-foot long stabilizers were placed above the down-the-hole hammer during borehole drilling.

Baker-Hughes INTEQ surveyed the alignment of each anchor hole using the Seeker™ Surveying System. This system computes direction by sensing the rotation of the earth using gyroscope and accelerometer readings. It is more tolerant of magnetic interference and more accurate than typical magnetic or gyroscopic systems, and does not require a reference direction for calibration.

Figure 13 summarizes the results of the borehole surveys. None of the boreholes met the 1:150 criterion. The two principal causes of alignment error are set-up error and drift. Drift was pronounced in the 45-degree boreholes, which drifted down and to the right. Set-up error contributed about 50% to the total error for the 45-degree and 8-degree holes.

Each hole was water-tightness tested using a falling head test, and then pre-grouted and re-drilled if necessary. Following borehole completion, the encapsulation pipe was lowered into place. Next, the annular space between the pipe and borehole was grouted in stages. Successfully grouting the encapsulation pipe into place required careful consideration and monitoring of the differential pressure acting on the pipe wall in order to prevent collapsing the pipe. After the encapsulation pipe was installed, the anchor tendon was uncoiled into the pipe and then grouted into place. Each anchor was performance tested after the anchor grout had cured at least seven days.

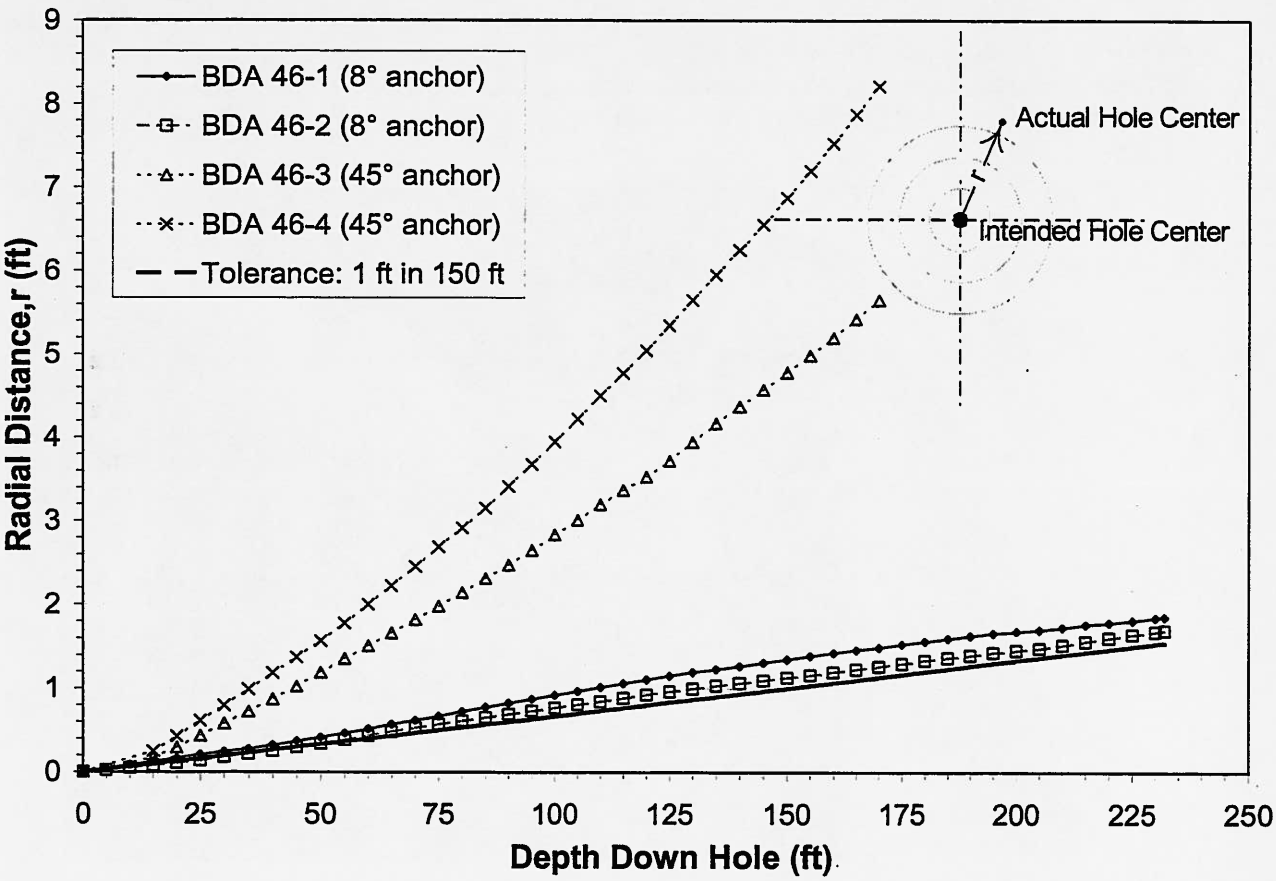

Figure 13. Borehole Survey Alignment Results.

Instrumentation

Instrumentation was installed on the two crest anchors to:

1) Determine the distribution of bond stress in single-stage and two-stage anchors, and

2) Monitor the axial load in several strands in the bond zone.

The fiber-optic strain gages, which are manufactured by FISO Technologies Inc., were used to monitor normal strain in the grout surrounding the encapsulation pipe. Fiber-optic instruments are completely immune to electro-magnetic and radio frequency interference. In addition, fiber-optic gages are stable (i.e. no drift tendencies); are easy to use; are relatively small; have high sensitivity and resolution of 0.01% of full scale; and are based on absolute measurements[2]. Another benefit of the fiber-optic instruments was the smaller profile of fiber-optic cable versus electrical cable. This was significant because the instrument cables were run through a limited number of 0.6-inch diameter holes in the anchor head. A multi-channel fiber optic signal conditioner was used to monitor the fiber-optic strain gages during testing. Fifteen fiber-optic strain gages were installed on the exterior of each anchor encapsulation pipe during field preparation. Figure 14 is a photograph showing a single gage mounted on the encapsulation pipe.

TENSMEG electrical strain gages, which are manufactured and sold by Roctest Inc., are the only commercially available instruments designed specifically for installation on seven-wire steel strand. Five TENSMEGs with a gage length of 48 centimeters, or 19 inches, were installed on two strands of each crest anchor during fabrication at Lang Tendon Inc. The five TENSMEGs were spaced over the top 15 feet of the bond length. Five TENSMEGs out of the total of ten were functional after installation. Some ceased functioning after the anchors were coiled at Lang Tendons, Inc.'s shop; while one ceased functioning during uncoiling and insertion into the anchor borehole.

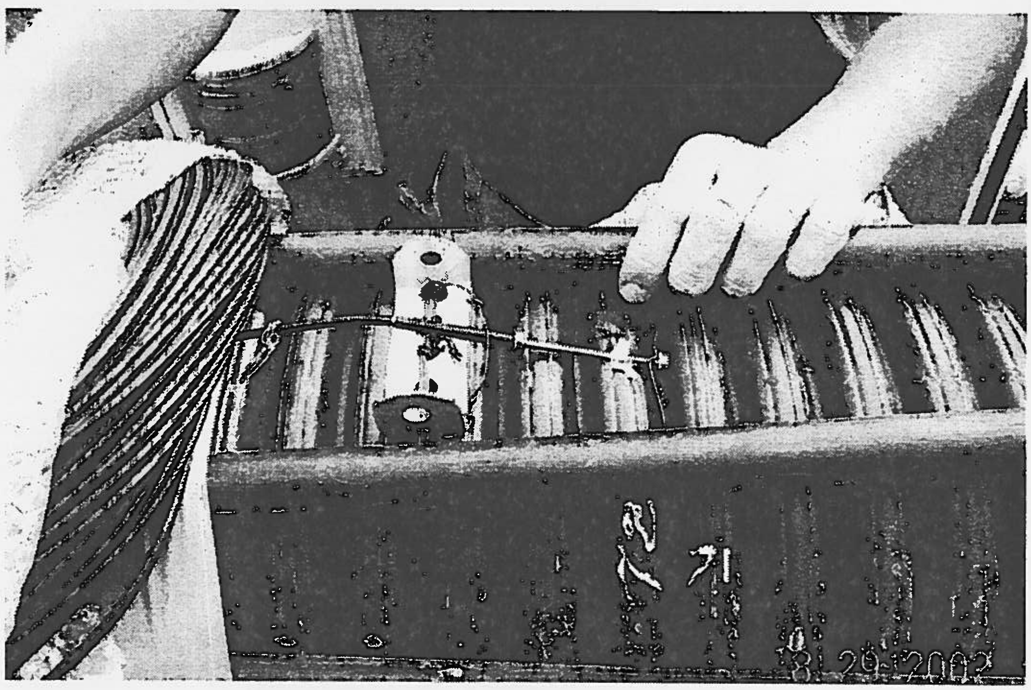

Figure 14. Fiber-Optic Strain Gages Mounted on Encapsulation Pipe.

Testing

Performance tests were conducted on all four anchors. The performance tests from the single-stage and two-stage anchors installed from the dam crest are discussed here. Figure 15 presents load-deflection data from both anchors. The theoretical elongation of each anchor's free length is shown as a dashed line on the graphs. Measured elastic deflections are shown as open boxes. The measured elastic deflections of the two-stage anchor lie very close to the theoretical free length elongation. The measured elastic deflections of the single-stage anchor are smaller than the theoretical free length elongation. This implies that there was load transfer in the free length of the single-stage anchor. The load-deflection curves for both anchors are very linear with the exception of the unloading portion from the single-stage anchor. This again was probably due to load transfer in the free length as it was noted during the test that the anchor would continue to recover deflection with time when unloaded. Figure 16 presents strain profiles from the fiber-optic gages for both anchors at the maximum test load (i.e. 133% GUTS). The strain from each gage is normalized by the average strain computed for the single-stage anchor over the instrumented length. The strain profile from the single-stage anchor is very similar to that predicted by the theory of elasticity[1,3]. The profile shows that almost all load transfer occurs over the top fifteen feet of the bond zone. The strain profile from the two-stage anchor shows that very little load was transferred in the top ten feet of the bond zone. Unfortunately, the anchor instrumentation only extended down to fifteen feet below the top of the bond zone. The lack of load transfer near the top of the bond zone may be due to poor grout quality in this region.

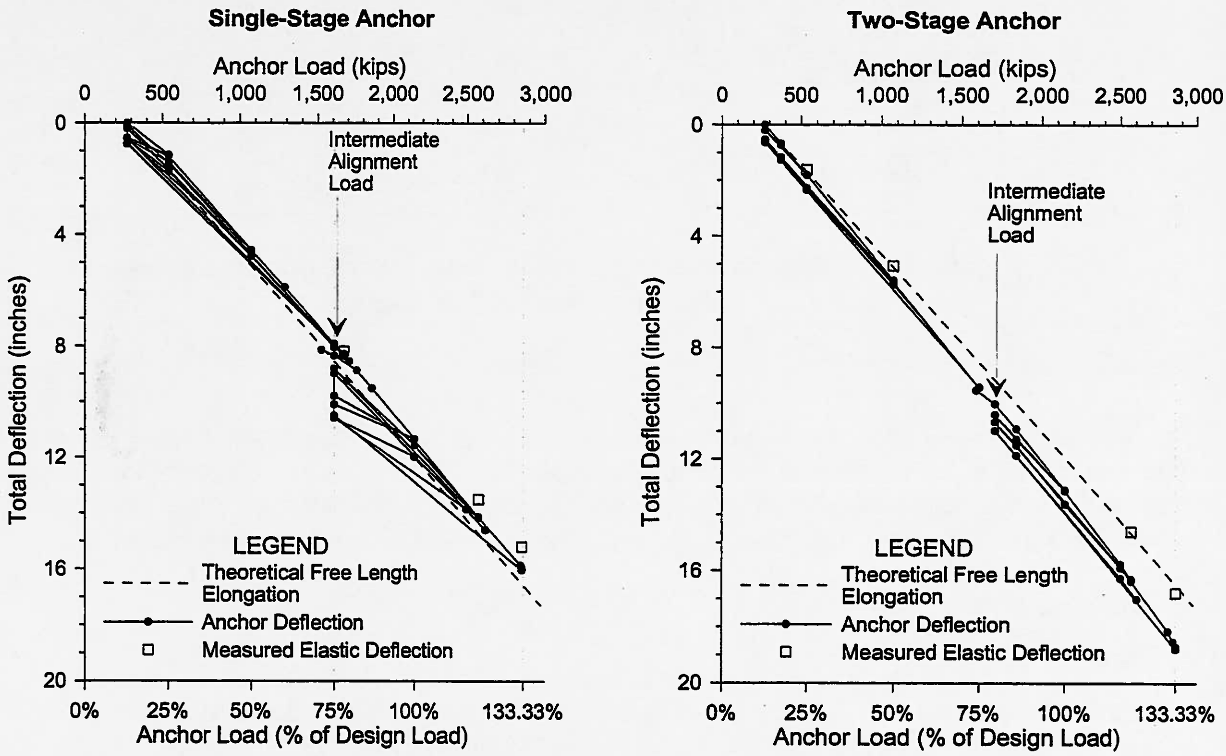

Figure 15. Load-Deflection Data from Performance Tests of Crest Anchors.

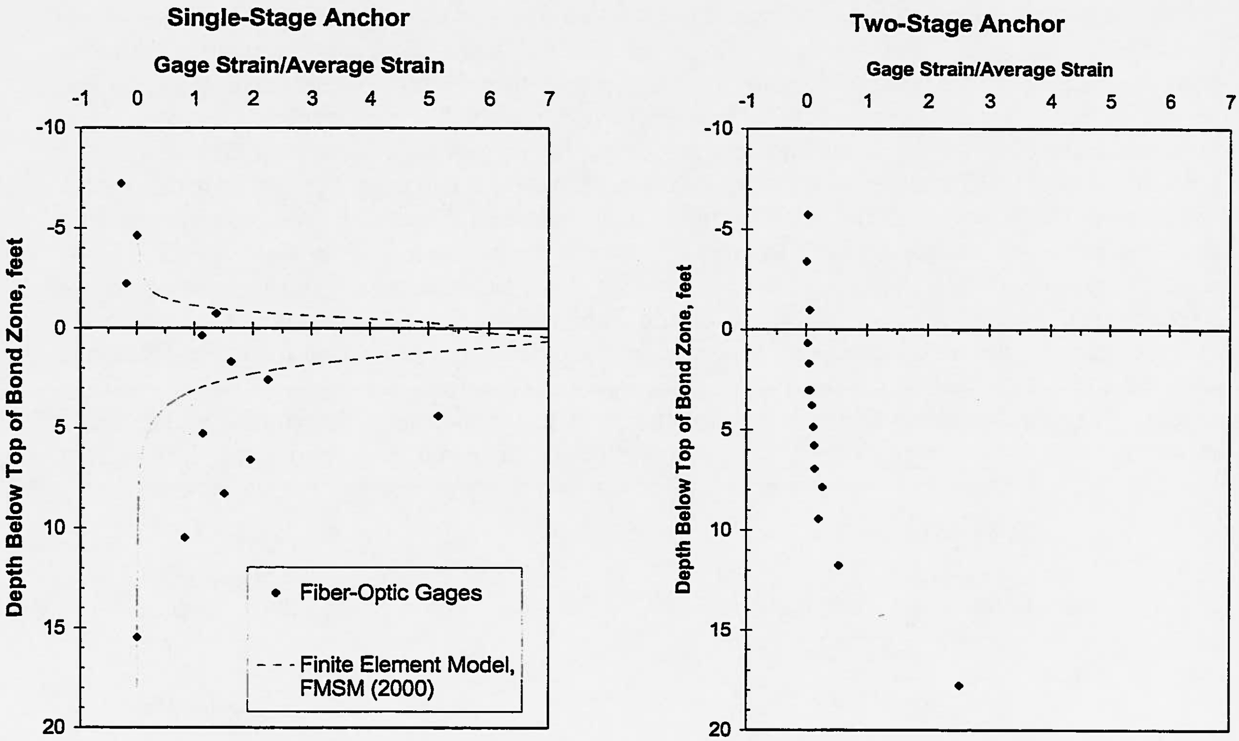

Figure 16. Normalized Strain Profile from Fiber Optic Strain Gages at Maximum Anchor Test Load.

Lessons Learned

The following sections provide a summary of some of the significant lessons learned during execution of the test anchor program. The lessons are drawn from aspects of the study that went well and unexpected problems that were encountered. Overall, the study was very successful due to the cooperation, teamwork, and diligence exhibited by everyone involved.

Anchor Design and Related Issues

Anchor quality, in the final constructed product, appears to be variable and caused by a number of factors that may not be apparent/predictable. Accordingly, bond zones should be longer than apparently necessary by a rigorous design to help accommodate field problems.

The Owner should require submittal of mix design by the Contractor for all grouted elements. These designs should include any admixtures that are proposed for use. A program and frequency for compression test breaks and related QC efforts should be stated in the specifications.

The anchor head design for the production work should consider the number of grout stages that will be employed in construction. Additional holes should be provided if possible to permit field correction of "lost" lift heights.

Drilling Capabilities and Tolerances

A detailed method for surveying the anchor positions and inclinations should be submitted by the contractor. Intermediate checks on the direction and inclination should be required if the anchor is in a location where these items are critical. The contractor should be required to submit methods to correct a borehole that has unacceptable drift. Based on the present study, hole alignments with stringent tolerances do not appear possible without significant cost and time. Therefore, it is prudent to avoid installing anchors in areas where there is little margin for error. It appears that with larger drill string less drift will occur on inclined holes. Furthermore, a better method for aligning the trumpet and sub-bearing plate needs to be developed. It is also believed that surveying was not sufficient to precisely align the drill equipment.

Assembly, Handling and Insertion of Tendons

Develop a plan for storage, handling and insertion of the tendons that provides an appropriate environment for the tendon to avoid corrosion prior to insertion and avoids nicks and cuts in the plastic sheathing. Submittals from the anchor installer should include a thorough and complete description of how the tendons will be manufactured, handled, packaged, loaded, transported, unloaded, stored, re-handled, and installed.

All steel rollers used to assist in tendon insertion should be covered with a material such as rubber that will help reduce the occurrence of nicks and cuts in the strand sheathing.

Anchor Corrosion Protection System Design and Installation

The likelihood of damage to the various components of the tendon corrosion protection system increases with the size of the tendon. The greased and sheathed portion of the strands on the exterior of the anchor tendon become increasingly vulnerable as the size of the tendon increases.

Rigorously design all corrosion protection elements, and consider the materials involved, crushing pressures, proposed construction methods, grout stages, lake/borehole water level conditions and manufacturers recommendations. The field engineer should keep a detailed record of the grout stages and borehole water levels and these should be monitored frequently during stage grouting. This should be one of the field engineer's primary responsibilities and stressed in the project specifications. Do not permit grouting of the encapsulation pipe unless specific tolerances are achieved on differential pressures.

Improper handling and installation can seriously compromise the effectiveness of corrosion protection elements. Therefore, the field installation of all corrosion protection elements should be subject to rigorous quality control.

Specify a minimum bending radius for the corrosion protection and stage grouting system that incorporates the manufacturer's recommendations. Strictly observe this minimum bending radius during installation of the system.

The realities of the probable success of corrugated corrosion protection pipe are significantly different than design idealizations and statements made in current PTI standards. Moreover, the installation of such systems becomes more difficult as the anchor length and diameter increase and as the inclination becomes closer to horizontal. Also, the time required to prepare and install these systems may make them a critical path item.

Stage Grouting of the Corrosion Protection

The stage grouting of the corrugated corrosion protection pipe produces a system of variable quality. In at least two to four feet of every lift, the grout is introduced into water. Moreover, final grout quality in the borehole is not adequately represented by cube or cylinder breaks on samples obtained at the grout plant. Eliminating stage grouting or minimizing the number of stages will produce a superior product.

Field Instrumentation and Data Acquisition

Interpretation of data from the 48-cm gage length TENSMEGs, used to measure strains in the individual strands was complicated because the resistance wires in the gages yielded at strand loads of about 20 kips, approximately 60% of the anchor working load. The strand load at which the gages yield can vary widely between gages due to variations in the manual pre-stress applied during installation. Although the electrical output of the gages remains proportional to total strain, the effect of yielding is to change the reference point of the gage (i.e. total strain is not equal to elastic strain of resistance wires). The uncertainty in the strand load indicated by each gage grows with each cycle of load during a performance test. However, the gages provide a useful indication of where load is being transferred in the anchor.

The gage factor of TENSMEGs is variable because each gage is manufactured by hand. The manufacturer provides a single typical gage factor to TENSMEG users. The gage factor should be determined for and supplied with each TENSMEG.

The fiber-optic gages worked very well and proved to be very robust. Data interpretation was qualitative due to uncertainty about individual gage orientation after installation.

Anchor Performance Testing and Interpretation

Load cells should always be an integral part of anchor performance testing and interpretation. Jack calibrations should be specified before and after the job, and at intervals during a project which lasts more than several months. The past five calibrations, if available, should be examined in conjunction with the current calibration. All jacks should be calibrated in series with their own load cell. The jack/load cell calibration should include cyclic loads (just as in performance testing) to help understand concepts such as jack friction.

Intermediate lock-off (i.e. re-gripping an anchor tendon) during anchor loading is undesirable. Post-tensioning jacks should have sufficient stroke to stress the tendon to 133% of the design load without re-gripping the wedges.

Two deflection measuring devices should be utilized during anchor proof and performance testing. These devices should conform to current PTI standards and have sufficient stroke to measure deflections without re-setting. The displacement and anchor load measurements should be compatible in terms of accuracy and precision.

It appears that there is some degradation of the bond zone accompanied with performance and extended creep tests. It is not believed that this degradation is sufficient to void the use of these test methods.

Single Stage and Two Stage Grouting Anchor Performance

Single stage grouting of long free lengths appears to cause significant load losses due to the sinuous nature of the tendon. These losses appear to be temporary (i.e. they are recovered with time).

Instrumentation on this project indicates that the load apparently goes deeper in two-stage than single-stage grouted anchors. Our hypothesis is that tensile cracking in the grout and/or poor quality grout near the top of the bond zone limit load transfer in this region. Therefore, the bond zone of two-stage grouted anchors should be over-grouted to reduce the portion of the bond zone lost due to this phenomenon.

In general, the instrumentation results support the bond shear stress distribution theories presented in the literature and numerical modeling previously conducted for this project. The distribution is non-uniform. Stresses are highest near the top of the bond zone and sharply decline with depth in the bond zone.

Conclusions

Although the ultimate capacity of the bond stress test anchors was not reached, the test results showed that the design working average bond stress values are sufficiently conservative. The installation, instrumentation, and testing of four 61-strand anchors in Monolith 46 of the dam provided information that will be very useful for the design and installation of rock anchors in Phase II of the DSA project. Successfully installing large and long rock anchors is a challenging undertaking that requires close communication and cooperation of the owner, designer, and installer.

Acknowledgements

We thank the United States Army Corps of Engineers, Huntington District for the opportunity to do this work. We also thank Nicholson Construction Company and their subcontractors who performed the anchor installation and testing. Roctest, Inc. provided technical support and installed the TENSMEGs.

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Geotechnical Conditions of the Pine Mountain Pilot Tunnel

Wayne Warburton, P.E.,P.G., *Golder Associates Inc. Atlanta, Georgia*
Richard Humphries, P.E., *Golder Associates Inc. Atlanta, Georgia*
Deana Sneyd, P.G., *Golder Associates Inc. Atlanta, Georgia*
W. Randall Sullivan, P.E., *Golder Associates Inc. Atlanta, Georgia*

Abstract

The Kentucky Transportation Cabinet plans to relocate US Highway 119 (US119) in a tunnel through Pine Mountain in Letcher County, Kentucky. The current alignment of US119 is over Pine Mountain. As part of the project, the Kentucky Transportation Cabinet plans to construct a Pilot Tunnel to evaluate the geologic and geotechnical conditions for the Main Tunnel. The geologic and geotechnical conditions and the design of the Pilot Tunnel are discussed in this paper.

Introduction

Pine Mountain is a dominant topographic feature in Southeastern Kentucky, rising more than 1000 feet above the valleys on either side. Currently, US Route 119 traverses Pine Mountain and is a principal arterial in extreme Southeastern Kentucky. It is on the National Highway System as designated by FHWA. The existing US 119 in the project area is a two lane road with narrow to non-existent shoulders. The US 119 Section between Partridge, Kentucky and Whitesburg, Kentucky is the last unimproved section in the State. Currently, spot improvements are being completed along US 119 to improve road conditions until the tunnel can be constructed.

Many alternative routes have been studied for the relocation of US 119, beginning as early as 1965. These alternatives have included upgrades to the current US119 over the mountain, various alignments with open cuts and tunnels through the mountain, as well as combinations of these. After a 30 year evaluation of alternatives, the Kentucky Transportation Cabinet (KYTC) decided to relocate US 119 in a tunnel through Pine Mountain. As part of the realignment project, KYTC intends to construct a pilot tunnel in the crown of the planned main tunnel to investigate geologic, geotechnical, and hydrogeologic conditions along the proposed tunnel alignment.

Project Description

The US119 Tunnel will extend through Pine Mountain in Letcher County, Kentucky at the location shown on Figure 1. The south portal is near the community of Oven Fork and the north portal is about two miles south of Whitesburg. The location of the proposed tunnel layout and alignment as shown on a USGS 7.5 minute quadrangle is included on Figure 2.

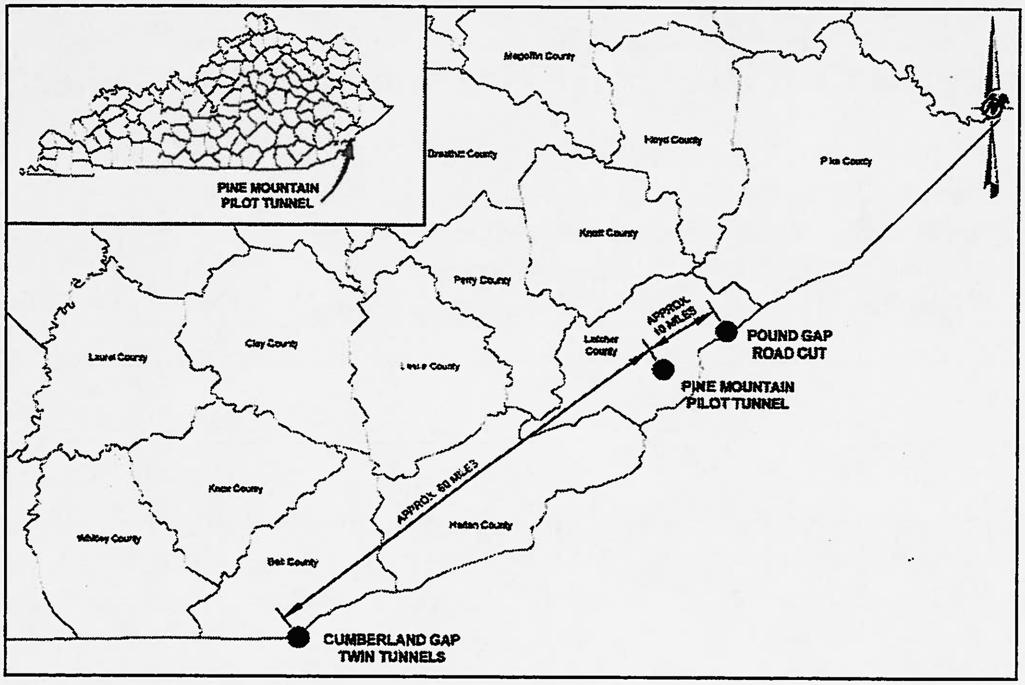

Figure 1 - Site Location

The proposed main tunnel is planned to be a single bore and will be bi-directional with one lane of traffic in each direction. The main tunnel will be approximately 10,400 feet long with dimensions of approximately 50 feet wide and 32 feet tall as shown on Figure 3. The tunnel slopes down to the north at a grade of 1.4% as shown on Figure 4. The tunnel interval is between approximately elevation 1656 feet above mean sea level (ft. MSL) at the south portal and 1538 ft. MSL at the north portal, with a maximum ground cover of approximately 1200 feet near the crest of Pine Mountain.

Collection of geologic data for design of the main tunnel using conventional drilling would be difficult for this project due to rugged terrain, depth of the tunnel and environmental concerns with regard to the Pine Mountain Wildlife Management Area. Consequently, KYTC has decided to construct a pilot tunnel in the crown of the proposed main tunnel to collect geologic and geotechnical data for the design of the main tunnel. The objective of the pilot tunnel is to reduce the total project cost by reducing uncertainties regarding the geologic and geotechnical conditions.. A pilot tunnel was used successful for the nearby Cumberland Gap Tunnels (Sullivan and Leary, 1987). The Cumberland Gap Tunnels extend through some of the same units as the proposed Pine Mountain Tunnel however the units dip to the northwest. These geologic units through Cumberland Mountain have been well explored with the pilot tunnel and the Twin Tunnels.

The lead designer of the Pilot Tunnel is Tetra Tech Inc., of Lexington, Kentucky. Golder Associates Inc. of Atlanta, Georgia is providing geotechnical design services, and Fuller, Mossbarger, Scott and May of Lexington, Kentucky provided geotechnical drilling, field testing, and laboratory testing services for the project.

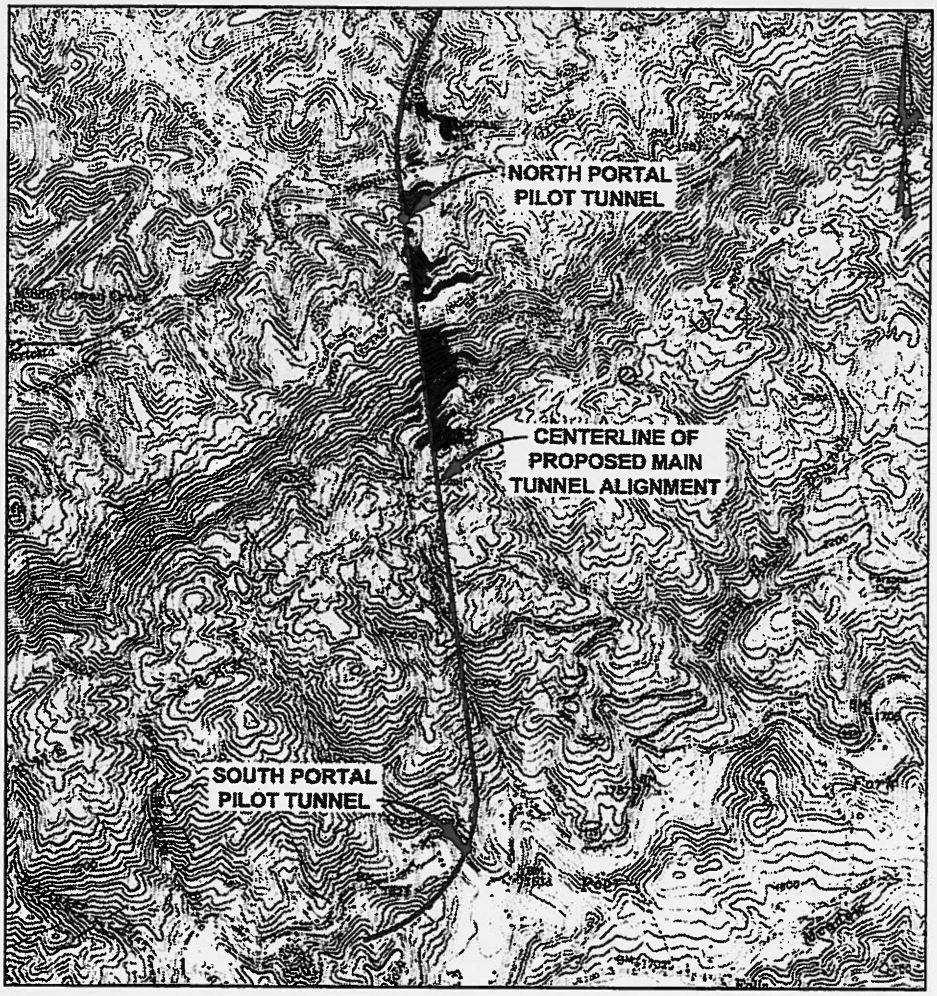

Figure 2 - Proposed Tunnel Alignment

Regional Geology

The Pilot Tunnel is located mainly within the Cumberland Overthrust Block in the Valley and Ridge Physiographic Province and in part within the Cumberland Plateau Physiographic Province of southeast Kentucky. The Cumberland Block is a mass of ground approximately 25 miles wide by 120 miles long that was pushed several miles to the northwest during the Appalachian Orogeny, around 250 million years ago (Wentworth, 1921). The carbonate, sandstone, siltstone and shale strata comprising this block range in age from the Upper Cambrian to the Pennsylvanian. The Cumberland Block is also known as the Pine Mountain overthrust block and is the westernmost major thrust sheet in the in the Southern Appalachian thrust belt in Virginia, Tennessee, and Kentucky (Mitra, 1988). The surface exposure of the Pine Mountain Thrust shows a minimum displacement of several thousand feet (Mitra, 1988). The block is divided into two major, northeast-trending structural features: 1) Middlesboro syncline and 2) Powell Valley anticline. Cumberland Mountain is located to the south of Pine Mountain and contains some of the same geologic units that will be encountered in the Pine Mountain Pilot Tunnel.

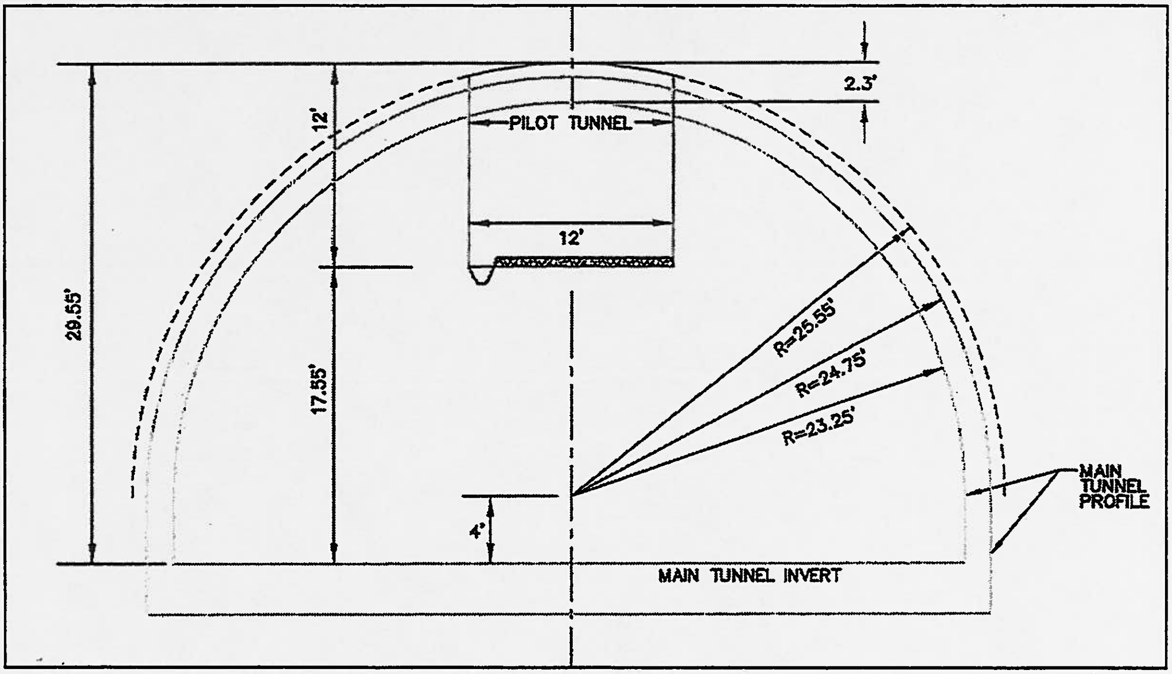

Figure 3 - Proposed Main Tunnel Layout and Pilot Tunnel

The proposed pilot tunnel transects the northwestern limb of the Middlesboro syncline which is manifested as Pine Mountain, the dominant topographic feature in the project area. The mountain is a hogback, the crest of which is maintained by a resistant sandstone within a sequence of dipping strata. The southeast flank of Pine Mountain is in part a dip slope and, in part, a series of lesser hogbacks reflecting the interbedded sandstones and shales of the Lee Formation. The bedrock dips moderately toward the southeast from about 20° to 40° on Pine Mountain in a continuous sequence of sediments of the Lee Formation on the southeastern slope. The northwest flank of Pine Mountain is a scarp slope comprised of Mississippian and Pennsylvanian-age carbonate and clastic rocks dipping from near horizontal up to about 40° near the Pine Mountain Thrust Fault. The Pine Mountain Thrust Fault represents the northwestern-most structure in the Valley and Ridge Physiographic Province. Northwest of the fault, flat-lying sedimentary rocks of the Cumberland Plateau Physiographic Province are present.

A zone of extremely folded and faulted sedimentary rocks occurs at the boundary between the Valley and Ridge and the Cumberland Plateau. This zone, referred to as the "Footwall Zone", occurs below the Pine Mountain Thrust Fault and is comprised of intensely folded and faulted rocks that are locally overturned. The Footwall zone is considered to transition gradually from intensely deformed rocks to flat-lying sedimentary rocks of the Breathitt Formation northwest of the Pine Mountain Thrust Fault. The thickness of this zone is not known in this project area, but is reported to be up to ¼-mile to ½-mile wide in the US 23 road cut (Chestnut et al, 1998) through Pound Gap shown on Figure 1. The same stratigraphic sequence expected to be encountered in the pilot tunnel through Pine Mountain is well exposed within the Pound Gap cut.

Geotechnical Exploration

The geotechnical exploration for the Pilot Tunnel consisted of geologic mapping, borehole drilling, and field and laboratory testing. The geotechnical exploration was limited for the Pilot Tunnel due to difficult access, and also because the purpose of the Pilot Tunnel is for exploration of geologic and geotechnical conditions along the proposed main tunnel alignment.

The geotechnical exploration was completed by Golder Associates Inc. (Golder) and Fuller, Mossbarger, Scott and May, Inc. (FMSM) of Lexington, Kentucky. Golder completed the geologic mapping, borehole logging and evaluation of the packer test data. FMSM provided site access, completed the drilling, packer testing, borehole logging, and laboratory testing. A discussion of the exploration program is included in the following sections.

Golder performed geologic mapping along the proposed tunnel alignment and portal areas to confirm previous interpretations and fill data gaps for the particular alignment that has been selected. The geologic mapping consisted of verifying the distribution of lithologies along the alignment, determining the nature and orientation of structural discontinuities, and evaluation of rock mass properties. As the subsurface explorations were limited, the geologic mapping formed the basis for most of the geologic and rock mass classification along the alignment.

Due to the limited exposure of the geologic units along the Pilot Tunnel alignment and limited subsurface data, information on the various geologic units was supplemented with descriptions of the same units as observed in the Pound Gap road cut along US 23 in Letcher County and mapped in the Cumberland Gap Pilot Tunnel. The locations of the Pound Gap cut on US 23 and the Cumberland Gap Tunnel relative to the Pine Mountain Pilot Tunnel are shown on Figure 1.

Five boreholes were completed for the Pilot Tunnel. The boring locations are shown on Figure 4. Boreholes C-1 and C-2 were drilled on the southeast flank of Pine Mountain to explore geologic and geotechnical conditions within the vicinity of the proposed south portal of the Pilot Tunnel. These borings were drilled to approximately 29 feet below the invert elevation of the Pilot Tunnel at this location. Boreholes C-4 and C-5 were drilled on the northwest flank of Pine Mountain within the vicinity of the proposed north portal of the Pilot Tunnel. These borings were drilled to approximately 33 feet below the invert elevation of the Pilot Tunnel at this location. Borehole C-3 was drilled near the projected location of the Pine Mountain Thrust Fault along the proposed tunnel alignment.

Geologic and Geotechnical Conditions Along the Alignment

Geologic Conditions

Based on the site mapping and regional geologic mapping, the rock units that will be encountered in the Pilot Tunnel include the Breathitt Group, Hance Formation, Lee Formation, Pennington Formation, Newman Limestone and Grainger Formation. Rock types vary from generally homogeneous sandstone, shale and limestone to interbedded sandstone, conglomerate, siltstone, shale, mudstone/claystone, and local coal beds. Portions of the Grainger Formation and Breathitt Group have experienced folding and faulting related to the Pine Mountain Thrust Fault.

The distribution of the geologic units anticipated in the Pilot Tunnel based on the exploration and geologic mapping completed to date is shown on Figure 4. The location and orientation of the geologic contacts and the thickness of the geologic units shown along the alignment were estimated from projection of geologic contacts mapped at ground surface in conjunction with up-dip projection of boring data, where appropriate.

Geotechnical Conditions

Important characteristics affecting design of the Pilot Tunnel include geologic structure, variation in rock type, groundwater conditions, presence of solution features or caverns, and the potential to encounter gas. The results of the borehole drilling, geologic mapping, and experience in the Cumberland Gap Pilot Tunnel were used to evaluate the anticipated geotechnical conditions along the Pilot Tunnel alignment.

Apart from the Pine Mountain Thrust Fault, bedding is the most evident and dominant geologic structure in the rock mass. Bedding surfaces may create discontinuities in the rock mass. However, bedding does not always create discontinuities in the rock and, in some cases, may be a slight color or texture change in an otherwise intact rock. Based on field measurements made during geologic mapping, bedding strikes about N66°E, which is variably about 55° to 75° to the tunnel alignment. Bedding generally dips to the southeast, ranging from near horizontal north of the Pine Mountain Thrust Fault, shown on Figure 4, and up to about 40° near the contact of the Pine Mountain Thrust Fault. The dip of bedding gradually decreases in magnitude from 40° near the Pine Mountain Thrust Fault to 19° near the south portal of the proposed Pilot Tunnel.

Jointing is also a significant structure in the rock mass. Based on the geologic mapping, several joint sets are well developed, but not all joint sets persist within a particular rock unit. Three major joint sets are present along the Pilot Tunnel alignment. The measured orientations of these joint sets generally range as follows:

- J_1: N65°E, 57°NW to N33°E, 82°NW (parallel to strike of bedding);
- J_2: N25°W, 87°NE to N45°W, 70°NE (parallel to dip direction); and
- J_3: N72°W, 85°SW.

The Pine Mountain Thrust Fault (PMTF) is the only major fault that is crossed by the tunnel alignment. The Pine Mountain Thrust Fault trends parallel to the strike and dip of bedding within the Grainger Formation along the northwest flank of Pine Mountain and is expected to cross the tunnel alignment south of the northern portal as shown on Figure 4. A majority of the movement of PMTF is expected to have occurred in the weak siltstone, shale and claystone beds of the Grainger Formation. The Fault Zone is underlain by a Footwall Zone that consists of carbonaceous shale and shaley siltstone beds of the Breathitt Group that have been complexly folded and faulted as described by Chestnut et al (1998). The expected location and approximate extent of the Fault Zone and Footwall Zone are shown on Figure 4.

Solution features and caverns are expected to be encountered in the Pilot Tunnel in the Newman Formation. Solution features and caverns result from solution of the carbonate rocks over periods of millions of years. Typically, the features develop along bedding or joint surfaces. Solution features and caverns are common in the Newman Limestone all along Pine Mountain (Chestnut et al, 1998). These features can range from solution-enlarged joints a few inches in width up to large caverns greater than the diameter of the Pilot Tunnel. In the Pound Gap road cut and in the Cumberland Gap Tunnels solutioning was observed in the Lower Newman at several locations along bedding and joint planes.

Anticipated Tunnel Ground Conditions

The general rock mass behavior in the Pilot Tunnel is expected to be controlled by the following mechanisms:

- *Loosening and block instability* - blocks or wedges of rock bounded by structural discontinuities (bedding planes, joints and shears) in the rock mass falling or sliding into the excavation; raveling is a form of progressive loosening where small rock particles fall into the excavation. Loosening and potential block instability is expected in all rock units. The amount of loosening that occurs depends upon the bedding and joint spacing, weathering, and water inflows, and timing of support installation. Figure 5 shows a schematic representation of potential block instability.
- *Slaking* is volume change and deterioration due to physical environmental changes, primarily moisture and temperature. Slaking is expected to occur in the shale, mudstone, claystone units.
- *Squeezing* - overstress of the rock mass due to stress concentrations around the tunnel. Squeezing is expected in relatively weak rocks such as the shale, claystone, mudstone units and in the Fault and Footwall Zones

Ground Support Design

The ground support design for the Pilot Tunnel has been developed to address potential tunnel instability resulting from block instability, slaking, and squeezing. In addition, the Pilot Tunnel will effectively be a crown drift for the proposed main tunnel, and the rock support installed for the Pilot Tunnel will assist in supporting the excavation of the main tunnel. However, additional rock support will be required during construction of the main tunnel as the support installed for the Pilot Tunnel will generally not be sufficient for permanent support of the main tunnel.

The ground support for the Pilot Tunnel will provide excavation stability and safety during the work, and will vary along the tunnel, depending upon the ground conditions encountered. The main ground support elements will consist of cement grouted rock dowels, tensioned and untensioned rockbolts, fiber reinforced shotcrete, and lattice girders or steel sets. The rockbolts will be used to support blocks and wedges of rock. Shotcrete will be used to retain smaller blocks of rock between the rock bolts and to prevent raveling and slaking of the exposed rock mass. A combination of lattice girders (or steel sets) and fiber reinforced shotcrete, and spiling if necessary, will be used in poor ground where rockbolts cannot be anchored and in squeezing ground.

Ground Categories

The range in ground conditions expected in the Pilot Tunnel was divided into five categories to develop ground support designs. The ground categories and typical rock support for the ground categories are shown on Figure 6. The support designs were based on experience from the Cumberland Gap Pilot Tunnel, empirical design methods using the Q-system and RMR rock mass classification systems, and analytical methods using the computer programs $Phase^2$ (V5, Rocscience) and UnWedge(V2.34, Rocscience).

Tunnel Construction Considerations

The Pine Mountain Pilot Tunnel will be constructed to coincide with the crown of the main tunnel as shown on Figure 3 and will be approximately 12 feet wide by 12 feet high. The drill-and-blast excavation method will be required for the Pilot Tunnel due to the variability in ground conditions expected. Excavation of weak zones in Category IV and V ground and potential infilled solution cavities may require some manual excavation or use of a small mechanical excavator. It is anticipated that the Pilot Tunnel can be driven full face except for potentially in squeezing ground (Category V ground), however, due to short standup times, smaller headings may be required locally.

At this time, the Pilot Tunnel is planned to be driven from both the North and South Portal. The drive from the south portal will be downgrade and groundwater inflows will have to be pumped out of the tunnel. The drive from the North Portal will be upgrade and water will flow out of the tunnel by gravity. Thus, the potentially highest inflows, from solution features in the Upper Newman and Lower Newman Limestone, will drain by gravity to the North Portal.

Due to the limited subsurface data and potential for solution features and caverns, the Contractor will be required to maintain a probe hole for a distance of at least 35 feet ahead of the tunnel face throughout the length of the tunnel. In the Upper and Lower Newman Units, four probe holes will be kept at least 35 feet ahead of the face to help evaluate the presence of solution features, and particularly water-filled solution features.

Pre-excavation grouting may be required depending on the amount of water encountered in the probe holes and also on field observations regarding ground conditions. The primary purpose of this grouting is to control groundwater inflows that may threaten tunnel stability.

Groundwater Inflows

Groundwater inflows into tunnels are difficult to estimate accurately due to the variability in geologic conditions along the tunnel alignment. Typically, the rock mass permeability is the most significant factor in estimating groundwater inflows. For the Pine Mountain Pilot Tunnel, there is little site-specific data to estimate groundwater flow because the tunnel is planned to be an exploration tool for the main tunnel. Consequently, estimates of rock mass permeability and groundwater inflow into the Pine Mountain were based on measured inflows into Cumberland Gap Pilot Tunnel. As noted previously, the Cumberland Gap Pilot Tunnel was excavated through some of the same units that will be encountered in the Pine Mountain Pilot Tunnel.

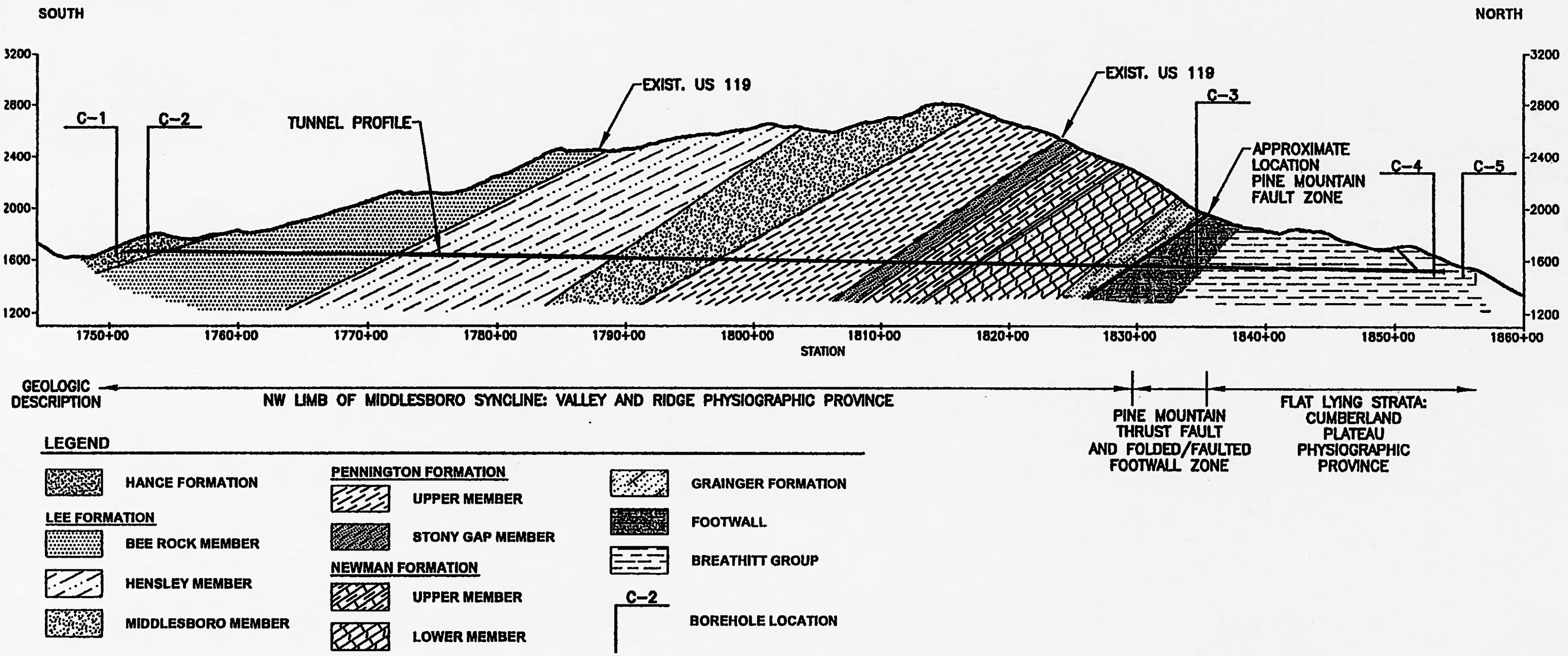
SOUTH
NORTH
3200
2800
2400
2000
1600
1200
ELEVATION (FT. MSL)
C-1
C-2
C-3
C-4
C-5
TUNNEL PROFILE
EXIST. US 119
EXIST. US 119
APPROXIMATE LOCATION PINE MOUNTAIN FAULT ZONE
1750+00
1760+00
1770+00
1780+00
1790+00
1800+00
1810+00
1820+00
1830+00
1840+00
1850+00
1860+00
STATION
GEOLOGIC DESCRIPTION
NW LIMB OF MIDDLESBORO SYNCLINE: VALLEY AND RIDGE PHYSIOGRAPHIC PROVINCE
PINE MOUNTAIN THRUST FAULT AND FOLDED/FAULTED FOOTWALL ZONE
FLAT LYING STRATA: CUMBERLAND PLATEAU PHYSIOGRAPHIC PROVINCE
LEGEND
HANCE FORMATION
LEE FORMATION
BEE ROCK MEMBER
HENSLEY MEMBER
MIDDLESBORO MEMBER
PENNINGTON FORMATION
UPPER MEMBER
STONY GAP MEMBER
NEWMAN FORMATION
UPPER MEMBER
LOWER MEMBER
GRAINGER FORMATION
FOOTWALL
BREATHITT GROUP
C-2
BOREHOLE LOCATION

Figure 4 - Proposed Main Tunnel Layout and Pilot Tunnel

Estimates of groundwater inflow into the Pine Mountain Pilot Tunnel were made by:

- scaling the inflow measured during the excavation of the Cumberland Gap Pilot Tunnel reflecting the similar geologic units, greater height of water over the Pine Mountain Pilot Tunnel, and the greater length of the Pine Mountain Pilot Tunnel; and
- following a method proposed by Heuer (1995) using estimated rock mass permeabilities based on the Cumberland Gap Pilot Tunnel inflows.

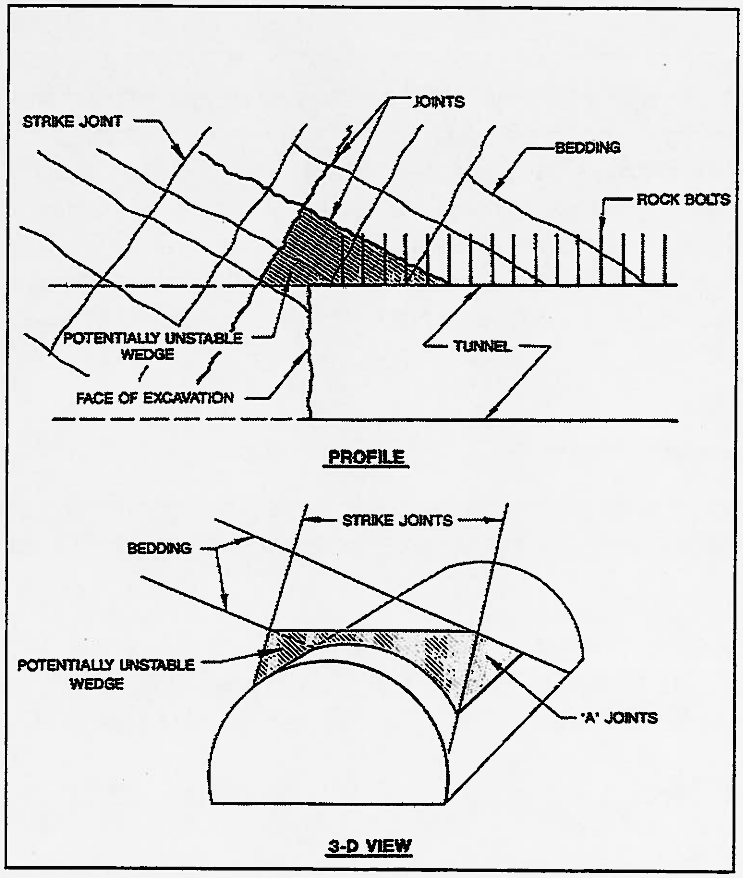

Figure 5 – Example of Potential Wedge Instability

Instrumentation and Evaluation during Construction

The purpose of the Pilot Tunnel is to provide geologic and geotechnical data for design of the main tunnel. Consequently, instrumentation and monitoring are an important component of the Pilot Tunnel work. A typical instrumented section of tunnel will consist of one piezometer to monitor hydrostatic head in the rock mass near the tunnel, 3-multiple-point rod extensometers to monitor deformation of the tunnel crown and sidewalls, and a pressure cell to monitor stress between the shotcrete and excavated rock surface. In situ stress measurements may be made in selected areas within the tunnel, depending upon the ground conditions encountered during tunneling. Groundwater inflows into the tunnel during construction will be monitored at the portals. Following tunnel excavation, stainless steel weirs will be installed in the tunnel at selected locations to monitor the long term inflows into the tunnel.

Geologic and geotechnical mapping is an important component of the Pilot Tunnel project. Geologic and geotechnical mapping will be completed after each round and the Contractor will be required to provide time for the geologists to map the heading.

Conclusions

The Pine Mountain Pilot Tunnel will be used as an exploration tool for the main tunnel and is intended to provide data for design of the main tunnel as well as information for Contractors for bidding of the main tunnel. The objective of the Pilot Tunnel is to provide cost savings on the main tunnel construction by reducing uncertainty in the ground conditions to be encountered. The design of the Pine Mountain Pilot Tunnel is based on experience from the Cumberland Gap Pilot Tunnel which has some of the same geologic units that will be encountered in the Pine Mountain Pilot Tunnel.

Acknowledgements

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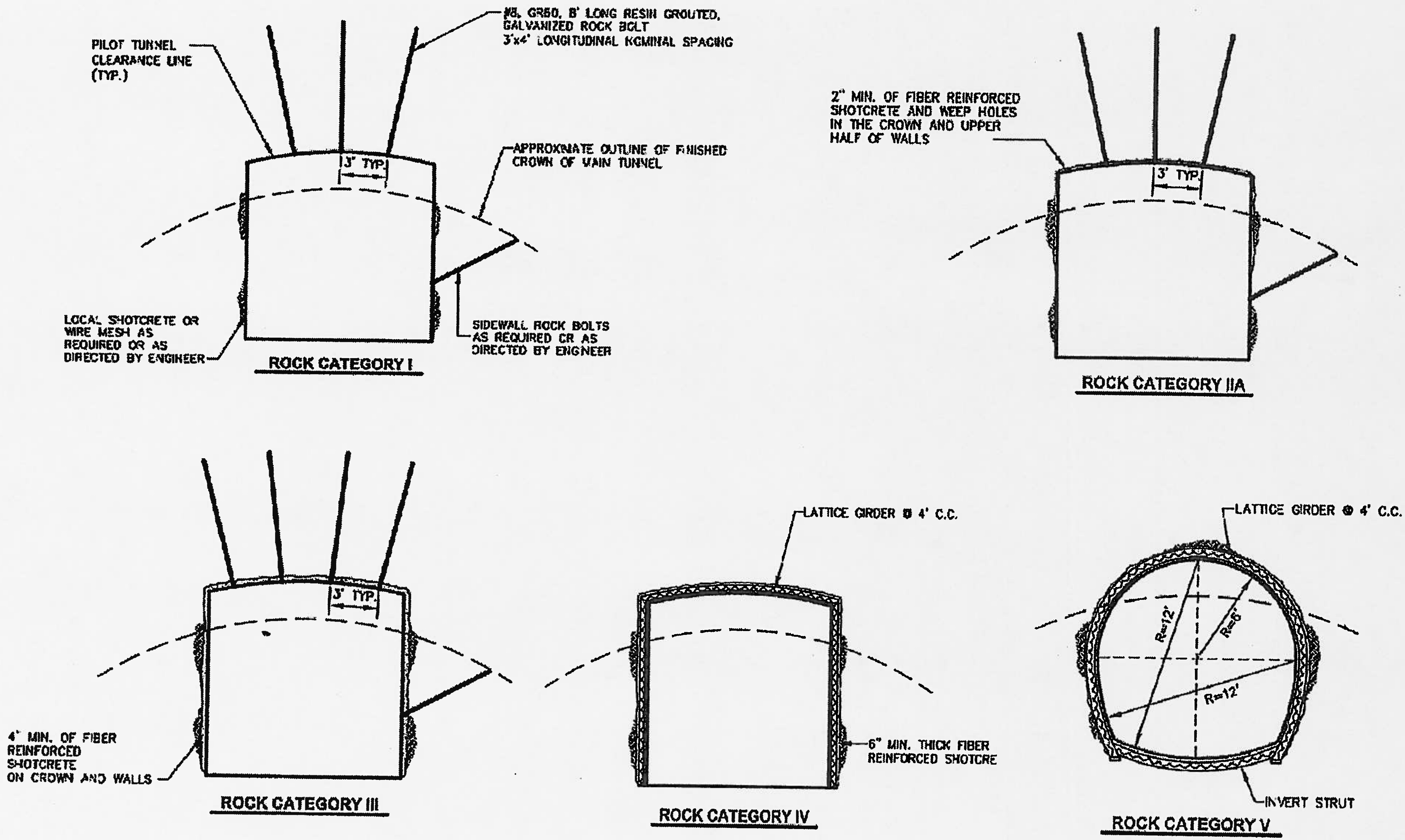

Figure 6 – Typical Rock Support for the Pilot Tunnel