OHIO RIVER VALLEY SOIL SEMINAR XXX

VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION

CINCINNATI, OHIO • OCTOBER 1, 1999

ORVSS XXX

VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION

OCTOBER 1, 1999

AGENDA

7:30 Registration

8:35 Welcome and Opening Remarks - Swaminathan Srinivasan, Chairman ORVSS XXX

8:50 Keynote Speaker - Dr. Jorj O. Osterberg, Northwestern University: ***"Value Engineering - A Great Concept Why Isn't It Used More Frequently?"***

9:40 G.T. Vandevelde - Atlanta Testing & Engineering: ***"Bootstrap Backfill - Actors Theatre Renovation, Louisville, Kentucky"***

10:15 Michael J. Cowell and Keith R. Moser - Geo Structures, Inc.: ***"Use of Rammed Aggregate Piers in Place of Deep Foundations for Settlement and Uplift Control of Buildings and Retaining Walls"***

10:50 Break

11:05 Dr. Mark T. Bowers - University Of Cincinnati: ***"Teaching Value Engineering in a Geotechnical Curriculum"***

11:40 Dr. Mark Meyers and Neil T. Schwanz - U.S. Army Corps of Engineers: ***"Mechanically Stabilized Earth Segmental Block Unit Retaining Wall in a Riverine Environment: A Value Engineering Success Case History"***

12:15 Lunch and Exhibitors Fair

1:20 Announcements

1:25 Keynote Speakers: Dr. F. McLean and C. Morrell: ***"Value Engineering, the Not So Secret Weapon to Improve Project Quality and Control Costs"***

2:15 Dr. W. Dotson - OGDEN Environmental and Energy Services: ***"Value Engineering of Liquefaction Mitigation"***

2:50 Wem-ping Chen - SVERDRUP Civil Inc.: ***"The Value Engineering Study of a Tunnel Segment of Detroit Metropolitan Wayne County Airport"***

3:25 Break

3:40 Frederick Slack, Douglas Keller, and Brent Grow - Richard Goettle, Inc.: ***"KY Route 9 - Licking Pike Widening Design/Build Slope Stabilization"***

4:25 Bruce G. Stegman -Rettew Geotechnical & Foundation Engineering and Dr. J. Darrin Holt - FDW, Inc.: ***"Determining the Capacity of Unknown Foundations Using Non Destructive Testing"***

5:00 Social Gathering and Exhibitors Fair

PROCEEDINGS

of the

THIRTIETH
OHIO RIVER VALLEY SOILS SEMINAR
(ORVSS XXX)

VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION

October 1, 1999
Sharonville Convention Center
Cincinnati, Ohio

Sponsored by:

Cincinnati Geotechnical Group, Cincinnati Section
American Society of Civil Engineers (ASCE)

Kentucky Geotechnical Group, ASCE

The Geo-Institute of ASCE

University of Cincinnati
Department of Civil and Environmental Engineering

University of Dayton
Department of Civil and Environmental Engineering
and Engineering Mechanics

University of Kentucky
Department of Civil and Environmental Engineering

University of Louisville
Department of Civil Engineering
and Center for Continuing Studies

ORVSS XXX

VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION

OCTOBER 1, 1999

TABLE OF CONTENTS

ORVSS XXX

VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION

OCTOBER 1, 1999

ORGANIZING COMMITTEE

Swaminathan Srinivasan, P.E., **Chairman**
H.C. Nutting Company

Anthony Amicon, P.E.
BBC&M Engineering, Inc.

Doug Batt, P.E.
Law Engineering and Environmental Services, Inc.

Andrew Bodocsi, Ph.D., P.E.
H.C. Nutting Company

Mark Bowers, Ph.D., P.E.
University of Cincinnati

Ron Ebelhar, P.E.
H.C. Nutting Company

Dusty Henricks, P.E.
Contech Construction Products

Joe Keiser, P.E.
Bickart Felton Associates, Inc.

Douglas Keller, P.E.
Richard Goettle, Inc.

John Kerr, P.E.
ATC Associates, Inc.

C.R. Lennertz, P.E.
H.C. Nutting Company

Chris Schewmaker, E.I.T.
H.C. Nutting Company

Ted Vogelpohl, P.E.
G.J. Thelen & Associates, Inc.

George Webb, P.E.
H.C. Nutting Company

ORVSS XXX

VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION

OCTOBER 1, 1999

ABSTRACT REVIEW COMMITTEE

Andrew Bodocsi, Ph.D., P.E.
H.C. Nutting Company

Mark Bowers, Ph.D., P.E.
University of Cincinnati

Ron Ebelhar, P.E.
H.C. Nutting Company

John Kerr, P.E.
ATC Associates, Inc.

C.R. Lennertz, P.E.
H.C. Nutting Company

REMEMBERING THIRTY YEARS OF ORVSS

What great achievement by our ASCE geotechnical groups of Lexington, Louisville, Dayton and Cincinnati in reaching the 30th Ohio River Valley Soils Seminar! This represents thirty years of service not only to our fellow geotechnical engineers, but also to fellow civil engineers, architects and contractors in Ohio and Kentucky, and all neighboring states.

ORVSS is rich in history and we remembered how well its past was summarized by Aubrey May in the Silver Anniversary Edition of the ORVSS Proceedings in 1994. We thought that we reproduce it in full length for you as follows:

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. 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.

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 KSMFO 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!

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 U.S. Army Corps of Engineers, C. A. Witte of Vogt, Ivers, Seaman & Assoc., 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 were 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 30 proceedings contain a great wealth of information for the practicing engineer in this region of the country. For that reason, we republished the Bibliography of papers that was distributed at ORVSS XXV and added the Bibliography of papers from ORVSS XXVI through XXIX. 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!

As mentioned in Aubrey May's article above, the Kentucky Transportation Research Center Library on the University of Kentucky Campus has a complete set of Proceedings, and so does the H. C. Nutting Company Library courtesy of their engineers and Larry Rayburn of Richard Goettle, Inc. The Cincinnati Geotechnical Group, ASCE, is offering to reproduce and send any article requested from the 29

Proceedings to within the Continental U.S. Please forward your request and $3.00 per article, to cover copying costs and postage, to Andrew Bodocsi c/o H. C. Nutting Co. 4120 Airport Rd, Cincinnati , Ohio 45226. Make checks payable to the "Cincinnati Geotechnical Group, ASCE".

Table 1. Past ORVSS Locations, Dates, and Topics

ORVSS No.	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	Cincinnati, OH

BIBLIOGRAPHY OF PAPERS PUBLISHED IN THE PROCEEDINGS OF ORVSS' I-XXV
October 21, 1994

ORVSS I

Deen, R.C. (1970). "And why do we do it this way?" Ohio River Valley Soils Seminar No. 1, .

Fuller, D. (1970). "A case study - Pipe piling for a deep foundation problem." Ohio River Valley Soils Seminar No. 1, Lexingtion, Kentucky.

Hagerty, D.J. (1970). "Innovations in foundation design and construction." Ohio River Valley Soils Seminar No. 1, Lexington, Kentucky.

Lennertz, R. (1970). "Design, construction and performance of spread footings on a uniform foundation." Ohio River Valley Soils Seminar No. 1, Lexington, Kentucky.

May, A.D. (1970). "Building foundation failures related to soil shrinkage." Ohio River Valley Soils Seminar No. 1, Lexington, Kentucky.

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

Barr, D.J. (1971). "Preliminary site selection studies with remote sensing techniques." Ohio River Valley Soils Seminar No. 2, Louisville, Kentucky.

Drnevich, V.P. (1971). "Exploration methods." Ohio River Valley Soils Seminar No. 2, Louisville, Kentucky.

Goettle, R. (1971). "Contractor's Problems." Ohio River Valley Soils Seminar No. 2, Louisville, Kentucky.

Hagerty, D.J., and Pavoni, J.L. (1971). "Environmental considerations in soils engineering." Ohio River Valley Soils Seminar No. 2, Louisville, Kentucky.

Hensey, M.D. (1971). "Site evaluation with remote sensing techniques applied to suspect karst topography." Ohio River Valley Soils Seminar No. 2, Louisville, Kentucky.

McGraw, W., and Tupman, D. (1971). "The selection of strength parameters and their use in the stability analysis of earth rock embankments." Ohio River Valley Soils Seminar No. 2, Louisville, Kentucky.

ORVSS III

Cowherd, D.C. (1972). "Factors to be considered in the design of backfill for large diameter flexible metal conduits." Ohio River Valley Soils Seminar No. 3, Fort Mitchell, Kentucky.

Drnevich, V.P. (1972). "Generalized sliding wedge method for slope stability and earth pressures analysis." Ohio River Valley Soils Seminar No. 3, Fort Mitchell, Kentucky.

Goettle, R. et al. (1972). "Deep excavation and building supported by tie-back system." Ohio River Valley Soils Seminar No. 3, Fort Mitchell, Kentucky.

Hagerty, D.J. (1972). "Lateral pressures and movements caused by pile driving." Ohio River Valley Soils Seminar No. 3, Fort Mitchell, Kentucky.

Lacroix, Y., and Jackson, W.T. (1972). "Design and construction fo supported temporary excavations in urban environment." Ohio River Valley Soils Seminar No. 3, Fort Mitchell, Kentucky.

Rizzo, P.C. et al. (1972). "Lateral pressures and prestressed tie-back walls." Ohio River Valley Soils Seminar No. 3, Fort Mitchell, Kentucky.

Wilson, S.D. (1972). "Instrumentation for stress and strain measurements." Ohio River Valley Soils Seminar No. 3, Fort Mitchell, Kentucky.

ORVSS IV

Baker, W.J. (1973). "Reliability analysis and cost optimization of embankments." Ohio River Valley Soils Seminar No. 4, Lexington, Kentucky.

Gedney, D.S., and Walkinshaw, J.L. (1973). "Reinforced earth - Design and construction." Ohio River Valley Soils Seminar No. 4, Lexington, Kentucky.

Gorman, C.T. et al. (1973). "In situ shear strength parameters by dutch cone penetration tests." Ohio River Valley Soils Seminar No. 4, Lexington, Kentucky.

Hagerty, D.J. et al. (1973). "Retrieval and use of geotechnical information." Ohio River Valley Soils Seminar No. 4, Lexington, Kentucky.

Huang, Y.H. (1973). "Applications of the finite element method to geotechnics and transportation engineering." Ohio River Valley Soils Seminar No. 4, Lexington, Kentucky.

Wu, T.H. (1973). "Stability analysis - Its potential and its limitations." Ohio River Valley Soils Seminar No. 4, Lexington, Kentucky.

ORVSS V

D'Appolonia, E. (1974). "Foundations on or in rock." Ohio River Valley Soils Seminar No. 5, Clarksville, Indiana.

Deen, R.C. et al. (1974). "A rock classification schema." Ohio River Valley Soils Seminar No. 5, Clarksville, Indiana.

Deere, D.U. (1974). "Rock engineering on some recent projects." Ohio River Valley Soils Seminar No. 5, Clarksville, Indiana.

Hagerty, D.J. (1974). "Rock excavation (a panel discussion): Consideration of effects on surroundings." Ohio River Valley Soils Seminar No. 5, Clarksville, Indiana.

Hagerty, D.J., and Coulson, J. (1974). "Exploration and testing in rock." Ohio River Valley Soils Seminar No. 5, Clarksville, Indiana.

Lennertz, C.R. et al. (1974). "Rock foundations." Ohio River Valley Soils Seminar No. 5, A panel discussion. Clarksville, Indiana.

Mahar, J. (1974). "Rock excavation (a panel discussion): Geologic and support considerations for rock excavations." Ohio River Valley Soils Seminar No. 5, Clarksville, Indiana.

Ross-Brown, D. (1974). "Design of open excavations in rock." Ohio River Valley Soils Seminar No. 5, Clarksville, Indiana.

Waddel, J.A. (1974). "Rock excavation (a panel discussion): The producer's viewpoint." Ohio River Valley Soils Seminar No. 5, Clarksville, Indiana.

ORVSS VI

Fleming, R.W. (1975). "Geologic perspectives--The Cincinnati example." Ohio River Valley Soils Seminar No. 6, Fort Mitchell, Kentucky.

Hopkins, T.C. et al. (1975). "Effects of water on slope stability." Ohio River Valley Soils Seminar No. 6, Fort Mitchell, Kentucky.

Krusling, J.R. (1975). "Cut and fill ordinance as adopted by the city of Cincinnati." Ohio River Valley Soils Seminar No. 6, Fort Mitchell, Kentucky.

Mathis, H.A. (1975). "Regarding failed slopes." Ohio River Valley Soils Seminar No. 6, Fort Mitchell, Kentucky.

Nethero, M.F. (1975). "Drilled pier retaining walls." Ohio River Valley Soils Seminar No. 6, Fort Mitchell, Kentucky.

Schuster, R.L. et al. (1975). "Importance of geologic structure in stability of rock slopes." Ohio River Valley Soils Seminar No. 6, Fort Mitchell, Kentucky.

ORVSS VII

Almes, R.G., and Butail, A. (1976). "Coal refuse: Its behavior related to the design and operation of coal refuse disposal facilities." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Bishop, C.S., and Rose, J.G. (1976). "Physical and engineering characteristics of coal preparation plate refuse." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Brummond, W.F. (1976). "Use of mine waste in tailings dams." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Cowherd, D.C. (1976). "Closed circuit coal refuse as a structural fill." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Drnevich, V.P. et al. (1976). "Geotechnical properties of some eastern Kentucky surface mine spoils." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Ellison, R.D., and Cho, Y.Y. (1976). "Dynamic design considerations of loose fine coal refuse." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Fetzer, C.A. (1976). "Use of compacted shale as dam embankments." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Hall, G. et al. (1976). "West Virginia experiences with review of coal refuse disposal facilities." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Khosla, V.K., and Murdoch, R.L. (1976). "Engineering evaluation of a Cleveland shale." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Leonard, G.A. (1976). "General report on design and construction." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

Parker, W.W., and Gray, R.E. (1976). "Observation evaluations of coal refuse embankment stability." Ohio River Valley Soils Seminar No. 7, Lexington, Kentucky.

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

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

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

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

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 Atmatzidis, 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

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 sealed 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

Cho, Y.Y. (1982). "Design considerations using SPT's." 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

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 penetration 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

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

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 (DPT) 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

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 stabilitzation 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

Ashar, M.K. (1987). "EEffective use of expert witnesses - The baking a cake approach." Ohio River Valley Soils Seminar No. 18, Fort Mitchell, Kentucky.

Baker, C.N., Jr. (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

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 incenerator 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, J.P. (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

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 insitu 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 ot 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

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.

Spencee, 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

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.

Sprauge, C.J. (1991). "Geotextiles in road structures." Ohio River Valley Soils Seminar No. 22, Lexington, Kentucky.

Wetzel, R. et al. (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

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-tunnelling to replace Boston's St. James Avenue sewers." Ohio River Valley Soils Seminar No. 23, Erlanger, 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 remediati[illegible] construction." Ohio River Valley Soils Seminar No. 23, Louisville, Kentucky.

ORVSS XXIV

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.

Bruce, D.A. (1993). "Contemporary practice in the stbilization 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.

Kessler, K.A., and McGarrah, R.E. (1993). "Stability anlaysis and remedial action for slab on a buttress dam." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.

Patel, A.K. et al. (1993). "Geotechnical aspects of bridge reconstruction." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.

Pohana, R.E., and Jamison, T.M. (1993). "Landslide remediation and prevention by the city of Cincinnati." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.

Rayburn, L.P., and Keller, D.J. (1993). "Infrastructure stabilization of the roads and hillsides in greater Cincinnati." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.

Scarborough, J.A. et al. (1993). "The use of jet grouting for undrepinning and temporary excavation support of a historic building." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.

Zoghi, M., and Aktan, E. (1993). "Technological innovation for infrastructure assessment and revitalization." Ohio River Valley Soils Seminar No. 24, Erlanger, Kentucky.

ORVSS XXV

Bell, K.R., and Davie, J.R. (1994). "Pile load testing - new and improved." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

Bruce, D.A. (1994). "Present researches into the behavior of high capacity pin piles." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

Goble, G.G. (1994). "Pile driving - An international state-of-the-art." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

Kuzmanovic, B.O., and Sanchez, M.R. (1994). "Design of bridge pier pile foundations for ship impact." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

Laefer, D. (1994). "Quality control for jet grouting." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

Long, J.H. (1994). "Databases for deep foundations." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

Ludlow, S.J. (1994). "Test pile program for a cable-stayed arch bridge." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

Murray, S.L., and Greer, D.J. (1994). "A drilled shaft experience - William H. Natcher Bridge, Owensboro, Kentucky." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

Thompson, W.R., and Brown, D.A. (1994). "Axial response of drilled shafts in intermediate geomaterials in the Southeast." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

Townsend, F.C. (1994). "Comparison of deep foundation load test methods." Ohio River Valley Soils Seminar No. 25, Lexington, Kentucky.

BIBLIOGRAPHY OF PAPERS - ORVSS' XXVI through XXIX

ORVSS XXVI
SITE INVESTIGATIONS
Geotechnical and Environmental

October 20, 1995
Holiday Inn Lakeview
Clarksville, Indiana

Microgravity Techniques for Detection of Karst Subsurface Features, by Nicholas C. Crawford

Geotechnical and Environmental Investigation Methods for New Tunnel Construction: A Case Study, by Daniel J. Hurst, James W. Martin, and Bernard H. Voor III

Accelerated Site Characterization Techniques Implemented at U. S. Army Corps of Engineers Contaminated Sites, by James D. Dzubay and Mark S. Meyers

Use of the Ground Penetrating Radar and Seismic Sounding System for Geotechnical Investigation, by M. Zoghi, P. J. Wolfe, B. H. Richard, G. F. Mitchell, and T. Nogami

Investigation and Instrumentation of Deep Lacustrine Clay Deposits in the Cuyahoga River Valley, by Paul H. Anderson and Stuart P. Ravary

Laboratory and Field Measurements of Coal Refuse Properties, by Mohamed M. Nofal and R. Michael Holbrook

Design and Implementation of a Multipurpose Groundwater Monitoring System at Sellafield, U. K., by Chris D. Eldred and John A. Scarrow

Site Characterization Methods for the Design of a Groundwater Extraction System in a Bedrock Aquifer, by D. Joseph Wessley, Richard H. Weber, and Daniel A. Otzelberger

Use of Existing Geotechnical Data to Supplement Site Investigations, by William J. Pfalzer

Site Characterization Aided by Evaluation of Pumping Test Data on Environmental Remediation Projects, by Barry K. Thacker

Geotechnical Characterization of the Waste Pit Material for the Fernald Environmental Management Project, by Mark T. Bowers and Dale A. Lutz

Geotechnical Engineering in Environmental Site Characterization and Restoration Projects, by Michael J. Saffran and Chris R. Karem

ORVSS XXVII
FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING

October 11, 1996
Sharonville Convention Center
Cincinnati, Ohio

Author(s)	Title
Clyde N Baker, Jr.	Earth Retention Design, Ground Movement Monitoring and Liability - A Case History
Mark T. Bowers	Of Drains, Shirts and Codes: Lessons Learned from Failures in Engineered Works
Fred W. Erdmann	Three Case Histories of Settlement Failure
Bengt H. Fellenius	Dispute Avoidance and Piling Specifications
James P. Gould	*Geotechnology in Dispute Resolution
Fred H Kulhawy and Kok Kwang Phoon	*Engineering Judgment in the Evolution from Deterministic to Reliability-Based Foundation Design
Gerald A. Leonards, J. David Frost, and Jonathan D. Bray	*Collapse of Geogrid-Reinforced Retaining Structure
James W. Niehoff	The University Apartments: An Olympic Settlement Case History
Jorj O. Osterberg	Geotechnical Failures - Case Histories An Analysis of the Causes
James P. Whittaker, Joseph P. Klein, III, and David P. Shiells	Stabilization of I-95 / U.S. Route 1 Ramp Embankment

* *Reprinted with permission of ASCE*

ORVSS XXVIII
UNCONVENTIONAL FILLS: DESIGN, CONSTRUCTION, AND PERFORMANCE

October 10, 1997
Holiday Inn North
Lexington, Kentucky

Design of a Failed Landfill Slope
Timothy D. Stark; University of Illinois at Urbana-Champaign, Urbana, Illinois
W. Douglas Evans; Ohio Environmental Protection Agency, Columbus, Ohio
Paul E. Sherry; Ohio Environmental Protection Agency, Dayton, Ohio

Construction of a Highway Embankment with Gasoline-Contaminated Soil
Fred W. Erdmann; Dames & Moore, Cincinnati, Ohio
Richard J. Hajer; Dames & Moore, Cincinnati, Ohio

Geotechnical Engineering Management of Unconventional Fill Materials in a Large Design-Build Highway Project
G.D. Prasad; Canadian Highways International Corp., Mississauga, Ontario, Canada
Dan Guistini; AGRA Earth & Environmental Limited, Mississauga, Ontario, Canada

Liquefaction Mitigation Procedures for Coal Refuse Dams Built by the Modified Upstream Method
Barry K. Thacker; Geo/Environmental Associates, Inc., Knoxville, Tennessee

Geotechnical Properties of Lightweight Aggregate
Thomas O. Keller; GEI Consultants, Inc., Carlsbad, California
Frank S. Archambeault; Norlite Corporation, Cohoes, New York

Crushed Glass Used as Structural Fill to Support Material Recovery Facility
Jon H. Gould; Raytheon Engineers & Constructors, Inc., Birmingham, Alabama

Settlement of Shallow Foundations on Uncontrolled Mine Spoil Fill*
J. Richard Cheeks; Stokley-Cheeks and Associates, Inc., Nicholasville, Kentucky

Embankment Construction Using Shale
Tommy C. Hopkins; Kentucky Transportation Center, Lexington, Kentucky
Tony Beckham; Kentucky Transportation Center, Lexington, Kentucky

Appendix Past ORVSS Dates, Topics, and Locations

* Reprinted with Permission of the American Society of Civil Engineers

ORVSS XXIX
PROBLEMATIC GEOTECHNICAL MATERIALS

October 16, 1998
Club Hotel by Doubletree
Louisville, Kentucky

Seismic Renovation of the South Carolina Statehouse
by Larry Rayburn, Terry Tucker, and Bill Heckman
Richard Goettle, Inc.

Railroad Subgrade and Slope Repair Along the Ohio River
by Gary T. Brill, Geo/Environmental Associates, Inc.,
Kenneth E. Darnell, Darnell Associates, LLC, and
Charles E. Gullakson, CSX Transportation, Inc.

Refined Field Methods for Identifying, Describing, and Testing Shale and Weak Rock
by Paul M. Santi
Department of Geological and Petroleum Engineering
University of Missouri-Rolla

Unconventional Earthwork Construction Minimizes Undercutting
by Jon H. Gould
Raytheon Engineers & Constructors

**Papa John's Cardinal Stadium:
Foundation Construction Over 100 Years of Railroad Repair Shop Debris**
by Timothy M. O'Leary and Nicholas G. Schmitt
Law Engineering and Environmental Services, Inc.

Railroad Tunnel Construction in the Kope Formation
by Mark J. Doller and Bernard H. Voor III
Ogden Environmental & Energy Services Company, Inc.

Landslide Repair at Mission Viejo Mall
by John R. Wolosick, Hayward Baker, Inc.
Paul B. Groneck, DBM Contractors, Inc.
Seth L. Pearlman, Nicholson Construction Company

Design and Performance of Foundations in Expansive Shale
by James J. Belgeri and Timothy C. Siegel
S&ME, Inc.

Problematic Soils Improved by Compaction Grouting
by John W. Reed and Dennis L. Boley
Denver Grouting Services

Soil Displacement Piles in Coastal Deposits: A Case Study
by Michael D. Smith and Daniel C. Brahana
Law Engineering and Environmental Services, Inc.

VALUE ENGINEERING - A GREAT CONCEPT WHY ISN'T IT USED MORE FREQUENTLY?

Jorj Osterberg[1]

ABSTRACT

Value Engineering is a great idea. When it has been used as intended, both economical and better designs have resulted. But for every successful case, there are many where it has not functioned well and far many other cases where it should have been used and was not used. This paper discusses instances where value engineering did not work and where many opportunities were missed for great savings and better designs. When value engineering is specified in a contract (mostly for government work), and the agency which prepared the design and specifications is the same agency which makes the decision on the value engineering report, it often looks upon the recommendations as an intrusion, and may think it reflects unfavorably on its original design So why should it accept someone else's ideas? In other cases, where value engineering is not an issue in the contract, why should the designer accept changes proposed by others which he thinks might reflect poorly on his engineering ability and would only cause more headaches and costs for him? There are other cases where the owner is not even aware of the possibility of a more economical and sometimes better design. If the owner was aware of the possibility, he probably would insist on a value engineering study. To rectify this situation, the agency responsible for design and construction should not be the same agency which judges the value engineering. An impartial third party should be the judge. For private work in which value engineering is not specified, there should be incentives for the design firm in the form of a percentage of the money saved and the additional cost incurred. The paper gives examples to illustrate the above.

INTRODUCTION

Value engineering is a great concept capable of potentially saving billions of dollars (yes, I mean *billions*) in construction costs and in many cases improved and safer designs. It is not likely that this potential will be reached in a very long time, if ever. However, it is within our power as engineers and contractors to save tens if not hundreds of millions a year if we get together and act. The

---

[1] Professor of Civil Engineering Emeritus, Northwestern University
Consulting Geotechnical Engineer, Aurora, Colorado

reasons for the present situation are many: the tendency of designers and particularly those in government to "play it safe" by being overly conservative; the reluctance to deviate from "accepted practice" for fear of later being sued; the antiquated and inflexible building codes; the attitude that "it's not my money, so why would I care"; do it the way it was always done; the ignorance or indifference of the owner about potential savings. If we engineers are honest, we should look at each project with the thought in mind - how would I do it if I were paying for it. Certainly safety and obeying the law is the primary and utmost consideration. Then the most economical, functional and practical solution should be sought, followed by a study of how costs can be cut. But don't forget, sometimes contractors have good ideas too! Often, an informal peer review within your own organization can be of great help. Then imagine you are the owner while you are reviewing your design for possible savings.

CASE HISTORIES

Following are case histories in which I have been directly involved in which substantial savings were possible but few in which the savings were actually realized.

Case 1 - Value Engineering Involved. The Corps of Engineers designed a new bridge to replace an old one which was built around the turn of the century. The new bridge was built close to the old one and the foundations were to be on drilled piers extending through a rather thick layer of very hard glacial till with boulders to rock. The foundations of the old bridge, which was a heavy stone-faced concrete bridge were on top of the glacial till and there was no evidence of settlement. The contractor, who realized the difficulties of drilling through the hard till and boulders, wondered why the new bridge, which weighed less than the old bridge, could not rest on drilled shafts resting on or a short distance into the hard till. He contacted a consulting engineering firm which had long experience designing bridges, which in turn consulted me for advice. I could see no reason why the drilled shafts needed to go to rock. There was no evidence of scour at the foundations of the old bridge. Furthermore there would be a substantial amount of overburden from the river bottom to the glacial till. A request was made to make a value engineering study but the Corps said absolutely no and would not even permit a meeting to discuss the matter. The bridge was built as the Corps had designed it. I did not make an estimate of what the savings might have been, but certainly it amounted to hundreds of thousands of dollars. Remember, this was your money and my money! This is a case of the owner who was also the designer ruling out value engineering out of hand.

Case 2 - Water Treatment Plant. A water treatment plant was designed to be next to a large river for a large mid-western city. The design firm for the city engaged a well-known geotechnical engineering firm to make a soil investigation and make a report giving recommendations for the foundations. After a comprehensive investigation, the firm recommended that a surcharge fill be placed over the areas which would be subjected to heavy loads. The surcharge fill was almost complete when the design firm went to the city engineer and said the structures must go on piles. The city engineer who was a civil engineer with some foundation experience wondered why the design firm changed its mind. When he couldn't get an acceptable answer, he contacted me. After studying the soil borings and report, I could see no reason why the structures needed to be on piles. The surcharge would consolidate the ground to loads larger than the structures would exert and the time-settlement curves would give a record to tell when and if the settlement at any time was sufficient. My recommendation was rejected and I was asked to come to the city and meet with the designers and the city engineer. The designers said, "If piles are used there will be no settlement." I refuted that statement and said there will be settlement, but if the piles were properly designed and installed, the settlement would be small and tolerable. Then I was asked, "Will you guarantee that if the piles were eliminated, there will be no cracks in the concrete?" I said I could not guarantee there would be no cracks but could give assurance that there would not be any detrimental settlement provided the surcharge was allowed to remain a sufficient time for the soil to consolidate to a load greater than to be applied. The designers then said the small additional cost of about $20,000 for piles would give additional assurances that there would be no settlement. I said, "What? I estimate that the additional cost of the piles would be about $200,000." The answer was, "But it would only cost the city $20,000 because it was an EPA project." and Uncle Sam would pay the rest. I said, "Who the h---'s money do you think this is?" When the city engineer was convinced that the surcharge method would work, he ordered the designers to eliminate the piles and use shallow spread foundations. I requested to have the settlement monitored as the project was under construction and when all the tanks were filled and to send me the results. The settlement did not exceed ¼ inch. Why the design firm changed its mind I will never know.

In most similar circumstances the owner would not have accepted my recommendation because the design firm would have refused to guarantee the work. Do you think this consultant wondered what he would do if it were his money that would be used to build the project?

Case 3 - Spread Footings. I was hired by Allstate Insurance Company to review a report making recommendations for the foundations and earthwork for a medium size office building in a large city on the West Coast. The report had sufficient borings and soils information to determine the allowable bearing pressure of footings. The consulting firm, a well-known one with a good reputation, recommended using 2,000 lbs/sq ft maximum bearing pressure. Soils information indicated to me that 5,000 lbs/sq ft could be used. I phoned the engineer who wrote the report and learned later that he was a new and inexperienced employee. I asked if there was anything unusual about the soil that made him recommend the low bearing pressure. He replied that there was nothing unusual. "Then, why did you recommend such a low bearing value?" He answered that using a higher bearing value would not save much. I pointed out that the savings would be in the tens of thousands of dollars. He then said he could not change the recommendation unless he went to higher authority. Well, I went to higher authority and talked to someone I knew and felt was well qualified. He agreed that a higher bearing value could be used but questioned my 5,000 lbs/sq ft recommendation. We finally agreed on 4,000 lbs/sq ft.

Even though very large sums of money were not involved, I give this example because I have experienced the very same thing a great many times. I am sure that every day many reports are made recommending bearing values which are much too low. When you multiply this by the number of footings which are constructed in a year, pretty soon it runs into a lot of money. Again, "It's not my money, so why would I care." However, high bearing values should only be recommended when the borings and soil tests clearly indicate that the bearing value you recommend can be justified. Even though this example does not justify a full value engineering study, there should be some mechanism whereby these wasteful practices do not occur.

Case 4 - Belled Piers. This is not a case of value engineering, but I can't resist telling it, because it defies all common sense. Actually, you might call it reverse value engineering. I wrote a report for Allstate recommending an office building be placed on belled piers and recommended the allowable bearing value. After the concrete frame and floors were completed, the structural engineer discovered that the shafts were drilled with a machine that made the angle to the vertical of the bells 45 degrees contrary to the specs which called for 60 degree bells. He stated to the owner that this was unacceptable and he could not approve it. The contractor was quite embarrassed since he had not read the specs carefully enough and missed the 60 degree requirement. I then looked at the concrete strengths from the test cylinders and found they were all about 5,500 lbs/sq in, whereas the specs called for 4,000 lb concrete. I presented this to the structural engineer but he said that made no difference. Then the contractor drilled a new shaft just outside the building and demonstrated that the angle of the bell was 53 degrees. This still made no impression on the structural engineer. Then a careful level survey of the entire building was made to determine if there was any differential settlement and there was none. Yet, the structural engineer could not be budged and said he could give no guarantee of the structural integrity of the building. When I asked him what the contractor should do to the building so he could give a guarantee, he had no answer. Since I had done a lot of previous work for Allstate and had good relations with their construction department, and since I convinced Allstate that the guarantee doesn't mean anything anyway, they took my advice to simply ignore the structural engineer. After the building was completely finished and in operation for more than a year, there was no evidence of differential settlement. Perhaps you might call this a case where value engineering would say, "Make no changes." I have had a number of cases in which similar incidences occurred, but in those cases the client would not accept my recommendation because the architect or design engineer would not make any guarantee if my recommendation were followed.

A great many cases have been encountered using the Osterberg method of load testing drilled shafts, in which if value engineering had been applied, many million of dollars could have been saved. In all these cases there was no mechanism I am aware of which could have initiated a value engineering review. To understand these cases a brief description of how the Osterberg Load Test is performed follows: Fig. 1 shows a hydraulic jack-like device placed on the bottom of a drilled shaft. After the concrete is poured and cured, pressure is applied to the device which exerts an equal upward and downward force on the shaft. The downward end bearing force is resisted at all times by the side shear (skin friction) and therefore no overhead load frame or dead weight reaction is needed. The upward movements of the bottom of the shaft, of the top of the shaft and the downward movement of the bottom are measured by telltales and are recorded on a data logger from which the movements can be plotted and/or shown directly on a monitor screen. Movements will continue until either the ultimate in side shear or the ultimate in end bearing is reached or the capacity of the device is reached, whichever occurs first. When this occurs, the test is completed. A method of constructing the equivalent top down curve (Ref.1) has been shown to agree with a top down test using a kentledge load made nearby.

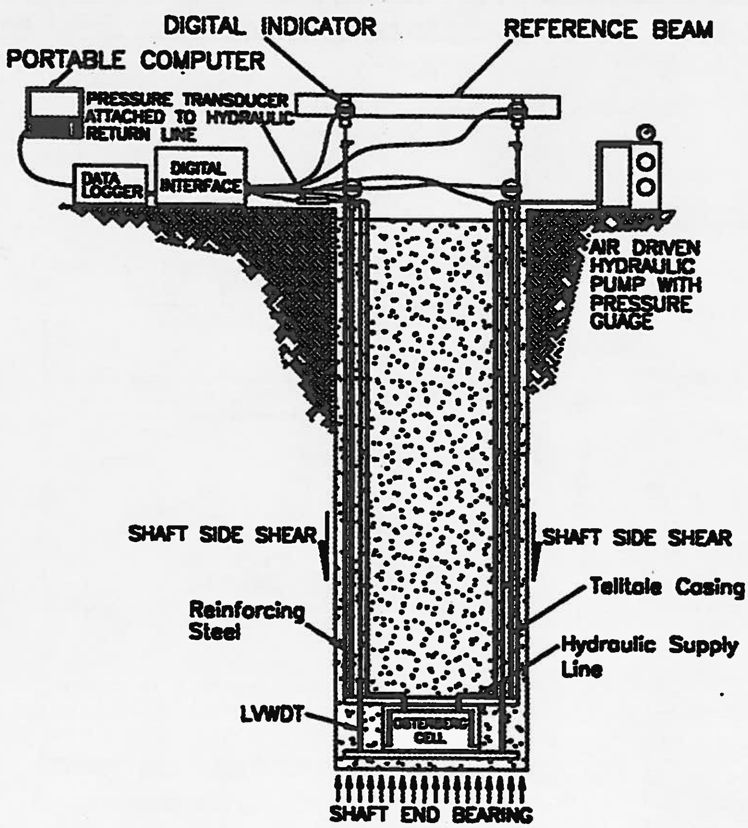


Fig 1 - Osterberg Load Test Method

Approximately 400 O-Cell tests have been performed. Load tests of up to 15,000 tons have been made on shafts up to 9 ft. in diameter and up to 200 feet deep. In the great majority of the tests, it has been found that the side shear (skin friction) is considerably larger than most designers assume and larger than estimated from compression tests. For rock sockets it has been found that the side shear is very much larger than that generally assumed and that compressive strengths of rock cores is a poor indicator of the sides shear. The cases discussed below illustrate these findings and indicate that a value engineering analysis could have saved very substantial sums of money. One case is given which shows how results on a test shaft used wisely and prudently for designing the working shafts is itself a form of value engineering.

LOAD TESTS ON DEEP SHAFTS USING THE OSTERBERG LOAD CELL

Case 5 - Test Shaft for drilled piers for a Bridge over the Ohio River at Owensboro, Kentucky.

Fig. 2 shows the soil and rock profile and the load- deflection curves for a test shaft at the site. Because of possible deep scour in the future, only the load capacity of the 19 ft of shale below the sand was to be considered in the design.

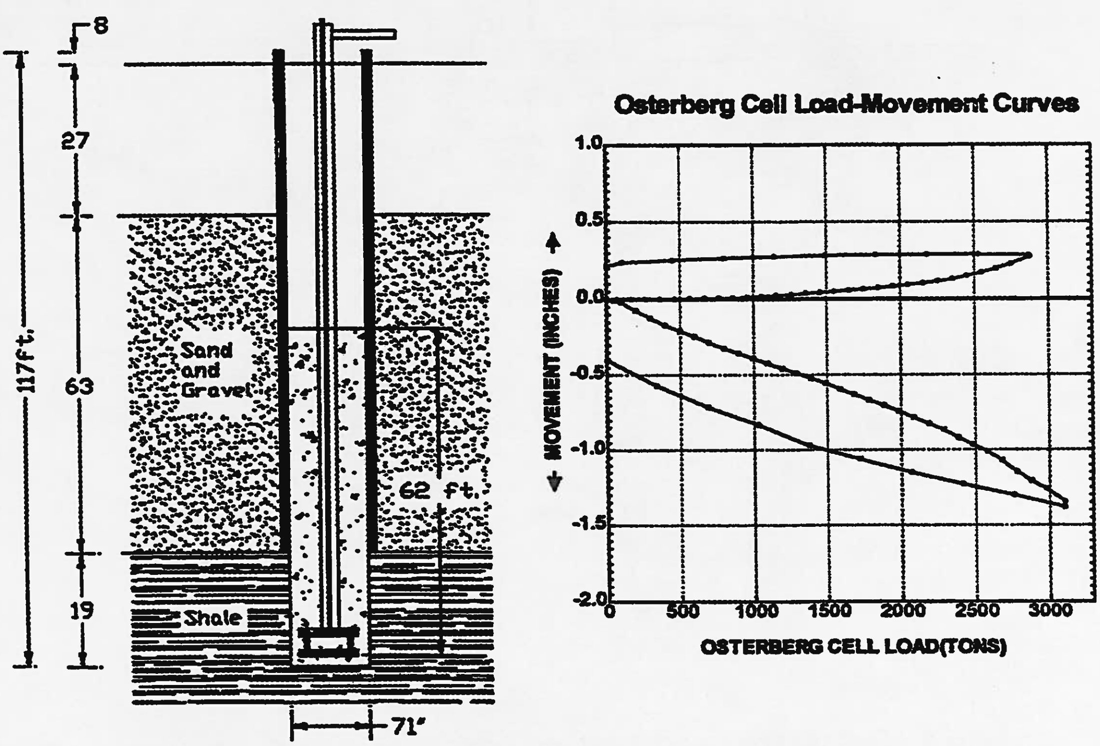


Fig 2 - Test Shaft for Bridge in Owensboro, KY

The rock consists of weathered shale (compressive strength varied from 350 to 500 lbs/sq in) with coal seams and sandstone seams. As shown in the Fig. 2, concrete was placed to some distance above the top of the shale. However, strain gage readings in the concrete above the shale showed that the load taken above the top of the shale was negligible. The test was designed to go to three times the design load.

However, the test went far beyond that and the ultimate load was not reached in either side shear or end bearing when the capacity of the device (3,000 tons up and 3,000 tons down) was reached as shown in the figure. At the design load of 1,000 tons, the total deflection was only 0.2 inch. The test showed that considerable savings (estimated to be in the millions) could be made if the working shafts would have been redesigned to take advantage of the actual measured strengths and deflections from the test results, but this was not done. For political and practical reasons, completion of the bridge on schedule was of the utmost importance and it was assumed that the time for redesign and renegotiating of the contract would delay completion. Though the redesign and renegotiating would have taken some time, it could easily have been made up by the reduction in construction time, but this was not even estimated. Value engineering as far as I know, was not even discussed. This is another case of using the load test results to demonstrate that the design was adequate but not to take advantage of what savings might result.

Case 6 - Test Shaft for a Tall Building in Hong Kong.

The drilled shaft, 3.3 ft. in diameter, went through 56 feet of overburden and 10 ft. into partially weathered granite. The hole was stabilized with bentonite slurry. The O-Cell was placed on the bottom and the shaft was filled with tremie concrete only to the top of the socket. Therefore no value was given to the side shear of the overburden. The test went only to twice the design load of the shaft even though the test could have gone to almost double that to reach the maximum allowable load of the O-Cell. The building code in Hong Kong does not allow the use of end bearing in shafts stabilized with bentonite slurry. At twice the design load in side shear, the upward deflection of the top of the shaft was only 0.07 inch. But since with the O-Cell, the upward load is equal to the downward load, the shaft was actually tested to four times the design load and at that load the downward movement of the bottom of the shaft was only 0.22 inch. Therefore at four times the design load the total deflection, if the load is applied to the top of the shaft, would be about 0.30 inch. Both the upward and downward load-deflection curves were linear. A conservative estimate of the lowest possible ultimate load indicated that, at the design load, the factor of safety would be at least 7. Wouldn't this be a great case for value engineering? Well, it wasn't. The designers were very happy that their design was safe. I wrote a critique which pointed out that considerable savings could materialize if the depth of penetration of the sockets into the rock would be cut in half. The reply was that it would be difficult to convince the building officials to change the design and to allow end bearing and also that time is money and the cost of delay would probably be more than the savings. What's the point in making a load test on a test shaft and not even considering any changes in design?

Case 7 - Test on a Working Shaft for a Bridge in the Midwest.

The shaft went through the overburden and was socketed 37 feet(!) into a shale formation. The drilling contractor and the geotechnical consultant told the state that the socket did not need to be so deep but it was drilled to that depth anyway. The design load was 500 tons and the test load was made to 3,000 tons up and 3000 tons down. Fig. 3 shows the load-upward movement of the shaft and the load-downward movement in end bearing. Fig. 4 shows the equivalent top-down load-deflection curve. It is seen that the upward movement and downward movement lines are linear and the equivalent curve is linear. At the design load the settlement is 0.01 inch and at 11 times the design load it is 0.10 inch. It is obvious that the great majority of the movement was elastic. The owners were very happy with the results and completed the bridge piers with the original design! This is the most extreme example of an over designed shaft I have ever encountered. Just think what could have been saved if the penetration into the rock could have been two diameters instead of twelve diameters.

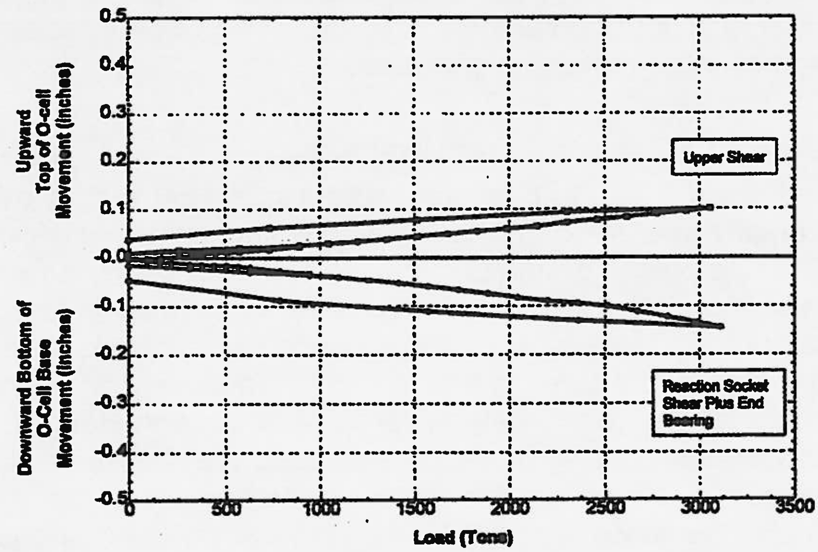


Fig 3 – Osterberg Cell Test Results – Load Movement Curves

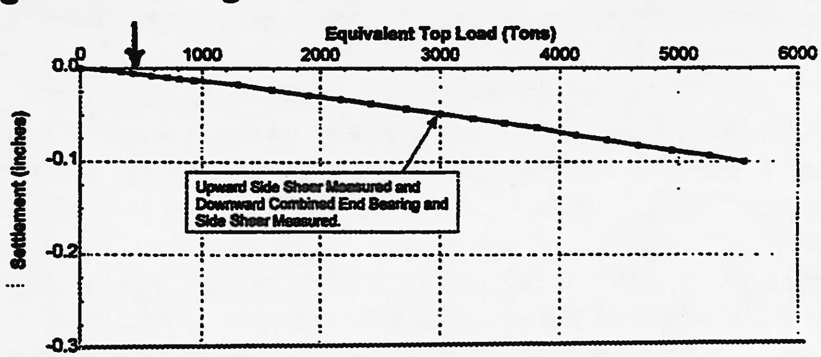


Fig 4 - Equivalent Top-Load Settlement Curve

Case 8 - Test Shaft for the Foundations of a Bridge in Massachusetts.

The soil profile consists of overburden underlain by shale. The purpose of the test shaft was to determine the ultimate side shear and end bearing of the rock socket and use these values for designing the working shafts. The O-Cell was placed on the bottom and the shaft grouted to the top of the socket. Figs 5 and 6 show the O-Cell movement curves and the equivalent top down curve. Coincidentally the ultimate in both end bearing and shear occurred at about the same load and at about the same deflection, 28 MN and 15mm (3150 tons and 0.75 inches). This is what you would call value engineering incorporated in the design, the way the previous examples should have been carried out.

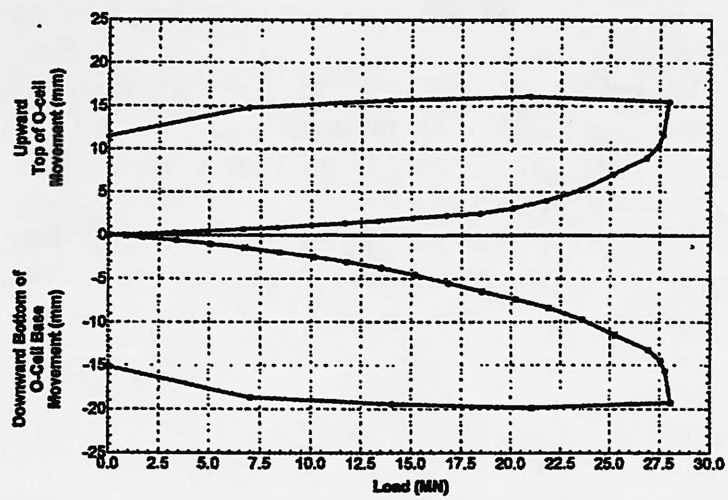


Fig 5 - Osterberg Cell Load-Movement Curves – Bridge in Massachusetts

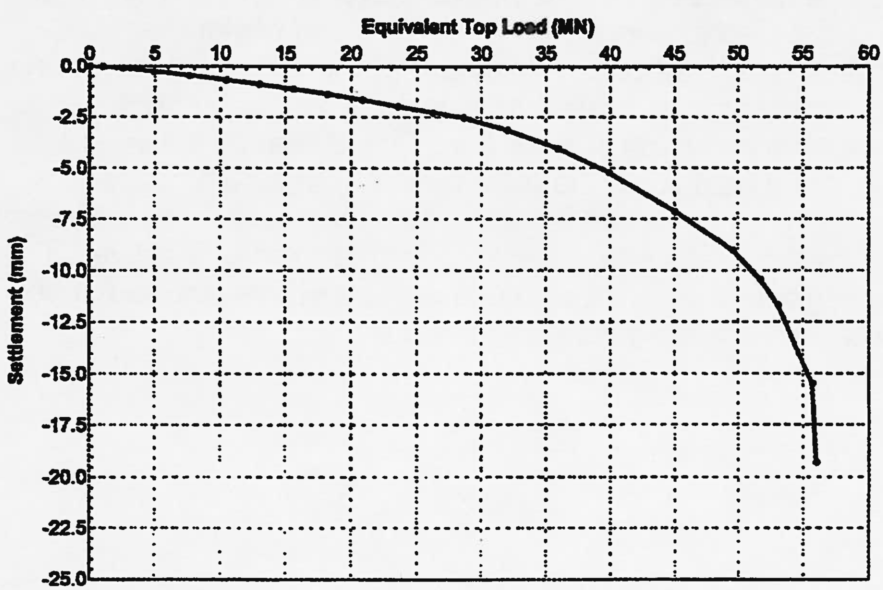


Fig 6 - Equivalent Top Load-Settlement Curve – Bridge in Massachusetts

CONCLUSIONS

1. Value engineering is not being used as frequently as it should be used. There are far more examples of where value engineering could save large sums of money than where value engineering did save money.
2. The reasons why value engineering is not used more frequently are:
   a. In many cases the group evaluating the value engineering is the same group that did the original design
   b. The limitations and obsolescence of building codes are obstacles.
   c. The fear of being sued if something should go wrong.
   d. There is no incentive for the designer to go through a value engineering assessment. In fact it is a disincentive because he generally does not get paid for the extra time involved.
   e. When the project is paid for with public money, then we tend to spend it more freely and to be "more safe" than if it is not public money.
   f. On many projects the owner frequently leaves it up to the designer and is not aware that there is a possibility to save on the design.
   g. Many design firms are not aware of the process of value engineering, and many of those who are aware believe it is used on only public works projects.

WHAT CAN BE DONE ABOUT IT

1. We geotechnical engineers should prepare a document explaining the process of value engineering for public projects, another document for private projects and a third document for smaller projects. These documents should be widely distributed to engineers, contractors and owners.
2. Owners and contractors should be made aware of the possibility of saving money and improving designs by using the value engineering process.
3. When a value engineering study results in savings, the engineer should share in the savings and be paid for the extra time spent in the process.
4. The value engineering study should be done by a competent independent third party and not by the designer, contractor or owner's representative.
5. We geotechnical engineers should be honest with ourselves and look upon each project as if we are spending our own money for it.

REFERENCES

1.OSTERBERG, JORJ O., "The Osterberg Load Test Method for Drilled Shafts and Driven Piles – The first Ten Years". Proceedings of the Seventh International Conference on Piling and Deep Foundations, Vienna, Austria, June 1998. Published by the Deep Foundations Institute.

2. OGURA,H., SUMI,M., KISHIDA,H. and YOSHIFUKA,T., (1996) "Application of the Pile Toe Test to Cast-In-Place and Precast Piles." Translated by MADAN KARKEE, Foundation Drilling Magazine, December 1996/January 1997.

3. SCHMERTMANN, JOHN, & HAYES, JOHN (1997), "Observations from Osterberg Cell Tests of Bored Piles." FULCRUM, The Newsletter of the Deep Foundations Institute, Winter '96-'97 pp. 11-14.

4. SCHMERTMANN, JOHN, & HAYES, JOHN, & MOLNIT, THOMAS, & OSTERBERG, J.O., "O-Cell Testing Case Histories Demonstrate the Importance of Bored Pile (Drilled Shaft) Construction Technique" (1998), Proceedings of the Fourth International Conference on Case HIstories, University of Missouri, Rolla, Missouri, March, 1998.

5. OSTERBERG, J.O., (1994) "Recent Advances in Load Testing Driven Piles and Drilled Shafts Using the Osterberg Load Cell Method", Proceedings of the Geotechnical Lecture Series, Geotechnical Division, Illinois Section, American Society of Civil Engineeers, Chicago, Illinois.

6. SCHMERTMANN, JOHN (1993). "The Bottom-Up, Osterberg Method for Static Testing of Shafts and Piles", Proceedings: Progress in Geotechnical Engineering Practice. 13th Central Pennsylvania Geotechnical Seminar, Hershey, Pennsylvania.

BOOTSTRAP BACKFILL

G.T. Vandevelde[1]

ABSTRACT

A mid project change in the planned construction schedule resulted in the need to backfill basement walls that were not designed as retaining walls. The change occurred at the point in the schedule when the walls needed to be backfilled. A low cost, quick solution was needed to prevent construction delays. Conventional bracing systems could not be used due to the future planned construction and site restrictions. A design was developed that utilized multiple layers of weak grout held off the wall by disposable forms, resulting in no lateral load on the wall. Soil nails were used to pin the grout to the slope. Sand was used to backfill the top four feet to allow for easy construction of utilities. The lateral load created by the sand was resisted by anchors in the upper wall tied to a reinforcing steel mat cast in the grout fill zones. During the year delay until construction of the adjacent floors permanently braced the walls, no wall deflections were detected.

IMPACT OF SCHEDULE REVISIONS

Most of the people who have had trouble at Actors Theatre in Louisville, Kentucky have been actors who have forgotten their lines or who have been embroiled in dramatic dilemmas. During construction of a nine-story parking garage and four-story theatre addition, difficulties of a more down-to-earth nature assailed the team of architects, engineers and constructors who were involved in the expansion. Basement walls in the structural addition had been designed essentially as two-way slabs, with relatively light reinforcing, to be supported by basement and first floor slabs at top and bottom, and by structural columns at both edges. Because of a change in construction phasing, the walls had been completed between the columns, but the floor slabs were not built. At that critical point, project planners decided to delay construction of these floors for at least a year, while the upper parts of the theatre expansion and garage were completed and put into use. In order to complete construction of the upper portions of the structure, it would be necessary to backfill the areas just outside the basement walls. Scaffolding, material stockpiles and equipment access all required a surface at grade outside the basement walls. Needless to say, the lightly reinforced walls were not designed to act as freestanding, cantilever retaining walls. The walls had been supported on narrow footings and pile caps built over augered, cast-in-place piles. Placement of backfill against the walls would surely cause collapse of the walls or at least intolerable deflection in the walls and supporting columns.

---

[1]G.T. Vandevelde, P.E., Vice President
QORE Inc. 13005 Middletown Industrial Blvd. Suite D
Louisville, Kentucky 40223

Acknowledgement is given to Dr. D. J. Hagerty, P.E.
University of Louisville, Louisville, Kentucky
for assistance in design development, construction monitoring,
and preparation of portions of this document.

EMERGENCY DESIGN

At this juncture, QORE the project geotechnical engineers (d.b.a. Ground Engineering), were contacted to devise a solution. In addition to the budgetary constraints presented by this magnitude design change in the middle of a project for the local theatre, backfilling of the wall was on the critical path for the entire project. A solution was needed immediately. The site was located in glacial outwash deposits near the Ohio River. Many sensitive historic structures exist in the area, including the adjacent theatre. Beneath approximately 15 feet of rubble fill, the site was underlain by loose sands to approximately 40 feet. Denser sands and gravels extended from that depth to rock at nearly 100 feet.

After some consideration of alternatives, QORE contacted Dr. Joe Hagerty of the University of Louisville, for assistance. Many designs were considered, but the unique combination of conditions and limitations required a true value engineering approach. An apparent solution to the problem would have been to construct a lateral bracing system of horizontal struts to restrain the walls. This alternative was unacceptable for several reasons: such a system would severely restrict construction operations within the walled enclosure; the lateral strut system would have been very expensive; and construction of such a system would have put the project weeks behind schedule. Consideration was given next to a system of inclined braces, or rakers, which would transfer lateral and vertical loads to a set of grade beams to be built across the floor of the excavation. This alternative would have been slightly less costly than the horizontal strut system but it would have hampered construction within the walls very severely and would have caused several weeks of delay in the project. The loads could not be transferred to the pile caps and auger-cast piles of the foundation because those piles were very lightly reinforced and were not designed or built to support lateral loads. Finally, the need to excavate utility trenches and access other areas in the basement floor virtually precluded any system of grade beams on the basement floor. Tie-back systems also were considered and found unacceptable for similar reasons. In essence, it was necessary to develop a self-supporting backfill, which would place no load on the basement walls. The engineers developed a solution, and then with additional insights of an experienced construction superintendent, Danny Overstreet, of Wehr Construction, Inc., refined the design to be economically and rapidly constructed. The design was termed "bootstrap backfill" since it essentially holds itself up.

DESIGN DESCRIPTION

The main elements of the self-supporting backfill include multiple layers of controlled density fill (CDF) in the deeper fill levels, soil nails to tie the CDF to the near 45 degree cut slope, a wall anchor at midheight secured in the mass of CDF, and inexpensive sand backfill for the uppermost 4 feet of the 12 ft-high wall. To avoid creating pressure on the walls, the CDF was placed in 2-ft layers and retained while fluid by Stayform (a corrugated light gauge expanded metal sheeting) forms maintained at least 2 inches from the face of the wall. The metal mesh forms were lightweight, and easily cut and shaped with hand tools. The forms were braced by vertical steel reinforcing bars, as shown in Figure 1, and the bracing bars were tied to the soil nails by heavy-gauge wire. The soil nails consisted of No. 8 reinforcing bars 8 feet long, driven at least 4 ft into in-situ soil; the soil nails were spaced at 2 ft, center to center, along the line of the wall. The retaining wires were tensioned to resist lateral movement of the form work and soil nails when the CDF was placed.

Figure 1. First Lift of CDF, Formwork, Soil Nails and Ties.

Before the CDF was placed in front of the walls, anchor bolts (4,000-lb working load) were installed in the basement walls at 3 ft center-to-center on a line 6 ft above the bottoms of the walls. A mat of No. 3 reinforcing bars was tied to these bolts and cast into the CDF fill, as shown in Figure 3. After the CDF was placed to a height of 8 ft above the bottom of the wall, a foam strip seal was placed between the wall and the top plane of the CDF, to prevent any material from entering the space between the wall and the face of the CDF. Then, sand backfill was placed over the CDF, to complete the 12 ft of fill depth against the wall. The bolt and mat anchor system restrained the wall against the lateral pressure exerted by the sand backfill. The weight of the CDF fill and the soil nails in turn restrained the CDF mass and embedded anchor mat.

CONTROLLED DENSITY FILL

A low cost, very fluid, weak grout was needed for this application. Such a material, frequently termed controlled density fill (CDF) or flowable fill is frequently used by local contractors for utility trench backfill, particularly in karst areas. The design of the CDF mix was done by Earl Kessler of I.M.I. to satisfy the following requirements: the mix was to be flowable and easily placed with a minimum of mechanical manipulation; the strength of the mix after 24 hr was to be at least 10 psi (sufficient to walk on and support dowels driven into the fill to tie the lifts together) and the 28-day strength was not to exceed 100 psi (to enable excavation with a backhoe). When the CDF was placed against the forms, little to no fill penetrated the form perforations. The CDF mix was so fluid that bleed water often built up on top of the pour lift. However, the water evaporated or bled off to the drain at the base of the wall. In a few locations shallow washouts were visible where the bleed water drained through the Stayform perforations. These washouts did not appear to cause any problems and were filled by the next CDF lift. The mix proved to be easily workable, and quickly developed sufficient strength to allow placement of the 3 foot long vertical dowels in the top of the CDF surface to key the fill layers together.

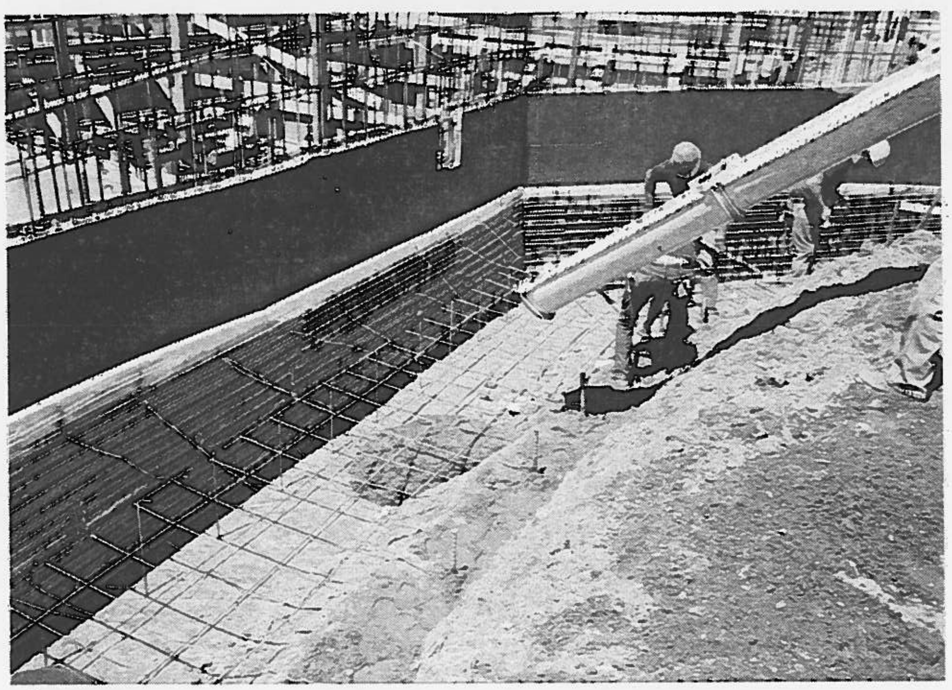


Figure 2. Placement of CDF By Chute

CONSTRUCTION SEQUENCE

The following sequence was used in construction of the system.

- Create a drain at the base of the wall to eliminate any water that accumulates.
- Establish reference marks on the wall to enable detection of movement.
- Shape cut slope to no steeper than 1H:1V
- Drive No. 8 bar soil nails 4 feet into slope.
- Construct formwork 2 inches from wall, tie to soil nails.
- Tighten tie wires to minimize deflection when CDF is placed.
- Place 2-foot thick CDF lift, confirm no contact between form and wall.
- When CDF can support foot traffic, drive in No. 4 dowels to tie layers together.
- Repeat construction of formwork, soil nails, and dowels (stager nail and dowel locations) for second and third lifts.

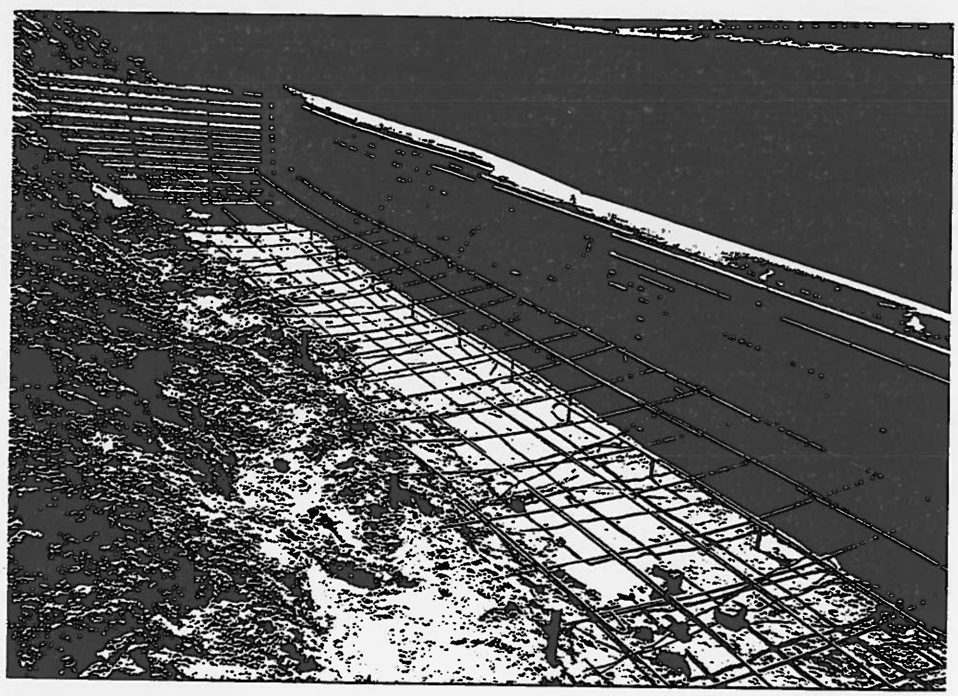


Figure 3. Forms and Reinforcing Steel Mat For Wall Anchors

- Set concrete anchors in wall.
- Construct mat of No. 3 bars and tie to wall anchors.
- Set fourth line of soil nails and place last CDF layer.
- When CDF achieves design strength, place foam blocking between wall and forms, and compact sand backfill to grade.

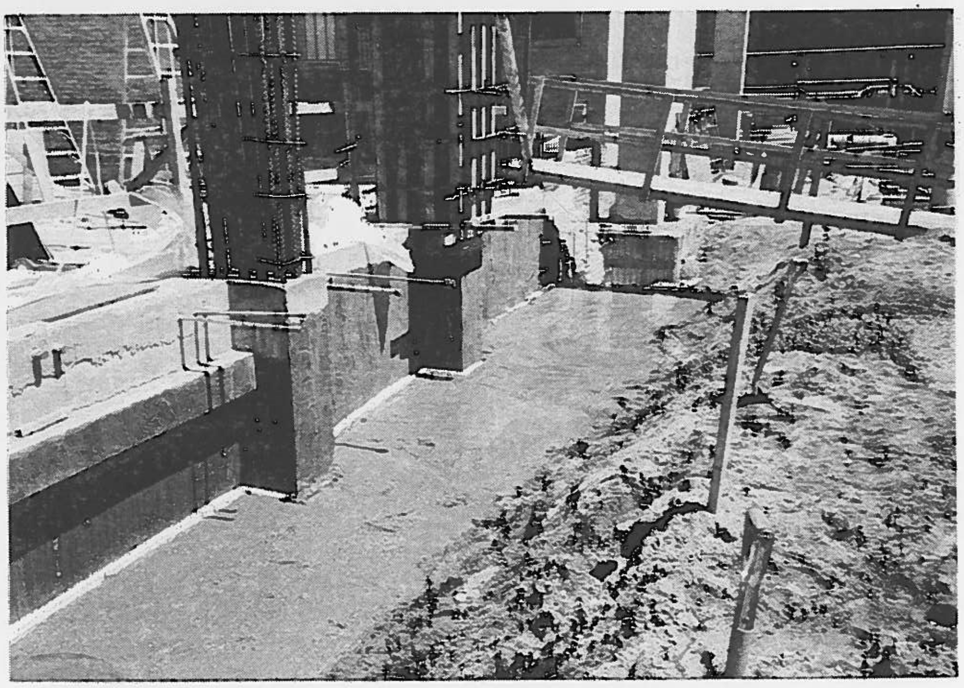


Figure 4. Completed CDF Fill With Foam Blocking At Wall

PERFORMANCE AND CONCLUSIONS

Precise location surveys on the basement walls before and after backfilling showed no significant deflections in the tops of the walls. Project constructors estimated that the self-supporting backfill system cost $ 50,000 to $ 70,000 less than the least expensive alternative bracing scheme, and saved at least a week in construction time. The CDF was of sufficiently low strength that utility trenches and other excavations could be made easily outside the walls, and the system interfered in no way with construction operations inside the walls. Approximately one year later, the expansion was completed without complications.

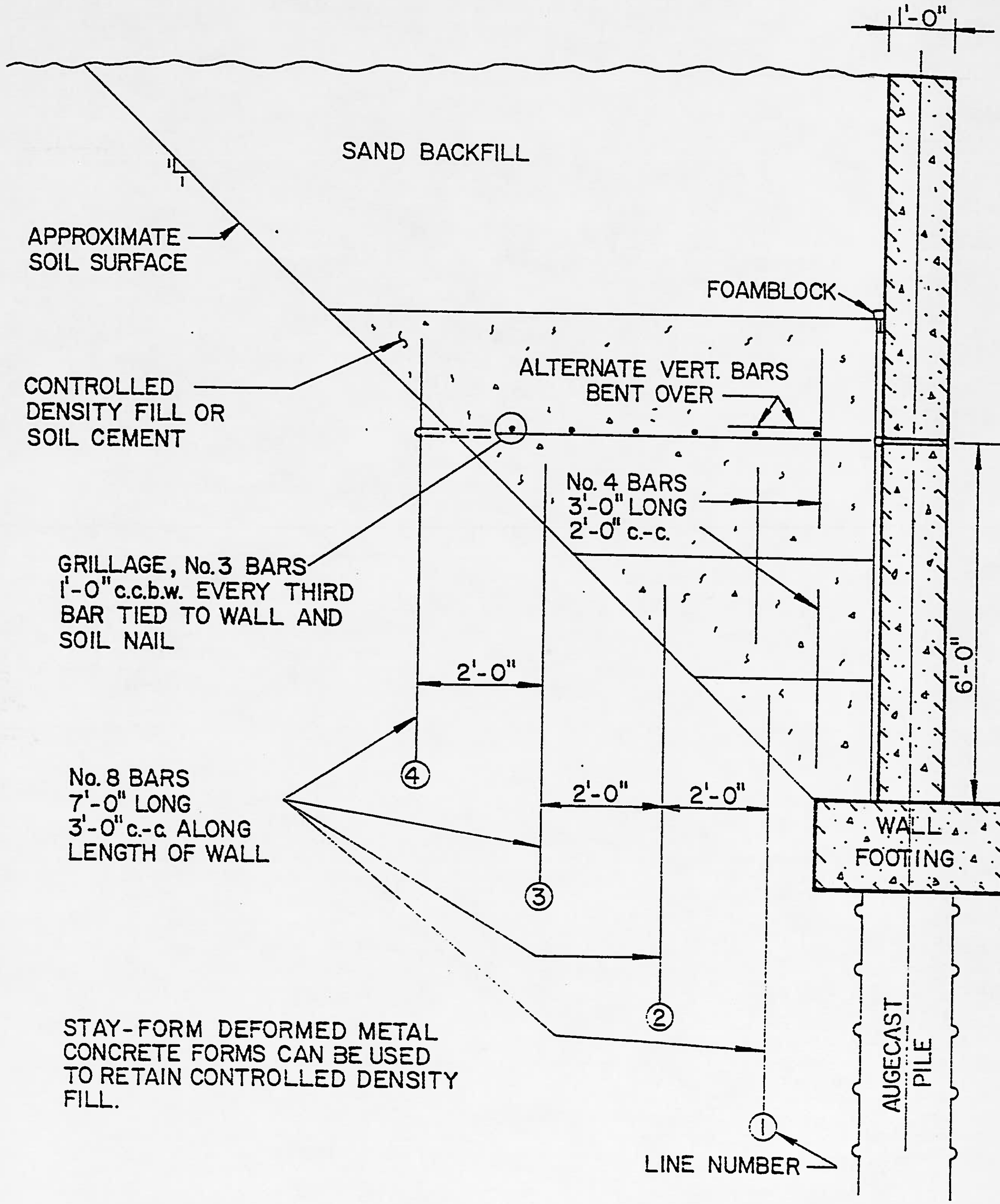


Figure 5. Bootstrap Backfill Design Details

USE OF RAMMED AGGREGATE PIERS IN PLACE DEEP FOUNDATIONS FOR SETTLEMENT AND UPLIFT CONTROL OF BUILDINGS AND RETAINING WALLS

Keith R. Moser, P.E.[1], Michael J. Cowell, P.E.[2] and Kord Wissmann, Ph.D.,P.E.[3]

ABSTRACT

Rammed Aggregate Piers™ are increasingly being used to replace deep foundations for settlement and uplift control of structures. Rammed Aggregate piers are constructed by excavating or augering a cavity, filling it with aggregate, and densely compacting the aggregate using a modified hydraulic hammer and a unique tamper foot. The Rammed Aggregate Piers are eight to forty times stiffer than in-situ soils, and typically provide three times the bearing capacity of in-situ soils by creating a composite layer of improved soil. This high modulus layer is used in a two layer settlement analysis method to estimate footing settlement.

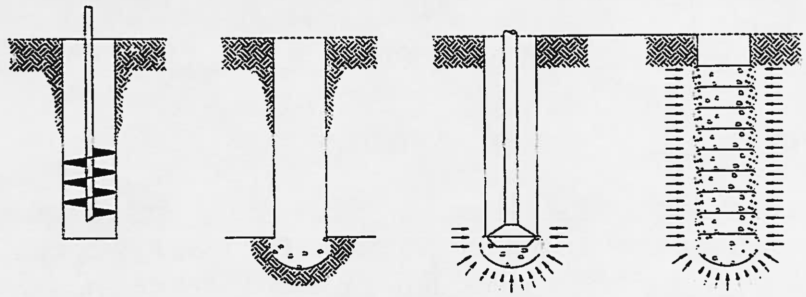
To control uplift, steel plate and threaded rod assemblies are placed within aggregate piers and connected to footings. The tamper applies high lateral stress approaching the passive soil limit, which allows development of high shear capacity on the aggregate pier shaft. Load tests on uplift piers have exceeded 140 kips.

The paper presents two value engineering case studies. In one study Rammed Aggregate Piers saved approximately 25 percent of foundation costs in comparison with caissons socketed into rock for a four-story commercial office and parking garage development. In the second study Rammed Aggregate piers saved more than 25 percent of foundation costs in comparison with minipiles to support retaining walls on a highway project.

INTRODUCTION

Rammed Aggregate Piers are constructed by first removing a volume of compressible material, either by drilling a hole or excavating a trench. A thin lift of opening graded stone is then placed at the bottom of the cavity. The soil at the bottom of the cavity is prestressed and prestrained by ramming the stone with a specially-designed tamper. A very stiff element is then constructed within the cavity using well-graded aggregate placed in thin lifts and highly densified by ramming with the same tamper used for bottom prestressing. The adjacent matrix soils are improved, not primarily by densification, but rather by lateral prestressing. The energy source applies impact ramming action, rather than a vibratory energy, to a 45-degree beveled tamping apparatus that maximizes lateral prestressing of the matrix soil. The buildup of lateral stresses in the surrounding matrix soils develops an over-consolidated soil surrounding each Rammed Aggregate Pier, resulting in a stiffened aggregate pier/matrix soil mass. In addition, the prestressing and prestraining of soils adjacent to the sides of the aggregate pier results in an undulated aggregate pier/matrix soil interface that provides excellent engagement of the aggregate pier with the surrounding soil. The lateral stress buildup

[1] Project Manager, GeoStructures, Inc., 107 Loudoun Street, SE, Leesburg, Virginia 20175
[2] President, GeoStructures, Inc., 107 Loudoun Street, SE, Leesburg, Virginia 20175
[3] Chief Engineer, Geopier Foundation Company, 515 Sunrise Drive, Blacksburg, Virginia 24060



a. b. c. d.

approaches the passive limit of the soil, thereby providing maximum shear strength along the aggregate pier shaft. Figure 1 shows the simple, four-step construction process that illustrates the basic concept of rammed aggregate pier construction.

Figure 1. Typical Construction Process – a. make cavity, b. place clean stone at bottom of cavity, c. densify aggregate and vertically prestress matrix soils beneath to create bottom bulb, d. make undulated-sided aggregate pier shaft with 12-inch typical lifts of graded aggregate, building up lateral stress in matrix soils during construction (Fox and Cowell, 1998).

KEY PROPERTIES OF RAMMED AGGREGATE PIERS

The stiffness modulus of a Rammed Aggregate Pier, k_s, is defined as the ratio of applied design stress divided by the corresponding displacement at the top of pier. The stiffness modulus is confirmed by performing a load test on an individual aggregate pier element at the project site. The load test measures the stress/strain behavior of the aggregate pier installed in the matrix soils.

Stress concentration occurs in the system due to the significance stiffness of the pier relative to that of the matrix soil. The stress ratio is the stiffness of the Rammed Aggregate Pier divided by the stiffness of the surrounding matrix soils. The concentration of the stress on the Geopier element is significant in analyzing its performance in terms of settlement, sliding and shear strength.

The stress on the Geopier element can be visualized by considering the analogy of a stiff spring within a matrix of less-stiff or soft springs. The stiff spring represents the Rammed Aggregate Pier and the less stiff spring matrix soils, respectively, as illustrated in Figure 2. Assuming that the footing and the stiff plate are perfectly rigid, if the footing or plate deflects under load, then the deflection must be the same at the aggregate pier/stiff spring as it is at the matrix soil/soft spring. For this example, a stiff spring constant of 10 and a soft spring constant of 1 result in stress ratio, R_s, of 10. Assuming that the footing/plate deflection equals 1, the load on the stiff spring equals the spring stiffness times the deflection, 10 x 1, or 10. Similarly, the load on each soft spring is 1 x 1, or 1, showing that the stiff spring carries ten times more load than each soft spring. Transferring this analogy to soil mechanics, the stress concentration has been measured, as predicted, in full scale, heavily instrumented footing load tests (Lawton, 1999).

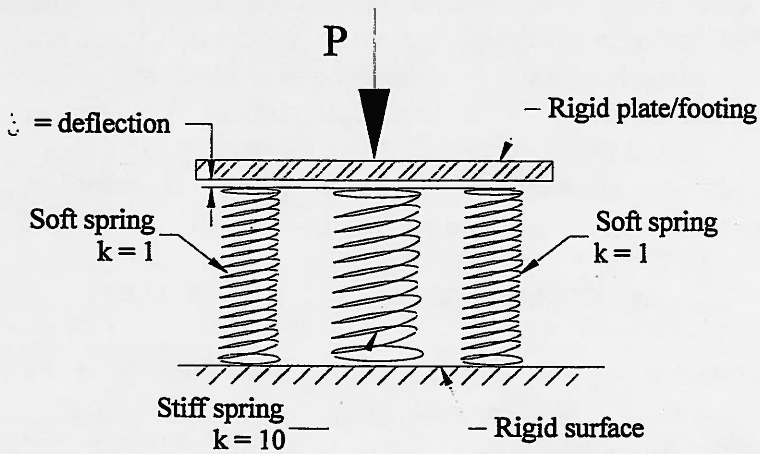


Figure 2. Stiff Spring Analogy (Fox and Cowell, 1998).

<u>The high shear strength</u> of a Rammed Aggregate Pier provides excellent resistance to sliding of footings under lateral loading conditions, such as for buildings with lateral seismic or wind loads, or for lateral retaining walls with earth pressure loads. The internal friction angle in excess of 50 degrees has been measured in full scale direct shear testing (Fox and Cowell, 1998). Combined with stress concentration, the shear strength provided by the Rammed Aggregate Piers substantially increases sliding resistance of footings subjected to lateral loads.

<u>Uplift resistance</u> is developed through shear along the interface between the Rammed Aggregate Pier and the surrounding matrix soil. Since the matrix soils have lower shear strength than Rammed Aggregate Piers, matrix soil shear strength controls uplift resistance. Where uplift resistance is required for a project, an uplift test is typically installed to confirm the design capacity. In early applications of Rammed Aggregate Piers for uplift resistance, when the prestressing effect was not considered in the design, the tested capacity was greater than anticipated by a factor of 4 to 5. The increase in capacity could only be accounted for by including passive earth pressure conditions within the calculations (Lawton, et. al., 1994) as illustrated in Figure 1.d. Rammed Aggregate Piers are more efficient for resisting uplift loads than are driven piles or drilled shafts because of this increase in lateral earth pressure and because of the high aggregate pier/soil friction angle. Further, relatively small deflections are required to mobilize shear strength at the passive soil limit, resulting in improved performance along with greater capacity.

GENERAL SETTLEMENT DESIGN APPROACH

To estimate settlement of footings supported by Rammed Aggregate Piers, the subsurface profile is divided into an upper zone and a lower zone. The upper zone extends from the bottom of footing to one pier diameter below the bottom of the drilled cavity, thus including the depth of prestressing and prestraining. Beneath the upper zone, the lower zone extends to a depth of two times the footing width for isolated spread footings, and four times the footing width for strip footings. Upper zone settlement is estimated by calculating the stress at the top of the aggregate pier and dividing by the pier modulus. For preliminary design purposes, modulus values are selected based on correlation to existing load test data in similar soils. Final design estimates of upper zone settlement are confirmed by full-scale modulus testing at the project site. Lower zone settlement is estimated using conventional soil mechanics approaches in conjunction with elastic or consolidation compressibility parameters. A complete discussion of the methods used for settlement estimate and design is provided by Lawton et al. (1994).

GENERAL SLIDING RESISTANCE DESIGN APPROACH

The first step in determining the sliding resistance of a footing supported by aggregate piers is to calculate the stress on the top of the aggregate piers. Often this has already been done to estimate footing settlement. The total vertical load supported by the pier is calculated by multiplying the contact stress by the sum of the cross-sectional areas of the piers. The total vertical load on the aggregate piers is then multiplied by the tangent of the internal friction angle of the aggregate piers to determine the component of sliding resistance within the aggregate piers. Because of the tendency for the aggregate to dilate during shearing, the angle of the internal friction is greater than 45 degrees and the tangent of this angle is greater than unity. The matrix soils will also provide some sliding resistance, which is calculated in the same way. However, the matrix soil stress is much lower than the aggregate pier stress, so the added resistance of the matrix soils provides a relatively small percentage of the total sliding resistance. Finally, the total sliding resistance is divided by the lateral load applied to the footing to determine the factor of safety.

GENERAL UPLIFT RESISTANCE DESIGN APPROACH

To determine uplift capacity, both the shear resistance of the aggregate pier shaft and the tensile capacity of the uplift anchor assembly, which includes steel plates and threaded rods. First, the shear resistance of the aggregate pier is calculated. Often it is possible to assign uniform soil parameters to the matrix soils in the upper zone, where the uplift capacity is developed. For a soil with given unit weight and drained shear strength, the shear resistance is calculated by assuming that the vertical stress at the middle of the aggregate pier shaft is the average vertical stress throughout the depth of the shaft. This vertical stress is multiplied by the Rankine coefficient of passive earth pressure to determine the average horizontal stress on the shaft of the aggregate pier. This stress is then multiplied by the tangent of the matrix soil's internal friction angle, and that product is multiplied by the surface area of the aggregate pier/matrix soil interface to arrive at the ultimate capacity of the uplift pier.

Once the shear capacity is determined and the working loads are established, allowable stress design techniques are used to size the structural steel elements that make up the uplift anchor assembly. It is important to note that allowable stress design often overestimates the size of the plate required. Therefore, the uplift anchor assemblies are sized based on acceptable load test results.

OFFICE AND PARKING GARAGE, DURHAM, NORTH CAROLINA

Rammed Aggregate Piers were used as foundation support for settlement control in the construction of a four story office building and adjoining parking garage. The cast-in-place concrete structures had typical column service loads ranging from 100 to 500 kips in the office building and 200 to 400 kips in the parking garage. The office building elevator core walls also served as shear walls with net uplift forces that had to be resisted by the foundation. Total settlement of the structure was required to be less than one inch with no more than one-half inch differential settlement. Construction was completed early in 1999.

SITE CONDITIONS

The site is located in Research Triangle Park in Durham, North Carolina, and the geology was of the Triassic Basin Geologic Formation of North Carolina. Upper Zone soils are residual clayey silts, silty clays, and silty sands. Partially weathered rock with $N_{SPT} > 50/6"$ was encountered at depths ranging from 4.5 to 10 feet below the ground surface. The sloping site required cuts and fills up to 15 feet to achieve proposed grades.

PROPOSED DESIGN

Project geotechnical engineers evaluated a variety of foundation alternatives including spread footings on virgin soil or controlled fill with 3,000 psf allowable bearing pressure, and moderate diameter drilled shafts bearing on partially weathered rock with 30 ksf allowable bearing pressure and 3,000 psf skin friction for resisting uplift loads. Shallow foundations were ruled out due to the potential for differential settlement and difficult excavation in the variable weathered rock surface. Structural drawings were prepared with drilled shafts providing the foundation support.

VALUE ENGINEERING PROPOSAL

A large, national design-build contractor was awarded the work, and was interested in evaluating foundation alternatives that could save time and money on the congested site. Potential for drilling overruns existed in the geologic conditions found at the site, and the hard rock would have required a large caisson drilling rig that might have difficulty accessing the site. Having heard about Rammed Aggregate Piers, the contractor investigated this system as an option for their project. GeoStructures, Inc., Leesburg, Virginia, was asked to prepare a value-engineering proposal for the design and installation of Geopier™ aggregate piers.

Initially, preparing the value engineering proposal involved discussions with the project geotechnical and structural engineers regarding the aggregate pier system and how it would be applied to their project. During the period after the geotechnical report was issued and before the project plans were drawn up, the geotechnical engineers had gained some experience with the aggregate pier system. They recognized the potential savings the system could provide, and suggested that they be allowed to review the final Rammed Aggregate Pier design.

One structural concern that arose during design was the net uplift loading on the shear walls. To resist the uplift loads, uplift assemblies were planned for the Rammed Aggregate Piers supporting the shear walls. Shear wall loading consisted of a 230-kip (±) couple that required a uplift anchors at each end of the wall, and the preliminary design included four uplift elements at each end of the shear wall footing. Since the wall was part of the elevator core, the bottom of the wall footing was much closer to the rock interface than the remainder of the footing locations at the site. Since conventional Rammed Aggregate Pier equipment can only drill in soil and not rock, the uplift capacity of aggregate pier elements would be severely limited by depth. As a result, grouted anchors were selected for resisting the uplift loads.

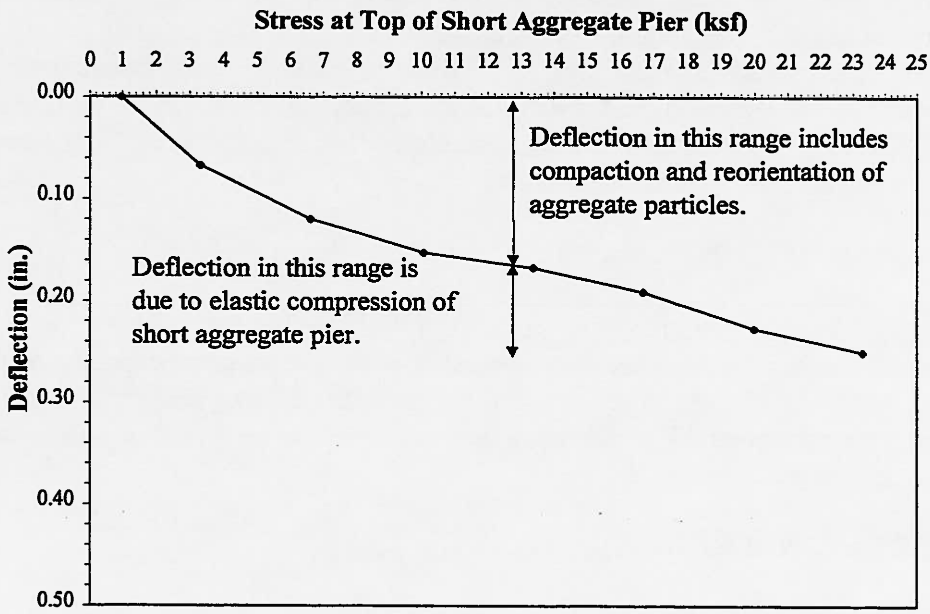
DESIGN ANALYSES

Prior to construction a detailed submittal was required, including analysis of potential settlement, bearing capacity failure due to high bearing pressures, and estimates of uplift anchor capacity. Settlement analyses were performed for two conditions. The first analysis considered the minimum required Rammed Aggregate Pier performance that would satisfy the requirements of one inch total and one-half inch differential settlement. The second analysis showed the anticipated performance based on correlation to other projects in similar subsurface conditions. The following design parameters were agreed upon by GeoStructures and the project geotechnical and structural engineers.

- Matrix soil modulus: 32 pci
- Maximum allowable bearing pressure: 10,000 psf
- Minimum Rammed Aggregate Pier modulus required for one inch settlement: 155 pci
- Anticipated Rammed Aggregate Pier modulus: 310 pci (results in 0.64 inch settlement)
- Maximum estimated Rammed Aggregate Pier design stress: 19,800 psf
- Rammed Aggregate Pier diameter: 36 inches
- Rammed Aggregate Piers to be founded in partially weathered rock with $N_{SPT} > 40$
- No lower zone settlement was included due to the proximity of weathered rock
- Grouted uplift anchor working bond strength in partially weathered rock: 30 psi

Rammed Aggregate Pier design modulus was confirmed by load testing a single, 36-inch diameter aggregate pier element at the project site. Since the test was installed in an area with deeper rock interface, aggregate piers were used as reactions for the load test. The load test was planned for a maximum test load of 100 tons, or about 150 percent of the maximum design stress acting on a 36-inch diameter aggregate pier. During the load test, which was performed in February, the jack equipment was affected by extreme change in ambient temperature, and experienced some leakage that made the maximum planned load unobtainable. However, at 116 percent of the design stress, or about 70 tons, the test pier exhibited far less deflection than anticipated. Stress vs. deflection was plotted, as shown in Figure 3, to determine if the test would be accepted. Typically a maximum safe design stress is taken as the point of inflection or steepening of the stress vs. deflection curve. Since no such inflection point was observed, the design was approved and crews began to install the aggregate piers at the site.

Figure 3. Rammed Aggregate Pier Modulus Load Test Stress vs. Deflection



RESULTING SAVINGS

Since the project required staged backfilling operations, construction was completed in two phases requiring two mobilizations. The tracked equipment used for aggregate pier installation had no difficulty moving around on the tight site, and construction was completed on time for a lump sum, except for the additional mobilization cost. Including that additional cost, the aggregate pier foundation resulted in about 25 percent savings over the drilled shaft alternative. Additional savings, although not quantified, resulted from the efficiency of the Rammed Aggregate Pier installation, and the time savings over the proposed caissons.

HIGHWAY RETAINING WALL, CLINTON, MARYLAND

A major highway widening project in the Washington D.C. metropolitan area required cast-in-place concrete retaining walls up to 14 feet tall for grade separation. The highway could not have lane closures during rush hour, so maintenance of traffic was a crucial part of the proposed construction schedule. Overhead power lines limited the foundation prospects to systems with low-overhead installation capability. Initial project documents included contractor incentives for providing foundation alternatives. Retaining wall construction was completed in late 1997.

SITE CONDITIONS

The site is located in Atlantic Coastal Plain, which locally includes the Potomac Formation consisting of interbedded, typically overconsolidated sands, silts and clays. The Potomac group soils are overlain by Pleistocene Terrace Deposits, which include layers of loose to dense rounded quartz gravel, or "bank run" gravel with varying amounts of silt and clay binder. Portions of the site are also located adjacent to a swampy area, so the near surface soils included recent, soft alluvial deposits including a weight-of-hammer clay layer ten feet thick. High groundwater and swampy conditions were major factors in determining the construction alternatives. Very tight site constraints were also serious considerations.

PROPOSED DESIGN

Original project documents required minipile foundations or an approved alternative. Eight-inch diameter minipiles were designed with compression capacity of 65 kips and uplift capacity of 15 kips to resist overturning loads. Sliding resistance was provided by battering the minipiles. The pile caps were designed to be five feet wide, and could not be enlarged on the narrow site.

VALUE ENGINEERING PROPOSAL

In order to satisfy the project design team, several iterations were required in the proposal process. Rammed Aggregate Piers were initially designed to provide settlement control and sliding resistance from each pier. Overturning resistance was provided by including uplift anchor assemblies in selected piers. Review comments generally favored the proposal, but made additional requirements with respect to the location of aggregate piers and their ability to serve multiple functions of settlement control, sliding and uplift resistance. It was agreed that only those aggregate piers that were located in the front two-thirds of the footing would provide settlement and sliding control. Due to the narrow footing width, that meant the uplift piers could not be used for settlement and sliding control, resulting in additional aggregate piers at closer spacing.

The additional design requirements caused a re-evaluation of construction feasibility, and the Rammed Aggregate Pier installer, GeoConstructors, Inc., decided to drill probe holes to provide a better assessment of the subsurface conditions. The probe holes encountered severe caving in the open graded bank-run gravel. Casing costs were included in the proposal to control the caving, and although the cost had increased after the value engineering proposal review, aggregate pier construction was estimated to be considerably less expensive than the minipile alternative.

DESIGN ANALYSES

A detailed submittal was prepared including estimation of settlement and analysis of sliding and uplift capacity. Providing an efficient design required sixteen design sections accounting for variable wall height and subsurface conditions. Additional consideration was given to the settlement influence from new backfill. On the north side of the site, swampy conditions controlled the design, and aggregate piers had to penetrate the soft clay. The south side had more favorable design conditions controlled by the bank-run gravel. Due to the layered subsurface profile, Upper and Lower Zone parameters for settlement analyses were correlated to minimum average N_{SPT} values. Uplift capacity was estimated by calculating the shear resistance contribution from each soil layer in the Upper Zone. Sliding resistance was calculated using the stress concentration and frictional resistance of the Rammed Aggregate Piers, and included the frictional resistance of the matrix soil. Footing bearing pressures ranged from 1,500 to 3,000 psf, which is considerably lower than typical Rammed Aggregate Pier foundations. The following design values were used. They were considered to be very conservative, since the project would be difficult to construct and would require a substantial financial risk on the part of the Rammed Aggregate Pier designers and installers.

- 36-inch diameter Rammed Aggregate Pier capacity: 65 kips
- 36-inch diameter Rammed Aggregate Pier capacity in soft clay: 45 kips
- 30-inch diameter Rammed Aggregate Pier capacity: 50 kips
- 30-inch diameter Rammed Aggregate Pier capacity in soft clay: 30 kips
- Rammed Aggregate Pier modulus: 180 pci
- Rammed Aggregate Pier modulus in soft clay: 80 pci
- Matrix soil modulus: 14 pci
- Lower Zone modulus, range of values (Bowles, 1988): 50 to 85 tsf
- Internal friction angle for uplift design: 28° silt, 20° clay
- Rammed Aggregate Pier internal friction angle for sliding: 37°, very conservative
- Maximum allowable settlement: one inch total, one-half inch differential
- Minimum factor of safety for sliding resistance: 1.5
- Minimum factor of safety for uplift resistance: 2.0

Actual load test results turned out to be far better than the values used for settlement control and uplift resistance analyses. Tested aggregate piers achieved modulus values of 240 pci in fair soil and 140 pci in very soft soil. Capacities were also increased to 110 kips and 90 kips, respectively. To reduce the risks associated with construction on a very soft subgrade in tight space constraints, the design submittal was revised to account for the improved performance and the number of Rammed Aggregate Piers was reduced. The total number of Rammed Aggregate Piers was revised and the final design was constructed.

RESULTING SAVINGS

Rammed Aggregate Pier installation was completed on time, in spite of the very challenging construction conditions. Caving soils were made worse during the rainy Fall season. The site was at a low point along the right of way, and any rain that infiltrated the soil would remain in the bank-run layer. Casing helped, but casing in the tight working conditions slowed production. Concrete crews were still able to work directly behind the Rammed Aggregate Pier crew, and often placed forms and concrete on the day following the aggregate pier installation. In the high pressure environment of highway construction under adverse site conditions, the Rammed Aggregate Pier system provided more than a $250,000 savings over the minipile alternative.

SUMMARY OF BENEFITS

- Rammed Aggregate Piers provide a cost-saving alternative for foundation support.
- Rammed Aggregate Piers can be used to replace deep foundation systems such as caissons and piles for settlement, uplift and lateral sliding applications.

REFERENCES

1. Fox, N. S., and M. J. Cowell, Geopier Foundation and Soil Reinforcement Manual, Geopier Foundation Company, 1998, 115 pages.
2. Lawton, E.C., seismic study
3. Lawton, E.C., N. S. Fox, and R. L. Handy, "Control of Settlement and Uplift of Structures Using Short Aggregate Piers," Proceedings of In-Situ Deep Soil Improvement Conference, ASCE National Convention, 1994, pp. 121-132.
4. Bowles, Joseph E., Foundation Analysis and Design, McGraw Hill, 3rd Edition.

TEACHING VALUE ENGINEERING IN A GEOTECHNICAL CURRICULUM

Mark T. Bowers[1]

ABSTRACT

Miles, an early proponent of value engineering, defined it as "a problem-solving system implemented by the use of a specific set of techniques, a body of knowledge, and a group of learned skills. It is an organized creative approach that has for its purpose the efficient identification of unnecessary cost. When applied to products, this approach assists in the orderly utilization of better approaches, alternative methods, newer processes, and abilities of specialized suppliers. It focuses engineering, manufacturing, and purchasing attention on one objective--equivalent performance for lower cost."

A typical geotechnical graduate curriculum is laid out showing the already crowded nature of the program. The point is made that value engineering needs to be taught throughout the geotechnical curriculum rather than in a single course.

The history of value engineering from its development in World War II to the present is summarized. The schools of thought (alpha and beta schools) within the value engineering profession are discussed. It is noted that the alpha school dominates United States' practice due to the strong influence of federal government agencies and departments. The process of function analysis, fundamental to the value engineering process, appears to be missing from today's application of value engineering. Reasons for this are discussed.

To be of most worth, value engineering needs to be implemented as early in the design process as possible. Five questions to prepare the mind for the value analysis approach are discussed. The question "What else would do the job?" is seen as being comprehensive and penetrating. Unless this phase of the work is effectively completed, it cannot be hoped that the final product will have more than an average degree of value.

Some thoughts on implementation of teaching value engineering in the geotechnical curriculum are presented.

[1]Department of Civil & Environmental Engineering, P.O. Box 210071, University of Cincinnati, Cincinnati, Ohio 45221-0071

BACKGROUND

The author states at the outset that this, without doubt, has been the most challenging paper that he has attempted to write. Why is that the case? Part of the difficulty stems from limited literature, another part from limited exposure to the concept (perhaps in the form of vocabulary), and another part due to a change in focus in the construction industry since the author entered the field after completing his academic instruction. The topic was seen as a challenge and as a demand, really, in that it was necessary to face this new world and prepare current students to read about, understand, and incorporate value engineering into their work.

Academia continues to prepare young engineers for the field by teaching fundamental principles, the state-of-knowledge, and then state-of-the-art practices. Due to time constraints and other mandated coursework, some topics are delayed until graduate school. Even then, there are demands on the credit loads taken. For example, in some graduate geotechnical programs leading to an MS thesis, only 36 quarter credits (approximately 12 classes) of course work are required beside the thesis. Of these, approximately one-half may be required in the form of:

Consolidation & Settlement
Shear Strength of Cohesive & Cohesionless Soils
Foundation Engineering II: Structural Design of Footings*
Advanced Foundation Engineering: Piles & Drilled Shafts

(*assuming the student did not complete this course as an undergraduate)

Supporting course work for graduate civil engineering students that may be required include:

Advanced Mechanics of Materials
Risk & Reliability of Civil Engineering Structures
An Advanced Course in Mathematics

Assuming for the moment that a student had only had basic Soil Mechanics and a course in Foundation Engineering as an undergraduate, it can be seen that half of the graduate course work could be mandated. The other half of the course credit requirements is usually fulfilled through:

Seepage: Flow Through Porous Media
Earth Dam Design
Slope Stability Analysis
Instrumentation
Rock Mechanics
Lateral Earth Pressures
Courses from Geology

It seems apparent that a separate course in value engineering may not be an appropriate choice. Rather, value engineering must be taught throughout the curriculum in appropriate courses. It is much the same with ethics. You cannot teach ethics in one class and hope that it will sink in and stick with the individual. Ethics must be taught throughout the curriculum. It must be emphasized and lived by the instructors. There must be assigned readings on the topic. There must be opportunities to role play, to brainstorm, to consider the issues from actual field experiences. Then there is a hope that it will be absorbed by the students.

During his undergraduate days the author heard that "an engineer was a person who could design a needed element for $1.00 which any damned fool could design for $2.00." The issues taught included principles of math and science and how they might be applied to the solution of the geotechnical problems; principles of mechanics; and principles of economics. Added to this was the need for experience and judgment. The concepts of practicality, constructability, and common sense were stressed. Each one of us left that undergraduate education experience knowing that we had only begun to learn. We could not learn all we needed from books and we probably never would.

The author believes it is appropriate to note that as an educator in the geotechnical engineering arena, he has not read the words "value engineering" within one of our speciality area texts...until this year. It appeared in Koerner's fourth edition text on Designing with Geosynthetics (1). In speaking of the large number of geogrid-reinforced retaining walls that have been constructed since 1988, Koerner wrote:

> ...[their] entry into the public sector has been through value engineering. This is the concept whereby the low-bid general contractor offers the state or federal agency an option for some particular segment of the project, for example a geogrid-reinforced wall in place of a conventional reinforced-concrete wall or steel-reinforced segmental wall. If the option is acceptable to the agency, the financial savings from the difference in the costs of the two different kinds of walls is shared equally between the agency and the contractor. It is a very effective vehicle for the introduction of new products and concepts like geogrid (and geotextile) reinforced walls.

The author puzzled over that definition. How was this different from the education I had received and that which I have been giving my students to this point? The author has stressed to his students that no design is finished until the project itself is complete and shown to be functioning as was expected. It is known that in geotechnical engineering, our designs may have to be modified once the ground is excavated and the soil profile is open to view. Our geotechnical engineering consulting firms are hard pressed when it comes to submitting bids for new work. The competition is keen. At our universities, professors continue to stress "economics" as a major role in a solution. What then of "value engineering"? How is it different from the training we have been giving to date? How might we approach our classes differently after having attended this conference?

DEVELOPMENT OF VALUE ANALYSIS OR VALUE ENGINEERING

Value engineering developed during World War II in the United States. Palmer et al. (2) note the following as concerns the birth of value engineering:

> It began as a search for alternative product components, a shortage of which had developed as a result of the war. Due to the war, however, these alternative components were often equally unavailable. This led to a search not for alternative components, but to a means of fulfilling the function of the component by an alternative method. It was later discovered that this process of 'function analysis' produced low-cost products without reducing quality and, after the war, the system was maintained as a means of both removing unnecessary cost from products and improving design. The process of value engineering based on analysis was therefore born.

As I embarked upon my own understanding of value engineering, I found the early work of Miles (3) to be very helpful. In the preface to his text Miles cites an expereince he had with his son:

> My son Robert, while 150 feet high on the magnificent four-mile-long Bay Bridge at Annapolis, looked down upon two dozen assorted craft in the water below. Some carried pile drivers, some cement mixers, some steel handlers, and some assorted materials--all working at building a parallel span. He said, 'I can't conceive of men being able to build such a bridge,' and then added, 'And the men who get the job done are no smarter than you and I.' Then his next comment struck home to me: 'It's the system that does it.'
>
> How often, when observing the startling results in ending costs that have brought no benefits, accomplished by people using the value analysis system, have I thought, 'Truly the results from

fully using the right system at the right time almost transcend understanding!'

Miles indicates that value analysis was created for one specific purpose--"efficiently identifying unnecessary cost before, during, and after the fact." He continues by writing that value analysis "is a disciplined action system, attuned to one specific need: accomplishing the functions that the customer needs and wants...at the lowest cost."

One of the fundamental issues about value engineering is that it is not "just traditional cost cutting" by another name. Palmer et al. (2) summarize the history of value engineering from its birth in World War II through 1995. Recapping this work quickly, they note that at the start of the 1970's "value engineering had developed into a three-pronged technique of function definition based on the verb-noun, function evaluation based on the lowest cost to achieve function, and creativity based on brainstorming. The means of organizing the technique into a systematic framework was the job plan." This status is illustrated in Figure 1.

The Job Plan			
Anytime	Function Definition	Verb-Noun	Team
	Function Evaluation	Lowest Cost to Achieve Function	
	Creativity	Brainstorming	

Figure 1. Value Engineering At the Start of the 1970s (Source: Palmer et al. (2).)

Palmer (4) completed a doctoral dissertation on the development of value engineering in the construction industry and found two major changes in its theory.

> First was the introduction of the 40 hour workshop as the method of carrying out a value engineering study. Second was the development of two separate schools of thought on how value engineering should be implemented.

These two schools of thought, termed "alpha" and "beta", are illustrated in Figures 2 and 3. These two figures represent

<table>
<tr><td colspan="4">The Forty Hour Workshop</td></tr>
<tr><td colspan="4">The Job Plan</td></tr>
<tr><td rowspan="3">35% Design Stage</td><td>Function Definition</td><td>Verb-Noun</td><td rowspan="3">External Team</td></tr>
<tr><td>Function Evaluation</td><td>Cost-Worth Ratio</td></tr>
<tr><td>Creativity</td><td>Brainstorming</td></tr>
</table>

Figure 2. The Alpha School of Thought in Value Engineering (Source: Palmer et al. (2).)

<table>
<tr><td colspan="4">The Forty Hour Workshop</td></tr>
<tr><td colspan="4">The Job Plan</td></tr>
<tr><td rowspan="3">Early Design Stage</td><td>Function Definition</td><td>Verb-Noun
FAST</td><td rowspan="3">Design Team</td></tr>
<tr><td>Function Evaluation</td><td>Value Mismatch</td></tr>
<tr><td>Creativity</td><td>Brainstorming</td></tr>
</table>

Figure 3. The Beta School of Thought in Value Engineering (Source: Palmer et al. (2).)

the position of value engineering in the construction industry in the U.S. as of 1995.

Palmer et al. (2) conducted an appraisal of these two schools of thought on value engineering by investigating current theory and practice. They surveyed value engineering consultants operating in the construction sector. Among their conclusions are the following:

a) Value engineering is a representation of practice. Value engineering programs were developed first and were then followed by texts on the subject. One of the outcomes of this is that value engineering theory in construction has developed without the benefit of academic scrutiny.

b) The main area where the latter is noticeable is in the area of function analysis, which appears, in practice, to have broken down completely. As far as the practitioners are concerned this is not important because they can achieve success in terms of cost reduction without function analysis. However, the presence of function analysis is important to the Society of American Value Engineers (SAVE), which feels that it is function analysis that makes value engineering what it is and that without it value engineering is simply cost cutting.

c) The development of value engineering in construction in the United States has been primarily from the U.S. Department of Defense. Other large users are the federal government, the USEPA, and the General Services Administration, all using the same system developed by the Department of Defense. The problem is that this system does not use function analysis.

d) Function analysis is extremely difficult and takes a great deal of time and expertise. In addition, at the 35% design stage [the alpha school used by DOD], it may be too late for an effective function analysis.

The Construction Management Committee of the ASCE Construction Division wrote a white paper on constructability and constructability programs (5). Their abstract includes the following:

> To receive maximum benefits, the construction input, or constructability, has to be started at the earliest stages during the conceptual planning stages. ...the integration of experienced construction personnel into the earliest stages of project planning as full-fledged members of the project team will greatly improve the

chances of achieving a better quality project, completed in a safe manner, on schedule, for the least cost.

Two figures from this white paper illustrate how constructability effort can result in the largest payoff during the early stages of a project with overlapping design and construction. Figure 4 depicts the ability of constructability to influence costs. For example, if constructability is initiated during conceptual planning, the ability to influence final costs over the project life is great. Figure 5 shows many of the factors of constructability and their relationship to other phases of the work in a typical large-scale project.

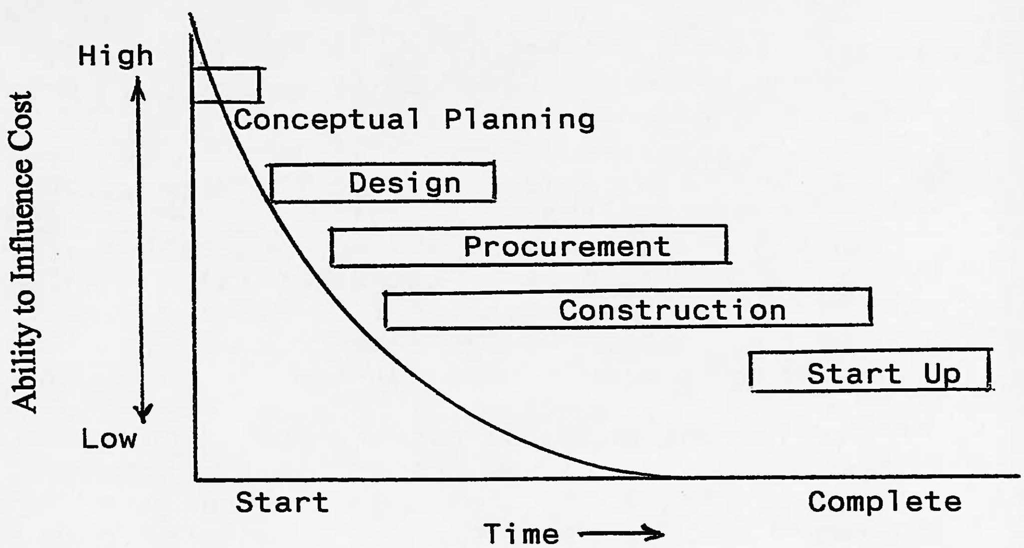


Figure 4. Ability of a Constructability Program to Influence Final Cost Over the Project Life (Source: (5).)

IMPLEMENTATION OF VALUE ENGINEERING IN THE CURRICULUM

It is appropriate to ask ourselves whether we currently practice value engineering or not. Let's look again at Miles' definition (3):

> Value analysis [or value engineering] is a problem-solving system implemented by the use of a specific set of techniques, a body of knowledge, and a group of learned skills. It is an organized creative approach that has for its purpose the efficient identification of unnecessary cost, i.e., cost that provides neither quality nor use nor life nor appearance nor customer features.

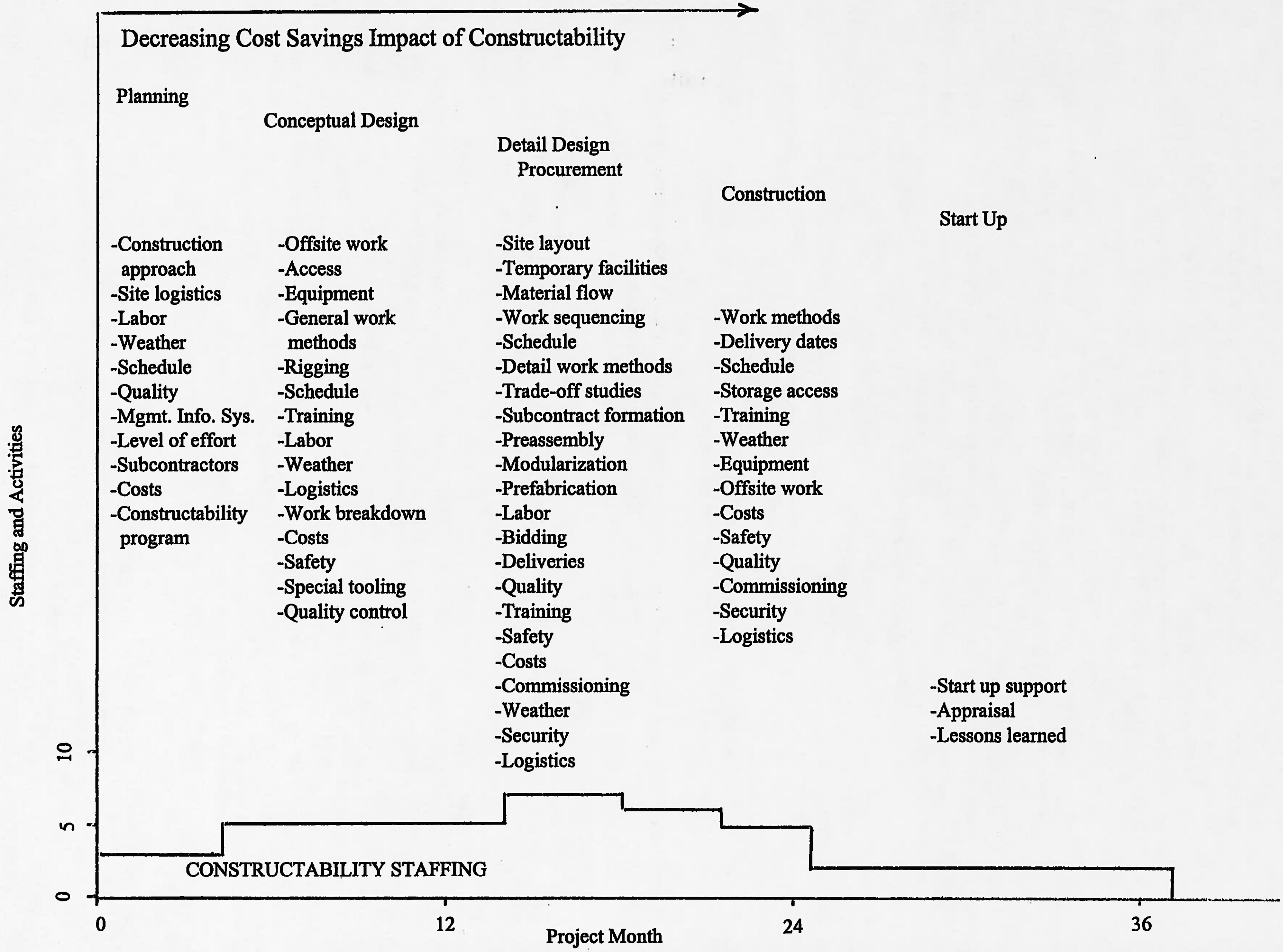


Figure 5: Typical Large-Project Constructability Profile (Source: (5).)

When applied to products, this approach assists in the orderly utilization of better approaches, alternative methods, newer processes, and abilities of specialized suppliers. It focuses engineering, manufacturing, and purchasing attention on one objective--equivalent performance for lower cost. Having this focus, it provides step-by-step procedures for accomplishing its objectives efficiently and with assurance.

What do I perceive my role to be as a mentor in implementing value engineering in my specialty area of geotechnical engineering? It is more than just teaching sound engineering fundamentals just as it is more than seeking an economic solution. I have learned that I must adjust my way of thinking when I consider teaching value engineering in our curriculum. Miles (3) gave an approach, which he acknowledged as being oversimplified, to prepare the mind for the value analysis technique. He listed five questions:

a) What is the item or service?
b) What does it cost?
c) What does it do?
d) What else would do the job?
e) What would that alternative cost?

Approaches to answer each of the five questions are then given. Objective data can be used to answer (a) while effective and meaningful cost data may be used for (b). Question (c) must be answered objectively if one is to establish sound criteria for decision making. Question (d) is seen as "comprehensive and penetrating" to use Miles' words. He continues:

> The comprehensiveness of the answer to that simple question governs, to a high degree, the effectiveness and the grade of value work being done. No matter how skilled the searcher and no matter how diligent and creative his search, there will always remain alternatives which he did not bring into focus, many of which would have accomplished the total performance reliably at very much lower cost. To ensure good value content, many of the special techniques and much of the special knowledge of value analysis must be used in getting substantial answers to this question. The approach here is to search intensively for alternatives by:
>
> Studying handbooks
> Perusing trade literature
> Telephoning people who might have pertinent information
> Writing to specialists and to companies who might

know of effective alternatives

Focusing intense creativity sharply on the precise task to be accomplished

Refining the results of these creative sessions and searching further for additional information

Unless this phase of the work is effectively and penetratingly done, it cannot be hoped that the product will have more than an average degree of value.

The author has tried to implement this approach in his teaching of graduate course work this academic year. He has particularly initiated efforts in this vein in the homework assigned and in the research work associated with theses. This coming academic year the author will promote the techniques associated with value engineering in a senior-level geotechnical capstone design course which he will be teaching with an emeritus faculty member and a geotechnical consultant. The five questions noted above will be utilized in examining retaining structures, foundations, and remedial action techniques. The case histories being presented today and included in the proceedings of this conference will be used as a part of that work and presentation in the classes.

CONCLUSION

Value engineering is a crucial part of the geotechnical engineering curriculum. It is more than just cost cutting; it is a change in the way we approach our problems. It is a problem-solving techanique that particularly examines and analyzes function. It is an effective vehicle by which new products and concepts may be introduced to our field of work. It is essential that the concept be integrated into the geotechnical curriculum through a cooperative effort of educators and industry professionals working together.

REFERENCES

1) Koerner, R.M., Designing with Geosynthetics, 4th edition, Prentice Hall, Upper Saddle River, New Jersey, 1998, 761 pages.

2) Palmer, A., J. Kelly, and S. Male, "Holistic Appraisal of Value Engineering in Construction in United States," Journal of Construction Engineering and Management, Vol. 122, No. 4, 324-328, December 1996.

3) Miles, L. D., Techniques of Value Analysis and Engineering, 2nd edition, McGraw-Hill Book Company, New York, 1972, 366 pages.

4) Palmer, A., "An Investigative Study of Value Engineering in the United States of America and Its Relationship to United Kingdom Cost Control Procedures," PhD Thesis, Loughborough University of Technology, Loughborough, England, 1992.

5) The Construction Management Committee of the ASCE Construction Division, "Constructability and Constructability Programs: White Paper," Journal of Construction Engineering and Management, Vol. 117, No. 1, 67-89, March 1991.

MECHANICALLY STABILIZED EARTH SEGMENTAL BLOCK UNIT RETAINING WALL IN A RIVERINE ENVIRONMENT: A VALUE ENGINEERING SUCCESS CASE HISTORY

Mark S. Meyers, PhD, PE[1]

ABSTRACT

Channel improvements for Stage 2B of the $97 million Rochester, Minnesota Flood Control Project would have impacted three greens in a golf course. A mechanically stabilized earth wall (MSEW) was selected for use in lieu of a conventional T-wall or anchored sheetpile wall for constructibility, cost, and aesthetic reasons. Excavation for a T-wall would have damaged the greens, closing the course for an additional season; this was not satisfactory to the local sponsor. The sheetpile wall was determined to lack the aesthetic quality required for the golf course setting. A Value Engineering Study in 1990 indicated a potential savings of $640,000 if a MSEW was used.

The St. Paul District had not yet utilized a MSEW at the time the plans and specifications for this project were being prepared (1991), much less in a riverine environment. Information was not available regarding performance and durability of MSEW products over time, especially in a riverine environment in a cold region; as such, MSEWs were not used. Factors considered to be potential problems included: connection strength; creep of reinforcement materials; movement of fines during rising and falling river stages; freeze-thaw characteristics of the segmental block units; and damage to the segmental block units from large trees or ice during flood events.

This paper discusses the selection, design, and construction of the MSEWs. Selection topics include the VE Study and internal technical decisions. Design topics include sliding, bearing capacity, seepage, global and compound stability, internal stability, and preparation of the contract documents for the MSEWs. Construction topics include submittal and shop drawing review, construction sequencing and timing, and lessons learned. A comprehensive discussion of cost comparisons between wall systems and advantages and disadvantages of MSEWs is included. This paper is directed towards project planners and government and private designers of earth retaining structures.

INTRODUCTION

The St. Paul District of the U.S. Army Corps of Engineers is regularly involved in major flood control projects in Minnesota, North Dakota, and Wisconsin. Project features typically include concrete and sheetpile earth retaining structures, which can be costly and may not provide an aesthetically pleasing project feature. A flood control project in Rochester, Minnesota, provided the agency with an initial opportunity to design and construct several mechanically stabilized earth wall (MSEW) systems utilizing geosynthetic reinforcement and

[1] US Army Corps of Engineers, St. Paul District, St. Paul, Minnesota 55101-1638

segmental concrete block units (blocks). This paper discusses the selection process, a value engineering study, design of the MSEW, presentation of the contract documents for bidding purposes, construction costs, construction of the MSEW, and lessons learned.

PROJECT DESCRIPTION

The Rochester Flood Control Project was a 10-stage, $97 million flood control project, designed by the St. Paul District, consisting of channel improvements and recreation features along the South Fork Zumbro River and two tributaries, Bear Creek and Cascade Creek. Design of the project was initiated in 1987 with passage of the Water Resources Development Act of 1986. The project was completed and turned over to the City of Rochester, the local sponsor, in September 1995.

Stage 2B of the Rochester project consisted of channel improvements through Soldier's Field Golf Course. The channel improvements, required to optimize the hydraulic efficiency and capacity of the channel, include widening and deepening the existing channel and placing riprap and bedding on the channel side slopes as erosion protection. Recreation and aesthetic features in Stage 2B consist of a serpentine channel alignment, four pedestrian bridges, and several weirs along the river. One of the weirs is required to create a pool as a source of water for the golf course irrigation system.

Soldier's Field Golf Course, owned and operated by the City of Rochester, is an 18-hole course with a course rating of 67.3. Three of the greens and one tee box are located immediately adjacent to the river. The construction activities required that the golf course be closed during construction. The course superintendent indicated that the construction of new greens, or the repair of the existing greens damaged by the construction activities, would require the course to be closed for one additional season to maintain the course rating. Loss of revenue for more than one season was considered to be unacceptable to the City of Rochester; therefore, a design constraint on the project was that the golf course could remain closed for one season only. All construction activities, including new sod, seeding, and the construction and/or repair of greens and tee boxes, were required to be completed by September of the year of construction.

ALTERNATIVES CONSIDERED

The extent of the channel widening and the geometry of the trapezoidal channel required to convey the 200-year design discharge of 16,800 cubic feet per second (cfs) would have disturbed the greens on the first, fifth, and sixth holes, and the tee box on the sixth hole, of the golf course. Earth retaining structures were required near the greens and tee box to allow the design discharge to pass through this reach of the river without creating water surface elevation increases upstream of the project, and to resist velocities of 8 to 9 feet per second (fps). A conventional concrete retaining wall system with architectural treatment and an anchored sheetpile wall system with a concrete cap were considered to be acceptable alternatives.

A concrete retaining wall would have been approximately 25 feet high for most of the length of the wall to meet frost protection and bearing capacity requirements, requiring a footing width of approximately 20 feet behind the front face of the wall. The excavation and required excavation slopes for this structure would have impacted the greens. The anchored sheetpile wall system would have functioned satisfactorily without affecting the greens, but lacked the aesthetic quality requested by the City of Rochester.

Although MSEWs were considered for use on several St. Paul District projects, including earlier stages of the Rochester project from 1987 to 1990, the District had not yet designed such a system. Flood control projects are typically designed for a minimum 100-year design life, requiring all critical features of the project to function for the life of the project. Earth retaining structures are generally considered to be a critical project feature with regard to function, providing a combination of earth retention and erosion protection.

Since detailed information was not available in 1991 regarding the performance and durability of geogrid reinforcement and block materials, especially in a riverine environment, MSEWs were not considered for use in the St. Paul District. Technical factors considered to be potential problems included connection performance, creep of geosynthetic materials, freeze-thaw characteristics of the block materials, damage to the blocks from debris in the river during high flows, and movement of fines from behind the wall during rising and falling river stages.

The anchored sheetpile wall system was initially selected for use on Stage 2B. This selection was based on the minimal impact of the construction of the sheetpile wall on the golf course features. The wale system was proposed to be placed behind the sheetpile wall to remove a non-aesthetic visible feature of the wall system. The concrete cap was used to soften the effect of the anchor assemblies on the front of the wall system. Figure 1 presents a schematic cross-section of the proposed anchored sheetpile wall system.

VALUE ENGINEERING

The Corps of Engineers is required to complete value engineering (VE) studies of completed designs of projects with high construction costs, focusing on project features with the highest costs. The VE study for Stage 2B of the Rochester Project was completed in July 1990. The VE study covered the entire Stage 2B project reach and considered VE alternatives to project features such as: riprap gradations; eliminating the concrete cap from the sheetpile retaining walls; alternatives to handrail, an expensive project feature at $75 per lineal foot; modifying the hydraulic design; and the earth retaining structure type to be used in the areas of the impacted golf course features.

The FAST Diagram, a critical component of a VE study, for the Stage 2B VE Study is presented in Figure 2 (USACE, 1990a). The retaining walls are included as a project feature related to excavation. The verb-noun combination for excavation was determined to be "create space"; the excavation of the channel is required to provide additional hydraulic flow area to pass the design flood discharge. The primary verb-noun combination for the retaining wall project feature was determined to be "retain earth". A secondary verb-noun combination for the retaining wall project feature was determined to be "maintain space". The retaining walls are required to retain the soil near the greens to provide (maintain) a maximum hydraulic flow area in locations where the trapezoidal hydraulic cross-section cannot be maintained without extensive damage to existing features adjacent to the channel (the greens and tee box).

The anchored sheetpile wall systems were proposed to be located from Station 255+64 to Station 259+75 on the right bank (looking downstream), from Station 255+73 to Station 261+43 on the left bank, and from Station 264+40 to Station 267+15 on the right bank, a total wall length of 1,256 lineal feet. The wall section included PZ27 sheetpile with a concrete fascia cap. Soil anchors were used to minimize excavation in the golf course. A decorative handrail, used throughout the Rochester Project, was proposed for used as fall protection along the entire length of the three wall segments. The cost of the proposed anchored sheetpile wall system, including the PZ27 sheetpile, the wale system, the concrete cap, the soil anchors, handrail, excavation, backfill, topsoil, and seeding, was estimated to be $1,270,300.

The VE alternative proposed replacing the anchored sheetpile retaining wall systems through the golf course with "modular masonry walls using a geogrid tieback" (USACE, 1990a). The VE study team recommended controlling seepage with an aggregate drain placed immediately behind the segmental block wall. Wrapping the aggregate drain in a geotextile material was also proposed to control loss of fines from behind the wall during rising and falling river stages. The expensive handrail was proposed to be replaced with a colored chainlink fence. The cost of the proposed MSEW, including segmental block units, geogrid reinforcement, excavation, backfill, offsite disposal of excess excavation, bedding for riprap, riprap, chainlink fence, topsoil, and turf establishment, was estimated to be $542,511. A schematic of the MSEW proposed by the VE study team is presented in Figure 3.

The financial analysis of the VE alternative indicated a proposed savings of $727,789. The St. Paul District reduced the proposed savings to $640,000 to account for unit cost uncertainties and the effort required to: become familiar with the design and specification of MSEWs; design the MSEW; prepare contract drawings for the MSEW; and prepare construction technical specifications for the segmental block units, the geogrid, and the geotextile. These activities were new to the St. Paul District. Further design efforts included conventional geotechnical analysis of the MSEW, such as sliding, overturning, bearing capacity, global stability, and design of the aggregate drain to function as both a drain and a filter.

At the time of the VE study, the use of MSEWs was becoming more popular and cost effective in both the private and public sectors, and detailed information addressing the technical concerns the St. Paul District had was becoming widely available. As the data base for these wall systems continued to grow, all or most of the technical concerns were quickly becoming invalid. The blocks are cast of air entrained concrete; a concrete compressive strength of 4,000 psi can be provided. Manufacturing and testing of the blocks and the geogrid are controlled by ASTM and other standards. Movement of fines during fluctuating river stages can be prevented or minimized through the use of a geosynthetic filter behind an aggregate drain, which is used to control seepage concerns; the aggregate drain can also be designed to function as both a drain and a filter. The consequences of movement of MSEWs are not as significant as compared to the use of conventional wall systems in channel work where there are buildings, foundations, and bridges adjacent to the wall system. MSEWs are flexible systems capable of withstanding and accommodating movements due to settlement, frost heave, and other environmental factors. Most importantly, any failure of the MSEW in the golf course environment would cause additional flooding only in the golf course, and would not result in loss of life or significant damage to property.

Since the technical concerns were on the verge of being solved, and, more importantly, the potential for loss of life was small if the MSEW failed, the risk of using the MSEW was low compared to the potential cost savings. A command decision was made to incorporate a MSEW into Stage 2B of the Rochester Flood Control Project.

DESIGN OF THE MSEW

The MSEW was designed by the St. Paul District Geotechnical Design Section. The design analyzed both the external stability and internal stability of the MSEW. External stability, which governs the required length of geogrid reinforcement, includes overturning, sliding, bearing capacity, seepage, and global and compound stability. Internal stability determines the required spacing of the geogrid reinforcement and may govern the required length of the geogrid.

The design also evaluated concerns regarding hydraulic effects, such as saturation and drawdown of the water level in the backfill, floating debris and ice impacts against the blocks, and a filter system to prevent or minimize the migration of backfill material through the aggregate drain and the block joints. A 250 psf surcharge was applied to the ground surface behind the wall to account for maintenance vehicle use or minor grading changes that might occur during the life of the project. A typical wall section for an exposed wall height greater than 5 feet is presented in Figure 4.

Geotechnical borings advanced in the MSEW segment reaches indicated the foundation soils, the reinforced backfill, and the retained earth soils generally would consist of relatively free-draining SP or SP-SM soils with less than 10 percent passing the No. 200 sieve. A transient seepage analysis compared the rate of rise and fall of the river level with the corresponding lag in groundwater level. The analysis indicated that the groundwater in the immediate vicinity of the wall would lag the drop in the river level by approximately two feet. This 2-foot head differential was applied to the water level behind the end of the reinforced backfill for the full range of river elevations checked when analyzing the external stability of the wall. To incorporate this head differential, sliding and overturning analyses were performed independently of the procedures identified in geogrid manufacturer technical references. Full active lateral earth pressures were applied on the driving side of the structure, with at-rest pressures applied to the resisting side. The coefficient of friction at the base of the geogrids was selected at 80 percent of the friction angle of the soil.

Bearing capacity was computed using Terzaghi's bearing capacity equation, with bearing capacity factors applied to account for eccentricity, ground slope in front of the wall, and the angle of the resultant load on the base of the structure. The base of the structure was selected as the elevation of the lowest geogrid with a width equal to the length of the lowest geogrid. Global slope stability was evaluated using Spencer's slope stability procedure, applying the 2-foot seepage head differential to various river elevations. The most critical stability condition occurred at low river levels. For this analysis, the entire reinforced zone was modeled as a rigid block of high shear strength material to force the automatic search for the critical failure surface behind this zone (we now know that compound stability analysis, where the search is conducted through the reinforced zone, can result in a more critical failure surface). Results of these analyses indicated that slope stability governed the required length of the geogrid reinforcement for walls greater than 14 feet in height and bearing capacity governed for walls less than 14 feet in height. Geogrid lengths were selected such that the length was equal to or greater than 70 percent of the design wall height.

Internal stability was analyzed using the tied-back wedge method following AASHTO-AGC-ARTBA Task Force 27 recommendations. Allowable geogrid reinforcement design strengths were selected by comparing and grouping the design strengths of three geogrid reinforcement manufacturers. The selected design strength was chosen to be a strength that each of these manufacturers could closely meet. Care was taken to insure that the design strength selected did not exceed the pullout capacity of the grid from between the blocks, as based on results from pullout testing performed by the University of Wisconsin-Platteville on a number of geogrid reinforcement and block combinations.

SEGMENTAL BLOCK UNITS

The block were specified to provide a concrete compressive strength of 4,500 psi to insure durability from freeze-thaw conditions for the design life. Specifiers assumed this high compressive strength would also decrease the potential for breakage of blocks resulting from floating debris, such as trees and ice, striking the wall. Although hollow core blocks were thought to have a greater potential for damage, the assumption that the floating debris would strike the wall at an angle, and a requirement to completely fill the voids with aggregate, minimized this concern.

The granular foundation soils at this site precluded the potential for long-term settlement of the MSEW. Lack of long-term settlement also reduces the potential for block joints to open, which has been observed to occur when excessive settlement has occurred under MSEW foundations.

Being in a river environment with seepage into and out of the wall backfill, however, the possibility for movement of retained earth through the blocks was considered, and a filter system was designed. A coarse drainage fill was selected to be placed immediately behind the blocks, with an aggregate size larger than the anticipated joint openings. The aggregate drain gradation was designed such that the drain also acted as a filter for the reinforced backfill materials. A geotextile, designed to function as a separator and as a filter, was placed between the drainage fill and the reinforced backfill materials to prevent movement of the backfill into the drain. A perforated drain pipe was placed within the coarse drainage fill to insure no head build-up behind the blocks. Although the joints between the blocks were expected to relieve the build-up of head, the collector pipe was used as a secondary, reliability measure of safety.

CONTRACT DOCUMENTS

The construction contract documents presented the government-designed MSEW, including alignment, maximum top-of-footing elevations, approximate top of wall elevations, block requirements, minimum geogrid reinforcement lengths, required geogrid reinforcement properties, connection strength requirements, and backfill and compaction requirements. The geogrid and block properties were determined by researching the specification requirements and material properties of various manufacturers, as well as the Task Force 27 recommendations. A generic specification was developed to allow as many potential suppliers to bid on the MSEW. The construction contractor could choose to use the government design in its entirety, or re-design the geogrid reinforcement locations/spacings and properties following the Task Force 27 recommendations. Without a formal VE proposal, the construction contractor could make no changes in alignment, maximum top of footing elevation, minimum geogrid reinforcement length, or backfill and compaction requirements.

Submittal requirements included: descriptive technical data on the geogrids, segmental block units, and caps; certificates of compliance for all geogrid properties; test results for the determination of connection and long-term allowable design load properties; shop drawings and profiles presenting geogrid reinforcement types and elevations, top-of-footing and top-of-wall elevations, and footing and drain construction details; and detailed descriptions of construction methods, procedures, and sequencing. If significant changes to the government design were proposed, computations or computer printouts identifying the input parameters and verifying the proposed design were required to be submitted, including the computed tensile force for each geogrid layer.

COSTS

Costs developed for the 1990 VE study indicated approximate unit costs of $37.50 per square foot for an anchored sheetpile wall system and $25.85 per square foot for a MSEW. The actual bid for the MSEW by the low bidder for the project was $15 per square foot, including geogrid reinforcement, segmental block units, geotextile, drainage fill, excavation, backfill, and fencing. This bid price represented a VE savings in excess of $1 million. The lower than anticipated unit cost was partially attributed to a very competitive bidding environment during the summer of 1991.

MSEW CONSTRUCTION

Wall construction, consisting of 16,800 square feet of exposed wall surface and 89,400 square feet of geogrid reinforcement, commenced in June 1992. Ames Construction of Burnsville, Minnesota was the general contractor. Ames selected TENSAR geogrid reinforcement and VERSA-LOK segmental block units for construction of the MSEW. Service Engineering of Maplewood, Minnesota provided Ames with the required submittals.

Several advantages of constructing MSEWs have been noted based on observations made to St. Paul District projects where MSEWs have been constructed. These advantages, discussed in the following paragraphs, contribute to the lower costs of MSEWs.

The sequencing of construction appears to be much more flexible than construction of conventional wall systems. The Rochester 2B construction contractor moved his crews from wall to wall, regardless of whether the wall was completed or not, depending on the adjacent construction activities, availability of equipment and materials, and whether the crew was needed elsewhere for a short period of time.

Dewatering costs can be minimized with MSEWs. The wall can be constructed to an elevation above which the dewatering system can be removed and/or above which flooding will not impact construction of the wall and adjacent areas. The Rochester 2B construction contractor utilized this scheme and left the walls in a partially completed state for several weeks without impacting the project.

A line and grade construction crew with unskilled laborers can be utilized instead of a typical wall forming crew. A line and grade foreman is typically used to insure line and grade requirements are met, especially for the first several courses of blocks. The unskilled laborers then place the blocks and geosynthetic reinforcement as specified in the approved shop drawings.

The St. Paul District geotechnical engineers have noted one disadvantage of MSEW construction. This is the often unfamiliarity of construction details by Corps inspectors and large construction contractors who generally are awarded Corps construction contracts. MSEW system suppliers are required by the specification to provide experienced installers to assist the construction contractor during the early stages of construction of the MSEWs. However, the installation learning curve for both the construction contractor and the inspectors can be substantial. Care for detail must be continuous to insure bulges or misalignments are minimized.

SUMMARY

The St. Paul District has successfully designed and constructed a MSEW in a riverine environment. The MSEW has performed as anticipated through several high river stage events on the South Fork Zumbro River, although the design event has not been experienced. The MSEW provided a cost-efficient and highly aesthetic alternative to conventional wall systems for minimizing the affect of the Rochester Flood Control Project construction of the Soldier's Field Golf Course.

REFERENCES

Meyers, Mark S., and Schwanz, N.T. (1993). "MSE Segmental Block Wall System in a Riverine Environment", Geotechnical Fabrics Report, Volume 11, Number 4, pp. 22-27.

US Army Corps of Engineers (1990a). "Value Engineering Proposal, Zumbro River at Rochester, Minnesota, Stage 2B FDM", St. Paul District, St. Paul, Minnesota.

US Army Corps of Engineers (1990b). "Design Memorandum Number 4, Flood Control Project, South Fork Zumbro River, Stage 2B, Rochester, Minnesota, St. Paul District, St. Paul, Minnesota.

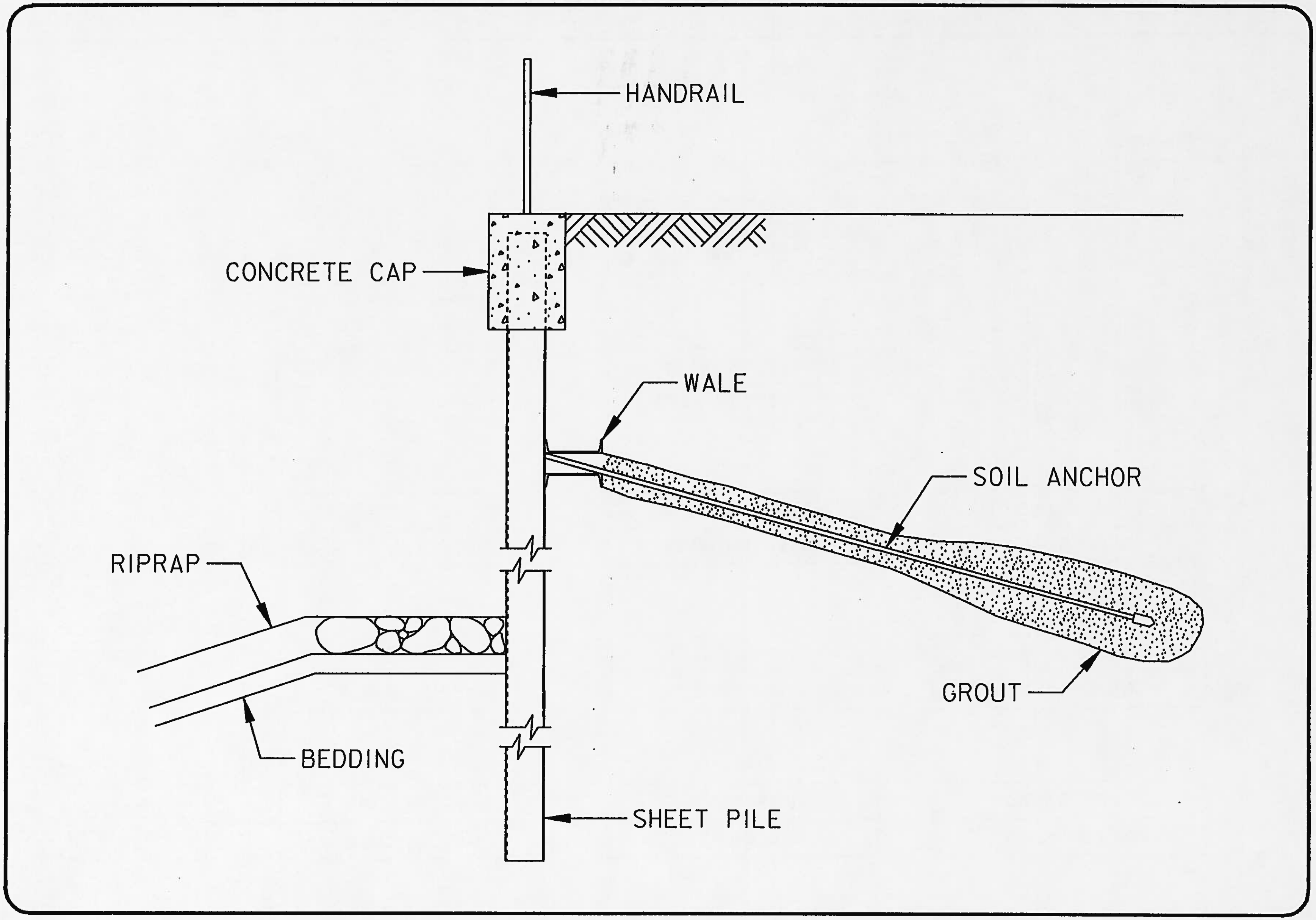


FIGURE 1: Anchored sheetpile retaining wall proposed in the Stage 2B Feature Design Memorandum

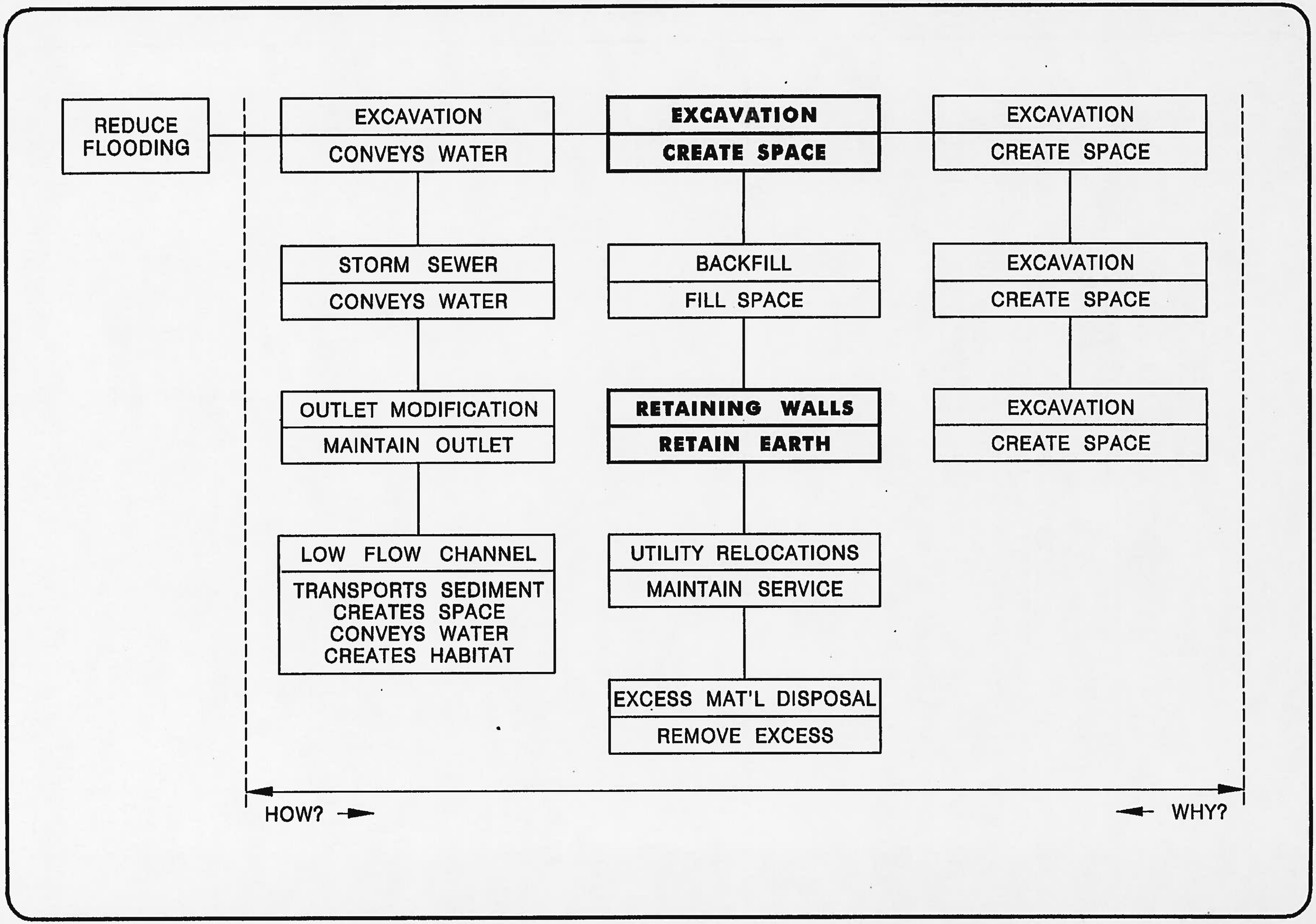


FIGURE 2: FAST diagram used in VE study

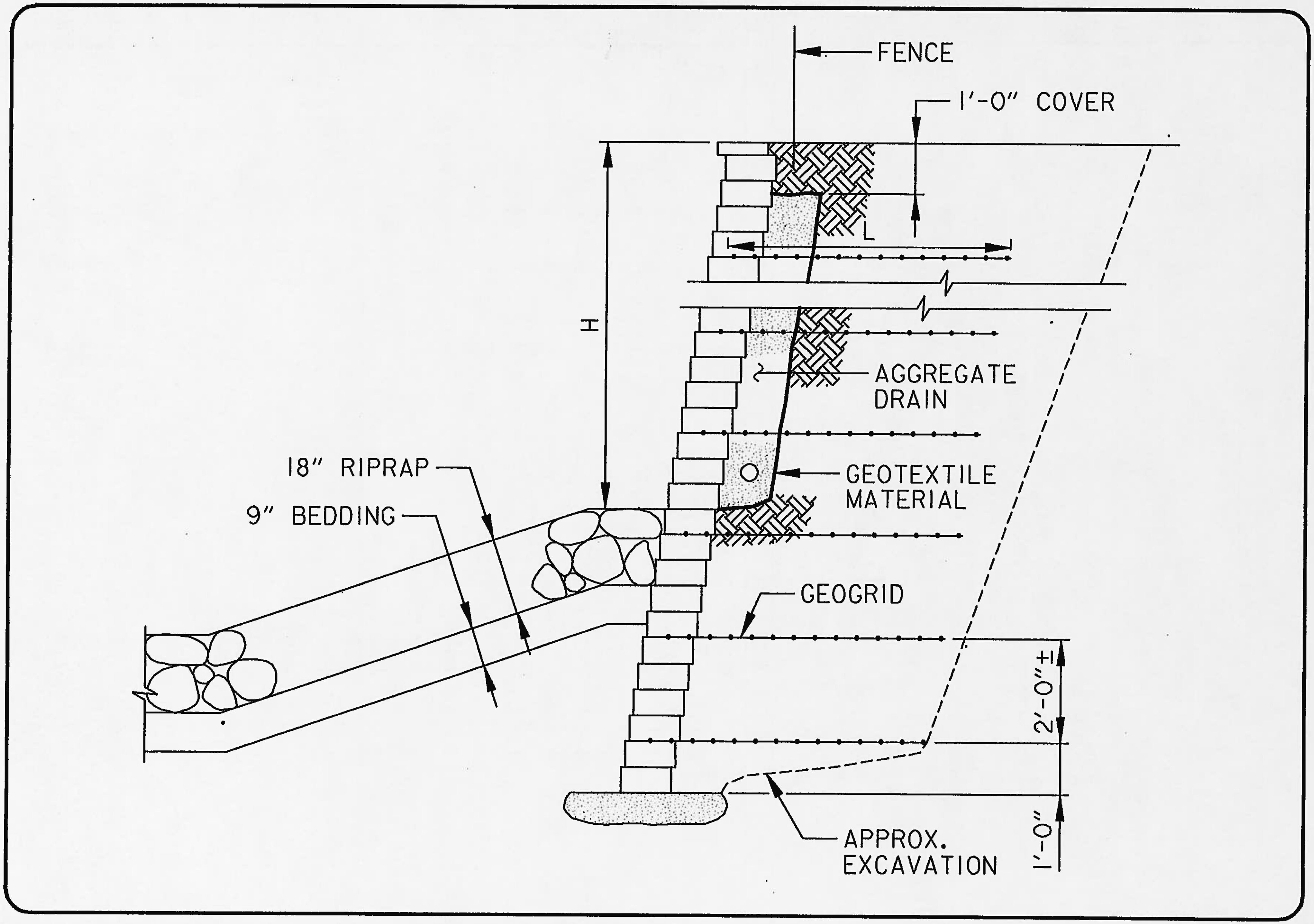


FIGURE 3: MSEW design used as a basis for the VE study

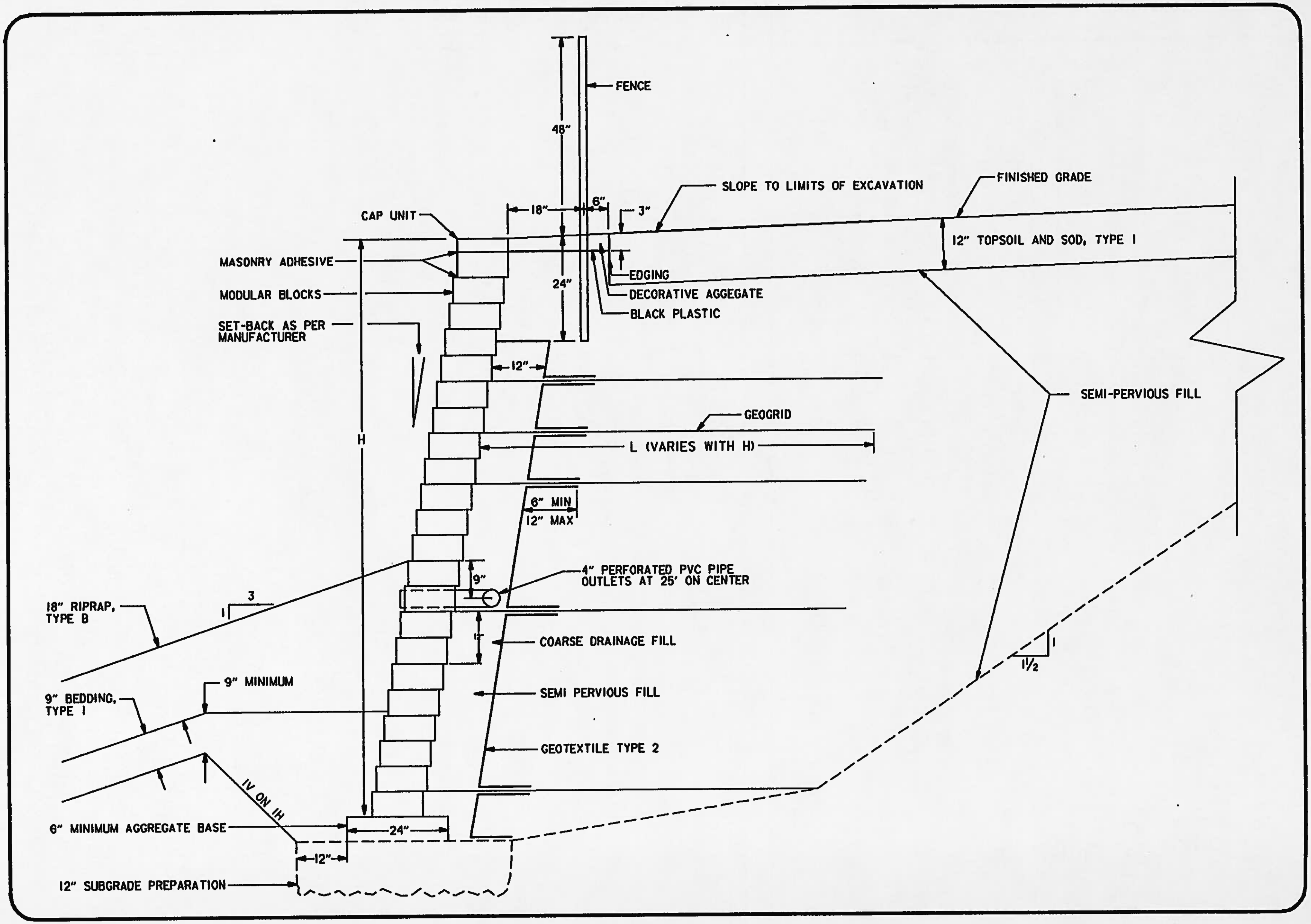


FIGURE 4: Typical retaining wall section for exposed wall 5 feet or higher

VALUE ENGINEERING: THE NOT SO SECRET WEAPON TO IMPROVE PROJECT QUALITY AND CONTROL COSTS

Francis G. McLean[1], Christopher C. Morell[2], and Norman Hyndman[3]

ABSTRACT

Conceived during the years of World War II, Value Engineering Methods have been refined to improve the process and enhance performance. The Value Method incorporates process elements that work synergistically to produce an outcome which leads to improved product function, and/or lower cost. These process elements are not unique to the Value Method, but the combination of elements and rigorous focus of the method invariably generate a successful outcome. Following the Value Method Job Plan forces a functional examination of the study object, and cost comparisons of identified functional alternatives provide a measure of value. Improving value, viewed as perceived worth versus cost, is the objective of the Value Method. Cost reduction, while a common outcome, is not the main purpose of the Value Method. Improved performance, safety or durability are other desirable results possible through value studies. This paper reviews the method, identifies the strengths of the process, presents the procedures that make it a strong and reliable decision process. It includes four case histories that demonstrate a breadth of possible applications, the power of the method, and typical results.

THE VALUE METHOD

Evolving from the asking and answering five basic questions posed by Larry Miles, the Value Method (VM) is usually accomplished by a team (although an individual can use the method) that commonly follows a Job Plan comprised of six steps: 1) Information Gathering and Functional Analysis, 2) Creative Idea Generation (Speculation), 3) Idea Analysis, 4) Idea Development (Proposal Preparation), 5) Presentation of Proposals, 6) Implementation. Proper selection of study subject and team, joined with faithful use of the Job Plan, rarely fails to increase the value of the product, process or activity, by producing an improvement or reducing cost. The team effort usually requires several days of dedicated work, depending on the size or complexity of the study subject. The method is

---

[1]Value Program Manager, US Bureau of Reclamation, POB 25007, Denver, CO 80225

[2] Value Engineering Coordinator, Reclamation Service Center, US Bureau of Reclamation, POB 25007, Denver, CO 80225

[3] General Engineer, Reclamation Technical Service Center, US Bureau of Reclamation, POB 25007, Denver, CO 80225

well documented, and continues to be the subject of improvement efforts.

STRENGTH OF THE METHOD

The following are perceived to be the strong points of the Value Method, and are covered in subsequent paragraphs: 1) The Job Plan, 2) The Team approach, 3) Functional Analysis System Technique (FAST), 4) Creativity Methods for producing ideas, 5) Rigorous Evaluation of identified alternatives.

The Job Plan

The value job plan guides the team, providing for an orderly progression through the activity and assuring that all the necessary process actions are performed. Thus, a thorough and thoughtful examination of criteria, issues, functions, alternatives, benefits and costs (value) is stimulated. In addition, the job plan meets all the requirements for a strong, reliable decision making process. Russo and Schoemaker (1989) reviewed and studied a wide variety of decisions and have identified the "key elements" in a decision process, Table 1. The second column contains a summary of the actions and requirements for each element of the decision process. While the key elements are few and simple, there are commonly a number of steps required to accomplish each of the elements. Depending on the nature of the problem, these steps can range from the simple and/or intuitive, to highly organized and extensive processes structured to achieve the objective of the individual elements. For example, broad and detailed market surveys or complex multi-variate analyses may be undertaken in the course of gathering data and evaluating

FRAMING	Selecting viewpoints of problem / future scenarios Determining what to decide (problem(s) to be solved) Establishing acceptable outcomes (criteria)
GATHERING INTELLIGENCE	Known factual data / information / requirements Identify unknown, but required, information Searching for "dis-confirming" information that invalidates views / data / assumptions / positions
COMING TO CONCLUSIONS	Approaches used range from "seat of pants" to highly developed, systematic Evaluate / analyze information Formulate and evaluate alternative solutions Decide / select alternative(s) to be implemented
LEARNING FROM FEEDBACK	Develop a process to monitor outcomes Compare expected versus actual results Incorporate "lessons learned" into ongoing or future decisions

Table 1. KEY ELEMENTS OF A DECISION PROCESS
(after Russo and Schoemaker, 1989)

alternatives. Table 2 presents the simple four phase decision process compared with the Value Method Six Step Job Plan. This demonstrates the basic strength of the Value Method as a decision process. Its successes in a variety of applications confirm this.

KEY ELEMENTS OF A DECISION PROCESS	GENERAL VALUE METHOD: PREPARATION AND "JOB PLAN" PHASES
FRAMING	Selecting Study Object and Objective Selecting the Study Team Setting Criteria and Requirements
GATHERING INTELLIGENCE	Information - Functional Analysis - Objectives/function - Stakeholders Views - Criteria (hard, soft) - Value (cost, efficiency) - Current method
COMING TO CONCLUSIONS	Speculation - Creativity (shift viewpoint) - Unconstrained generation of numerous ideas - List all ideas
	Analysis - Meets Criteria/Needs - Advantages/Disadvantages - Cost Comparison
	Development - Confirm information - Insure functionality - Insure adequacy - Insure compatibility - Identify barriers
	Presentation - Sell identified alternatives - Overcome resistance
	Implementation
LEARNING FROM FEEDBACK	Accountability Report

Table 2. VALUE METHODOLOGY AS A DECISION SUPPORT PROCESS

The Team Approach

The VM incorporates a "high performance team" approach in the process. The benefits of this are well known. For example, the Northrop Corporation's use of the "skunk works" in producing special aircraft with short development times has been broadly publicized.

Several aspects of team activity that benefit the VM are: 1) using experienced team members from several disciplines, 2) the various educational and experience backgrounds of individual members combines to bring significant knowledge to team capability, 3) synergism between team members during their creativity efforts and development deliberations, 4) lack of previous individual involvement avoids vested interests in past decisions and choices, 5) sole focus on the study activity and objective, 6) suppression of organizational turf issues and bias by team interaction, 7) relieving the team members of other responsibilities during the study. Finally, the team approach takes advantage of the power of people working together for a common goal.

Functional Analysis

The Functional Analysis System Technique (FAST) is a powerful way to identify the basic function of components or processes, allowing the subsequent creativity activity to focus on alternate ways to achieve the basic function and meet criteria and needs. This is accomplished by developing two word pairs (verb-noun) that describe function, and later linking them in graphical form through "how-why" relationships. Necessary and critical functions are quickly identified, as are unnecessary functions and potential value mis-matches, where functional importance, effort and/or cost appear to be unbalanced.

Creativity Methods

A strict creativity process is incorporated in the VM, usually group brainstorming. This method is very effective, in that group synergism leverages ideas and allows both vertical and lateral movement in generating ideas. Other techniques are available, and appropriate, but must be unconstrained, with the judgmental phase occurring after the idea generating phase. Because the purpose of VM is to seek and develop innovative alternatives, the object of the creativity session is to generate as many ideas as possible for later evaluation and subsequent development into viable alternatives.

Rigorous Evaluation

Before developing the ideas into proposals, they are subjected to evaluation regarding how well they meet criteria, perform the function, and what they cost. Competing ideas are rated and ranked to see which best fulfill the functional, intangible and fiscal needs of the activity. Multiple criteria can be developed and multi-variate analysis methods can be used for evaluation, but the "paired alternative" method is commonly used. It is simple and can handle a sufficient number of variables to usually meet the requirements of the VM.

VALUE METHOD APPLICATIONS

As a sound decision support process, the VM can be used for almost any application. The focus of the study can be general or specific, while the complexity of the object of study can be great or simple. Complex study subjects are frequently approached by performing several studies on sub-systems, sub-processes, or components. These multiple studies may be performed simultaneously, or in sequence. The method has been commonly used for facilities, construction projects, processes, manufactured products, organization function and structure, and services. The method rarely misses returning substantial benefits, in both functional improvements and lower costs.

EXAMPLES OF VALUE STUDIES

Four case histories of applications are presented. They demonstrate possible applications and typical outcomes from using the Value Method. Product improvements are not generally documented for posterity in the same manner as cost savings, but frequently the benefits of shortening performance or cycle times, efficiencies in production, quality or functional improvements, etc., exceed actual savings. In heavy and facilities construction the experience in savings is commonly 4% of project cost, or more. The return on investment (money saved versus money spent on the study and implementation of proposals) from Value Studies is routinely in excess of 20:1, sometimes returning well over 1000:1.

Case History #1 - Computer Support Services for a 1500 Person Site

A study of a centralized service center's computer support group was called for because of repeated and consistent user complaints about the nature and quality of their service. The center was comprised of engineers, scientists, and program management staff in support of offices distributed throughout 17 states, and the financial, human resources and administrative services support staff for the center. Since the productivity of almost every employee and program in the center depended on the reliability of the center's computer systems (desktops, minis, mainframe, LAN's, WAN) the group's failure to deliver fast, cost effective service was having a significant impact on the organization.

The team decided early in the study to refine the scope of the problem, detail the number and variety of variables, quantify customer attitudes, and identify specific problem areas. Three techniques the team used to acquire this information were; (1) "Target Opportunity Panels," a survey questionnaire, and an "Expert Day." The team held Target Opportunity Panels (similar to customer focus groups) to identify areas of customer satisfaction and dissatisfaction, and to highlight the more common computer problems. The Panel findings were used as the basis for a questionnaire distributed throughout the center for comment. Finally, the team held an "Expert Day" to meet with outside computer service experts to discuss their experiences and get their comments on the team's findings and initial proposal concepts. Based on these inputs, the Team found that:

✓ Contrary to the pervasive beliefs about customer service at the start of the study, over

60% of the center felt that they were receiving good or better service.

✓ The organizational complaints were inevitable partially because of (1) the broad range of user expertise and needs, (2) the wide variety of computer hardware and software used in the center, (3) the absence of agreements to define the expectations for service, and (4) the range of personnel aptitude and skills in the support group.

✓ The gap between individual service demands and support group staff abilities, the lack of any agreement to define service expectations, and the unconstrained growth of different computer hardware and software, guaranteed the support group's failure.

✓ Experts outside the center confirmed that lack of standardization and adequate controls eventually leads to poor service and high cost. They stated that service degrades because, while an unconstrained system may enhance individual productivity, it presents complex problems in execution, management and service. Additionally, an open system's complexity results in higher operation, management, and training costs because there are no limits.

✓ Most of the center's managers were unwilling to accept a standard system and wanted to retain control over their group's computer hardware and software decisions.

✓ While a study objective was to achieve the best customer service at lowest reasonable cost, there was no clear agreement on current computer or computer service costs. Even among computer industry professionals, there was no accepted standard cost model for determining or evaluating computer operation or service costs.

✓ The center believed that service to the individual customer's desktop computer was the true measure of support group success. Based on the individual customer's expectations for service, success could be translated as, "If the desktop computer failed to perform satisfactorily, regardless of cause, it was the fault of the computer support group."

✓ A small minority strongly believed that the current support group staff could not provide the desired support under any circumstances.

Twenty seven of the 34 proposals presented to management for improving service were accepted. Although no specific cost savings could be identified at the completion of the study due to lack of detail for implementation, the study has clearly made a difference and the new monthly newsletter (a proposal) tracks the implementation of the study's proposals. Several contract studies have been performed in order to fully implement some of the proposals. To date 16 of the 27 proposals have been put into action.

Case History #2 - Dam Safety Remedial Actions

A value study was commissioned to evaluate six conceptual designs to eliminate the safety discrepancies of two dams on a federal wildlife refuge and recommend alternate corrective solutions. The estimated cost of the preferred design concept was $4.18 million.

The value team of three engineers and the study leader, visited the site with the design team leader and refuge manager. Team discussions with the design team leader, the refuge manager and the project manager, indicated that there were differences in the criteria and weighting that should be used to evaluate and compare the solutions presented by the six design concepts and the proposals developed by the study. To overcome these differences, the team used a criteria matrix technique to score each design concept and proposal against specific weighted criteria.

During the information phase and site visit, the team determined that, based on functional analysis of the criteria, the problem did not require that the structural deficiencies be corrected. The preferred design concept, however, focused on correcting the structural deficiencies of two dams rather than on the larger issues of dam safety, downstream inundation and loss of life that were highlighted in the team's functional analysis. By refocusing their attention, the team developed two proposals that would eliminate the need for major structural changes to the dams. By changing dam operations and restoring the flows feeding Hollis Pond to their natural course in Hollis Canyon, the flood hazard was eliminated and only minor work was required.

The team prepared a criteria matrix from each of three points of view to evaluate the proposals and confirm that the criteria were met. The scores from all three matrices confirmed that all three solutions satisfied the criteria and the study proposals ranked slightly higher than the preferred design concept regardless of which matrix was used.

The agency decided to implement the study team's proposal which resulted in an estimated savings of $4 million, which was more than 90% of the initial project cost and about 100 times the cost of the study.

Case History #3 - Environmental Clean up

A three day value study was conducted of one of five operable units for overall site cleanup of a mine Super Fund site on the National Priority List. The study evaluated the 60% concept design to determine whether there were alternatives to improve the quality or cost of actions to reduce the non-point source acid generation. The proposed design included elements to treat heap leachate, acid rock drainage and acid mine drainage to prevent the off-site release of cyanide solutions. The primary techniques used in the project design were drainage ditches and controls, to convey and channel surface runoff and water emitted by seeps, and area re-vegetation, to limit the infiltration of storm water into the ground, waste piles and mine workings. The estimated cost of the project was $25 million.

The team consisted of seven engineers and geologists and a study team leader. The engineers and geologists represented various government and state environmental agencies, the downstream residents and the technical design agency. The study team was designed for success from the outset because it represented both the range of perspectives of the project's stakeholder groups and the full range of technical diversity and expertise necessary to attack the problem. An added advantage of the team's diversity of technical expertise and representation was that the members had immediate and intimate access to technical information and stakeholder concerns and opinions impacting the project.

An unusual advantage of this study was that the design team leader had participated in another successful value study that not only saved time and money but was acceptable to the surrounding community. The design team leader encouraged the lead agency to conduct and participate in the study, actively consulted with the team as they developed ideas into proposals, and championed acceptance of the study's proposals into the final design.

The team developed nine proposals to adjust and refine the design concept. Six of the proposals were adopted for the final design. The estimated savings of the six proposals was $2.2 million, almost 10% of the initial project cost or 50 times the cost of the study.

Case History #4 - Water Diversion and Conveyance System

The original concept design (approximately 30% stage) included a pipeline and pumping plant system to transfer water from a river to an aqueduct system, a few miles to the west. The conceptual estimate for the project was $14.5 million, almost double the preliminary estimates.

The five person team included civil, structural, and mechanical engineers, a water agency representative and a study team leader. The team met for five days to develop and consider alternate ways to capture the water and transfer it to the aqueduct. In that time they developed and presented nine proposals with a maximum potential for capital savings of $8.1 million, and the potential for life-cycle savings of about $30.0 million.

During the information gathering step, the team determined some of the design criteria were not as firm as originally perceived. The location and alignment could be significantly changed for the right proposal. Also, the system design capacity had been arrived at somewhat arbitrarily. Following the job plan allowed the team to question criteria and assumptions used for the design process, and to develop several proposals that otherwise would have been overlooked.

The team came up with 29 ideas in the brainstorming step of the study. Nine of these ideas were selected as being "very promising" and were developed into proposals. The team selected six more ideas as having "good promise," but did not have time to develop them. These ideas were forwarded to the designers for further consideration and possible use. Several of the proposals presented by the team included less expensive canal sections in lieu of the 22,000 feet of 42 inch steel pipeline in the design. The team learned that in many of the early planning and scoping meetings the term "pipe" had been used so often that it became the design concept, and the concept of using canal sections just got overlooked.

In an initial review of the nine proposals, the designers recommended that four be pursued for use in the final design, or use with modifications. The designers also recommended that two of the "good," undeveloped ideas be developed for use. There is little doubt that the proposals from this study will result in a project that is easier to construct, more durable, safer and more efficient. The savings are almost certain to be from 20 to 200 times greater than the cost of the study itself.

CLOSING

In spite of the long and successful record of the VM, and its adaptability to almost any study subject, application of it is not wide spread. Certainly society is not benefitting to the extent it could from applying the method. Both the private sector and government could benefit from the improved products and reductions in cost that the method routinely provides.

Unfortunately, common motives for using the VM are: when final design costs exceed early estimates; a process is a disaster; manufacturing costs too much and takes too long; construction bids are well above the budget; or "you just can't get there from here - nothing will work." It's amazing that a method that is capable of resolving these kinds of situations after-the-fact is not routinely used pro-actively to avoid them in the first place. This is an example of the proven fact that people will risk more (in this case invest in a value study) to get out of a bad situation than to avoid it. To derive maximum benefit from using the Value Method requires a decision to invest money up front, i.e., to do the study. With rates of return commonly exceeding 15:1, this shouldn't be a difficult decision.

There are many barriers to applying VM, and more barriers to implementing proposals after a study has been performed. These are identified and described in many of the books on VM. They include maintaining past practice ("we've always done it this way"), organizational and individual egos, organizational, cultural or personal preconceptions, and a variety of other reasons. Underestimating the barriers will predestine some, if not all, of the study proposals to non-acceptance. The study team must develop reasons for adopting the proposals which overcome the barriers, and target their presentation to gain acceptance.

A successful VM Program requires a "champion" in the organization, preferably one that has significant input into the financial operations. Similarly, it is important that the decision maker be vested in the outcome of the decision and prepared to deal with resistance to change.

SELECTED REFERENCES

Federal Construction Council Consulting Committee on Value Engineering, Technical Report No. 92, Elements of an Effective Value Engineering Program, National Academy Press, Washington, D.C., 1990.

Hensey, M., Collective Excellence: Building Effective Teams, The American Society of Civil Engineers, 1992.

Keeney, R.L., Value-Focused Thinking, Harvard University Press, 1992.

Kleindorfer, P.R., H.C. Kunreuther and P.J.H. Schoemaker, Decision Sciences: An Integrative Perspective, Cambridge University Press, 1993.

Lumsdaine, E. and M. Lumsdaine, Creative Problem Solving, McGraw-Hill, 1995.

Michalko, M., Thinkertoys, Ten Speed Press, 1991.

Parker, D.E., Management Application of Value Engineering for Business and Government, The Lawrence D. Miles Foundation, 1994.

Russo, J.E. and P.J.H. Schoemaker, Decision Traps, Simon and Schuster, 1989.

Russo, J.E. and P.J.H. Schoemaker, "Managing Overconfidence", Sloan Management Review, Vol. 33, No. 2, Winter 1992.

Value Analysis Final Report, PC/LAN Support, U.S. Bureau of Reclamation, Denver, CO, October 20, 1997.

Value Engineering Final Report, Grama & Comanche Dams, Wichita Mountains Wildlife Refuge, Oklahoma, U.S. Bureau of Reclamation, Denver, CO, September 10, 1997.

Value Engineering Final Report, OU-4 Site Wide Reclamation, Summitville Mine Super Fund Site, Colorado, U.S. Bureau of Reclamation, Denver, CO, June 5, 1998.

Value Engineering Final Report, Lower Owens Pumpback System and Pipeline, U.S. Bureau of Reclamation, Denver, CO, May 21, 1999.

Whyte, G., "Decision Failures: Why They Occur and How to Prevent Them," Academy of Management Journal, Vol. 5, No. 3, August 1991.

INTERNET RESOURCES

www.valuefoundation.org
Lawrence D. Miles Value Foundation, 2107 National Press Building, Washington, D.C. 20045.

www.value-eng.com
SAVE International, 60 Revere Drive, Suite 500, Northbrook, IL 60062.

www.usbr.gov/valuprog
US Bureau of Reclamation, POB 25007, Denver, CO 80225.

VALUE ENGINEERING OF LIQUEFACTION MITIGATION

Don W. Dotson[1] and David E. Ott[2]

ABSTRACT

To expand manufacturing capacity, CPS Chemical Company contracted with a design/build contractor to construct new manufacturing facilities at their West Memphis, Arkansas plant. The plant was located near the New Madrid Fault and the possibility of earthquake damage was considered. A subsurface exploration program revealed the presence of loose, saturated sands beneath the proposed structures that could potentially liquefy during an earthquake. The original design called for a deep foundation system to bypass the liquefiable soils and found the structures on deeper, firmer stratum. A specialty foundation contractor submitted a value engineering proposal to use stone columns instead of the deep foundation system. The challenge was to predict how the stone columns would react to a 7.5 magnitude earthquake. This was especially difficult since stone columns have been primarily used to increase the strength of soft soils under static loading; their use in liquefaction mitigation is a fairly recent innovation. To substantiate the design parameters, post-installation testing was conducted. After installation of over 6.5 miles of stone columns, the foundation portion of the project was completed on schedule at a net savings of $1.5 Million.

INTRODUCTION

A proposed chemical plant expansion, consisting of process structures, additional storage tanks, and an administrative building was planned for a site in West Memphis, Arkansas. The subsurface exploration program encountered loose, potentially liquefiable, saturated sands interbedded with soft clays. The soils at the site were characterized through a number of geotechnical borings. Drive-sampling was performed in each boring at 2.5 foot intervals to 10 feet below ground surface (bgs), then at 5 foot intervals thereafter. The fines content (FC, percent of soil passing the No. 200 sieve), clay fraction (percent of soil particles smaller than 5 microns), and moisture content (w%) were determined for each sample. Standard Penetration Testing (SPT) provided the field blow counts per foot, N_{field}, for each sampling interval. The N_{field} values obtained from the SPT were converted to a standardized blow count, $(N_1)_{60}$, using procedures presented by Seed, et al, (1).

---

[1] Ogden Environmental and Energy Services Company, Inc. 3800 Ezell Road, Nashville, Tennessee 37211. Adjunct Professor of Civil Engineering, Tennessee State University, Nashville, Tennessee 37209.
[2] Ogden Environmental and Energy Services Company, Inc. 161 Mitchell Road, Oak Ridge, Tennessee 37830.

The borings indicated that the site is underlain by clean, loose sands to depths of about 15 feet bgs. N_{field} values typically ranged from 2 to 16 blows per foot, indicating very loose to loose soils, susceptible to liquefaction. Beneath these sands to depths of 40 feet bgs is a layer of compressible silty clay; underneath the clay is denser, silty sands. The groundwater table was found at approximately six feet bgs.

For the proposed structures, static soil loadings of up to 1,750 pounds per square foot (psf) were anticipated. Two additional design requirements had been set by the Owner: post-construction consolidation settlements must be less than 1.5 inches, and post-earthquake settlements must be less than 1.0 inch. The design earthquake consisted of a 7.5 (Richter) moment magnitude event with a peak ground acceleration (α_{max}) of 0.22g.

To meet the design criteria and mitigate the effects of potential liquefaction from the design earthquake, the structures were to be founded on a deep foundation system that would bypass the liquefiable sands and carry the loads to a layer that was considered to be non-liquifiable. The original plan called for H-piles driven into the dense sands encountered at approximately 40 feet bgs. The estimated cost for the H-pile foundation was $2.5 million.

The Fort Worth office of Hayward Baker, Inc., a specialty foundation contractor, submitted a value engineering proposal to use stone columns instead of H-piles for liquefaction mitigation. Stone column construction consists of the partial replacement of weak subsurface soils with a compacted vertical column of stone. When jetting water is used, the process is called *vibro-replacement*. When no jetting water is involved, the process is called *vibro-displacement* (2). Typically, vibro-displacement is used to increase relative densities in cohesionless soils; while vibro-replacement is used to increase drainage and provide strength to clayey soils.

To construct stone columns, a vibratory probe penetrates the ground, either under its own weight (dry) or by jetting (wet). Horizontal vibrations produced by the equipment cause densification of the surrounding sands. Stone backfill is introduced in controlled lifts, either down the annulus (top feed), or through feeder tubes (bottom feed). Placement of the stone backfill is followed by compaction with the vibrating probe. This densifies the stone in the stone column as well as densifies the surrounding soils by forcing the stone outward.

The ability of stone columns to increase soil resistance to liquefaction occurs through three mechanisms. First, the vibratory process densifies cohesionless soils. Second, the stone columns provide drainage paths for rapid dissipation of excess pore water pressures as they develop. Third, the stone columns provide reinforcement through their own structural integrity. (3).

Use of stone columns for mitigation of liquefaction potential has significantly increased since 1987. The limited experience with stone columns in actual earthquakes has been positive. Studies conducted following the Loma Prieta, California earthquake in 1989, for example, showed that sites previously improved using stone columns successfully resisted liquefaction, while untreated adjacent ground showed signs of distress, settling, and/or sand boils (4).

The analysis and design process involves three steps. First, the liquefaction potential of the existing subsurface soils is determined by calculating a factor of safety. If the factor of safety is less than unity, the soils are considered liquefiable and further analysis is required. Second, for soils with a liquefaction factor of safety less than unity, the liquefaction-induced settlement is calculated. If the expected settlement is greater than the allowable maximum, liquefaction mitigation is required. Third, the stone columns are designed to mitigate liquefaction potential.

CALCULATION OF FACTOR OF SAFETY

For each depth, the cyclic stress ratio (CSR) is the seismic load that would be generated by the design earthquake. This value is determined from procedures outlined by Youd and Idriss (5):

$$\mathrm{CSR} = \frac{\tau_{ave}}{\sigma'} = 0.65\,\frac{\alpha_{max}}{g}\,\frac{\sigma}{\sigma'}\,r_d$$

where τ_{ave} is the average shear stress in the layer, σ and σ' are the total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient which is a function of depth that varies from 1.0 at the ground surface, to 0.5 at depths of 30 meters or greater.

The cyclic resistance ratio (CRR) required to resist liquefaction is determined for each sample depth by comparing the corrected blow counts per foot, $(N_1)_{60}$, to observed cyclic stress ratios. For soils with fine contents above 5%, the energy-corrected blow counts, $(N_1)_{60}$ is first adjusted to account for measured CRR increases with increasing fines content. For 5%<FC<35%, this is given as follows:

$$(N_1)_{60cs} = e^{(1.76 - \frac{190}{FC^2})} + (0.99 + \frac{FC^{1.5}}{1000})(N_1)_{60}$$

For fines content greater than 35%, refer to Youd and Idriss (5).

Youd and Idriss (5) report a fourth degree polynomial which approximates the relationship between CRR for a standard 7.5 magnitude earthquake and $(N_1)_{60cs}$ values less than 30:

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4}$$

where $x = (N_1)_{60cs}$, a = 0.048; b = -0.1248; c = -0.004721; d = 0.009578; e = 0.0006136; f = -0.0003285; g = -1.673×10^{-5}; and h = 3.714×10^{-6}.

The factor of safety for liquefaction, FS, is then determined for each sampling interval by the following formula (6):

$$FS = \frac{CRR}{CSR}$$

Overall, several locations were noted that were susceptible to liquefaction (FS<1). The majority of these areas were located just above the silty clay layer, at approximately 10 to 15 feet below grade. At these locations where FS<1.0, the relative soil density required to prevent liquefaction, based on correlations to $(N_1)_{60}$, was back-calculated from the above by setting the factor of safety, FS, equal to one. Increases in $(N_1)_{60}$ from 4 blows/foot to 21 blows/foot were required in some cases.

CALCULATION OF POTENTIAL LIQUEFACTION-INDUCED SETTLEMENT

The expected liquefaction settlement for each sample interval where the factor of safety against liquefaction was less than unity was determined using procedures set forth by Tokimatsu & Seed (7). These procedures relate $(N_1)_{60}$ and CSR to the expected volumetric strain (in percent), based on empirical data.

The expected percent strain for each sampling interval was determined using a graph from reference (7). The settlement for each interval was then summed to determine the total expected liquefaction settlement at that location. Volumetric strains of up to 4% for some intervals were calculated. Using this procedure, total calculated liquefaction settlements varied from 0 to 3.8 inches with an average settlement of 1.3 inches. Therefore, liquefaction mitigation was determined to be necessary.

STONE COLUMN DESIGN

In addition to the liquefaction potential, other concerns at the site included long-term consolidation settlement of the clayey soil layers. Since stone columns are frequently used to reduce potential settlement (both by providing drainage paths to speed consolidation, and by providing structural support), it was anticipated that stone columns could be a cost-effective means of mitigating the liquefaction potential while also minimizing post-construction consolidation. To provide an additional factor of safety, the soil columns were designed to minimize liquefaction settlement based primarily on the densification of the liquefiable soils. Reinforcement and drainage benefits were counted as secondary.

An important concept in stone column design is the Area Replacement ratio, A_r. This is defined as the fraction of soil tributary to the stone column replaced by the stone and is defined as:

$$A_r = \frac{A_s}{A}$$

where A_s is the area of the stone column after compaction and A is the total area within the unit cell. The area ratio is often expressed in terms of the diameter of the stone column, *D*, and the center-to-center spacing, *s*. For instance, for a square pattern,

$$A_r = \frac{\pi}{4}\left(\frac{D}{s}\right)^2$$

The A_r required to achieve a relative density that would prevent liquefaction (that is, the corrected blow counts (N_{corr}) to achieve FS=1.0, as determined above) was calculated for the site using a design equation presented by Baez (4):

$$A_r = -\frac{1}{20.61}\ln\left[\left(\frac{N_{field}}{0.776 - 0.0194N_{field}}\right)\left(\frac{1}{(N_1)_{60}} - 0.025\right)\right]$$

By determining the required A_r for each critical zone, it was determined that the maximum A_r necessary was 9%. With a proposed stone column diameter of approximately 3 feet (based on the experience of the contractor and the size of the available equipment), an initial spacing of 9 feet was determined. The design installation depth was to the underlying dense sand layer at about 40 feet. The column grid was proposed to cover the entire building area, with an additional row placed just outside the building footprint to provide a transition zone between loaded and unloaded soils.

Additional analyses were performed to ensure that the proposed stone column installation would sufficiently limit post-construction consolidation. An axisymmetric finite element program was developed and a model was created using estimated post-construction elastic parameters. This analysis showed that post-construction settlement were within design requirements.

STONE COLUMN INSTALLATION

The stone columns were installed in the late summer and early fall of 1998 using a "bottom-feed, dry" method for stone insertion. The equipment consisted of a large crane especially equipped with a generator to power the vibratory motor of the insertion probe and pneumatic equipment to insert the stone under air pressure. A hopper was installed on top of the vibratory probe to hold the stone prior to insertion and the crane was used to suspend the vibratory probe. A feed bin was incorporated into the system to facilitate filling the hopper with stone. The bin would be lowered by the crane so that a front-end loader could fill it with stone. The bin would then be raised by the crane and dumped into the hopper. The hopper lid would close and the hopper would be pressurized so that the stone could be inserted under pressure after the probe had been vibrated into the soil to the required depth. Using this method, over 16,000 tons of stone was inserted into 1,160 stone columns installed to depths of up to 40 feet.

Pressure and amperage gauges are located inside the cab of the crane so that the operator can keep track of stone insertion pressure and vibratory motor effort, respectively. Although these can be useful as quality control indicators, post-installation testing is the most reliable gauge of the effectiveness of the stone columns.

POST-INSTALLATION TESTING

Following the installation of the stone columns, verification testing was performed using Standard Penetration Tests (SPT). In general, the verification borings were located between the stone columns and sampling was performed at least every five feet in the borehole. Four borings (B-1, B-3, B-4, and B-5) were advanced in the tank farm area; one boring (B-2) was installed in the control structure area.

The corrected $(N_1)_{60}$ blow counts in the upper sand layer averaged around 60 and some were as high as 106, although some moderately low values of around 10 were noted. Overall, a significant increase in relative density, and corresponding decrease in liquefaction potential, was obtained. As expected, little increase in blow counts in the silty and clayey material was noted. The post-installation maximum potential liquefaction settlement was calculated to be significantly less than the design requirement of 1 inch in all borings except B-3, which had a single loose sand layer at 10 feet deep (see Table 1).

Table 1
Post-Installation Testing

Boring No.	Area	Max $(N_1)_{60}$ (Sands)	Min $(N_1)_{60}$ (Sands)	Total Expected Dynamic Settlement
B-1	Tank Farm 1	103	39	0.0"
B-2	Control Building	79	37	0.0"
B-3	Tank Farm 2	97	9	1.6"
B-4	Tank Farm 3	105	28	0.0"
B-5	Tank Farm 4	42	16	0.6"

Based on the verification testing, additional secondary stone columns were installed in the vicinity of boring B-3 midway between the primary stone columns that had been installed at a spacing of 9 feet. This reduced the center-to-center spacing in this area to 4.5 feet.

Additionally, a load test was performed in the area of Tank Farm 4. A mound of soil seventeen feet high, and 20 foot across the top edge was placed over a portion of the treated area and three markers were installed to monitor settlement. These markers were monitored for one week. No settlement was noted, indicating a significant increase in the relative density and a reduction of the expected post-construction settlement.

CONCLUSION

As can be seen by Table 1 above, the stone column program successfully reduced expected dynamic settlement across the site. The upper sand zone corrected blow counts were increased from typical values 8 to 12 blows per foot, to 28 to 109 blows per foot (except for the remaining loose area which received additional treatment). In addition, post-construction settlement was reduced to negligible levels.

Vibro-replacement stone columns can be an effective means of reducing the liquefaction potential of loose sands; while simultaneously mitigating consolidation settlement of interbedded cohesive soils. Since installation of vibratory stone columns is still very much an art more than a science; and because the various design calculations are based on a relatively small database of empirical evidence, a carefully developed quality control and testing program is essential to ensure that the desired effects have been achieved. In this instance, the installation of over 6.5 miles of stone columns resulted in a net savings of $1.5 million over a conventional deep foundation system.

REFERENCES

(1) Seed, H. B., K. Tokimatsu, L. F. Harder, and R.. M. Chung, "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Journal of Geotechnical Engineering, ASCE, Vol. 111, No. 12, pp. 1425-1445, 1985.

(2) Barksdale, R.D., and R. C. Bachus, Design and Construction of Stone Columns, Volume I, Report No. 1 FHWA/RD 83/026, Federal Highway Administration, 1983.

(3) Glover, J. C., "Sand compaction and stone columns by the vibroflotation process", Recent Developments in Ground Improvement Techniques, A.A. Balkema, Rotterdam, Netherlands, 1985.

(4) Baez, J. I., A Design Model for the Reduction of Soil Liquefaction by Vibro-Stone Columns, Ph.D. Dissertation, University of Southern California, Los Angeles, California. 1995.

(5) Youd, T. L. and I. M. Idriss, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Technical Report NCEER-97-0022, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, New York, 1997.

(6) California Division of Mines and Geology (CDMG), Guidelines for Evaluation and Mitigating Seismic Hazards in California, Special Publication No. 117, 1997.

(7) Tokimatsu, K. and H. B. Seed, Simplified Procedures for the Evaluation of Settlement in Clean Sands, Report No. UCB/GT-84/16, Earthquake Engineering Research Center, University of California, Berkeley, 1984.

THE VE STUDY OF A TUNNEL SEGMENT OF DETROIT METROPOLITAN WAYNE COUNTY AIRPORT

Wern-ping Chen[1] and Linn Day[2]

ABSTRACT

A forty-hour Value Engineering (VE) study was performed for the South Access Road - North Tunnel and Bridge Segment (Element 3) of Detroit Metropolitan Airport on August 12 -16, 1996. This paper will focus on discussions of the tunnel construction methods and the foundation support alternatives proposed in the study. The goal, as in any VE, was to creatively furnish technically sound alternatives to satisfy the user's needs at the lowest life-cycle cost. The VE processes from the development /evaluation of alternatives, the criteria weighting process, the criteria/idea matrix, the cost estimates, to the life cycle cost analysis are presented as a case study. The recommended tunnel construction alternative would result in a potential saving of 3.4 million dollars (in 1996 value).

INTRODUCTION

In early 1990's, the Detroit Metropolitan Wayne County Airport began an airport expansion program, which includes a second entrance, the South Access Road. This road will provide a new access to airport facilities and a connection between Eureka Road (which connects to I-275) and I-94. An element of this access road is the 880-meter (2,890-foot) long North Tunnel and Bridge Segment that was a subject of the VE study. This element includes a bridge at the Taxiway Fox and a tunnel that passing under Runway 9R/27L, Taxiway Victor and Hotel, and a portion of Concourse A terminal apron. Figure 1 shows the location map and project elements.

The Sverdrup Civil performed a VE study on August 12 to 16, 1996 (at Detroit Metropolitan Wayne County Airport) for the North Tunnel and Bridge Segment by request of Wayne County Airport and the Program Manager, Sverdrup Facilities; however, this paper focuses only on discussions of the tunnel construction alternatives of this study. The design engineer of this contract is Parsons Brinckerhoff Michigan.

[1]Sverdrup Civil, Inc., Two Center Plaza, Boston, Ma 02108
[2]Detroit Metro Wayne County A/P, Detroit, MI 48242

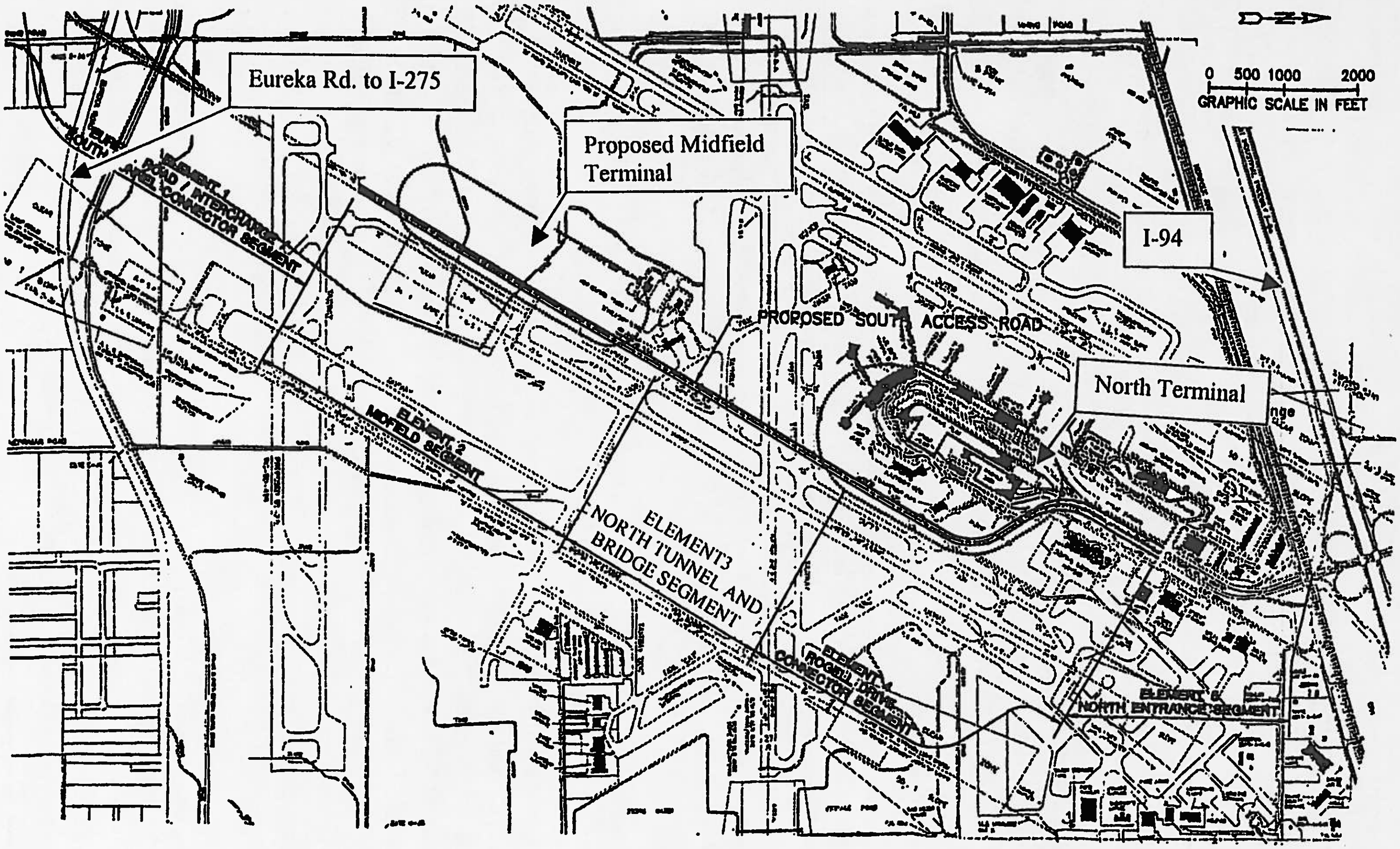


Figure 1. Location map and project elements (2)

GEOTECHNICAL SUMMARY

The geology in the project site could be distinguished by six strata [1], including the topsoil, the brown silty clay, the gray silty clay, the gray silt, the very hard sandy clay/clayey sand (locally called "hardpan."), and the bedrock (about 80 feet below existing ground surface). Figure 2 shows the geological profile along the North Tunnel alignment. As shown in Figure 2, the entire tunnel alignment is within gray silty clay.

Regional Groundwater Condition

The geotechnical investigation performed by Somat Engineering [4] suggested that the piezometric head of the deep confined aquifer system is normally 20 to 25 feet below the ground surface. Approximately 20 feet beneath the ground surface, groundwater conditions are influenced by the piezometric head of the deep aquifer in the soil-rock transition zone and in the upper part of the bedrock. In the upper reaches of the clay formation, the influence of the deep aquifer is minimal; as the depth below grade is increasing, the influence of the deep aquifer becomes pronounced.

Design Considerations

Major geotechnical design considerations are:

- Groundwater inflow during excavation – It could be controlled by
  a) limiting the time the excavation remains open; however, it would impose a difficult construction constraint during tunnel excavation,
  b) slurry wall construction, which is an effective means of groundwater cut-off during excavation, but is relatively expensive and without local experience, and
  c) dewatering in the soil/rock transition zone, which would require excessive pumping and would possibly lead to undesirable settlement.

- Long term groundwater conditions/Consolidation settlement - Groundwater drawdown that would depress the ambient piezometric level and would result in an increase in the effective stress on the deeper soft clays and lead to undesirable settlement must be minimized. Consolidation settlement may be a concern at distances away from the roadway if an effective vertical cut-off is not provided along the roadway. The Geotechnical Report [4] recommended that excavated side slopes be limited to no steeper than 3h:1v if open cut methods are used.

- Long term drainage and uplift pressure - Appropriate planning and design, such as raising the tunnel vertical alignment to reduce the potential hydrostatic pressure beneath the tunnel base slab, may resolve this issue. If the tunnel is designed as a watertight box, by open cut method without positive drainage system, the hydrostatic uplift pressure may approach the full depth of the roadway beneath the ground surface. In the case of slurry

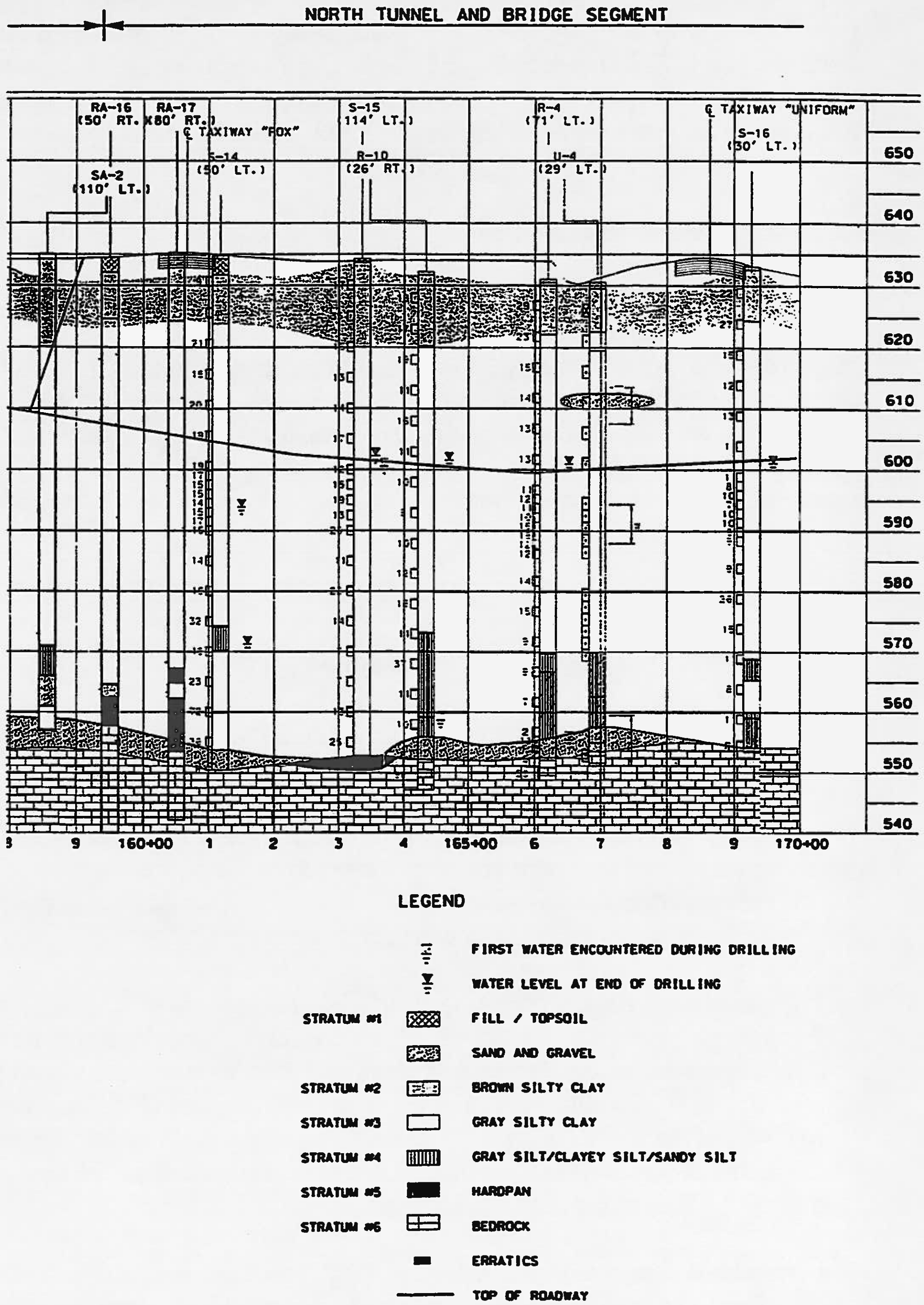


Figure 2. Geological profile (1, 3)

wall construction, the effect of near-surface groundwater may not develop, and the uplift pressure may be equivalent to the distance between the bottom of the slab and the phreatic surface of the deep aquifer.

PRE-DESIGN STUDY

Two final tunnel construction alternatives were considered for the North Tunnel Segment during the Pre-Design Study Supplement II stage. They were the deep slurry wall construction method and the conventional open cut concrete box construction method. In the slurry wall method, shown in Figure 3, the exterior walls would provide the main vertical support for the final tunnel structure. Coupled with three rows of tiebacks, these exterior slurry walls also serve as the support of excavation during tunnel excavation. The interior tunnel vertical support would be provided by three rows of piles beneath each interior wall. The conventional open cut box structure, shown in Figure 4, would be vertically supported by a mat foundation sitting on soil. The design engineer has performed a cost estimate of these tunnel construction alternatives [2]. The results suggested that the slurry wall construction method is slightly more cost-effective than the open cut construction method. From evaluations of the cost, the reduction in the risk of settlement, and the minimization of construction impacts to adjacent structures, the design engineer recommended using the slurry wall method.

The cross section of the 428-m (1400-foot) long North Tunnel Segment, to be constructed approximately 9 meters (30 feet) beneath the airfield facilities, consists of three cells to contain northbound and southbound roadways and an access service road. Both the northbound and the southbound roads have three lanes each. The service road is a two-lane road, one lane in each direction. The width of each cell is about 13.4 meters (44 feet). The vehicle clearance in the tunnel is 14'-9". The project is funded 80% by the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) and 20% by Wayne County. The proposed construction schedule of the North Tunnel Segment is from June 15, 1998 to December 1999.

EXPERIENCE FROM THE CONSTRUCTION OF ADJACENT CONTRACT

The Pre-Design Study Supplement II Report states that the Airport Engineering staff is not aware of any settlement of existing facilities due to construction excavation. No dewatering wells or recharging wells were used for any airport construction at the time the report was written. The Midfield Segment contractor used a pump within a caisson for construction of the pump station; however, the observed quantity of water inflow was minimal. This work involved pumping from a 24.4-meter (80-foot) depth, which is a good test to determine if groundwater table drawdown affects the buildings. No damage or settlement to buildings was observed during or after the pump station construction.

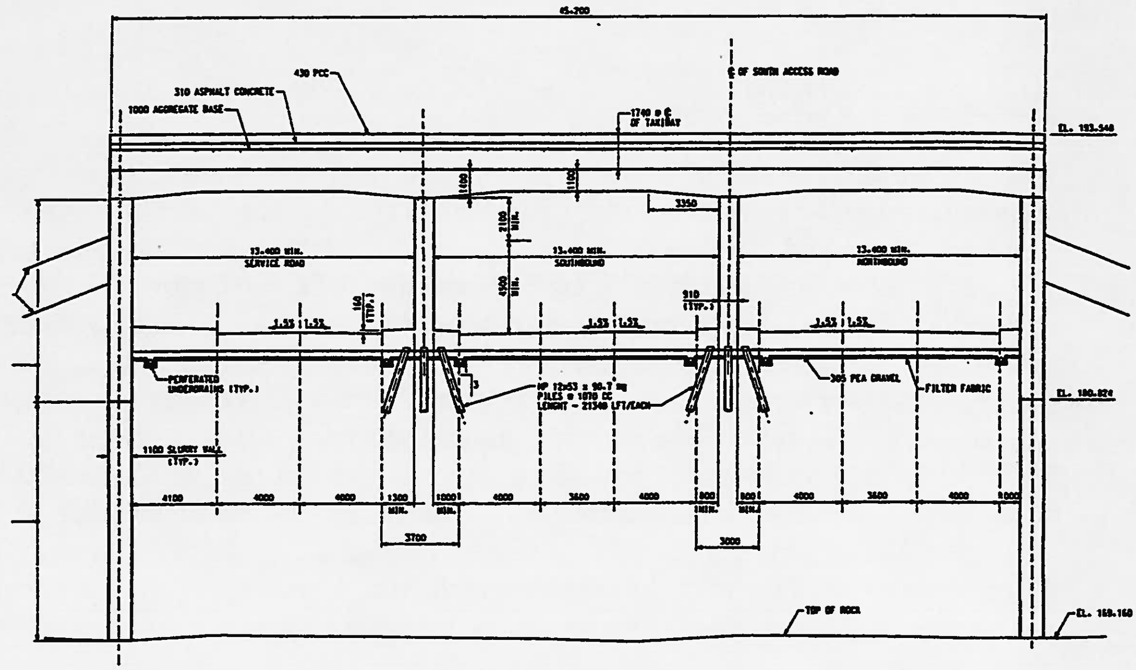


Figure 3. Slurry wall original concept (2)

Notes: 1. Two rows of tie-backs are required during tunnel excavation
2. The deep slurry wall panel into rock is required only at every fifth panel

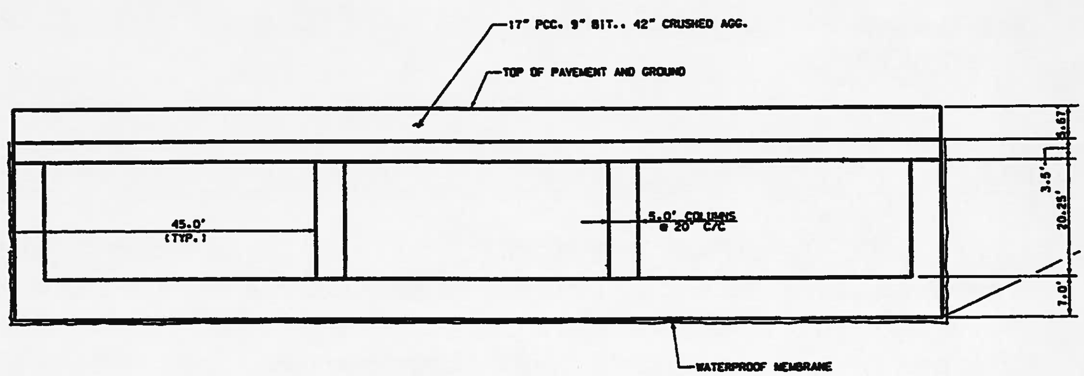


Figure 4. Open cut alternative

Please also note that two conditions must exist for groundwater drawdown to cause surface settlement under buildings. The first condition is a significant lowering of water levels at the building locations. The second condition is a significant thickness of compressible material that must exist under the buildings.

VE STUDY METHODOLOGY

Prior to the briefing by the design engineer and the site visit, the VE team discussed the Functional Analysis Systems Techniques (FAST) diagram, the Cost Models, and the Function Analysis. It was determined that study efforts would concentrate on those functions involving the largest dollar amounts because of their impacts on potential savings.

The FAST diagram illustrated the function hierarchy (How?) and construction activities (Why?) to meet the mission of this project, that is to provide a second entrance to airport and meet (the airport's internal expansion) requirements. The ITEM/Function Cost Model showed that the cost of tunnel construction, by slurry wall method, is about 65.5 % of the total project cost at $34,260,000. The high percentage to the total cost provided a good indication that the tunnel construction should be studied in detail. The Function Analysis and Cost/Worth (C/W) Worksheet also showed the Value Index (V.I.) of the slurry wall (at a cost of $6,847,500) to the exterior walls of the open cut method (at an estimated cost of $5,000,000) is 1.37. This 1.37 V.I. further suggested that the slurry wall construction method is a poor value item and should be studied. Following the designer's briefing and site visit, the VE study commenced.

Information

Costs were reviewed taking into account the size of the project, its complexity, geographical location, and site constraints [6]. The team reviewed the bid tabs on local projects received recently, and checked the local bidding climate and availability of general contractors in the area and their experiences on the type of construction the project requires. Labor rates were based on Davis-Bacon rates, required for federally funded projects such as this, although Union members were not required.

After reviewing the construction cost of the tunnel by slurry wall construction method, the VE team performed an independent cost estimate. This estimate was $5,160,000 more than that of the design engineer's. This difference could possibly be from the following reasons; the design engineer's cost estimate neglected the cost of the piles beneath the interior walls, and the VE Team included a separate cost for demolition of existing runway and taxiway pavements and the disposal of debris. Please also note that the VE estimate did not include costs for removal of items or extra work which may result from unknown site conditions, escalation to the midpoint of construction, engineering fees, owner's administration, and post contract contingency; if these items were included, the estimate would be further higher.

Creativity

The VE study identified five tunnel construction alternatives. They are

1. Original concept – cut-and-cover tunnel with slurry wall construction method and interior pile foundation,
2. Open cut with pile foundation,
3. Open cut with mat foundation,
4. Slurry wall shallow foundation without interior piles – shallow slurry wall panels (exterior wall) and spread footings under the interior wall, and
5. Precast slurry wall and interior pile foundation – original concept, except for the precast slurry wall panel.

Other alternatives that were proposed and have been investigated by the design engineer include roof-first construction, secant pile wall, and freeze wall. The design engineer has eliminated these alternatives from considerations of cost or function deficiency. The VE study did not evaluate these alternatives.

Evaluation

Alternatives were evaluated through a Criteria/Idea Matrix analysis. In this matrix, alternatives were compared using various criteria that will satisfy the tunnel function requirements. The top-three ranking alternatives were subjected to an advantage/disadvantage analysis for final ranking.

Prior to the Criteria/Idea Matrix analysis, a criterion weighting process was performed to assign a weight for each criterion. Table 1 lists the proposed evaluation criteria, including the cost, the surface settlement during construction, the long-term surface settlement, the water tightness, the ease of finish, the local construction experience, and the construction impact to airport operations. Each criterion was compared, in Table 2, against other criteria for its importance, and a raw score (weight) was assigned. The weighting rule is a 4-point for a major preference, a 3-point for medium preference, a 2-point for minor preference, and a 1-point (Letter/Letter) for no preference, where each criterion scores one point. For example, the cost criterion A is a minor preference to the long-term surface settlement criteria C; therefore a 2-point (indicated in Table 2 as 2A) is assigned to the cost criterion.

Table 1. Criteria weighting process

Criteria	Description	Raw Score (weight)
A	Cost	11
B	Surface settlement during construction	3
C	Long-term surface settlement	8
D	Water tightness	10
E	Ease of finish	0
F	Local construction experience	4
G	Construction impact to airport operations	5

Table 2. Criteria comparison

	B	C	D	E	F	G
A	3A	2A	2A	1A/1E	1A/1F	2A
B		2C	2D	2B	2F	1B/1G
C			1C/1D	2C	1C/1F	2C
D				3D	2D	2D
E					3F	3G
F						1F/1G

Table 3. Criteria/Idea Matrix

		ALTERNATIVES				
	Weight	1	2	3	4	5
Satisfies Functions	11	4/44	4/44	4/44	3/33	4/44
Cost	11	3/33	2/22	4/44	4/44	3/33
Water Tightness	10	3/30	4/40	5/50	3/30	4/40
Long-Term Surface Settlement	8	4/32	4/32	5/40	1/8	4/32
Construction Impact to Airport Operations	5	4/20	3/15	3/15	4/20	4/20
Local construction Experience	4	2/8	4/16	4/16	2/8	2/8
Surface Settlement During Construction	3	4/12	3/9	3/9	4/12	4/12
Total Weighted Value		179	178	218	155	189

Table 3 summarizes the criteria/idea matrix analysis, and assigns a final weighted value for each alternative. The top-three ranking alternatives are the open cut with mat foundation alternative (with 218 points), the precast slurry wall construction alternative (with 189 points), and the original slurry wall construction alternative (with 179 points). They were then subjected to an advantage/disadvantage analysis for final ranking in Table 4.

Development and Implementation

Because of the lack of local construction experience and the requirements of special slurry wall construction techniques, the precast concrete slurry wall construction alternative, Number-3 in the final rank, was ruled out for further evaluation. Cost estimates were performed for the final top-two ranking alternatives, the open cut tunnel with a mat foundation method and the original slurry wall construction method. Their estimated costs are $35,860,000 and $39,260,000, respectively.

Table 4. Advantages/Disadvantages Analysis

Rank	Alt. #	Advantages	Disadvantages	Final Rank
1	3	• Water tightness • Standard construction Method • Easy to finish • Cost less	• Relative high construction impact • Possible local settlement during construction	1
2	5	• Same as Alt. No. 1 • No finish work for wall is required	• Requires special construction technique • Water inflow to the final structure • Requires special joint constructions between slurry wall panels • Might require distribution head beams on top of the slurry walls • Lack of construction histories in North America	3
3	1	• Minimum settlement during construction • Less construction impact to adjacent structures • Accommodate water inflow during construction better	• Requires special construction technique • Water inflow to the final structure • Requires final finish work for the walls • Might require distribution head beams on top of the slurry walls	2

CONCLUSION

The intent of this tunnel VE study is to revisit the tunnel support elements/structures that represent the intentions of the design and to offer additional or new alternatives, without impairing the quality, the reliability, and the function of the original design intention.

Five tunnel construction alternatives were identified in this study. They were evaluated based on the cost, the geotechnical design considerations (surface subsidence during construction and long-term consolidation settlement), the construction consideration (construction impact to airport operation and available local construction experience), and the structure performance (water tightness and ease of finish) criteria. The final top-two alternatives, which are both functional, technically sound, and feasible for construction, were evaluated in detail by a cost estimate.

This study concluded that the cost of the open cut construction method would be cheaper than the slurry wall method for this proposed tunnel, though the design engineer's conclusion might be different. Also, from a technical point of view, the proposed tunnel is relatively shallow with the bottom elevation of the base slab at 9-meter (30-foot) below grade, and with a low groundwater table at about 20 to 25-foot below grade; therefore, the slurry wall construction method (that is embedded into bedrock 80 feet below grade) with tiebacks, might not be the best option. In conclusion, the VE study recommended using the open cut construction method in final design. The potential saving of implementing this alternative is about $3,4000,000 (in 1996 value), not considering the redesign cost.

ACKNOWLEDGEMENT

The VE Team leader of this study is Steven Kautz of Sverdrup Civil in Seattle office. Tom Mahoney, a former Structural Engineer of Sverdrup Civil in Seattle office also participated in the tunnel alternative evaluations of the VE study.

REFERENCE

[1]. South Access Road Pre-Design Study Report Supplement, Parsons Brinckerhoff Michigan, Inc., January 1996.
[2]. South Access Road Pre-Design Study Report Supplement II, Parsons Brinckerhoff Michigan, Inc., August 1996.
[3]. Geotechnical Investigations for South Access Road, North Tunnel and Bridge Segment, Testing Engineers and Consultants, Inc., December 1995.
[4]. Geotechnical Investigation, Air Rescue Fire Fighting Facility, Somat Engineering, August 1989.
[5]. New Runway 9R-27L and Taxiway, Wayne County Department of Public Services, April 1991.
[6]. Value Engineering Study Report – South Access Road, North Tunnel and Bridge Segment, Sverdrup Civil, Inc., August 1996.

Kentucky Route 9 – Licking Pike Widening
Design Build Slope Stabilization

by

Frederick W. Slack, PE [1]

ABSTRACT: This paper will discuss the inherent value engineering associated with the design and construction of approximately 184,000 SF of anchored tieback wall, installed as part of the widening and drainage improvements for KY9 in Campbell County, Kentucky. The total project is 2.2 miles in length. The anchored walls, excavation grading and drainage total to a complete project cost of $28.4 million. This is one of the largest design-build projects undertaken by the Commonwealth of Kentucky. It also features another first for the Transportation Cabinet– the general contractor is responsible for quality control/quality assurance.

The project is located just south of Cincinnati, OH and is situated along the east bank of the Licking River. The slopes along this stretch of the river have had a long history of movement and instability.

The Kentucky Transportation Cabinet advertised the anchored wall portion of this project to be designed and constructed by pre-approved specialty contractors. The Transportation Cabinet provided the basic design parameters to each specialty contractor.

Aspects to be discussed include the site and project history, previous attempts at slope stabilization along existing KY9, the technical proposal and bidding process, and the actual design and construction to date.

Project Location and Geology

The project is located in the Greater Cincinnati Area. Specifically, the work site is across the Ohio River from downtown Cincinnati in Campbell County. The project falls within the city limits of Wilder, Kentucky. The entire project lies along the right descending (east) bank of Licking River, which flows to the north into the Ohio River. (See Figure 1)

The soil mantle consists of an alluvial layer of silts and clays deposited by the flooding of the Licking River. This layer is found below elevation 500 NGVD and is about 60 foot thick. The alluvium is also found in several drainage features above the flood plain.

Ancient river deposits, referred to as terrace deposits, are found above the alluvial deposits. These deposits also range up to 60 feet thick.

The predominant soil at the site is a colluvial layer of clay, mixed with weathered shale and limestone floaters. This gravity deposited material is found at base of slopes along the project alignment.

[1] Chief Engineer, Richard Goettle, Inc., 12071 Hamilton Avenue, Cincinnati, OH 45231

KY Route 9 - Licking Pike Widening

SITE LOCATION MAP

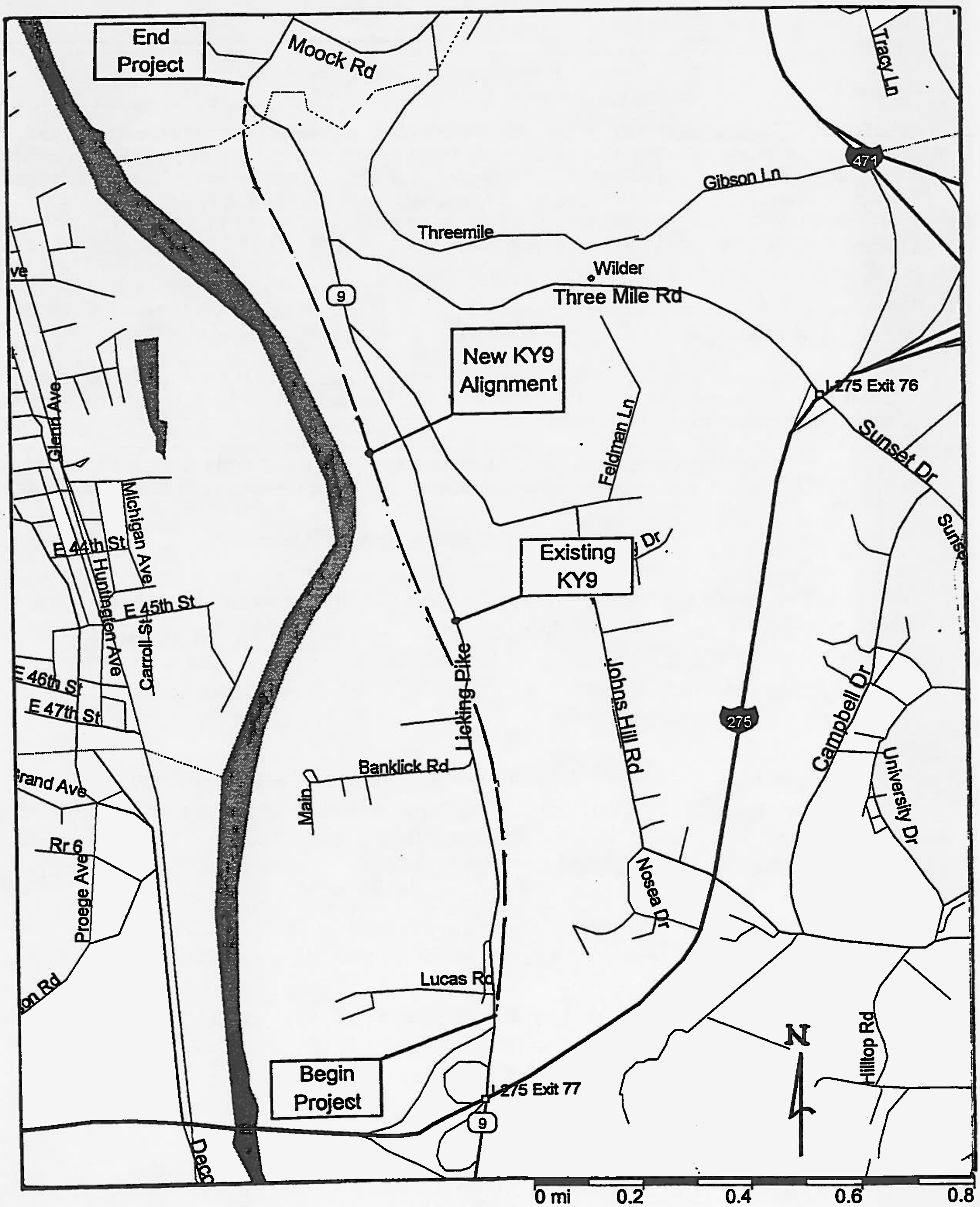


Figure 1

The second most predominant soil found at the site is a residual layer of shale and clay derived from the in place weathering of the underlying shale and limestone rock formations.

Directly beneath the residual soils is a layer of weathered rock referred to as the Rock Disintegration Zone or RDZ. This is a material that has rock-like appearance and structure, but the shale in this matrix has weathered to a clay material.

The rock stratigraphy, from highest to lowest is as follows:

1. Bellevue Tongue of Grant Lake Limestone. This tops out at approximately 830 NGVD and is 12 to 20 feet thick.

2. Fairview Limestone. Located between approximately 830 to 740 NVGD, this layer is 100 to 120 feet thick and contains about 40% limestone at the base and 65% limestone near the top of the formation.

3. Kope Shale. Roughly located between 500 to 470 NGVD, this layer is 220' to 250' thick and consists of 75% to 80% shale. (The rock anchorage is formed in this material).

4. Point Pleasant Limestone. This 30' thick layer is found between 500 to 470 NGVD and consists of 45% to 65% limestone.

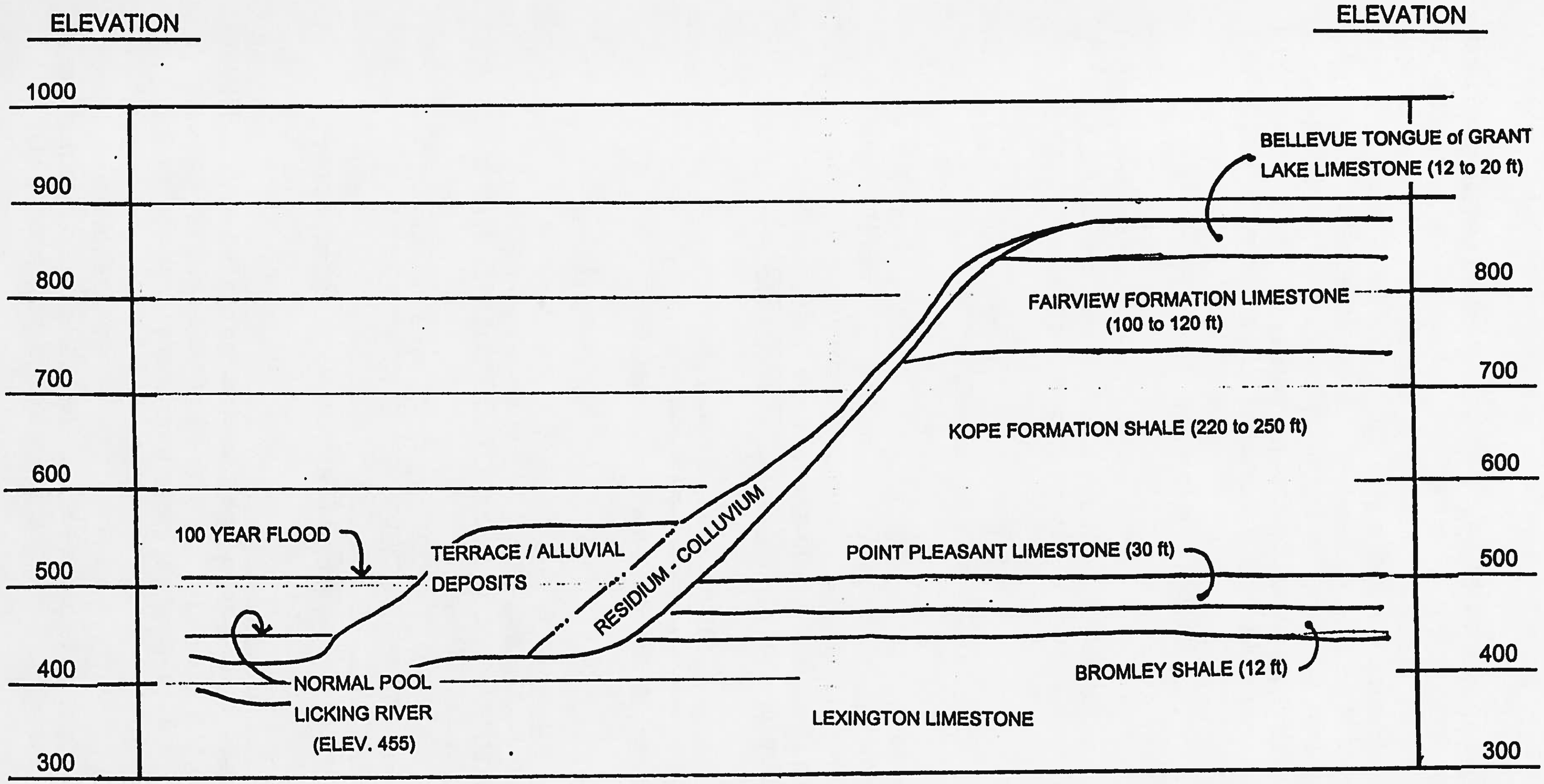
5. Bromley Shale. This layer is similar to the Kope material and tops out at 470 NGVD. It is 12' thick and is 70% shale.

6. Lexington Limestone. This basal material is an interbedded shale and limestone.

Figure 2 shows a generalized section of the site. Note that along the length of the project, the top of wall ranged between approximately elevation 550 to 620.

Project Site History

Prior to 1970, the land surrounding the project area was mainly rural, with limited commercial or residential development. With the construction of the interstate highway interchange (I-275) at the south end of the project in the 1970's, the project area started to develop. Several site development projects were attempted along the length of the project, however in each case, slope movement stopped the project. Each new attempt at development seemed to set off another round of slope movement. A 1979 study determined downhill movement ranged between 4 to 15 inches per year, with a mean rate of 8 inches



Data from U.S.G.S. Geologic
Data Sheet; Newport, KY

GENERALIZED SECTION
(LOOKING NORTH)

Figure 2

per year. This study also identified seven different landslides totaling 2,800 plan LF within a one-mile stretch of the KY9 alignment. Localized slope stabilization methods kept KY9 open to traffic. However, the constant difficulties with site development severely limited future development projects.

Over the years, the Kentucky Transportation Cabinet (KTC) undertook several attempts to stabilize the slopes along the project alignment. These stabilization methods included:

1. <u>Buttress Fills</u>. These were typically temporary measures to stabilize the roadway shoulder. The process generally involved random fill dumped over the edge of the road.

2. <u>Horizontal Drains & Drainage Gallery</u>. The Kentucky Transportation Cabinet (KTC) developed this method as a permanent solution for the slope instability. A 100-ft test section installed in 1985 showed that lowering the ground water table would slow the rate of ground movement. This test section showed a reduction from 8" per year to 0.35" per year. Encouraged by these results a new roadway and stabilization project using this method was let in 1992 and consisted of drilled shafts, 36" in diameter socketed 10' into rock. The sockets were interconnected by hand and machine tunneling. The shafts and tunnels were backfilled with uncompacted granular material to form a groundwater collection gallery. Horizontal drains were installed from downslope to intercept drainage gallery. A new embankment for the roadway would then be placed above the drainage gallery. Embankment construction created large hillside movements in June 1993 that led to a stop in construction activities and ultimately the cancellation of the project in December 1993.

3. <u>RR Rail and Metal Panels</u>. Heavy rains in the spring of 1996 caused rapid hillside movements that required weekly maintenance. A quick and effective solution was needed to slow the slides and reduce maintenance requirements. Recycled railroad rails were inserted into 12" dia. drilled holes. Old guard rail lengths were installed as lagging at the top of the rails to re-establish the shoulder. This project was bid in a special letting in July 1997 and completed in early August 1997.

4. <u>Elevated Structure</u>. The KTC has plans sitting "on the shelf" for an elevated structure for the entire length of this project. Due to cost considerations (construction costs were estimated to be approx. $40,000,000), this scheme was never advertised for bids.

It was finally decided that a permanent stabilization program was necessary. The KTC elected to follow a system utilizing the collective knowledge of the industry; namely a design-build project provided by a pre-qualified specialty contractor.

Pre-qualification and Design-Build Bidding

To assure that only qualified specialty contractors would be allowed to bid on this project, a pre-qualification process was established. A notice was posted informing interested parties that a pre-qualification meeting would be held. This meeting was held in May 1997. At this meeting, a pre-qualification package was distributed to the various respondents. This package requested that the specialty contractor provide prior experience and their approach to project means and methods, project organization and project scheduling. If the KTC was satisfied with the information provided, your firm was considered qualified and was thus able to continue in the pre-bid process. For this project, the pre-qualification package was due in June 1997.

The next step in the pre-bid process was a Request for Technical Proposal (RFTP). This request was distributed to the pre-qualified specialty contractors in July 1997. The RFTP provided a basic overview of the project and the basic design parameters and specifications, referred to by the KTC as the Special Note for Anchored Walls. The specialty contractor was required to provide the following information:

- Overview, Site Map, Comments on the Special Note
- Sample calculations at project stations 108+00, 120+00 & 130+50
- Anticipated construction sequence & procedures
- Material Data and Specifications

The Technical Proposal was submitted in September 1997.

The KTC reviewed each Technical Proposal and made comments. Each specialty contractor had the opportunity to reply to the comments and modify the Technical Proposal accordingly. Once a basic agreement as to concept and design procedure was reached, the KTC listed each such specialty contractor as qualified to submit a design-build proposal to the bidding general contractors for this project. Also pre-qualified for this project were Schnabel Foundation Co., Malcolm Drilling Co., Inc. and Nicholson Construction Co.

It should be noted that the design approach provided in the Technical Proposal was not binding. The KTC was interested in examining how each contractor approached the problem and applied the engineering principles required. They did not want to obligate a contractor to a design if he developed a more cost-effective approach during the bidding process.

A pre-bid meeting was held on June 25, 1998 and a bid date set for July 24, 1998. For a variety of reasons beyond the scope of this paper, a Joint Venture (JV) was formed for this project to act as both the general and specialty contractor. The joint venture partners include Richard Goettle, Inc. (lead partner) and Hayward Baker, Inc.

During the month between the pre-bid meeting and the bid date, a preliminary design was completed, using the basic design philosophies from the Technical Proposal. This preliminary design attempted to minimize the components which cost the most or which entailed the highest risk. It was also used to establish bid quantities. As it turned out, the time and added detail spent in this preliminary design process greatly minimized the additional work required for the final design.

The project was finally bid on July 24, 1998 at which time the JV was the apparent low bidder. The JV ultimately received the Notice to Proceed in September 1998.

Final Design

The final design concept established for this project contained the following elements:

- One anchor per pile, adjusting spacing to achieve reasonable anchor loads
- Drilled-in double elements
- Limit number of structural sections used.
- Minimum rock sockets required by the Special Note
- Treated temporary wood lagging
- Corrosion protection by means of sacrificial steel, resulting in reduced section properties
- Double corrosion protected anchors
- Geotextile drainage material and weep holes

This project consisted of four distinct walls, two right or uphill walls, R1 and R2 and two left or downhill walls, L1 and L2. The right walls used only one coefficient of lateral pressure K_L, while K_L varied along the length of the left walls. The top of wall to the bottom of the RDZ defined the design height of the wall. The RDZ varied from several feet below the bottom of facing to several feet above the bottom of facing. (See Figure 3).

Wall heights ranged from 6 feet to 39 feet, resulting backwall pressures ranging from 0.500 ksf to 3.30 ksf. The soldier piles were analyzed as a simple beam having supports at the bottom of the RDZ and at the anchor location. Beams were sized for the bending moments produced by the lateral earth pressure and for the axial load induced by the inclined rock anchor and weight of the concrete facing.

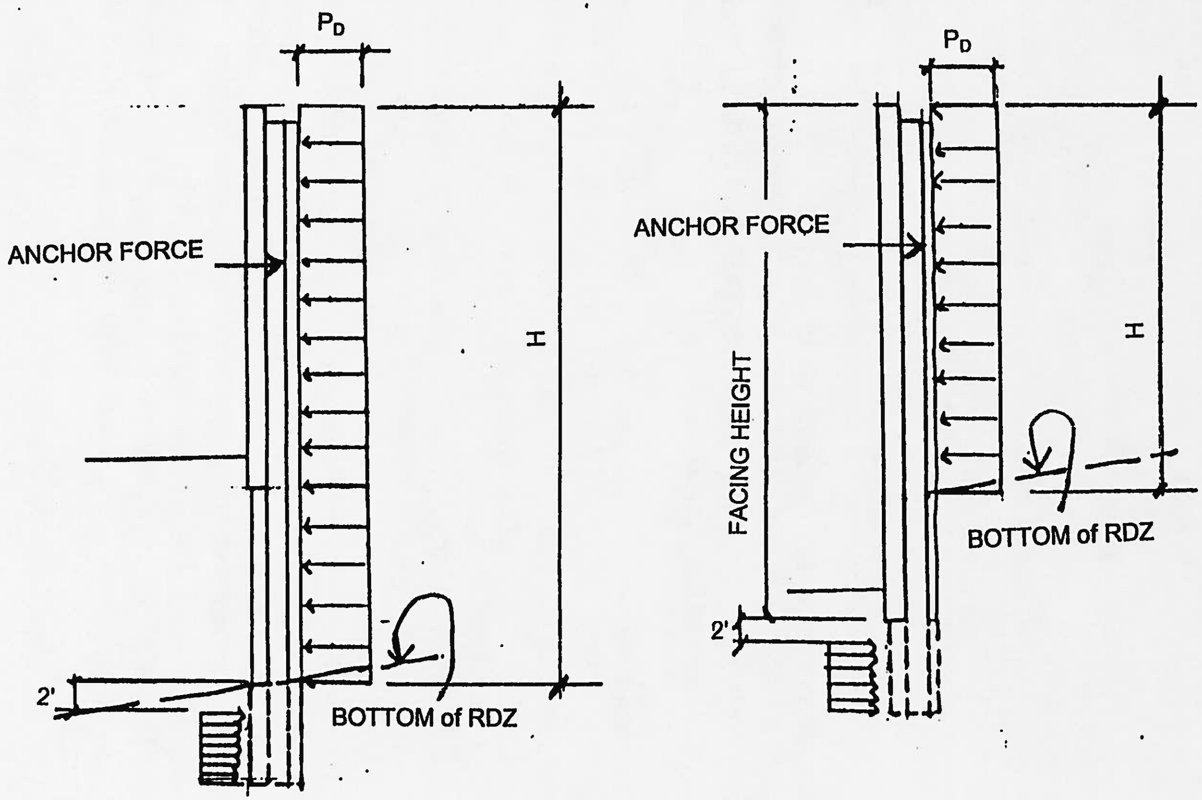
The depth of the toe rock socket required was determined by satisfying rotational stability about the lowest anchor and ensuring that the resultant embedment provided adequate vertical capacity. An allowable lateral resisting pressure of 8 ksf was used. The allowable bearing pressure was 40 ksf and no side friction in the socket could be considered. Finally, since the boundary between weathered and sound rock was not distinct, the top two feet of the socket were ignored in the embedment calculations.

Anchor tendon count was determined by using 53% of the strand materials guaranteed ultimate strength. Using a 0.6"ϕ strand yielded a value of 31.1 kips per strand. Anchor lengths were determined using an ultimate grout to rock transfer stress of 80 psi and a factor of safety of 2.0.

It should be noted that the 40 psi working transfer stress in the Kope Shale formation is somewhat higher than the 20 to 30 psi typically used in this area. Anchor testing to date has been very satisfactory. As of this writing, over 1,000 anchors have been installed. Only two anchors required the use of supplemental anchors or replacement anchors.

Soldier pile elements consisted of C15, W14 and W18 structural shapes in both Grade 36 and Grade 50 steel. The temporary facing consisted of 3" nominal thickness CCA treated hardwood lagging. All anchors were installed into the gray shale and limestone (Kope) rock formation. Strand counts in the anchors varied from 3 to 15 strands per anchor. Each lagged bay received a four-foot wide chimney drain of a geotextile drainage material. Reinforced concrete facing was placed over the lagging and geotextile drainage mats. The facing had a structural thickness of 13" and an overall thickness of up to 19". The geotextile drains led to a 3" weep hole, 12" above final grade in front of the wall.

Immediately after we were awarded the project, work started on the final calculations and working drawings. Due to project phasing requirements, a package for each of the four walls was developed. Each wall package was submitted in the order it would be built. The packages for this wall were submitted:



BOTTOM of FACING ABOVE B/RDZ

BOTTOM of FACING BELOW B/RDZ

LEFT (DOWNHILL) WALLS:
K_L VARIES ALONG
LENGTH of WALL
$0.032 < K_L < 0.061$

RIGHT (UPHILL) WALLS:
K_L CONSTANT;
$K_L = 0.040$

P_D = DESIGN PRESSURE = $1.4\ K_L\ H$

Figure 3

- Wall R1 – Oct, 1998
- Wall L2 – Nov, 1998
- Wall R2 – Dec, 1998
- Wall L1 – Dec, 1998

Each package was submitted to the KTC for their review. Once any difficulties or problems were resolved, we were authorized by the KTC to provide "For Construction" mylar reproducibles. The KTC printed and distributed the drawing to the various parties involved. The "For Construction" mylars were submitted:

- Wall R1 – Dec, 1998
- Wall L2 – Jan, 1999
- Wall R2 – Jan, 1999
- Wall L1 – Feb, 1999

Construction

After receiving the Notice to Proceed in September 1998, utility work, clearing, grubbing and other preliminary work started at the site in October 1998. Retaining wall work started on Wall R1 on December 7, 1998. The project documents call for completion in May 2000.

One of the contract requirements was that a Partnering Agreement be developed between the owner, KTC and all the contractors. A subcontractor hired by the JV provided this service. To date, this process has been working very well, with a true spirit of co-operation by all the parties involved.

Another unique element of this project is the Quality Assurance/Quality Control aspect. The owner is typically responsible for this portion of the work. However, the KTC decided to have the general contractor provide these services. The H. C. Nutting Company of Cincinnati, Ohio was hired to 1) develop quality control procedures for each operation required for the completion of the work and 2) provide the necessary field inspection required by the procedures. The QA/QC information was submitted to the KTC for their approval. Once a QA/QC procedure was approved, the JV was obligated to follow the means and methods developed. The KTC is using this project as a pilot for this method of QA/QC. To date, this process has performed quite well and the KTC is quite satisfied with the process.

As far as the actual construction is concerned, two project requirements dictated the original construction phasing and traffic control for this project. First, traffic could not be stopped or detoured. Accordingly, traffic had to be maintained on existing KY9 until it could be diverted onto completed portions of the new work. Second, all paving had to be accomplished during the paving season, which runs from April 1 to November 15 each year.

Keeping the proceeding in mind, the initial work plan consisted of the following steps:

- Build Wall R1 and Wall L2, maintaining traffic on existing KY9
- Construct the new left lanes up to the planned "S" switchover
- Construct a temporary road from exist KY9 to the new left lanes north of the planned "S" switchover
- Abandon exist KY9 north of the "S" switchover
- Construct the new right lanes to point when "S" switchover can be constructed
- Divert traffic onto the new right lanes south of "S" switchover so that traffic is now travelling on the new right lanes, through the "S" switchover, to the new left lanes
- Abandon exist KY9 south of the "S" switchover
- Build Wall R2 and Wall L1
- Complete excavation, drainage and paving to complete project

As work progressed on phase 1, it became apparent that the bulk of the work could be completed by the end of November 1999 if existing KY9 could be closed and the traffic re-routed. The only items to be completed during the next paving season (spring 2000) were the final wearing coarse of asphalt and the final pavement striping. A four-month closure would allow the entire length of the new highway to be re-opened full width six to eight months ahead of the original schedule. There is a very good possibility that the entire project can be completed by the end of November 1999, 10 months ahead of the original completion date.

A detailed schedule for this approach was developed and presented to the KTC for the comment and approval. The plan was ultimately accepted and KY9 was closed to traffic on June 23, 1999 and will remain closed until November 30, 1999.

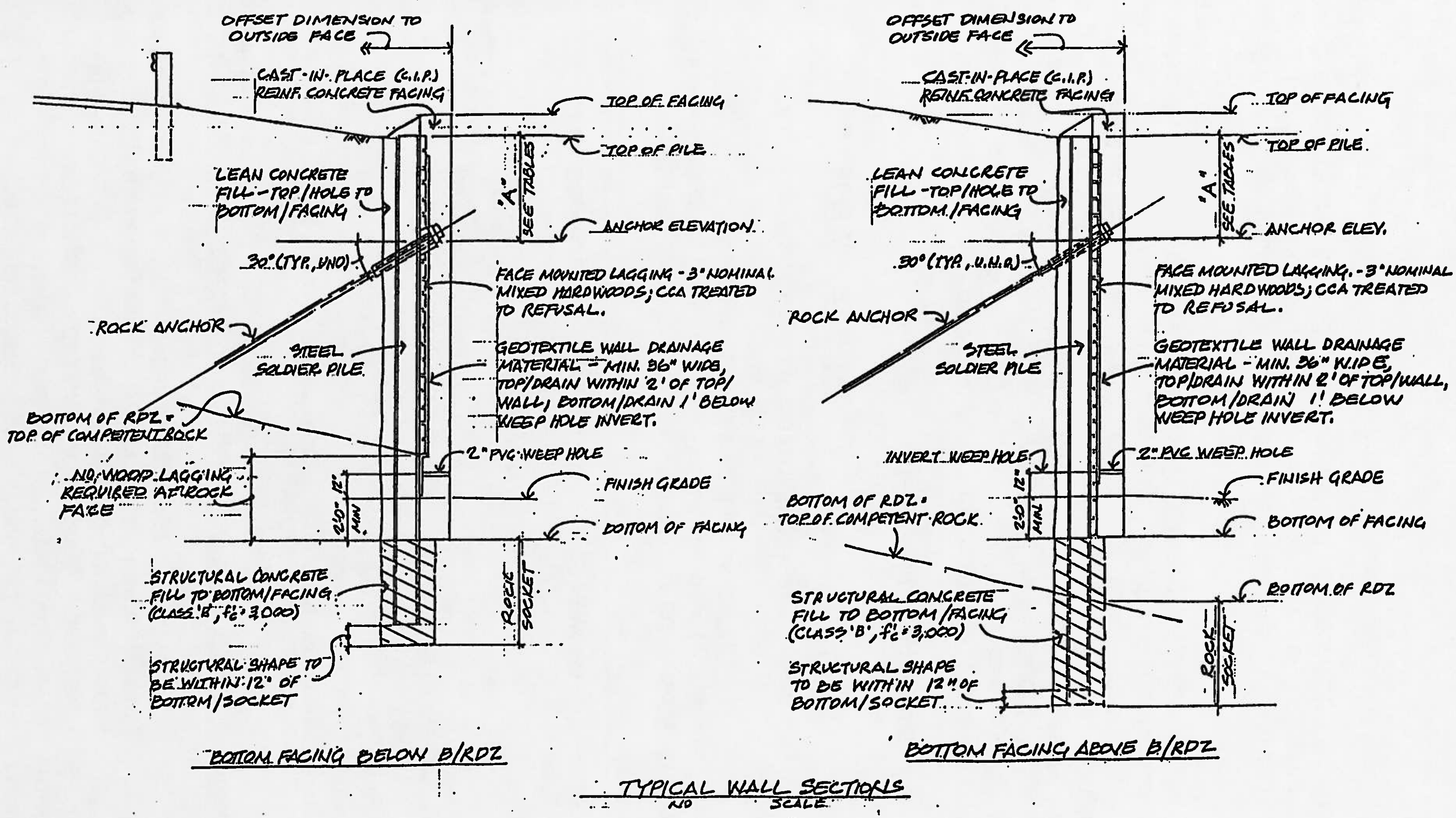

OFFSET DIMENSION TO OUTSIDE FACE
CAST-IN-PLACE (C.I.P.) REINF. CONCRETE FACING
TOP OF FACING
TOP OF PILE
LEAN CONCRETE FILL-TOP/HOLE TO BOTTOM/FACING
"A" SEE TABLES
ANCHOR ELEVATION
30° (TYP., UNO)
FACE MOUNTED LAGGING - 3" NOMINAL MIXED HARDWOODS; CCA TREATED TO REFUSAL.
ROCK ANCHOR
STEEL SOLDIER PILE
GEOTEXTILE WALL DRAINAGE MATERIAL - MIN. 36" WIDE, TOP/DRAIN WITHIN 2' OF TOP/WALL, BOTTOM/DRAIN 1' BELOW WEEP HOLE INVERT.
BOTTOM OF RDZ = TOP OF COMPETENT ROCK
NO WOOD LAGGING REQUIRED AT ROCK FACE
2" PVC WEEP HOLE
FINISH GRADE
2'-0" MIN
12"
BOTTOM OF FACING
STRUCTURAL CONCRETE FILL TO BOTTOM/FACING (CLASS 'B', f'c = 3,000)
ROCK SOCKET
STRUCTURAL SHAPE TO BE WITHIN 12" OF BOTTOM/SOCKET
BOTTOM FACING BELOW B/RDZ
OFFSET DIMENSION TO OUTSIDE FACE
CAST-IN-PLACE (C.I.P.) REINF. CONCRETE FACING
TOP OF FACING
TOP OF PILE
LEAN CONCRETE FILL-TOP/HOLE TO BOTTOM/FACING
"A" SEE TABLES
ANCHOR ELEV.
30° (TYP., U.N.O.)
FACE MOUNTED LAGGING - 3" NOMINAL MIXED HARDWOODS; CCA TREATED TO REFUSAL.
ROCK ANCHOR
STEEL SOLDIER PILE
GEOTEXTILE WALL DRAINAGE MATERIAL - MIN. 36" WIDE, TOP/DRAIN WITHIN 2' OF TOP/WALL, BOTTOM/DRAIN 1' BELOW WEEP HOLE INVERT.
INVERT WEEP HOLE
2" PVC WEEP HOLE
FINISH GRADE
2'-0" MIN
12"
BOTTOM OF RDZ = TOP OF COMPETENT ROCK
BOTTOM OF FACING
BOTTOM OF RDZ
STRUCTURAL CONCRETE FILL TO BOTTOM/FACING (CLASS 'B', f'c = 3,000)
STRUCTURAL SHAPE TO BE WITHIN 12" OF BOTTOM/SOCKET
ROCK SOCKET
BOTTOM FACING ABOVE B/RDZ
TYPICAL WALL SECTIONS
NO SCALE

Figure 4

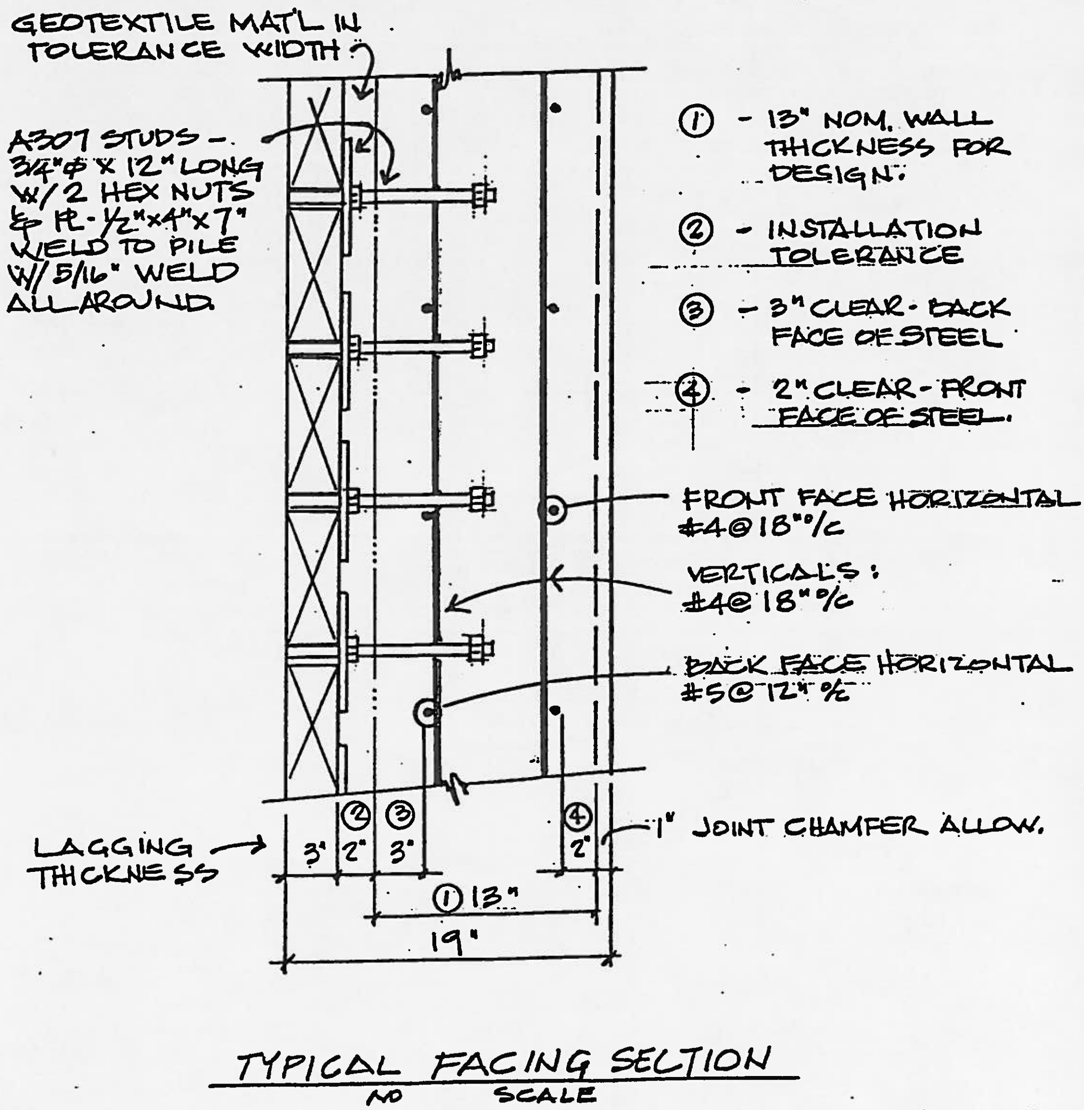

GEOTEXTILE MAT'L IN TOLERANCE WIDTH
A307 STUDS - 3/4"Φ x 12" LONG W/ 2 HEX NUTS & PL-1/2"x4"x7" WELD TO PILE W/ 5/16" WELD ALL AROUND.
① - 13" NOM. WALL THICKNESS FOR DESIGN.
② - INSTALLATION TOLERANCE
③ - 3" CLEAR - BACK FACE OF STEEL
④ - 2" CLEAR - FRONT FACE OF STEEL.
FRONT FACE HORIZONTAL #4@18" o/c
VERTICALS: #4@18" o/c
BACK FACE HORIZONTAL #5@12" o/c
LAGGING THICKNESS
1" JOINT CHAMFER ALLOW.
3" 2" 3" 2"
① 13"
19"
TYPICAL FACING SECTION
NO SCALE

Figure 5

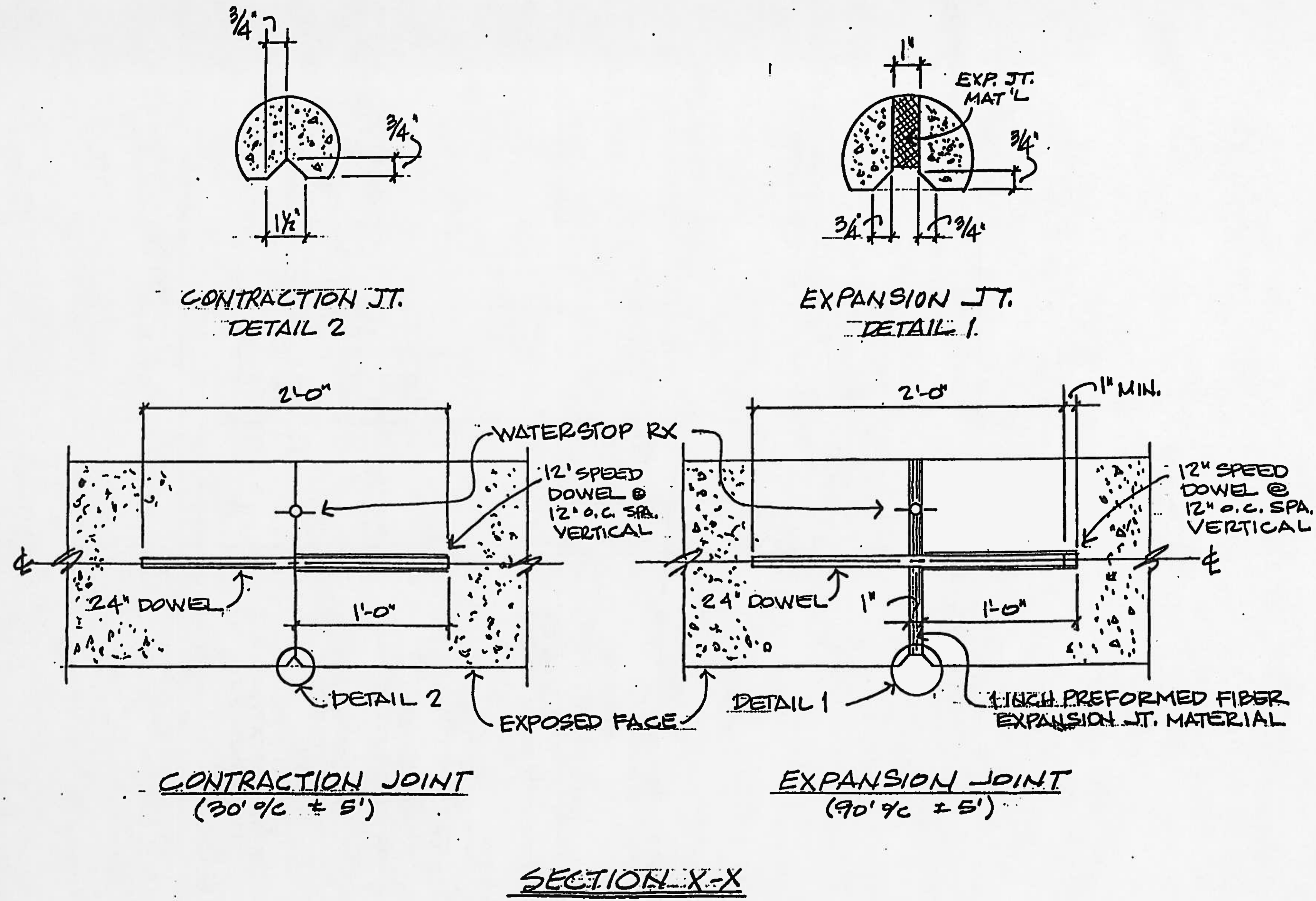

3/4"
1½"
3/4"
CONTRACTION JT.
DETAIL 2
1"
EXP. JT. MAT'L
3/4"
3/4"
3/4"
EXPANSION JT.
DETAIL 1
2'-0"
WATERSTOP RX
12' SPEED DOWEL @ 12" o.c. SPA. VERTICAL
24" DOWEL
1'-0"
DETAIL 2
EXPOSED FACE
2'-0"
1" MIN.
12" SPEED DOWEL @ 12" o.c. SPA. VERTICAL
24" DOWEL
1"
1'-0"
DETAIL 1
1 INCH PREFORMED FIBER EXPANSION JT. MATERIAL
CONTRACTION JOINT
(30' o/c ± 5')
EXPANSION JOINT
(90' o/c ± 5')
SECTION X-X

Figure 6

Soldier piles were installed in 30" or 36" diameter drilled holes, socketed into the gray shale and limestone. These holes were filled with structural concrete to the bottom of facing elevation. The balance of the hole was filled with lean concrete fill. Rock socket lengths varied from 5 to 8 feet. The piles were made of a pair of identical structural shapes. Typically, an 8" gap between the shapes was created for the installation of the rock anchors. However, some of the larger/heavier shapes required the anchor gap to be narrowed. These piles had to be notched and/or coverplated. (See Figure 4).

The temporary facing consisted of CCA treated hardwood lagging. The planks were a nominal 3" thick. The lagging was attached to the front face of the soldier piles using threaded studs and plates. (See Figure 5). Note that the studs used to attach the lagging also acted as the structural attachment for the concrete facing.

Rock anchors were made up of 0.6" diameter, 7-wire strand. Strand count per anchor varied between 3 and 15 strands and anchor loads ranged from 72 kips to 420 kips. The Special Note called for a minimum stressing length of 15 feet and a minimum bond length of 15 feet. Actual stressing lengths ranged from 15 to 40 feet. Actual bond lengths ranged from 15 to 45 feet. The rock anchors were also furnished with double corrosion protection details, consisting of greased and sheathed strands; full length corrugated sheathing and cement grout. The bond length was drilled in the rock using either 6 or 7 inch drill bits. Anchors were either proof, performance or creep performance tested to 150% DL and then locked-off at 75% DL. Due to existing grades and extensive fill requirements behind several stretches of the walls, no-acceptance test areas were designated. Anchors in these areas were simply loaded to 50% DL and locked-off at that point. No formal testing was performed on the "no-acceptance test" anchors.

The permanent facing for the wall was constructed of reinforced concrete. A total facing thickness of 19" from the face of the steel soldier pile could be subdivided into the following segments:

- 1" allowance for the architectural treatment
- 2" allowance as an installation tolerance
- 3" for the lagging thickness
- 13" structural wall thickness

The main steel was located horizontally, as the wall spanned in the stiffer, short dimension between the piles. This steel consisted of $^{\#}5$'s @ 12" $^{o}/_{c}$ horizontal in the back face and $^{\#}4$'s @18" $^{o}/_{c}$ horizontal in the front face. Vertical temperature steel consisted of #4's @ 18" $^{o}/_{c}$ vertical in both faces. (See Figures 5 and 6).

Since the right or uphill walls would be visible from the roadway, they received an architectural treatment consisting of alternating plain and vertically striated panels. The use of form liners created the vertical striations. Since the left or downhill walls would be hidden from the traffic's view, they were detailed to have a plain concrete finish. Both the uphill and downhill walls were scheduled to receive a concrete coating and anti-graffiti coating. It is interesting to note that while the walls received a special coating to counter the work of our urban artists, the KTC stated the most effective deterrent for the graffiti was the vertical architectural treatment.

The facing was placed using one-sided forms, supported by clips and heavy duty form ties welded to the soldier piles. The facing forms rested on a leveling pad placed prior to the start of form installation. Construction joints were placed at roughly 30-foot intervals and expansion joints were detailed at about 90-foot centers. The joints contained both waterstop and dowels.

The top of the walls were detailed to have a drainage ditch running behind the entire length of the wall. These ditches emptied into concrete drainage structures located behind the wall. At each drainage structure the wall was penetrated to allow an outlet pile to pass through. The outlet was then piped across the highway or allowed to empty into a ditch in front of the wall. Also, each lagged bay was drained by means of a 3" weep hole.

Buried utilities crossing the wall line at various locations were 8" and 12" sanitary sewers, a 12" water main and a 24" high-pressure gas line. Overhead utilities crossing the wall line were high and low voltage electric services, telephone, CATV and fiber-optic cables. The overhead utilities required extensive co-ordination so as to maintain service to the surrounding area while these utilities were moved to allow construction to proceed. It should be noted that a complex web of utilities complicated the construction phasing greatly. In fact, utility maintenance often dictated the locations that were available for various starting and stopping points for the work phases.

Closing

A project of this magnitude and high profile could not be completed without the cooperation of all the various parties – both owner and contractor. The following organizations deserve recognition for their part in the development and construction of this project.

The owners, the Kentucky Transportation Cabinet, had the following consulting team:

- Wilber Smith & Assoc. – Primary design engineer
- Law Engineering – Geotechnical engineering
- Roenke-Bates Consultants – Partnering

The Joint Venture had the following team of subcontractor and suppliers:

Subcontractors:

- Soldier Pile Installation – Case Foundation Co.
- Excavation – Carlisle Construction Co.
- Facing – Baker Concrete Construction, Inc.
- Paving – Barrett Paving Co.
- Drainage & Utilities – Spartan
- Traffic Control – A&A Safety, Inc.
- Quality Assurance – H. C. Nutting Co.
- Horizontal Drains – Jensen Drilling Co.
- Painting – Kretan Painting, Inc.
- Fence & Guardrail – Security Fence Co.

Suppliers:

- Concrete – Turnbull Concrete Co.
- Tendons – Lang Tendons, Inc.
- Steel – R. W. Conklin; Skyline Steel
- Lagging – Johnson Doppler Lumber Co.
- Misc. Metal – O'Neal Steel, Inc.
- Anchor Bulk Cement – Nurre Building Supplies

The joint venture is continually grateful for the dedication and efforts provided by these firms.

DETERMINING THE CAPACITY OF UNKNOWN FOUNDATIONS USING NON DESTRUCTIVE TESTING

By
Bruce G. Stegman, PE [A]
J Darrin Holt, Ph.D., PE [B]

Using a program of mostly non-destructive-testing, the foundation capacity of a nearly 100-year old bridge was determined at a fraction of the cost of what a conventional approach. Initial proposals to determine the foundation size, depth, and integrity using an invasive/destructive approach of test pits, borings, and coring was substantial. The municipality accepted an alternative proposal to perform a combination of non-destructive testing and limited destructive testing. The program incorporated Ground Penetrating Radar (GPR), dispersive technology, and limited borings. GPR was used to determine the reinforcing in the abutments, thickness of the abutment, and if abutments and piers had footings. Dispersive wave technology was used to determine the thickness of the abutment, the depth of abutment, and the general quality of the concrete. Conventional borings were done to sample the concrete, calibrate the NDE, and determine the founding materials. Using this combination of complementary NDE and limited destructive testing, very few assumptions were necessary to determine the foundation capacity and integrity of the nearly 100-year old foundations at a fraction of the cost of an invasive/destructive evaluation.

INTRODUCTION

Using a program of mostly non-destructive-testing, the foundation capacity of a nearly 100-year old bridge was determined at a fraction of the cost of a conventional approach. A local municipality found itself in the unusual position of having to justify replacing an obsolete bridge over a railroad. When the municipality concluded that the single lane, weight limited bridge should be replaced, some local residents mounted a campaign to salvage the old bridge, citing it's ambience with the surroundings. Aside from the superstructure, the municipality needed to demonstrate that the substructures were unsalvageable. The initial proposal specified determining the foundation size, depth, and integrity using an invasive/destructive approach of test pits, borings, and coring; the cost for this work was substantial. Faced with the prospect of "throwing good money after bad", the municipality accepted an alternative proposal to perform a combination of non-destructive testing and limited destructive testing at a fraction of the cost. The program included the use of Ground Penetrating Radar (GPR), dispersive wave technology, and limited borings through the bridge.

GPR was used to determine if reinforcing was present in the abutments, determine the thickness of the abutment, verify that abutments and piers did not have footings, and check for anomalies in the concrete. Dispersive wave technology was used to determine the abutment depth, thickness, and general quality of the concrete. Conventional borings were drilled (at night) through the abutments to sample the concrete, verify/calibrate the Non-Destructive Evaluation (NDE) methods, and determine the founding materials. Using this combination of

[A] Manager, Rettew Geotechnical & Foundation Engineering, 24 Coupler Drive, Stewartstown, PA 17363, 888.333.1566, stegman@nfdc.net.
[B] Vice President & Principal Scientist, FDH Inc. , 521 Uwharrie Court, Raleigh NC 27606, 919.755.1012, fdh@mindspring.com.

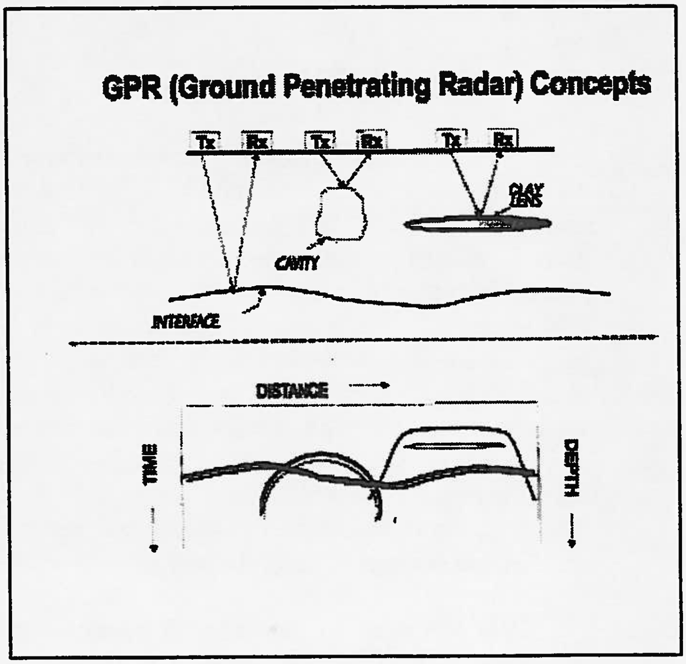
complementary NDE testing and limited destructive testing, very few assumptions were necessary to determine the capacity and integrity of the nearly 100-year old foundations.

CURRENT TECHNOLOGIES

Although there exists numerous other Non-Destructive techniques/technologies, this paper describes the use of Ground Penetrating Radar, Dispersive Wave and other stress wave methods, and magnetics. New technologies such as Electrical Resistivity Tomography, Cross-Hole Seismic Tomography, and Spectral Analysis of Surface Waves are also given a brief discussion at the end.

Ground Penetrating Radar (GPR)

GPR has been used to identify reinforcement (or verify the absence of reinforcing) in the concrete, determine concrete thickness, and locate anomalies in the concrete. GPR can also be used to find dimensions and depths below grade of buried footings. GPR transmits UHF and VHF frequencies (10 MHz to 2 GHz); at a very low radiated power (0.6-100 milli-watts). Antennas transmit electromagnetic energy and receive reflections from boundaries between materials that have a dielectric contrast, see Figure 1 [1].



The depth of penetration and resolution are dependent upon antenna frequency but more importantly the subsurface dielectric permittivity and conductivity. The greater depth of penetration sacrifices resolution. The lower the frequency, the greater the depth of penetration; however, the larger antenna has less resolution and is physically larger. Conversely, the higher the frequency, the greater the resolution; however, the depth of penetration is greatly reduced.
Objects with diameters on the order of millimeters (such as wire mesh) can be detected using high frequency (~1 Ghz) antenna. Objects such as geologic anomalies that could be as large as hundreds of meters can also be detected tens of meters deep using low frequency (~100mhz) antenna.

Unlike seismic reflection, radar waves travel at near the speed of light. Antennas "fire" or transmit 44,000 times a second; and the radiated beam is directional or focused. By focusing the beam, reflections are primarily due to objects directly beneath the antenna. A simplified radiation pattern is given in Figure 2.

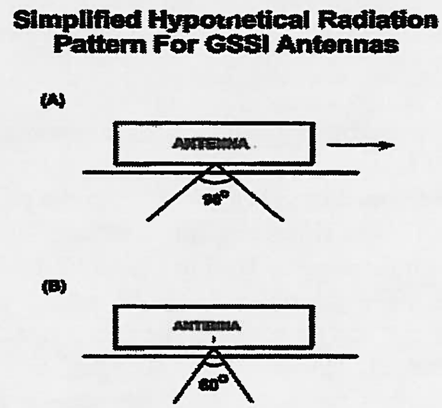


Figure 2

Penetration and resolution are most effected by:

- Dielectric Permittivity or Dielectric Constant; a dimensionless measure of the capacity of a material to store a charge when an electric field is applied. The lower the dielectric constant, the greater the radar signal will penetrate.
- Electrical Conductivity: This is the inverse of resistivity, higher the conductivity deeper radar signal penetration difficult (it is noteworthy that conventional resistivity or conductivity testing is a good indicator of a sites suitability for GPR).

For estimating depths and thickness from GPR to the required accuracy, it is almost essential to calibrate the system using objects of known depth or thickness. Standard velocities and published dielectric constants are adequate for field checking the data, but could introduce appreciable uncertainty.

Dispersive Wave

Dispersive wave propagation methods to nondestructively determining the length and condition of piles was pioneered by FDH [2]. FDH through their licensees, offers dispersive wave technology for pile testing and other types of substructures.

Dispersive wave technology incorporates the change in shape of the stress waves or signals as they propagate through materials. Dispersion is an almost unavoidable phenomenon wherein individual frequencies in a signal travel at their own velocity. Conventional signal analysis techniques for dispersive behavior are based upon the Fourier transform. Such methods find relative phase angle for individual frequencies between two gage locations. This relative phase is used to determine the time required for the frequencies to travel a know distance, thus allowing a phase velocity to be computed. There is no way, however, to tell if the computed phase is the actual value or whether the actual values is the computed value plus some integer multiple of the frequency's period. For this reason, the Fourier transform method is inherently difficult to use for finding wave speeds and travel times of frequency components in a dispersive signal.

A typical set up for side impact is given in Figure 3.

An example signal trace in presented in the Figure 4. In the A-A' region, the wave is recorded as it travels directly from the point of impact towards the bottom of the element; the lower trace slightly behind (in time) the upper trace. The signal in the B-B' region is the wave that travels towards the pile top, reflects then travels downward. The signal in the C-C' region is the wave, originally seen in A-A', but after reflecting from the bottom of the element to then travel upward. The bottom reflection signal can be identified as the reflected wave because the bottom trace now leads the top trace in time.

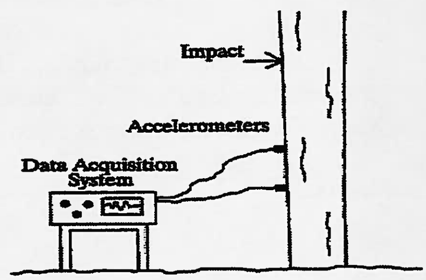


Figure 3

Digital signal processing of these signals by Fourier transform and other methods yields wave speed and distances of travel of selected frequencies inside the signals, see Figure 5 which gives an example of the signal from above reduced to one frequency component.

By applying this advanced digital signal processing to the individual frequencies, the effects of wave "*dispersion*" can be eliminated or minimized to improve accuracy, and also "see" quantitatively anomalies that conventional methods could only infer.

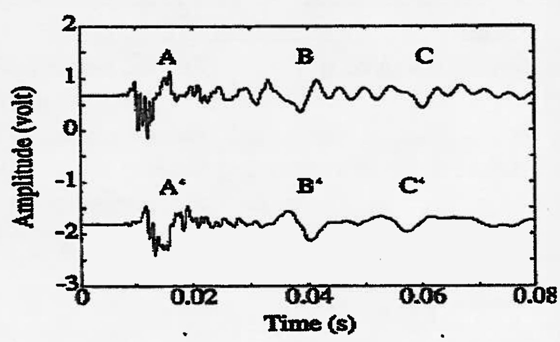


Figure 4

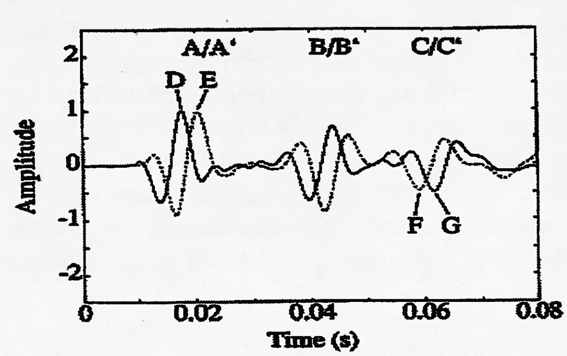


Figure 5

CASE HISTORY & EXAMPLES

90-Year Old Bridge Local Road over CSXT

The subject bridge is believed to have been constructed in 1903. The initial plans indicate the abutments are tapered pedestals with no footing. Using a combination of complementary NDE and limited destructive testing, very few assumptions were necessary to determine the structure dimensions and integrity. Not only were the dimensions of the structure determined, but serious defects in the concrete (cold joints) were clearly identified.

Abutments

A typical trapezoidal abutment sub-structure is shown on Figure 6. Based on the assumption that the slope of the front and back were constant, the dimensions at the base were approximated by:

- Measuring the front slope;
- Using NDE to measure the thickness at various locations (longitudinally and vertically) and thereby calculate the back slope or change in thickness with depth;
- Using NDE to determine the depth of the concrete;
- Drilling through the structure to calibrate the NDE, and also provide quantitative data on depth and quality of concrete and quality of founding material.
- Calculate the width at the base using the parameters above.

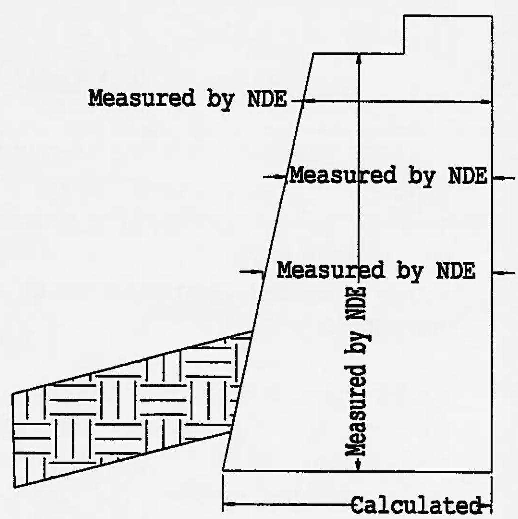


Figure 6

West Abutment- About 7 to 8-feet of the face of the west abutment was exposed. This face exposure allowed for three radar scans and three Dispersive wave profiles. Based on nearby subway borings, it was suspected the abutment extended down 18 feet below grade and was founded on medium dense residual soil.

East abutment – This substructure consisted of a very shallow pedestal (about 2-feet) on about 3-feet of sub-foundation concrete, the cover of which had eroded over time exposing both the bottom of the footing, and bottom of sub-foundation concrete. Based on rock outcrops, it was suspected that the bottom of sub-foundation concrete was exposed and founded on completely weathered rock. The testing on this abutment was similar to the West Abutment, except that the thickness of the abutment was determined at fewer locations because much less face was exposed. There was only enough face width to perform one line of GPR. The NDE depth analysis looked for both the bottom of footing, but also bottom of sub-foundation concrete.

Pier Footings

The initial (1903) plans indicate the footings were a tapered pedestal, the absence of a footing was confirmed with GPR. Based on the assumption that one side slope was constant for each face of the four piers, the dimensions at the base of the footing were approximated by:

- Measuring the side slope;
- Determine the depth of the pier;
- Calculate the width at the base using the parameters above, see Figure 7.

The depth of the pier was determined at each pier location using NDE. To determine the depth of the old pier footings required access to the old top of pier, thus, a hole several feet deep was excavated, see Figure 7.

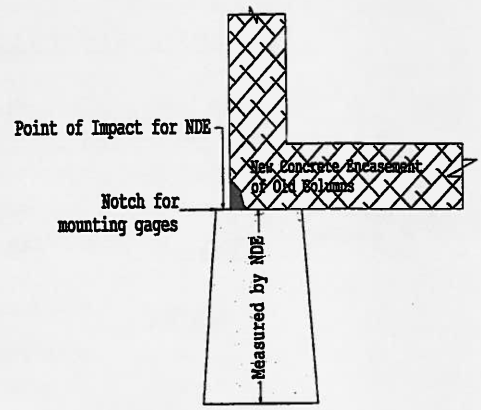


Figure 7

Ground Penetrating Radar (GPR)

GPR was used to locate reinforcing, identify anomalies in the concrete, and determine if a footing was present. GPR surveys were taken across the face of the abutment using a 400-mhz antenna. Where possible, three parallel lines at different elevations were performed. The 400-mhz antenna is small and light enough to be held overhead. The antenna was equipped with a survey wheel that created horizontal control (or scale) by inserting a tick mark (in the data field) every meter of scan.

The same antenna was used to look for footings. Scans were done parallel to the abutments and piers at a progressively greater distance from the substructure. The penetration using a 400-mhz antenna can be somewhat limited in materials with a high dielectric constant. A 200-mhz antenna was also on site should the 400-mhz proved to have limited penetration.

Dispersive Wave

For this application, dispersive wave technology was used to measure the thickness and depth of the structure, and determine the general quality of the concrete. Along the face of the west abutment, tests were performed over a three by three grid pattern; rows were typically 4-feet apart horizontally and 8-feet apart vertically. Along the top of the abutment, three tests were also performed. Three tests were performed along the top of the east abutment. There was inadequate face width for any tests on the face.

At two footing locations, east and west piers, tests were performed to determine the depth of the concrete, i.e. footing embedment. The taper of the footing was determined by exposing a portion of each side of the footing.

Conventional Borings

Conventional borings were drilled, one through the east and west abutment, to allow for verification and calibrate both NDE methods. Samples from the borings also provided specific data about the concrete quality, strength, and thickness. Borings were also drilled (through the bridge deck) adjacent to each of the piers.

Drilling required closing the bridge, but daytime bridge closure was not permitted. Thus, borings were drilled at night, typically 9:00 PM to 5:00 AM. Working at night afforded several major benefits. Closing of the bridge allowed borings to be drilled through the existing abutments, thus allowing for determination of the abutment depth, consistency of the concrete, and direct evaluation of the bearing materials beneath the foundations. A conventional (less costly) truck mount drill rig was used, and the premium costs for night drilling are minor.

Abutment borings were drilled through the deck, avoiding bridge girders, floor beams, and stringer beams. Pier borings were drilled through the deck also.

Cost Analysis

The NDE program described herein was done for about $12,000.00. This included the cost of the borings, GPR, dispersive wave, limited traffic control, and a final report. For comparison, the traditional approach was 50 to 100 percent more costly.

OTHER TECHNOLOGIES

New technologies such as Electrical Resistivity Tomography, Cross-Hole Seismic Tomography, and Spectral Analysis of Surface Waves are also given a brief discussion below. These technologies also have potential to locate unknown foundations.

Spectral Analysis of Surface Waves

The propagation of surface waves have been used to estimate the elastic parameters of surface materials based on the propagation of Rayleigh or surface waves [3].

Electrical Tomography

Electrical resistivity has long been used as a geophysical method, with newer switching and processing ability; subsurface profiles can be developed using Tomography technology. The technology is based on the conventional resistivity survey principles, using Swift Smart Electrode System, manufactured by Advanced Geosciences, Inc. (AGI). The system is designed for efficient acquisition of large amounts of resistivity data when performing resistivity imaging (Tomography) surveys. A complete system consists of an interface box and up to 254 electrode switches (smart electrodes) placed on electrode stakes and connected by a multi-lead cable to the central interface unit. The switches allow for innumerable combinations of dipole-dipole survey or any programmed array (i.e. Schlumberger, Wenner, pole-pole, pole-dipole, square array, etc.). Measurements are taken by the Sting and are stored in the internal memory or on a PC hard disk. The resistivity inversion program, RES2DINV, provided by AGI, is used to simulate the resistivity data to create a 2-dimensional model (profile) of the subsurface resistivity for each resistivity spread.

Seismic Tomography

Like electrical resistivity, seismic reflection and refraction have long been used as a geophysical method. With newer processing and acquisition ability, subsurface profiles can also be developed using cross-hole seismic methods and Tomography technology. Cross-hole seismic consists of multiple geophones locations and multiple source locations are used to develop a velocity profile. The velocity image generated for each of the pair of boreholes can be combined to produce maps of the subsurface between the boreholes, and when combined with other boreholes, it can be used to produce a three dimensional image. Velocity images can be correlated to density/stiffness images.

Magnetics

A relatively inexpensive hand held "Pach-O-Meter" can be used to find near surface reinforcing and tendons, see Figure 8[4]. This is a small hand-held digital magnetic device that generates a field and measures the changes in the field due to the presence of magnetic materials. A digital display records the distance from the probe to the bar or tendon. Closely spaced parallel bars and/or bars that lie above each other (such as a splice) can make data interpretation difficult. The depth of penetration is generally 8 to 12 inches depending upon bar size.

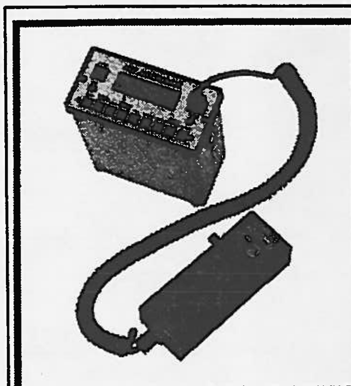


Figure 8

It has also been used to locate floor beams beneath a concrete floor. The floor consisted of about 2 to 3 inches of concrete over a corrugated metal pan floor. Interestingly, the magnetic device could "see through" the metal floor whereas GPR would not have been able to penetrate the corrugated metal floor.

SUMMARY

Using a combination of complementary NDE and limited destructive testing, very few assumptions are necessary to determine the structure dimensions and integrity at a cost of two-thirds to one-half the cost of an invasive/destructive evaluation. A project using this type of approach was recently completed where not only were the dimensions of the structure determined, but serious defects in the concrete (cold joints) were clearly identified.

REFERENCES

1 Geophysical Survey Systems, 13 Klein Drive, North Salem, NH 03073-0097, 800.524.3001, gssisales@gssi.com

2 FDH Inc. 521 Uwharrie Court, Raleigh NC 27606, 919.755.1012, fdh@mindspring.com.

3 Nondestructive Testing for Geotechnical Applications, Richard D. Woods, Proceeding Central Pennsylvania ASCE Seminar, April 15, 1999, Hershey Pennsylvania.

4 REBAR DATASCAN C-4974, James Instruments Inc, 3727 North Kedzie Avenue, Chicago, IL 60618, 800.426.6500

5. Bendat, J.S. and Piersol, A.G., Engineering Applications of Correlation and Spectral Analysis, John Wiley and Sons, Inc. New York, 1980.

6. Douglas, R.A., and Holt, J.D., Determining Length of Installed Timber Piling by Dispersive Wave Propagation Methods, Research Project Number 23421-92-2, North Carolina Department of Transportation, June 1993.

7. Holt, J.D., Chen, S., and Douglas, R.A., Determining Lengths Installed Timber Piles by Dispersive Wave Propagation, Transportation Research Record No 1447, TRB, National Research Council, Washington, D.C., 1994, pages 110-115.

8. Holt, J.D., Finding the Length of Installed Steel H Piles by Dispersive Wave Propagation Methods, Proceedings of the Fifth International Conference on the Application of Stress Waves Theory to Piles, page 1172, September 11-13, 1996, Orlando, FL.

9. Jalinoos, F., Aouad, M., and Olsen, L., Three Stress-Wave Methods for the Determination of Unknown Pile Lengths, Proceedings of the Fifth International Conference on the Application of Stress Waves Theory to Piles, page 673, September 11-13, 1996, Orlando, FL.

10. Yew, C.H., and Chen, C.S., Study of Linear Wave Motions using FFT and its Potential Application to Non-Destructive Testing, International Journal of Engineering Science, Volume 18, 1980, pages 1027-1036.

APPENDIX
PAST OHIO RIVER VALLEY SOILS SEMINARS

ORVSS I	*BUILDING FOUNDATION DESIGN AND CONSTRUCTION* October 16, 1970, Lexington, KY
ORVSS II	*EARTHWORK ENGINEERING, START TO FINISH* October 15, 1971, Louisville, KY
ORVSS III	*LATERAL EARTH PRESSURES* October 27, 1972, Fort Mitchell, KY
ORVSS IV	*GEOTECHNICS IN TRANSPORTATION ENGINEERING* October 5, 1973, Lexington, KY
ORVSS V	*ROCK ENGINEERING* October 18, 1974, Clarksville, IN
ORVSS VI	*SLOPE STABILITY AND LANDSLIDES* October 17, 1975, Fort Mitchell, KY
ORVSS VII	*SHALES AND MINE WASTES:* *GEOTECHNICAL PROPERTIES, DESIGN, AND CONSTRUCTION* October 8, 1976, Lexington, KY
ORVSS VIII	*EARTH DAMS AND EMBANKMENTS:* *DESIGN, CONSTRUCTION, AND PERFORMANCE* October 14, 1977, Louisville, KY
ORVSS IX	*DEEP FOUNDATIONS* October 27, 1978, Fort Mitchell, KY
ORVSS X	*GEOTECHNICS OF MINING* October 5, 1979, Lexington, KY
ORVSS XI	*EARTH PRESSURES AND RETAINING STRUCTURES* October 10, 1980, Clarksville, IN
ORVSS XII	*GROUNDWATER: MONITORING, EVALUATION, AND CONTROL* October 9, 1981, Fort Mitchell, KY
ORVSS XIII	*RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE* October 8, 1982, Lexington, KY
ORVSS XIV	*FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION* October 14, 1983, Clarksville, IN
ORVSS XV	*PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING* November 2, 1984, Fort Mitchell, KY
ORVSS XVI	*APPLIED SOIL DYNAMICS* October 11, 1985, Lexington, KY
ORVSS XVII	*NATURAL SLOPE STABILITY AND INSTRUMENTATION* October 17, 1986, Clarksville, IN

APPENDIX
PAST OHIO RIVER VALLEY SOILS SEMINARS (continued)

ORVSS XVIII	*LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION* November 6, 1987, Fort Mitchell, KY
ORVSS XIX	*CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES* October 21, 1988, Lexington, KY
ORVSS XX	*CONSTRUCTION IN AND ON ROCK* October 27, 1989, Louisville, KY
ORVSS XXI	*ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING* October 26, 1990, Fort Mitchell, KY
ORVSS XXII	*DESIGN AND CONSTRUCTION WITH GEOSYNTHETICS* October 18, 1991, Lexington, KY
ORVSS XXIII	*IN-SITU SOIL MODIFICATION* October 16, 1992, Louisville, KY
ORVSS XXIV	*GEOTECHNICAL ASPECTS OF INFRASTRUCTURE RECONSTRUCTION* October 15, 1993, Fort Mitchell, KY
ORVSS XXV	*RECENT ADVANCES IN DEEP FOUNDATIONS* October 21, 1994, Lexington, KY
ORVSS XXVI	*SITE INVESTIGATIONS: GEOTECHNICAL AND ENVIRONMENTAL* October 20, 1995, Clarksville, IN
ORVSS XXVII	*FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING* October 11, 1996, Cincinnati, OH
ORVSS XXVIII	*UNCONVENTIONAL FILLS: DESIGN, CONSTRUCTION, AND PERFORMANCE* October 10, 1997, Lexington, KY
ORVSS XXIX	*PROBLEMATIC GEOTECHNICAL MATERIALS* October 16, 1998, Louisville, KY