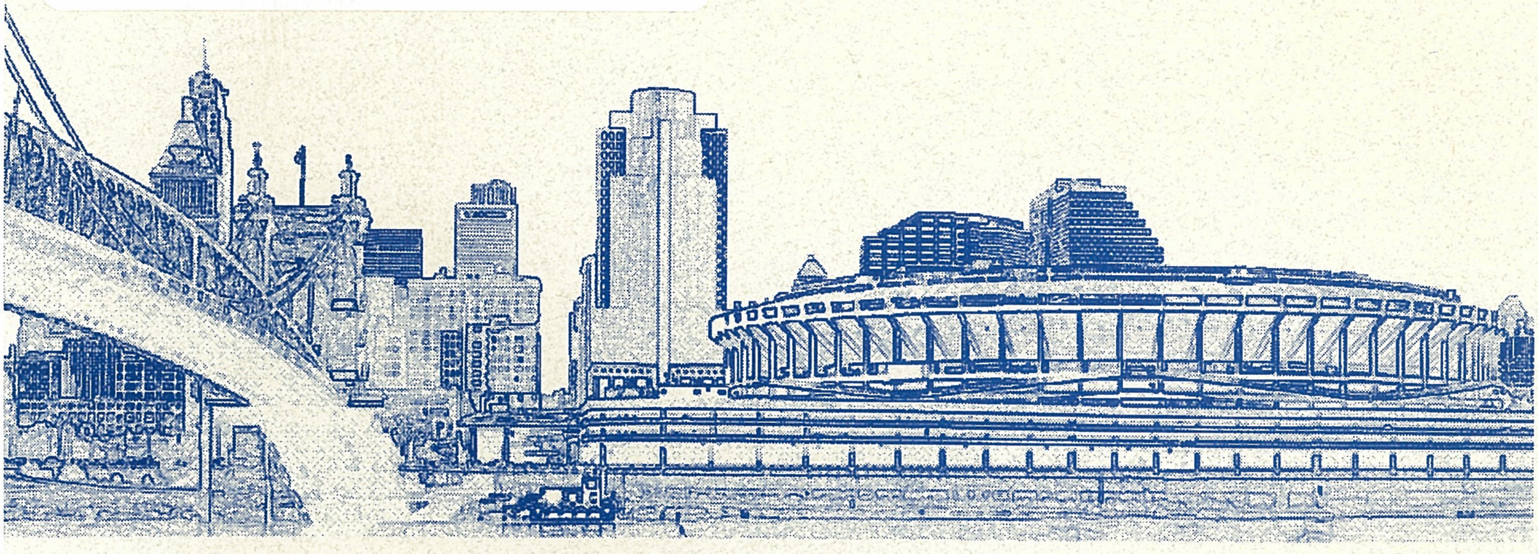
XXIV

GEOTECHNICAL ASPECTS OF INFRASTRUCTURE RECONSTRUCTION



Cincinnati, Ohio
October 15, 1993

PROCEEDINGS OF THE TWENTY-FOURTH OHIO RIVER VALLEY SOILS SEMINAR

GEOTECHNICAL ASPECTS OF INFRASTRUCTURE RECONSTRUCTION

October 15, 1993
Holiday Inn - Cincinnati Airport
Erlanger, Kentucky

Sponsored by

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GEOTECHNICAL ASPECTS OF INFRASTRUCTURE RECONSTRUCTION

OCTOBER 15, 1993

8:45 a.m.	Welcome and Opening Remarks

MORNING SESSION
Dan Holder, Moderator

9:00 a.m.	Keynote Address - "Technological Innovation for Infrastructure Assessment and Revitalization" by Dr. A.E. Aktan, Interim Director Ohio Infrastructure Institute, University of Cincinnati and Dr. Manooch Zoghi, University of Dayton
9:45 a.m.	"Landslide Remediation and Prevention by the City of Cincinnati" - Richard E. Pohana, City of Cincinnati
10:15 a.m.	"Micro-Tunneling to Replace Boston's St. James Avenue Sewers" - James R. Lambrechts, Haley & Aldrich, Inc.
10:45 a.m.	"Impact of Utility Cuts on Performance of City Street Pavements" - Dr. Andrew Bodocsi, University of Cincinnati
11:15 a.m.	Break
11:30 a.m.	"Stability, Analysis and Remedial Action For Slab on a Buttress Dam" - Keith A. Kessler, Ebasco Infrastructure
12:00 p.m.	"Rehabilitation of Roads Across Soft Soils" - Imtiaz Ahmed, Purdue University
12:30 p.m.	Lunch

AFTERNOON SESSION
Peggy Esterle, Moderator

1:30 p.m.	"Infrastructure Stabilizations for Roads and Hillsides in Greater Cincinnati" - Larry P. Rayburn, Richard Goettle, Inc.
2:00 p.m.	"Geotechnical Impacts on the Design of an Urban Highway Relocation" - Paul G. Gruner, Woolpert Consultants
2:30 p.m.	"Soil-Structure Interaction Relationship for Buried Conduits: Modulus of Soil Reaction" - Dr. Mark T. Bowers, University of Cincinnati
3:00 p.m.	Break
3:15 p.m.	"Hemlock Tunnel Rehabilitation" - James D. Smith, Haley & Aldrich, Inc.
3:45 p.m.	"The Stabilization of Concrete Dams by Post-Tensioned Rock Anchorages: The State of American Practice" - Donald A. Bruce, Nicholson Construction Company
4:15 p.m.	"The Use of Jet Grouting for Underpinning and Temporary Excavation Support of a Historic Building" - Jessee A. Scarborough, Ogden Environmental and Energy Services
4:45 p.m.	Social Hour

PROGRAM SPEAKERS

ORVSS XXIV

GEOTECHNICAL ASPECTS OF INFRASTRUCTURE RECONSTRUCTION

Dr. A.E. Aktan, University of Cincinnati, Interim Director Ohio Infrastructure Institute

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Manooch Zoghi, University of Dayton

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TECHNOLOGICAL INNOVATION FOR INFRASTRUCTURE ASSESSMENT & REVITALIZATION

M. Zoghi[1] and E. Aktan[2]

ABSTRACT

It is well known that infrastructure, the underlying foundation or basic framework of a society, made up of transportation network, water systems, sewage systems, ... is a critical index of a nation's economic vitality. As the infrastructure system deteriorates due to age and lack of maintenance, inevitably the productivity of economy and quality of life decline.

Although the United States' investment in its infrastructure system was significant during the last century, however, age, neglect, misuse, and excessive demand have taken their toll, jeopardizing the U.S. economy's future growth. Since the cost of rebuilding the nation's public works is prohibitive, intelligent renewal, utilizing the limited resources available more efficiently, is indispensable. Technologies for infrastructure assessment and revitalization are presented in this paper. Specifically, an overview of the Ohio Infrastructure Institute (OII), a statewide consortium with membership from academia, government and industry whose goals are to improve the quality of infrastructure through intelligent renewal and cost-effective preservation while at the same time creating new industries and jobs, will be presented herein.

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[1] Assoc. Prof. of Civil Engrg., Univ. of Dayton, Member of Ohio Infrastructure Institute (OII) Exec. Comm.

[2] Prof. of Civil Engrg. Univ. of Cincinnati, Dir. of Cincinnati Infrastructure Inst., and Interim Dir. of Ohio Infrastructure Inst.

1. INTRODUCTION

1.1 Definition of Civil Infrastructure Systems:

Infrastructure is defined as the basic fabric on which continuance and growth of a community, state, and a nation depend, including, roads, schools, power plants, transportation and communication systems (1).

The rise and fall of many civilizations in history have been related to the life-cycle of their civil infrastructure systems. According to the National Science Foundation (2), the U.S. passed through the up-cycle of civilization-building in the last century. The system, however, has rapidly degraded due to age, neglect, misuse and excessive demand. The current decline in the nation's productivity and its increasing deficit are partially attributed to the deterioration in the quality of our infrastructure.

The six-year $151 Billion ISTEA Bill of 1992 (3), and the additional spending that has now been proposed by the new administration, indicate an awareness of the urgent need to "rebuild America." According to the Office of Technology Assessment (4), the Federal Government spent about $24 Billion on infrastructure in 1989 in terms of 1982 dollars. [It is important to note that three times the amount expended by the federal Government is expended by State and local governments (5)]. However, according to a recent NYT analysis (6), "the nation needs to spend twice what it has been spending." NSF contends, however, that given the national dept, the added burden of such an expenditure could be detrimental to the U.S. economy. Obviously, intelligent renewal is required which will use the limited resources available in a cost-effective manner.

The importance of the condition of civil infrastructure systems and the effort on the quality of life and safety to the public can be exemplified by recent events. The Great flood of 1993 in several mid-western states, the Chicago flood, the DC downtown flood, the 1989 earthquake in the San Francisco Bay area, telephone systems outage in New York, clogged freeways in LA, Hurricane Andrew, and other events directed public attention to the consequences of infrastructure systems failure and the direct impact of such failures on the safety and well-being of the public. Despite the significant effect of infrastructure quality on public safety and well-being, there is little public awareness and recognition of the impact of infrastructure on our general welfare. Unfortunately preservation has long been mistakenly neglected; instead, more new construction has been financed (7).

1.2 Impediments To Effective Infrastructure Preservation

The impediments to effective and reliable infrastructure preservation have been classified by the National Science Foundation (8) as "technical," "institutional," "social," and "legal," all closely intertwined. Technological advances resulting from research and technology transfer can have a significant long-term impact on infrastructure conditions. However, this can only occur when it is recognized that technological advances must be coupled with appropriate management strategies, reorganization and training. Moreover, it is essential to recognize the "complex systems" nature of infrastructure, as opposed to "isolated elements." For example, advances in bridge evaluation and maintenance procedures will have greater benefit if these advances are integrated in a management system that considers bridges and highways as components within a network.

It has been recognized by NSF that the knowledge and technology required for constructing new infrastructure components is not exactly the same as that required to effectively preserve existing construction. Advances in the state-of-engineering, particularly civil engineering education, research and training, are needed to generate new knowledge for correctly assessing the conditions of existing construction and for their intelligent renewal. These efforts should be coupled with efforts directed to overcoming the barriers to knowledge transfer across disciplines and for developing and taking advantage of innovative strategies for intelligent infrastructure preservation.

Obviously, to overcome institutional and social barriers, new knowledge and strategies must be customized to fit local state-of industry, as well as organizational realities and management practices of state and local governments in charge of public works. Considering that state and local governments spend three times what the federal government spends on public works, it is not possible to improve infrastructure conditions without understanding and formulating solutions to region-specific problems.

2. THE RESEARCH THRUST

Civil infrastructure system requires a multidisciplinary research activity integrating cross-cutting research in "all engineering disciplines, and earth, physical, chemical, biological, mathematical, computer, and social sciences, as well as education and human resources". Accordingly, NSF has proposed a cross-disciplinary research program involving three major elements; i.e., deterioration science, assessment technologies, and renewal engineering. These will be discussed briefly herein, with a special emphasis on geotechnical system.

2.1 Deterioration Science

The science of deterioration process is fundamental in understanding why constructed facilities deteriorate in the first place. "Failure processes, materials science and processing, fabrication, manufacturing and assembly, corrosion, fatigue and environmental hazards, performance criteria, extension of service life, strength and durability" are the basis for deterioration science.

Nearly all construction materials are complex and their performance are time-dependent. This complexity increases with repair or upgrading of the material. Generally, proof-tests are recommended as an indicator of the material's performance. However, due to restraints imposed during testing, they may not simulate the actual conditions.

2.2 Assessment Technologies

The remaining life of existing structures can be determined using the assessment technology. The state of the art technology for evaluating the health conditions of constructed facilities is, however, primitive and unreliable, providing the basis for conservative and costly decisions. On the other hand, vital elements may not be accessible due to large mass of structure involved.

Some of the parameters involved in assessment technologies include (from NSF): "loads and natural hazards (service, fatigue, strength and extreme-event limit states), nondestructive evaluation, smart materials, damage process mechanisms, advanced instrumentation systems, evaluation of remaining service life and long-term monitoring, system evaluation, characterization of performance under extreme events, acceptable risk, interdependence and collocation of infrastructure systems, geographical information systems, and social and economic effects". The assessment technology may be enhanced enormously by adopting the existing advanced technology from electronic, medical, space, defense, and manufacturing fields. The state of the art instrumentation has been acquired by the Ohio Infrastructure Institute (OII) recently which will be described later in this manuscript.

2.3 Renewal Engineering

Majority of constructed facilities were built at the turn of century and are deteriorating. Consequently, they need renewal, modification and upgrading to meet today's standards. Although the cost and disruption associated with replacement of these existing infrastructure is prohibitive, but valuable lessons can be learned regarding their performance for constructing new facilities for maximum life and environmental adaptability.

Renewal engineering will then entail (by NSF) "performance criteria, system modeling, prioritizing, repair strategies, demolition, disposal and recycling, selecting repair materials and methods, preserving national resources, hazard mitigation new materials, design for safety and system performance, innovative technologies, structural control and robotic applications, integrating structural design, processing and fabrication, expert systems, optimization, future demand, reliability, functionality, and longevity of new systems". Two demo projects will be presented in ensuing sections to illustrate the applications of above concepts.

The elements discussed in the preceding paragraphs are applicable to each infrastructure system and are supported by cross-cutting programs such as: "geotechnical engineering; environmental engineering and science; fluid mechanics and water resources; hazard mitigation; materials science and mechanics; systems engineering; smart materials and structures; computational, mathematical, and information science; societal, behavioral, and economic studies; urban studies". The detailed description of each discipline, along with its applications, are presented elsewhere (8). Geotechnical systems will be the major topic of discussion herein.

3. GEOTECHNICAL SYSTEMS

Geotechnical engineering, principally a subset of civil engineering, provides a base for constructed facilities and maintaining the infrastructure. As such, there is a close relation between this field and structural engineering and construction. The major components of geotechnical systems are summarized below (NSF).

3.1 Geo-environmental Engineering

Geo-environmental engineering has received much attention recently and will continue for some time in the future. Some typical examples include: landfills and waste containment facilities, contaminated ground water flow, and remediation and isolation of contaminated sites. NSF has proposed the following areas of research within geo-environmental; i.e., "definition and characterization (particularly non-invasive procedures), environmentally benign construction, fluid transport phenomena, and robotic techniques for remediation and exploration".

3.2 Soil and Site Improvement

Since majority of better construction sites have been used in the past, advanced techniques will be required to convert an otherwise uneconomical site into one which is economical. Proposed topics requiring further research may include finding techniques to extend the life of existing landfills, as well as, deal with "nonacademic" soils such as silts and organic materials.

3.3 Material Inclusions

Material inclusions in soils such as reinforcement, fluid flow control, drainage, filtration, and separation are being used more frequently. Extensive research is being conducted in this area, including life cycle extension, use in less ideal environments, effects of special loadings, repair and replacement, and use of materials possessing variable properties (depending on the environment); i.e., "smart materials."

3.4 Natural Hazards Performance

The natural hazards such as earthquakes, wind, landslides may have a devastating effect on civil infrastructure systems in general, and geotechnical systems in particular. This was demonstrated by the most recent earthquake of Loma Prieta in 1989. While a great deal of research efforts has been expended in this area in the past, much remains to be done. For instance, the predictive capabilities shall be refined so that movement levels can be determined and controlled, and design criteria for new technologies, such as reinforced earth need to be developed.

3.5 Interactions with Surrounding Media

Many civil infrastructure systems involve interactions with surrounding soil in the form of loadings, and with fluids in the form of environmental effects such as corrosion. While significant research activity has been attributed to this area, developing state of the art numerical analysis tools, there needs to be ways of facilitating the use of such tools. Also, in some cases it is required to bring the tools to the level of utility needed for practice.

3.6 Field Instrumentation of Prototype Structures and Systems

In the past, extensive research has been expended in developing numerical models, or testing scaled models of prototype systems. At times, this may not be adequate and instrumentation of actual system will be required to capture all the variables and validate the laboratory experimental programs as well as numerical analysis. This type of information will be useful for future design improvement of civil infrastructure systems.

3.7 Underground Space

As the construction sites become scarce in the urban areas, sometimes it is viable to utilize underground space. Since U.S. is lagging behind other nations in this field, it may be logical to focus on facilitating technology transfer considering the realities of the situation.

3.8 Computational and Visualization Techniques

Mechanical properties of soils plays an important role in number of infrastructure problems. Thus, advances in constitutive modeling and coding techniques should be at the heart of geotechnical infrastructure research. Also, advances in computational technology, visualization techniques using dynamic imaging should be considered in geotechnical system.

3.9 Existing Structures

Since the majority of infrastructure issues deals with existing structures, evaluation of the earth pressure distribution on a structure that has been in place for many years is an important parameter and should be investigated. This type of in-situ data will be specially valuable to a designer undertaking a rehabilitation project.

4. THE OHIO INFRASTRUCTURE INSTITUTE

4.1 Background

Recognizing a need for education, training, research and technology transfer to address the state and region-specific nature of the infrastructure preservation problem, the Ohio Engineering Dean's Council established the Ohio Infrastructure Institute (OII) in 1992. OII is a consortium of civil and environmental engineering programs and other academic units related to infrastructure preservation in the state, in partnership with state and local governments and related industries.

The Ohio Engineering Dean's Council financed a study by the Urban Center of the Cleveland State University entitled "Assessing the Need for an Infrastructure Institute in Ohio." The work culminated in a report by the Urban Center to the Ohio Engineering Council (9).

4.2 Infrastructure Preservation in Ohio

The Urban Center study revealed significant facts regarding Ohio's infrastructure conditions and expenditures relative to other states. For example, according to 1988 Highway Statistics (10), the condition of Ohio's urban interstates ranked 37th in the nation, while bridges on the Federal-Aid system ranked 17th. In 1987, Ohio ranked 41st in per capita state and local government spending, and 49th in per capita federal expenditures for the transportation infrastructure as obtained from Highway Trust Fund. The Urban Center study revealed that in 1987, the annual capital expenditures for Ohio's infrastructure was about one Billion dollars, while the need was estimated at $1.9 Billion. This pointed to an accruing shortfall over a ten year period far more significant than the one time $1.2 Billion provided to local governments in Ohio for infrastructure construction through the Bond Issue two (11). This data should be placed in context with the recognition that Ohio's GNP ranks eighth amongst the 50 states.

The Urban Center's analysis of Ohio's infrastructure followed the analyses that had been previously presented by many federal agencies (12, 13). Taking full advantage of innovative strategies and emerging technologies for more effective and reliable infrastructure preservation is found to be essential to alleviate adverse effects of the financial shortfall of infrastructure spending on public safety and well-being of Ohio's citizens. Further analyses of the Urban Center report and justification for establishing OII are presented by Papadakis (14).

4.3 Structure, Mission and Organization of OII

OII will be conducting applied cross-disciplinary, knowledge-transfer type research as opposed to basic research. It is believed that there is already an abundance of technologies, generated through basic research, that have yet to find their way into infrastructure preservation and condition assessment. OII researchers will utilize existing Non-Destructive Evaluation (NDE) research in the basic sciences areas of the medical, electronic, space, defense and manufacturing field for application to infrastructure assessment as well as explain fundamental issues (15).

OII's missions, drafted by the OII Task Force, are to: a) further civil engineering and public works education at all professional levels and continuing education; b)promote and disseminate innovative and cost effective infrastructure technologies through a consortium of Ohio universities; c)position Ohio to successfully compete for education, research and demonstration funds; d)promote the transfer of technology from research to application; and develop infrastructure repair and maintenance strategies and technologies to save taxpayers' money.

The organizational structure of OII consists of: a) a director and an executive committee that will coordinate the functions of the institute. The functions are carried out by various focus groups drawing upon resources of member academic institutions, government agencies and industry; b) a governing board comprised of six each of university, government and industry representatives, which will appoint and monitor the performance of the director and focus groups; c) a technical steering committee with three members each from academia, government and industry, to promote and evaluate the applied research activities; d) a technology transfer committee with three members each from academia, government and industry, to promote and evaluate the information dissemination and training; e) focus groups and demonstration projects team that will conduct education, research, technology transfer and knowledge depository functions of the institute.

4.4 OII Demonstration Projects:

OII executive committee members have initiated two demonstration projects that are scheduled to last for several years. These will become vehicles for multi-collegiate and cross-disciplinary interactions to advance the state-of-the-art in assessment technologies and renewal engineering, while contributing to the national recognition of OII.

The first demonstration project is entitled "Instrumenting, Testing and Monitoring a Newly Constructed Reinforced Concrete Deck-on-Steel Girder Bridge." The subject constructed facility for this proposal, namely, steel-stringer bridges, comprise 35% of the bridge population in the National Bridge Inventory. The replacement value of Ohio's medium-span steel-stringer bridge inventory is estimated to exceed $2.5 Billion. The research team will have a diverse background, pooling their expertise in field structural testing, scale-model testing and field geotechnical-foundation testing.

A steel-stringer bridge in Ohio's construction program will be used to investigate the long-term performance os such structures and to study the main design and maintenance parameters. The test bridge will be instrumented and monitored through its construction and service phases using traditional and new types of sensors. Researchers will conduct structural identification, condition assessment and rating and explore the problems that may arise in instrumented long-term health-monitoring and in-motion weighing. These are problems that stand in the way of creating "smart structures."

The second demonstration project is designed to explore the performance of advanced composite materials in bridge construction. This project aims at designing, constructing and monitoring a pedestrian bridge made in whole or in part from fiber reinforced polymer composites. One monitoring component of the bridge may be realized through the use of fiber optics which may also serve as fiber structural reinforcements. Researchers will explore problems in design, manufacturing and construction of a major infrastructure component using material applied in the aerospace industry. This promises many advantages in infrastructure use over traditional construction materials (16). The significance of exploring and demonstrating the use of polymer composites for major infrastructure components is clear when considering that Ohio's polymer industry accounts for more than sixty percent of the national production. Moreover, there is already a significant accumulation of knowledge on the manufacture and use of composites from aerospace and defense-related research in several Ohio universities.

5. <u>CULTURE AND INFRASTRUCTURE</u>

While the technology program component is fundamental in improving the infrastructure system; social, economic, political, and environmental elements are critical as well. This becomes apparent when civil infrastructure systems of other countries are considered. Subsequently, highlights of Japanese and European ways of dealing with infrastructure problems will be presented herein.

The most critical factor which is notable in European and Japanese infrastructure technology that is lacking in the U.S., is the active involvement of the construction industry in undertaking and funding research (17). There appears to be a close cooperation between the construction industry in Europe and universities as well as research institutes. Japanese construction companies, on the other hand, have well-established in-house research and development programs, along with well-equipped laboratories.

Japanese culture favors long-term solutions; as such, lifetime employment and long-term relationships with customers are preferred. Due to the long-term relationships between the owner and contractor and between the contractor and subcontractors, there is reduced claims. In Japan, the Ministry of Construction (MOC), along with the Ministry of International Trade and Industry (MITI) and two subsidiary organizations, provide assistance to the construction industry. Annual construction expenditure in Japan is approximately $500 billion, which represents 18% in terms of GNP.

In European countries, various schemes are utilized to generate funds needed for infrastructure related research. For instance, in France, Belgium and Sweden a tax is levied from construction contracts to provide funds to various administrative bodies. In Denmark, similar to Sweden, there is a research, or technological council that finances applied research.

In Norway, a close relationship exists between industry and universities. Specifically, oil companies are encouraged to participate and fund research. In Germany, there are twenty Materials Testing Institutes (MPA) that are created, accredited and certified by the state. An extremely close relationship exists between the industry, MPA and in turn the universities. While the state finances MPAs which have permanent personnel, the testing and research work are funded by the industry.

6. SUMMARY AND RECOMMENDATION

The scope and complexity of infrastructure systems are presented in this paper. It is evident that the nation confronts a crisis caused by years of neglect and underinvestment in the maintenance of public works. In dealing with challenging infrastructure problems, a multidisciplinary research initiative has been recommended to address the parallel issues of implementing existing knowledge and developing appropriate new scientific and engineering knowledge.

As for the geotechnical aspects of infrastructure reconstruction is concerned, a number of broad research areas are identified in this report that could contribute to more efficient and improved infrastructure design, construction, and rehabilitation. The research projects are interdisciplinary in nature which will require the involvement of various engineering programs for a successful implementation.

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LANDSLIDE REMEDIATION AND PREVENTION BY THE CITY OF CINCINNATI

Richard E. Pohana[1] and Timothy M. Jamison[1], P.E.

ABSTRACT: Landslides are a common and long standing problem in the Greater Cincinnati area. In 1989 the City of Cincinnati, acting upon the recommendation of the Infrastructure Commission, formed a Geotechnical Office, staffed by a geotechnical engineer, an engineering geologist and two civil engineering technicians, to manage a landslide repair and prevention program. Landslide prevention methods include maintaining a landslide database, engineering geologic mapping, assistance in the enforcement of excavation, fill and zoning regulations and the preparation and review of geologic and geotechnical reports.

Other recommendations of the Infrastructure Commission resulted in an inventory and inspection of retaining walls that impact the right-of-way and a separate Capital Improvement Project fund for landslide and retaining wall correction projects. A total of $2.1 million was allocated to the fund between 1988 and 1992. A total of $4.6 million is proposed for stabilization projects between 1993 and 1995. Landslide correction projects mainly involve the stabilization of historic slides which occur on roadways older than 50 years. Cantilevered drilled pier walls are typically the most economical solution and are the predominate method of stabilization.

INTRODUCTION

The City of Cincinnati is presently taking major steps to restore and maintain its infrastructure. In 1987 an independent commission studied the city's infrastructure. The Infrastructure Commission Report [1], issued in December of 1987, contained 100 specific recommendations for improving the condition of the city's infrastructure. These recommendations were adopted by city council and funded by a voter approved 0.1% earnings tax in May of 1988. The infrastructure tax will only stay in effect if 90% of the calculated base amount is expended each year. The base amount was set at $42 million in 1990.

Four of the 100 infrastructure recommendations deal specifically with improving the stability of the city's retaining walls and landslides.

I. Budget $200,000 annually to cover the expenses of a geotechnical engineer, an engineering geologist, and staff. They would develop an accurate database including a mapping program in order to set priorities for repairs, enforce excavation and fill regulations, and review geologic and geotechnical reports submitted to city agencies.

II. Develop an inventory of the city's retaining walls to assess the city's im-

[1]City of Cincinnati, Department of Public Works, Division of Engineering, Room 410-City Hall, Cincinnati, Ohio 45202

mediate and long-term liability, and to determine priorities.

III. Provide catch-up capital of $10,000,000 to repair and replace inadequate walls and repair landslide damage.

IV. Require engineering geologic and geotechnical investigations before the design of any new development is approved [1].

Following the recommendations of the Infrastructure Commission the city instituted a retaining wall repair and landslide remediation and prevention program. The benefits of this program include: assuring the safety of the public right-of-way, reducing the city's potential liability from either injury to the public or damage to adjacent private property, reducing the long term cost of correction projects by stabilizing problem areas before major damage occurs, and aesthetically improving the hillsides and public right-of-ways by eliminating unsightly deteriorated walls and landslide conditions.

In 1989 the City of Cincinnati formed a Geotechnical Office, staffed by a geotechnical engineer, an engineering geologist and two civil engineering technicians, to manage the landslide repair and prevention program. The Geotechnical Office is part of the Structures Section of the Engineering Division, Department of Public Works. The primary function of the Geotechnical Office is to provide geotechnical expertise concerning landslide stabilization and prevention within the public right-of-way and other property controlled by the city. The staff also provides limited geotechnical support to private property owners by assisting the Building and Inspections Department and the Planning and Zoning Department review projects in hillside areas.

RETAINING WALLS

The city established a retaining wall inventory in 1951. This inventory and subsequent revisions in 1956 and 1960 were supposed to include all retaining walls constructed or maintained by the Department of Public Works. The length of walls in the inventory totaled about 25 miles in 1960. The Infrastructure Commission concluded that the city's information on the condition, location, and ownership of retaining walls was incomplete and sometimes unreliable [1].

Database

An inventory and inspection of retaining walls that impact the right-of-way was performed in 1990. The geotechnical staff and engineers of the Structures Section developed the criteria for the inventory. The staff then performed pilot studies to test and refine the inventory and determine the work force required to collect the information. The actual inventory was performed by several private consultants under the direction of the geotechnical staff.

Inventory information was gathered on both public and private walls. Walls judged to be the responsibility of the Department of Public Works were inspected in detail. The inventory currently contains information on 3,652 walls. About 30 percent of the walls, totalling about 38 miles in length, are the responsibility of the city. All walls maintained by the city are inspected at least once every five years.

A private consultant was also contracted to develop a computer database to store and access the information obtained by the inventory. The database allows prompt determination of the ownership of a specific wall, and the type, dimensions and condition of the wall as well as other descriptive information. The overall condition of the retaining walls is rated from zero (0) to four (4), four being the worst condition.

The inventory and periodic inspections allows the staff to monitor the performance of the walls and forecast future expenditures for maintenance and repair. The inspection records in the database are queried each year to generate lists of walls for replacement or rehabilitation. These walls are then inspected again to develop an annual priority list. In the past, wall repair and replacement priority was based on citizen complaints or chance observations of walls in bad condition. The initial retaining wall inventory was performed at a cost of $165,000. Periodic inspection is critical to maintain the long term accuracy of the information. Future plans are to link the retaining wall records to the

Cincinnati Area Geographic Information System (CAGIS).

Monitoring

The Engineering Division uses optical survey methods and tilt measurements to monitor the performance of retaining walls. The surveys are performed by city survey crews. Tilt measurements provide the best information on wall performance if the footing is fixed in stable material. Optical survey methods including leveling and offset measurements are used if the wall footings are not founded on stable material. Standard survey techniques allow reestablishment of lost reference points with a minimal loss in accuracy for measurements taken over several decades. The primary disadvantage of optical surveys is that the overall accuracy is generally much lower than that obtainable with sensors used in most inclinometers and tiltmeters. Another disadvantage is that it is not possible to monitor the performance of buried structures, such as tiebacks. Finally, the number of locations monitored by optical surveys is limited by the availability of survey crews. The city currently monitors 14 walls by survey methods.

Inclinometers and strain gauges are used to monitor the performance of tied back pier walls along Columbia Parkway. Inclinometers to monitor structural behavior are also installed in at least one pier of each wall constructed by the city.

The city uses permanently fixed tiltmeters to monitor a bridge pier within a landslide mass. Because of the successful use of these tiltmeters, the city will obtain a portable tiltmeter sensor and reference plates to monitor retaining walls and structures.

Maintenance

An annual retaining wall maintenance budget of $500,000 was approved to repair and refinish walls in 1988 through 1992. In 1993 the wall maintenance budget was reduced to $200,000 due to a reduction in the general fund. Approximately 75% of the retaining wall maintenance budget is spent on refinishing exposed surfaces with gunite or miracote. Other maintenance work includes railing replacement, patching, drainage corrections and minor wall replacement. Maintenance projects are done by the city's Highway Maintenance Division and private contractors. Wall replacement projects are funded from the Capital Improvement Program.

LANDSLIDE CORRECTION

Funds to correct the backlog of damage caused by landslides is an essential element of the landslide correction program. The Infrastructure Commission estimated that deferred repairs to landslide damage was over $15,000,000 and that since 1983, the annual expenditures for emergency repairs for landslide damage to city streets have been approximately $500,000 [1]. Because of inadequate funding, the city's past policy was to temporarily stabilize landslides, or maintain the utilization of roadways by leveling the pavement with asphalt. Landslide correction projects were not a capital improvement priority. Although the city knew the location of landslides it did not stabilize the areas until roadways had completely failed, necessitating costly emergency repair. As a result of the Infrastructure Commission's recommendation, a separate Capital Improvement Project fund for landslide and retaining wall correction projects was established. A total of $2.1 million was allocated to the fund between 1988 and 1992. A total of $4.6 million is proposed for stabilization projects between 1993 and 1995. Landslide and retaining wall projects are also funded by State Issue 2 funds and from the city's Capital Improvement Project fund as part of street rehabilitation projects.

Field Investigation

Implementation of the landslide correction program began with inspecting and ranking the severity of known landslides. During the winter of 1989 and 1990 the geotechnical staff inspected and described over 100 sites listed by the city's Highway Maintenance Division as areas affected by slope movement. Based on these observations forty areas are targeted for monitoring and/or remedial measures.

The subsurface investigations performed in the areas of slope movement typically consist of several test borings to determine the subsurface conditions. The staff establishes the scope of the subsurface investigation and coordinates the drilling and laboratory phases of the investigation that are performed by private geotechnical consultants. The staff then prepares reports relating findings, conclusions and recommendations. The reports are submitted to the engineers in the Structures Section who design the remedial measures.

Monitoring

The installation of slope inclinometers and piezometers is typically included in the scope of the subsurface investigations. This allows the staff to monitor the problem areas to determine the rate, depth, lateral extent and mechanisms of movement. The depth of movement is determined from cumulative displacement plots of the inclinometer data. Displacement versus time plots assist in determining the priority of stabilization projects. In most cases the installation of the instrumentation in the areas targeted for repair is performed at least one year prior to construction. Studying the slide in this detail assists in determining the most economical method of stabilization. The city monitors 85 inclinometers and 21 piezometers distributed over 31 sites. Most of the inclinometers are associated with landslide stabilization projects, but some are used to monitor hillside movement in areas adjacent to major construction projects.

Extensive and continuous long term monitoring of two landslides is scheduled to begin in the fall of 1993. The proposed monitoring involves instrumenting the sites with piezometers, soil moisture blocks and inclinometers. The purpose of this instrumentation program is to study in-depth the interrelationship of soil moisture, groundwater, precipitation and hillside movement. If a strong correlation relating hillside movement to specific changes in soil moisture and/or groundwater fluctuations is found, it may be possible to use moisture control systems to inhibit hillside movement.

Stabilization Projects

Landslide correction projects mainly involve the stabilization of historic slides which occur on roadways constructed before 1940. City records indicate a history of movement in the slide areas generally dating back to shortly after construction. The predominate cause of the landslides is improper construction of fill embankments during the original grading for the roadway. The landslides move relatively slowly at overall rates of one-quarter to one-half inch per year during the summer and fall increasing to rates of 2 to 3 inches per year in the winter and spring. This rate of movement does not cause major problems with the infrastructure because there is no interruption of services. However, during years of high rainfall these relatively dormant slides are quite active and can cause disruption of underground and overhead utilities and road closures.

Table 1 summarizes the city's landslide correction programs from 1974 to projects in design as of July, 1993. Projects funded entirely with State and/or Federal funds are not included. Examples of such projects are the Mt. Adams Retaining Wall and the stabilization of Columbia Parkway from Baines Street to Torrence Avenue and from Tusculum Avenue to Beechmont Avenue. Table 1 starts with 1974 because the first pier wall was constructed during that year. The city, however, did perform landslide correction projects prior to 1974 in response to major movement. Notable projects on record include the repair of Hillside Avenue and River Road in 1927 to 1930 [2], Elsinore Avenue north of Wareham Drive in 1900, [3], Glenway Avenue at Carson in 1942 [4] and Warsaw-Waldvogel 1949 to 1953 [5].

The cost of the projects listed on Table 1 include, all construction, engineering, real estate, management and inspection costs associated with the project. The degree of repair associated with each project various considerably from the construction of a simple pier wall without panels to complete roadway restoration, including curbs, sidewalk, full pavement removal and replacement, underground utilities and hillside steps. Final cost information is not yet available on all projects. When only the contract bid price was available it was increased by 24% to cover associated engineering, real estate, management and inspection costs associated with the project. Estimated costs were used for walls currently in design.

Drilled Pier Walls

Cantilever drilled pier walls are the predominate method of landslide stabilization. Drilled pier walls are reinforced concrete retaining

Table 1. CITY LANDSLIDE CORRECTION PROJECTS

Year	Project	No. Piers	Pier Diam (in)	Pier Spacing(ft)	Overburden Total (ft)	Overburden Avg (ft)	Rock Total (ft)	Rock Avg (ft)	Earth/ Rock (3)	Wall Length (ft)	Cost
1974	Anderson Ferry	25	36	5	350	14	225	9	2.59	120	$31,970
1974	Elberon Ave	151	36	5	2114	14	1210	8	2.92	750	$115,253
1974	Groesbeck Rd	41	36	5	615	15	410	10	2.50	200	$35,119
1977	Colerain Ave	23	36	5	460	20	299	13	2.56	110	$52,203
1977	Hillside Ave	305	36	5	6405	21	3965	13	2.69	1520	$557,040
1978	Wareham Dr	41	36	5	533	13	492	12	1.81	200	$56,276
1980	Eastern Westway	36	36	5	576	16	432	12	2.22	175	$170,606
1984	McMicken Ave	Inverted toe wall									$103,988
1984	West Fork Rd	55	30	5	825	15	550	10	3.00	270	$186,859
1984	Elberon Ave	21	36	5	147	7	168	8	1.46	100	$67,382
1986	Sutter Ave	22	36	8	308	14	220	10	3.73	168	$298,539
1986	Hillside Ave @	29	30	7	406	14	435	15	2.61	207	$301,407
	McGinnis	28	36	6	308	11	280	10	2.20	162	
1986	Colerain Ave	73	36	5	2190	30	365	5	10.00 (4)	360	$405,000
1986	Mt. Echo Park (1)	59	24	5	590	10	472	8	3.13	290	$202,385
1989	Monestary St	28	24	8	280	10	252	9	4.44	224	$1,380,000
		39	30	8	663	17	741	19	2.86	304	
		80	36	8	1040	13	1040	13	2.67	640	
1989	Delhi Pike (1,5)	69	36	8	1449	21	966	14	4.00	544	$237,100
1991	Liddell St	13	36	8	182	13	143	11	3.15	96	$127,782
1991	West Fork Rd (1)	20	36	8	300	15	240	12	3.33	152	$94,949
1991	Groesbeck Rd (1)										
	Slide 1	50	30	5	650	13	700	14	1.86	245	$220,000
	Slide 2	21	30	8	273	13	378	14	2.97	160	
1991	Kirby Rd (1)	18	30	8	288	16	180	10	5.12	136	$118,380
1991	Elberon Ave (1,5)	33	30	8	528	16	396	12	4.27	256	$172,798
1992	Brighton St (1)	12	36	10	228	19	180	15	4.22	110	$83,295
1992	Columbia Pkwy -	54	24	8	486	9	486	9	4.00	424	$1,917,405
	Torrence to Delta	214	24	10	2782	13	2140	10	6.50	2130	
	(1,5)	70	36	8	1050	15	770	11	3.64	544	
		127	36	10	1778	14	1524	12	3.89	1250	
1992	Sutter Ave (1)	Keystone faced MSE wall									$513,866
1993	Kirby Rd (1)	24	30	8	408	17	336	14	3.89	184	$149,571
1993	Hillside Ave @										
	Anderson Fy. (1,5)	121	30	8	1694	14	1694	14	3.20	928	$183,668
1993	Ruther Ave (1)	31	30	8	465	15	527	17	2.82	232	$147,409
1993	Brestle Rd (1)	20	30	10	260	13	280	14	3.71	190	$106,778
1993	Elsinore Ave (2)										
	Slide 1	14	30	8	168	12	140	10	3.84	104	$210,000
	Slide 2	10	30	10	340	34	150	15	9.07	90	
1993	Elberon Ave (2)	38	30	10	380	10	494	38	1.05	360	$150,000
1993	Hill St (2)	16	30	10	400	25	200	12.5	8.00	150	$140,000
1993	Stanley Ave (2)	Embankment									
1993	Warsaw Ave (2)	47	30	10	752	16	658	14	4.57	460	$300,000
1993	Shepherd Rd (2)	64	24	8	832	13	832	13	4.00	504	$350,000
1993	McMicken Ave (2)	31	36	10	775	25	620	20	4.17	300	$250,000
1993	Montana Ave (2,5)	Embankment									$225,000
	TOTALS	2173			34278		25590			15349	$9,662,028

(1) Cost estimate based on actual construction cost + 24% for real estate, design, testing, etc.
(2) Cost estimate only
(3) Earth /Rock = (Pier Spacing x Avg Overburden) / (Pier Diam x Avg Rock)
(4) Unweathered rock socket not recorded or known
(5) 70% to 90% State funds, 10% to 30% Local funds

structures consisting of a single row of drilled shafts as shown on Figure 1.

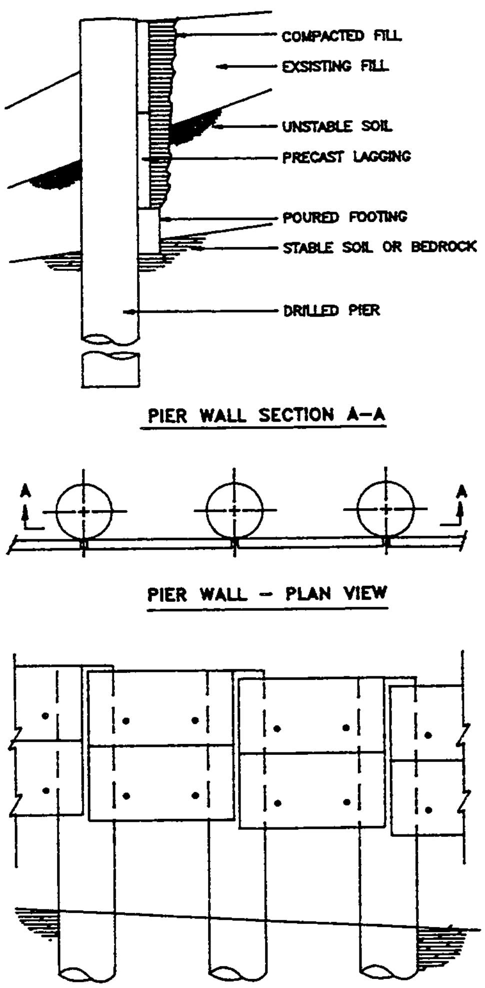

FIGURE 1 – TYPICAL PIER WALL

The shafts range in diameter from 2 feet to 4 feet and are drilled on 5 feet to 10 feet centers. The piers are reinforced to provide sufficient moment and shear capacity to resist lateral earth pressures associated with hillside movement and traffic surcharge.

Piers extend through the unstable soils and into the bedrock to a depth where the lateral stresses in the rock socket and the deflections are not excessive. Lagging consisting of reinforced concrete panels is often used to retain the upper soils between the piers. Deeper soils are retained by arching action between the piers.

Table 1 lists the number of piers, the center to center spacing between piers, the amount of overburden and rock drilled, the average overburden and rock drilled per pier and the total length of the pier wall for each project.

Pier walls are well suited to infrastructure stabilization in the Cincinnati area. Only four of the thirty-six projects listed on Table 1 involve stabilization by methods other than drilled piers; Montana Avenue and Stanley Avenue where earth buttresses will be constructed, McMicken Avenue where an inverted cantilever wall was used and Sutter Avenue where a Mechanically Stabilized Earth Wall was constructed. Some of the benefits of pier walls are listed below:

- Constructed within the right-of-way avoiding real estate acquisition.
- Constructed without open cut or temporary shoring.
- Constructed with minimal disturbance and lane closure of existing roadways.
- Vibration, perceived or actual, due to drilling equipment is less than other types of construction equipment, notably driven piles and tiebacks drilled with air-track rigs.
- Cincinnati bedrock is easy to drill, has acceptable strength for resistance of lateral loads and is often found at shallow depths (< 20 foot).
- Typically are the most economical solution.

The lateral load that a pier wall must resist is primarily a function of overburden depth. Numerous inclinometers show that the overburden soils move along a shear plane at the soil-rock interface, above the upper surface of the brown weathered shale and limestone. Most city pier walls are designed assuming active earth pressures extending from the ground surface to the bedrock and a traffic surcharge. It is also assumed the each pier behaves as a rigid pole [6]. The length of the rock socket in this method is taken as the minimum length where the stresses applied by the shaft are just less than the allowable stresses of the socket material.

There is a tendency for the pier walls designed by the city to become less conservative with time. A measure of this is inferred from the ratio of the retained earth to the rock socket (e/r ratio), Table 1. This is a dimensionless number calculated by dividing the product of the pier spacing and overburden depth by the product of the pier diameter and rock embedment. From 1974 to 1986 the e/r ratio for 13 walls ranges from 1.46 to 3.73 and averages 2.57. From 1987 to 1993 the e/r ranges from 1.86 to 9.07 and averages 4.13 for the 26 walls designed or constructed. The 60% increase in e/r ratio is attributed to the use of lower design safety factors justified by field observations.

The two walls designed with e/r ratios from 8 to 9, Hill Street and Elsinore slide 2, are experimental. In both cases the walls will be instrumented with inclinometers and vibrating wire strain gauges. Pier spacing for these walls is large enough to accommodate additional piers should future deflections or strains become excessive.

LANDSLIDE PREVENTION

The fourth recommendation of the Infrastructure Commission, requiring engineering geologic and geotechnical investigations before the design of any new development is approved, intended to prevent the occurrence of landslide damage in both public and private property. Strict implementation of this recommendation required changing current codes and regulations which were viewed as adequate by the governing city departments.

The geotechnical staff is available for consultation with all city agencies. The staff has provided consultation on a variety of geotechnical issues to the Department Neighborhood Housing and Conservation, the Planning and Zoning Department, the Building and Inspections Department, the Department of Economic Development, Water Works and the Metropolitan Sewer District.

Database

The geotechnical staff is presently developing a computer database that will assist in the management and documentation of past, present and future landslides. The location of the areas of instability, both public and private, identified in city files are plotted on City of Cincinnati and Hamilton County Topographic Maps and on USGS 7.5 minute quadrangles and are cross referenced to summaries of information in the city's files. Four hundred and ninety two areas suspected as being unstable are plotted on the maps. It is the staffs intention to field check and describe each of the areas.

In addition to the documentation of landslide areas the staff plans to document the location and results of key test borings drilled within the City Limits of Cincinnati. The database will provide information for planning, implementation, and review of geotechnical investigations for private or public construction within the city. Future plans are to link the database to the Cincinnati Area Geographic Information System (CAGIS).

Mapping

As part of the development of a database, relative stability analyses are performed in key areas of the city. These analyses present geotechnical and geological information such as boundaries of individual landslide masses, location of known test borings, man-made excavations, bedrock units, and surface deposits on engineering geologic maps. A relative stability map is derived from the engineering geologic map. The relative stability map classifies areas of ground with respect to slope stability, explains the expected engineering behavior of the materials in each category and gives recommendation for further geotechnical study prior to development. This information is presented in descriptive terms that are meaningful to nontechnical users. A written report which further explains the information presented on the engineering geologic and relative stability maps, including the logs of all test borings identified, is prepared for each study area.

The staff has completed engineering geologic and relative stability mapping of the East End Riverfront Development for the Department of City Planning and of the Sycamore Hill Area in Mt. Auburn for the Department of Neighborhood Housing. Mapping of the East Price Hill area is currently in progress.

Project Review

Earth stability of private hillside development is under the control of the Department of Buildings and Inspections through the excavation and filling of land regulations of the Cincinnati-Ohio Basic Building Code. The direct responsibility for the administration of the regulations is performed by the Building Plans Examiners in conjunction with their review of plans for building permits.

Since October of 1991, the geotechnical staff has assisted the plans examiners in the review of projects in landslide sensitive areas. It is the responsibility of the geotechnical staff to assure that each geotechnical investigation, and the resulting report, adequately addresses the geologic conditions and that the conclusions and recommendations are reflected in the design plans. This procedure closely parallels that suggested by the U.S. Geological Survey [7]. This study presented a procedure for utilizing geotechnical and topographical parameters for enforcement of a cut and fill ordinance.

The staff also reviews developments in Environmental Quality Hillside Districts (EQHD) for the Planning and Zoning Department. These districts have overlay zoning regulations superimposed on existing land use regulations. The basic philosophy of the EQ districts is to carefully evaluate development in the designated districts in order to determine whether they conform with and do not detract from the special environmental features of the district. Development control is exercised in EQ Districts by requiring development permission for new structures, additions, or major alterations prior to the issuance of a building permit. Decisions on development permits is made by a Hearing Examiner.

The geotechnical staff reviews the preliminary plans submitted with the application for development and submits their recommendations to the Hearing Examiner. The review typically includes an independent site inspection. If a geotechnical report is submitted with the application, the staff reviews the report and comments on its adequacy. If a geotechnical report is not submitted, the staff recommends whether the applicant needs to retain a geotechnical consultant.

Inquires from the public regarding slope instability are also investigated by the staff. Although the majority of these calls concern private property, they are able to advise the private individual as to the seriousness of the situation and refer them to consultants that can provide assistance.

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MICRO-TUNNELING TO REPLACE BOSTON'S ST. JAMES AVENUE SEWERS

James R. Lambrechts[1], Dennis J. Doherty[2], and Edward Duggan[3]

ABSTRACT

Groundwater lowering by sewer pipes installed 80 years ago in Boston's Back Bay caused problems for wood pile foundations in the 1930's. Sewer construction methods of the early 1900's allowed the leaky pipe to affect a large area, and required that the sewer be operated under surcharged conditions to maintain local groundwater levels. Trenchless constructions methods are being planned for reconstructing the sewers because St. James Avenue is now a busy arterial street. Considerations in the planning to use micro-tunneling are discussed.

INTRODUCTION

Trenchless construction techniques using micro-tunneling are being planned to replace approximately 10,000 ft of structurally and hydraulically deteriorated sewers and storm drains located beneath busy arterial streets in the high-rise business district of Boston's Back Bay. Another 5,000 ft of existing sewer pipes will be rehabilitated in-place using cured-in-place structural renovation methods. This paper reviews the history of the St. James Avenue sewer system and geotechnical problems which have affected it. A summary of issues in selection of trenchless replacement/ rehabilitation method is presented. Construction is planned for late 1994.

HISTORICAL BACKGROUND

St. James Avenue lies in the middle of Boston's Back Bay, an area of land created in the second half of the 19th century by filling former tidal marshes, described by Aldrich (1). Figure 1 shows the sewer location and indicates present streets and major adjacent buildings. In the St. James Avenue area, fill was first placed in mid-1830's for construction of embankments for early railroads, one of which crossed the sewer alignment and terminated at a station, as indicated in Figure 1. Remnants of this construction, in the form of differing fill materials, wooden trestles and station foundations (wood piles with granite block pile caps), still exist and must be expected to be encountered during the proposed sewer replacements.

A formal filling program of this area of Back Bay did not occur until

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[1] Haley & Aldrich, Inc., 58 Charles Street, Cambridge, MA 02141
[2] Bryant Associates, Inc., 77 North Washington Street, Boston, MA 02114
[3] Boston Water and Sewer Commission, 425 Summer Street, Boston, MA 02210

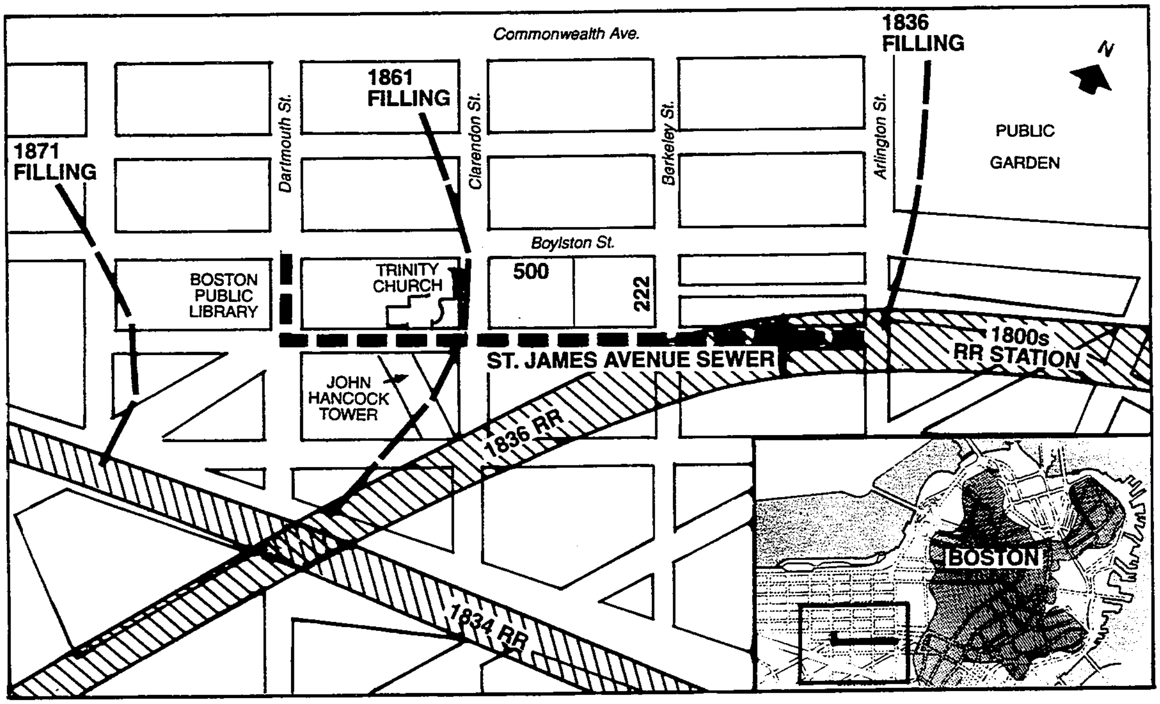

Figure 1 - St. James Avenue sewer location and area historic development.

the second half of the 19th century. The progress of filling is indicated on Figure 1 for 1836, 1861 and 1871. Initially rowhouses were built, founded on wood piles penetrating 25 to 40 ft through the fill and organic deposits of the former Back Bay mud flats to a dense sand layer or the crust of the Boston Blue Clay. Within the last 70 years, most rowhouses have been replaced by large office buildings. A variety of deep end bearing and "floating" mat foundation have been used.

SEWER CONSTRUCTION

The St. James Avenue sanitary sewer and storm drain were constructed in 1910-1912 to improve the original system that often used wooden box conduits. Figure 2 shows configurations of clay and concrete pipes constructed, and the trench and underdrain used for

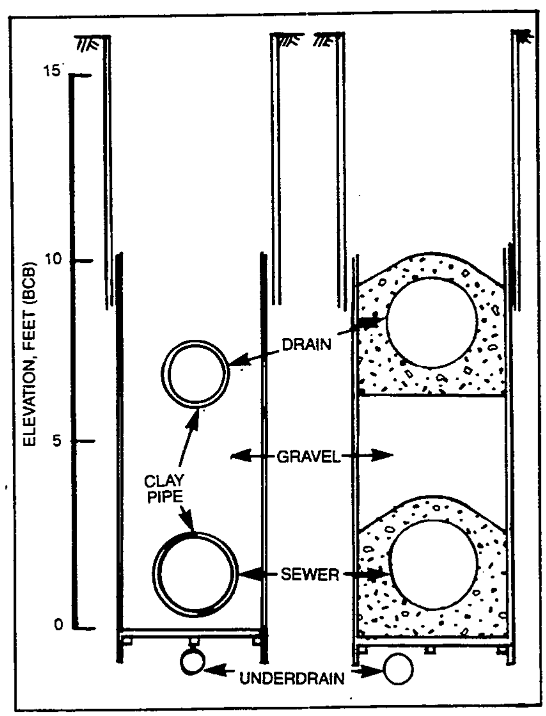

Figure 2 - Typical drain and sewer pipe cross-sections.

the installations. Combinations of clay and concrete pipe also exist at some locations. The ground surface is relatively flat at about Elev. 16, Boston City Base (BCB). The depth to invert of the storm drain is 8 to 10 ft, and about 15 ft for the sanitary sewer. Groundwater in the Back Bay area is typically between Elev. 6 and 8, slightly lower than the nearby Charles River.

Sewer construction used methods and materials typical for the period. The underdrain pipes were installed for trench dewatering and were run to sump pits where pumping occurred. There are no records to indicate that the underdrains were plugged. Wood sills were reported in field notes to have been placed above the underdrain pipe, and concrete pipes were cast on the sills and against wood sheeting.

In some areas, notably between Berkeley and Arlington Streets, clay pipe was used for the deeper sewer pipe. Gravel refill was used for bedding and backfill; compaction was by "ramming or wetting" during placement. Field records indicate that some pipe leaks and settlement were observed during construction. Pipe joints were sometimes flush, with only cement mortar. Recent analyses using Marston's procedures have indicated that the weight of trench backfill probably overstressed the clay pipe, which had much lower strength than modern clay pipe.

GROUNDWATER PROBLEMS

Although problems with the sewer were noted during construction, impacts were not apparent until 15 years later. In 1929, evidence of settlement of the front steps of the Boston Public Library led to investigation of wood piles which revealed extensive decay in the tops of many of the piles in the area nearest the sewer. The rotting had been caused by lowered groundwater which exposed the wood to air. The tops of wood piles were at or below Elev. 5, three feet below Charles River level. The nearby Trinity Church became very concerned about the 4500 wood piles on which it is founded. Investigations soon revealed that tops of wood piles at the church were submerged below groundwater, being lower than constructed due to settlement.

Study of groundwater levels by installation of numerous observation wells led to development of contour plans like the one shown in Figure 3.

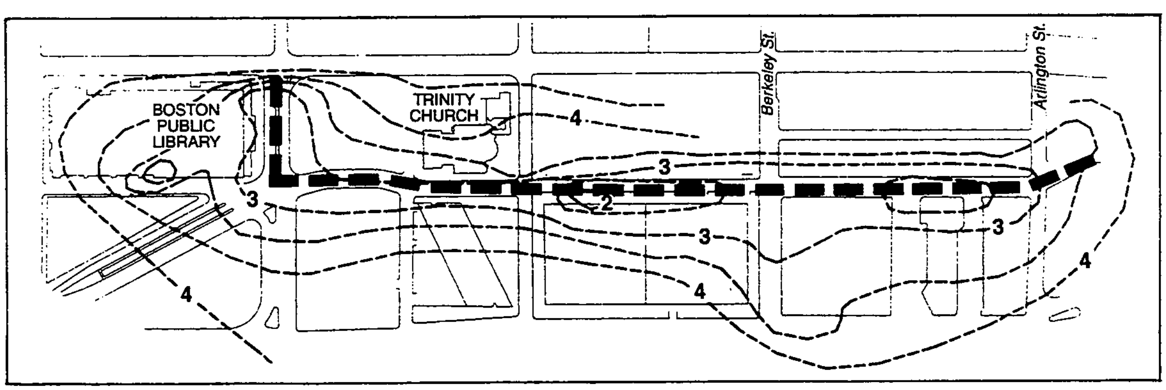

Figure 3 - St. James Avenue area 1932 groundwater contours, from Paine (2).

Further study showed that when sewer levels were surcharged several feet, groundwater levels rose similar amounts. Since 1933, the Sewer Department (now the Boston Water and Sewer Commission) has maintained a dam in a manhole in front of the Boston Public Library to artificially surcharge the lower sanitary sewer. This has kept groundwater levels at acceptable levels. A leak in the sewer was never specifically identified.

BUILDING CONSTRUCTION IMPACTS

Along the length of the St. James Avenue sewer, building construction projects have impacted the existing sewer, or pose obstacles to proposed replacement. In the late 1960's, the construction of the John Hancock Tower caused 2 to 4 feet of ground and street settlement when the lateral earth support system for this 30 ft deep excavation deformed. The concrete sewer pipes are within the zone of influence, and inspections and invert surveys have identified a sag of as much as 1-1/2 ft in the concrete pipe near the building. There was however, no significant lowering of groundwater associated with the excavation.

In the 1980's, office buildings built at 500 Boylston Street and 222 Berkeley Street used tiebacks to support lateral earth support systems (for 30 ft. deep excavations). Figure 4 illustrates the locations of the tiebacks that supported concrete slurry wall for the 500 Boylston Street building, which had to be located to bypass the existing pipes. However, there is little area available for a replacement sewer, and access from the surface is difficult due to the extensive network of other utilities.

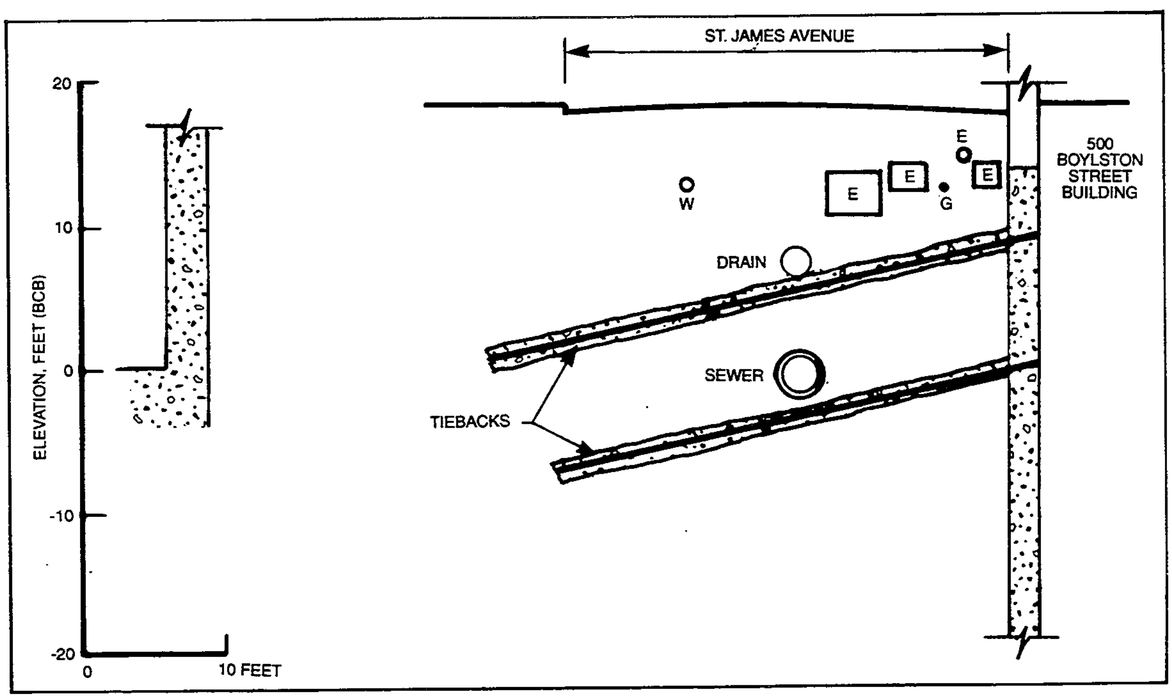

Figure 4 - Obstructions to micro-tunneling at 500 Boylston Street.

NEED FOR RECONSTRUCTION

With construction of the new Deer Island sewage treatment plant for the Boston Harbor clean-up project, local sewer authorities have renewed emphasis on reducing infiltration. The St. James Avenue sewer rehabilitation project began in 1990 with a charge to identify the problems of the existing system and propose corrective actions, with particular emphasis on eliminating the groundwater withdrawal problem. A study was made to determine areas of groundwater withdrawal by measuring changes in groundwater levels when the artificial sewer surcharge was removed. Findings have shown that groundwater drawdown began within a day when the surcharge condition was eliminated. In the Arlington Street area where the pipe is of clay tile, the greatest and most rapid lowering occurred, but this area is furthest from the dam.

The pattern of groundwater lowering now measured is much the same as measured in the 1930's. Significant lowering of groundwater occurs along most of the sewer's length, but it is suspected that most leakage occurs in the clay pipe area. The underdrain is thought to convey groundwater to "leaky" sections of sewer pipe. An important lesson to be learned is that construction techniques using underdrains can later cause remote problems to have far-reaching effects.

Due to poor structural condition and settlement of the sewer, and the groundwater lowering impacts, the Boston Water and Sewer Commission has decided to rebuild this portion of the sewer system. Final design of replacement systems is to be completed in the Spring of 1994.

SEWER REPLACEMENT METHOD

Prime concerns for designing the sewer replacement are the impacts that construction will have on groundwater, business activities, service connections, existing utilities and traffic. Figure 4 illustrates utility density typical along most of the alignment, and it is more intense at street intersections. Most of the utilities are within 5 ft of ground surface, well above sewer levels. Extensive utility relocations would have been necessary if open trench excavation were used.

Given the density of high-rise office buildings and hotels along the street, and the heavy traffic volume on St. James Avenue, which will be an auxiliary arterial during the upcoming Central Artery Project construction in Boston, a key component in the sewer reconstruction design is to minimize open street excavation. Micro-tunneling methods, in which non-man entry sewer pipe emplacements is accomplished using laser guided tunneling machine pushed ahead from a jacking pit, can be utilized to minimize construction impacts.

A number of trenchless methods to replace the St. James Avenue sewers are being considered, including; horizontal earth boring, and micro-tunneling along the existing alignment, and abandonment and new alignment construction using micro-tunneling. There would be only limited open excavation. Both geotechnical and construction considerations influence the selection of trenchless construction methods.

MICRO-TUNNELING DETAILS

The use of micro-tunneling for trenchless sewer pipe installation must address the particular subsurface conditions of the proposed alignment. For the St. James Avenue project, a number of issues enter into the sewer alignment and jacking pit location planning, and specifications development.

Replacement of both the existing sewer and drain means there will be micro-tunneling both above and below the groundwater table. The fill stratum gradation is generally medium to fine sand with some coarse sand and gravel and only trace silt, although substantial variations occur. Because of the area's sensitivity to groundwater lowering (see Aldrich and Lambrechts, ref.3) extensive dewatering is not an option for pipe construction. As such, the micro-tunneling method used to install the lower sewer pipe may require slurry shield techniques.

Selection of the machine will be a contractor option within specification requirements. There will be emphasis on maintaining positive face support to prevent loss of ground that could lead to surface settlement. It is expected that the same requirements will be used in the specifications for upper storm drain line micro-tunneling to prevent loss of ground in the clean granular fill. Many details of micro-tunneling rely on contractor experience. Throughout construction, the contractor must determine appropriate slurry for boring shield support and spoil transport, depending on ground conditions.

THE NEED FOR EXCAVATIONS

Open excavations will be required for jacking/receiving pits and to make service connections. Excavations may also be needed for obstruction removal, and in emergency situations for recovery of shield if it were to become inoperative. Jacking pits will generally be located at mid-block locations with receiving pits at intersections where manhole structures will be needed for connections to other tributary sewers. Again, design must both consider sewer hydraulic needs and minimize conflicts with existing utilities. Jacking pit excavations must be designed to provide sufficient thrust reaction for the jacking forces and prevent transfer of loadings to adjacent structures.

Pipe alignment must avoid the tiebacks emanating from the 500 Berkeley and 222 Boylston Street buildings, which leave very limited available space for installation of the pipes. If a tieback were encountered, it would obstruct the micro-tunneling machine. Other obstructions may also be encountered such as wood piles and granite blocks associated with the railroad and facilities that existed in the 1800's. Although most available boring machines can drill through wood piles, granite blocks would be difficult and require special rock cutting faces, or emergency removals by open excavation.

Small local excavations extending down about 20 ft and requiring localized dewatering will be needed for making service connections to the new sewer and catch basin connections to the storm drain. Several large office buildings and hotels are serviced by the

St. James Avenue sewer, which cannot be interrupted. The existing sewer must remain in operation during construction of the new line.

BORING THROUGH EXISTING PIPE

Special techniques of boring may be applicable for one block length of the sewer where clay tile pipe was not encased in concrete. This technique is especially applicable in the area of the former railroad station where the risk of encountering obstructions from previous foundations is considered high.

Special micro-tunnel pipe crushing machines which remove the existing pipe and insert the new pipe directly behind the micro-tunneling machine may be feasible. Another trenchless method of pipe rehabilitation that can be used on small diameter pipes, bursts the pipe by outward expanding pressure, pushing broken pieces of pipe out into surrounding ground and pulling replacement plastic pipe behind. When this method is used, special design of systems is needed to provide for bypass of sewer flows either through the machine and following new pipe, or by pumping through auxiliary bypass pipes.

It would be difficult to use these methods or to drill out existing concrete encased pipes. Concrete pipes will probably be abandoned because their size and type of construction (Fig. 2) would require very large replacement bores.

PIPE CONSIDERATIONS

A final, but very important consideration for design and construction using micro-tunneling is the strength of the pipe to be used and the detail of its joints. The pipe must be designed to satisfy the jacking thrust loads. Joints are particularly susceptible to damage under jacking forces. Due to the sensitivity to groundwater lowering, the new pipe must be water-tight with certain provisions to prevent infiltration. Also, future settlement is expected, and there may be differential settlement along the length of the sewer.

REHABILITATION WITH LINERS

Sewers in three blocks adjacent to St. James Avenue are not considered in need of reconstruction, based on initial examinations of the conduits. These will be rehabilitated with liners designed to carry full structural loads. Methods of relining that are being considered are generally the modified slip-lining methods. The existing conduit's inner surfaces must be cleaned prior to lining insertion. Although the new linings will reduce interior areas by 5 to 8 percent, the reduction in roughness will increase flow efficiency and capacity will not be reduced. Relining methods can not however correct out-of-shape or sagging conditions.

SUMMARY

The St. James Avenue sewer reconstruction project illustrates innovative trenchless technology construction methods that will find greater applicability and desirability as aging sewerage systems in older congested cities must be rehabilitated or replaced. Micro-tunneling new conduits to replace older pipes must address project and site specific conditions and requirements, such as those conditions discussed for the St. James Avenue project.

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IMPACT OF UTILITY CUTS ON PERFORMANCE OF STREET PAVEMENTS

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ABSTRACT

Often, utility companies dig open a small area of city street pavements to inspect utility services or install new ones. It has generally been observed that, the pavements in and around the cuts deteriorate at a greater pace. Currently no standard guidelines are available for evaluating the quality of restoration and the extent of maintenance required. This paper reports the results of a study being carried out at the University of Cincinnati for the City of Cincinnati and the American Public Works Association to develop a methodology for evaluating the quality of restoration of cuts. The preliminary results indicate that, for the City of Cincinnati, the additional maintenance cost due to poor restoration amounts to one million dollar per year.

General

The street network of a city represents one of the single largest investment of public funds. Pavements comprise the most important component of streets. Unless maintained properly, pavements deteriorate and disintegrate at a rapid rate. The problem is further compounded with the presence of utility cuts which introduce discontinuities and weaken the pavement. Although the utility companies fill up the cuts and restore the pavement sections, it is observed that the road sections in and around the vicinity of such cuts not only fail at an accelerated pace, but the failures extend to adjacent sections, thereby progressively weakening larger areas of the pavement. Currently, cities recover the administrative and inspection cost from the utilities, based on the surface area of the cuts made. If a cut is ineffectively restored and causes damage to a wider area of the pavement, the city has traditionally borne all future maintenance costs from its own resources. With increasing number of cuts, a greater demand for more frequent maintenance of streets, and increase in maintenance costs, many cities are now questioning their cost recovery policy.

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Local Agency Issues

Local governments are faced with higher community expectations for public services and amenities. With a large number of cuts made every year, either for installing new services or inspection and maintenance of existing ones, controlling the quality of opening and restoration becomes an uphill task. The current procedures followed by Cincinnati and other cities for recovering the additional costs of pavement maintenance due to the long-term effects of utility cuts have been inadequate. However, cities cannot establish a new cost recovery policy unless it is based on scientific investigations. Many cities have developed guidelines for utility cut opening and restoration procedures. However, there are no standard procedures for the field evaluation of the quality of restoration and for assessing related costs in the event of a poor restoration.

Present Study

Recognizing the need for a scientific, reliable and yet practical approach for the field evaluation of utility cuts, a research project was initiated at the University of Cincinnati Infrastructure Institute in February 1991. This research was based on a request by the City of Cincinnati, following the City's joining a nationwide research effort by the American Public Works Association, focusing on this problem. The study is directed towards developing field techniques appropriate for the evaluation of utility cuts using objective measurement of strength (deflection) and subjective assessment (visual inspection of distresses). This paper deals with cuts in asphalt and macadam pavements and reports on the following:

- deflection testing procedure;
- extent of pavement affected by cuts;
- seasonal deflection correction factor; and
- computing additional maintenance costs.

Deflection Measurements: Instrument, Procedure and Test Sites

The deflection measurements were made by using the standard Benkelman Beam along with a truck having a rear axle load of 18,000 lbs and 70 psi tire pressure on the rear dual wheels. The deflection test involved measuring maximum rebound deflection under a truck wheel load as per the Canadian Good Roads Association Procedure [1]. The deflection tests were carried out in two phases. The first phase involved a comprehensive study around utility cuts to find the areal extent of pavement weakening, and the critical points for deflection measurement. The second phase involved routine measurements of deflections at the critical points, as identified in the first phase. Figure 1 illustrates the location of deflection observation points. Deflection measurements were made at close intervals near the cut and on a control point at a distance of 20 feet away from the edge of the cut. This control point

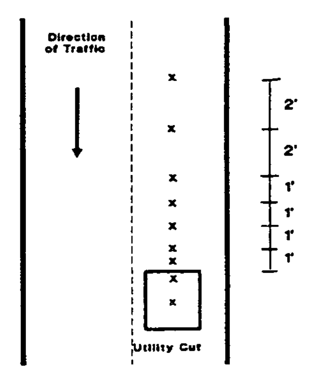

Figure 1. Location of Deflection Observation Points

was assumed to be in a zone of no influence. The deflections measured in and around the cut were utilized to establish the extent of influence. In all, 36 cuts in asphalt and macadam pavements were tested. Figure 2 shows a typical plot of deflections and the surface condition in and around the cut.

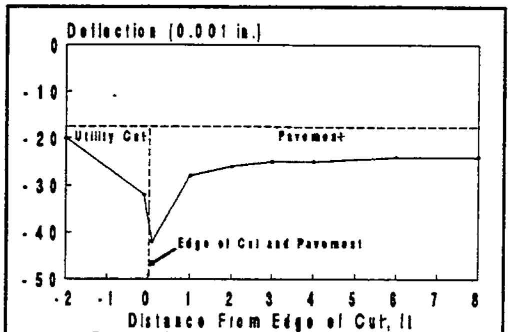

Figure 2. Surface Deflection Plot

Data Analysis

Temperature and Seasonal Correction

The pavement surface temperature can have significant influence on the behavior of pavements. At higher temperatures, asphalt pavements are less stiff and hence deflect more. At cold temperatures, due to increase in stiffness, they deflect less. Hence, Asphalt Institute [2] recommends that the deflections measured be corrected for a standard temperature of 70^0F, using the charts provided. Pavement deflections also vary with the season. Deflections will usually be larger during the rainy spring season or spring thaw. Hence the deflection measurements made at any time of the year should be corrected for the critical season using a seasonal correction factor. In order to do this, 12 cuts tested in summer were retested during the spring. The deflections were initially corrected for temperature and then a ratio of deflections during the two seasons was computed for each cut. A statistical analysis was carried out to determine the most representative value of the seasonal deflection correction factor.

Results

Three sets of results were obtained from the analysis of field observations. They were: (i) the seasonal deflection correction factor, (ii) lateral extent of damage, and (iii) additional overlay thickness.

Seasonal Correction Factor

The average seasonal correction factor was found to be 1.26. All the deflection data collected at times other than spring were multiplied by this factor after applying the appropriate temperature correction.

Lateral Extent of Damage

Using the deflection plots similar to Figure 2 for each cut tested, an analysis was made to estimate the average extent of pavement area affected by a cut. This was done by systematically measuring the deflection of points at and near the cut and comparing them to the deflection of the pavement at the control point. If the deflection at a point was found to be greater than the deflection at the control point, then that point in the pavement was considered to be adversely effected by the cut. The aggregate of such points made up a zone of influence in and around the cut. The boundary of the zone was given by points where the deflection was equal to that of the control point. The width of the zone of influence around each cut was tabulated. This varied with the size of the cut, traffic level and existing condition of the pavement. Thus the typical area of weakened pavement at and near a 4 ft by 5 ft cut was found to be (4 + 3) x (5 + 3) = 56 sq ft. To restore the strength of this area (or reduce its deflections to that of the control point), an overlay over the whole area of the weakened pavement may be applied.

Overlay Thickness Computation

The Asphalt Institute Method [3] was employed to compute the required overlay thickness. The key inputs for the overlay design were the maximum rebound deflection and the traffic in terms of Equivalent Single Axle Loads (ESAL). The overlay thickness at the control point and a critical point near the cut was computed. If the overlay thickness required near the cut exceeded the thickness required at the control point, the difference was computed as the additional overlay thickness required.

Cost of Overlay

Finding the cost of overlay is a complex task requiring the determination of the exact area over which the overlay needs to be built and the time of construction. The average area of influence of a 4 ft by 5 ft cut was found to be 56 sq ft. Constructing an overlay of this size over a cut would result in an elevated area which would affect the safety of motorists and would also be aesthetically objectionable. To solve this problem, two options are available. The rideability can be improved by levelling off the overlay over a larger area and providing a smooth transition. The second option is to remove a certain thickness of the existing pavement in the affected area and replace it with a superior material that can strengthen the cut area and also be in level with the surrounding pavement. The second option would be the preferred option.

For either of the two options, the time of execution becomes critical. With nearly 6,000 cuts made every year in the City of Cincinnati, the cost of individual overlays on utility cuts would be high in comparison with the normal cost of pavement resurfacing. The estimated cost of an overlay over the affected area and the additional material and labor required to provide a levelling course was computed. The summary of required overlay thicknesses is presented in Table 1. The overlay thickness ranged from 0 to 3.5 inches, with an overall average of 1.75 inches. The range of cost per cut was estimated to be \$500 to \$750. Approximately 35% of the cuts in Cincinnati are in asphalt or macadam pavements. The test results indicated that about 75% of the cuts needs strengthening. Hence for the City of Cincinnati, with about 6000 cuts a year, the additional maintenance cost per year amounts to approximately one million dollars. It should be recognized that these figures are preliminary and need refinement.

Conclusions and Limitations

The study carried out at the University of Cincinnati has resulted in an objective evaluation of impact of utility cuts on surrounding pavements. The study demonstrated that the Benkelman Beam can be used for the strength evaluation of pavements at utility cuts. The study also presents a methodology to identify the lateral extent of area affected by a cut based on deflection measurements. Construction of an overlay over the affected area requires the identification of feasible materials and construction methods. Certain policy decisions could affect the overlay costs. A city may decide to resurface the street in a complete block instead of a small area around the cut. The restoration may also be deferred to a future date when the whole block is considered for resurfacing. Thus computing the actual overlay costs is a complex task involving technical as well as managerial decisions. The cost figures presented in the present study are, therefore, only tentative. The writers are looking at viable alternatives and are in the process of refining the cost model.

Acknowledgements

The authors express their sincere appreciations to the City of Cincinnati and the American Public Works Association for sponsoring this study.

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Table 1. Additional Overlay Thickness Required Around Utility Cuts

Utility Cut	Max. Defl., in.	Control Point Deflection, in.	Additional Overlay Thickness, in.	Utility Cut	Max. Defl., in.	Control Point Deflection, in.	Additional Overlay Thickness, in.
1	0.064	0.045	1.50	15	0.039	0.030	2.50
2	0.048	0.045	0.50	16	0.050	0.047	0.50
3	0.049	0.020	3.50	17	0.139	0.119	1.00
4	0.059	0.030	3.50	18	0.040	0.030	2.00
5	0.051	0.035	2.00	19	0.062	0.021	3.00
6	0.036	0.028	2.00	20	0.042	0.024	2.00
7	0.161	0.087	2.50	21	0.058	0.042	2.00
8	0.109	0.094	1.00	22	0.039	0.032	1.50
9	0.050	0.047	1.00	23	0.147	0.130	0.50
10	0.095	0.030	4.00	24	0.113	0.076	1.50
11	0.082	0.054	2.50	25	0.130	0.111	0.50
12	0.140	0.119	0.50	26	0.109	0.079	1.00
13	0.057	0.052	0.50	27	0.127	0.085	1.50
14	0.091	0.073	1.00				
Average Overlay Thickness = 1.75 in. (Note: Overlay thickness computed using Asphalt Institute Method for a 20 year analysis period)							

STABILITY ANALYSIS AND REMEDIAL ACTION FOR SLAB ON A BUTTRESS DAM

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ABSTRACT

After review of the 1986 Part 12 inspection report for the C.H. Corn Hydroelectric Station, the Federal Energy Regulatory Commission (FERC) requested additional analyses be performed on the stability of the station powerhouse and spillway structures. A renewed effort was made to determine the existing condition of the structures and the foundation materials. Then, after establishing the foundation material strength properties, stability analyses were performed in accordance with FERC guidelines for the evaluation of hydropower projects. For those cases being analyzed which did not have a Factor of Safety as required by the FERC guidelines, remedial measures were proposed to increase the stability to the extent necessary to satisfy the regulatory requirements.

INTRODUCTION

The C. H. Corn Hydroelectric Station is a 12.3 MW low head hydro facility located on the Ochlochonee River 35 miles upstream of the Gulf of Mexico and 20 miles southwest of Tallahassee, Florida. The Ochlochonee River is restricted by the Jackson Bluff Dam thus creating Lake Talquin. The dam consists of the powerhouse, a 196-foot concrete spillway, and 3600 feet of earth embankment. The spillway is a hollow, slab on buttress, reinforced concrete structure with seven radial gates. The powerhouse is 132 feet long with a reinforced concrete substructure and brick superstructure.

The left abutment of the dam as well as the powerhouse and spillway foundation consists of sandy limestone and sandstone of Middle Miocene Age. The primary focus of the analyses was to determine the Factor of Safety against the structures sliding on or moving with sections of underlying rock.

PROJECT HISTORY

The project was licensed to the Florida Power Corporation by the Federal Power Commission (FPC) in 1927 and was constructed in 1928-29. In 1931, 1940, and again in 1948 erosion of the limestone rock and riverbank slopes downstream of the

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spillway required modification and repair of the right training wall and spillway apron. In 1957 a portion of the earth embankment failed as a result of saturated slopes following a period of heavy rain. The embankment was repaired and the plant was put back into operation in 1958.

The FPC license expired in 1970 and the Florida Power Corporation transferred the operation of the reservoir to the State of Florida, Department of Natural Resources. The reservoir, Lake Talquin, was then operated solely for recreation and flood control. In 1981 the City of Tallahassee, Florida, filed with the Federal Energy Regulatory Commission (FERC) for a license to reconstruct the hydroelectric generating project. Following repairs and modifications to the dam structures and installation of new generating equipment, the plant was again producing electric power by the end of 1985.

STABILITY CONSIDERATIONS

A safety inspection of the project was performed in late 1985. This inspection was the initial inspection to be made in compliance with Part 12 of the FERC regulations. The criterion and method of analysis for the stability of dam structures are presented in publication FERC 0119-2, "Engineering Guidelines for the Evaluation of Hydropower Projects". In this publication the recommended Factor of Safety for the usual loading condition (plant operating with normal headwater and tailwater pool elevations) is 3.0. For unusual conditions (flood conditions) the recommended Factor of Safety is 2.0.

In the preparation of the safety inspection report it was determined that the normal loading condition was the most critical condition. It was found that during flooding, the water elevation downstream of the dam rose faster than the headwater. This was due to restriction of the flood plain downstream of the structures. With the water rising faster downstream, the differential head across the dam decreased, thus increasing the stability of the structures. Through discussions with the FERC it was agreed that a Factor of Safety of 2.0 would be used for both the usual and unusual loading condition.

The subsequent reduction of the Factor of Safety for the usual loading condition was significant in ultimately satisfying FERC requirements. However, with this reduction it became apparent that an extra effort was being made by the FERC to develop a thorough understanding of the actual site conditions and to have the analyses performed using conservative material properties and boundary conditions. The following sections discuss the site and laboratory investigation programs and the ultimate selection of the material properties and boundary conditions required by the FERC.

SITE AND LABORATORY INVESTIGATION

Safety inspections have been performed throughout the life of the project. These inspections have

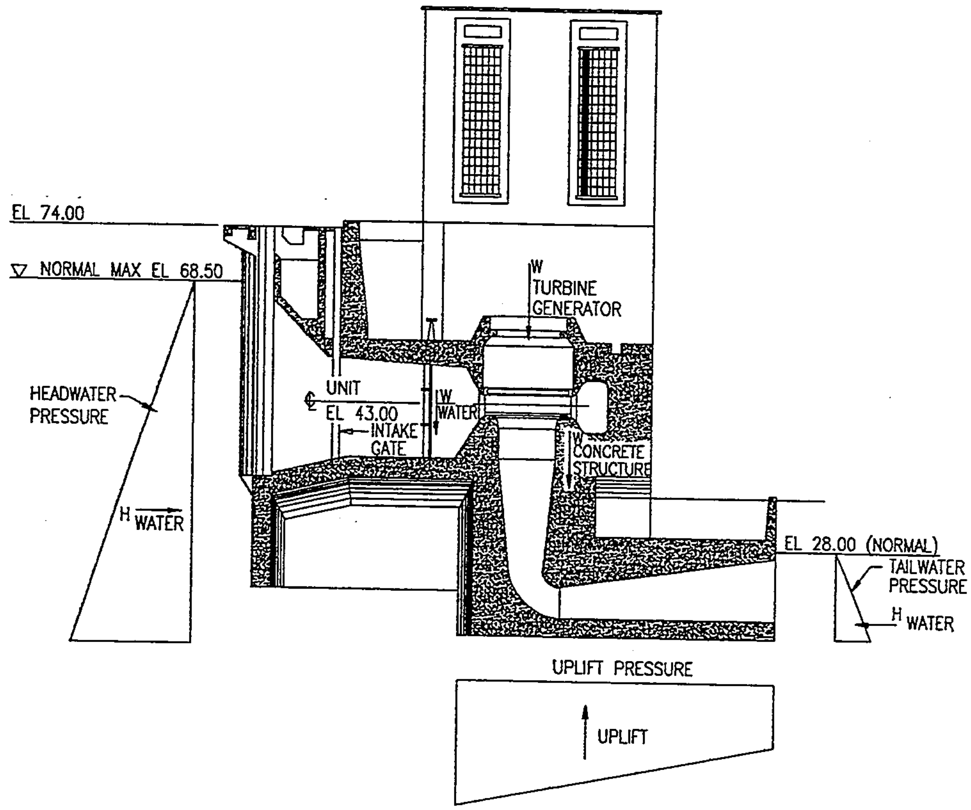

Fig. 1 POWERHOUSE LOADING DIAGRAM

included visual inspection of the structures, underwater inspections and review of instrumentation monitoring data. One of the more comprehensive investigations and reports was performed in 1967. The investigation included site and laboratory investigation of the structure and foundation materials. Analyses based on the information obtained indicated stable structures with adequate Factors of Safety. Additional material testing was performed on concrete cores in 1984.

For the 1985 Part 12 inspection all available test data was used to establish the material properties as well as the lower bond of published representative test data. The resulting analysis did satisfy the stability requirements, however, the FERC requested further verification of the site conditions and the foundation material strength properties used in the analysis. A drilling and testing program was conducted in 1988 and again in 1990. Strength tests of rock cores obtained during the investigation indicated properties similar to those used in the analyses. The interpretation of the new data, however, suggested the possibility of multiple sand seam in the foundation rock. Ultimately the FERC required the stability analysis be performed with sand seams 5, 8, and 15 feet below the concrete structure. Sand strength properties were also to be used at the base of the structure.
Rock properties were to be used only for crossbed shear which was limited to the sloping surface of the passive wedge.

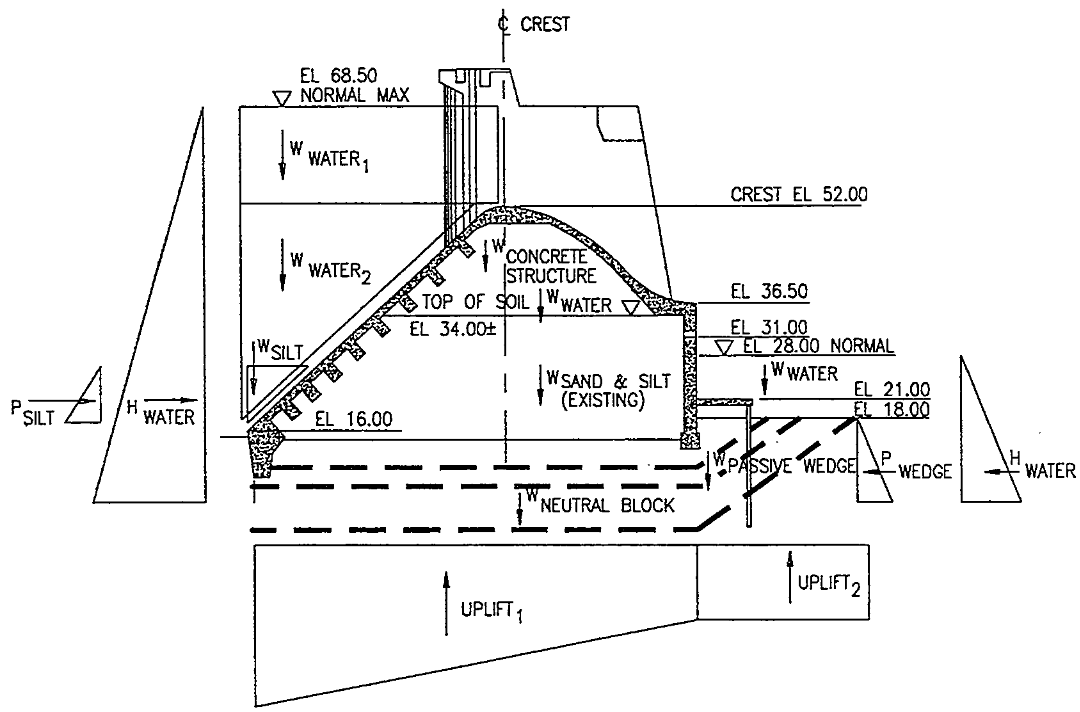

Fig. 2 SPILLWAY LOADING DIAGRAM

The strength properties were conservatively set at $\phi=36°$, C=0 for the sand and $\phi=21°$, C=17psi for the rock. The rock strength represented the lower limit of all strength tests performed during the 1990 investigation program.

STABILITY ANALYSES

Figures 1 and 2 represent the loading conditions analyzed for the powerhouse and spillway respectively. The driving forces consisted of the water load and silt load upstream of the structures. The resisting forces being the sliding friction along the horizontal sand seams and the passive wedge at the downstream edge of the structures as well as the downstream water load. For the powerhouse the friction force was controlled by the weight of the rock and structure overlying the failure surface. This load was reduced by the full uplift pressure calculated at the failure plain. For the spillway the load consisted of the rock, the structure, and the soil in the spillway bays. These loads were also reduced due to the uplift pressure.

Not illustrated in Figures 1 and 2 is the unique condition for the spillway sliding at the base of the structure. For this condition friction force could only be developed at the base of the buttress and support walls and at the passive wedge in front of the upstream shear key. With this loading condition the weight of the soil within the spillway has only limited stabilizing influence on the structure. At the same time, however, the stability is enhanced since uplift pressure cannot develop in the sand at the base of the walls.

The analysis of each failure case for the powerhouse and for the case analyzed at the base of the spillway resulted in Factors of Safety equal to or greater than 2. Those cases analyzed for failure along the sand seams underlying the spillway indicated stable conditions, however, several cases failed to satisfy the required Factor of Safety of 2.

PROPOSED REMEDIAL MEASURES

Several remedial measures were considered which would result in an increased Factor of Safety. Of these measures one method was found to be significantly more appropriate in terms of constructibility and the ability to obtain the desired result. By adding sand to the existing fill inside the spillway bays the load over the sand seam was increased. This increased loading resulted in higher friction values along the failure surface and increased the resisting force used to calculate the Factor of Safety. It was found that bringing the fill to elevation 34 feet, an increase of 5 to 13 feet, a Factor of Safety of 2 or more could be calculated for each of the cases analyzed. It must be noted that at the time of this writing, final FERC review has not been completed and therefore the analysis has not received final acceptance.

CONCLUSIONS

The reduction of the recommended Factor of Safety from 3 to 2 for the usual loading condition was in part justified since the usual loading condition was also the most critical condition. With the reduction, FERC required additional site subsurface investigations and laboratory testing of foundation and construction materials. This was done in order to obtain a thorough understanding of the site conditions and to verify strength properties. A conservative interpretation of both the site conditions and the material strength properties was then assigned for the stability analysis.

The stability analysis was then performed using only those driving and resisting loads which are given in the FERC guidelines and which can be reasonably defined through site investigation and laboratory testing. Resisting forces which do exist but which are difficult to quantify were not used in the analyses.

The required Factor of Safety was not obtained for all cases analyzed using the existing site conditions. The remedial action selected consisted of placing additional soil fill within the spillway structure. By placing 5 to 13 feet of fill in the spillway bays, a Factor of Safety of 2 or greater was obtained for all cases analyzed. It is anticipated that the remediation can be constructed in a relatively short time and at a reasonable cost.

The approach taken to the stability analysis of the C. H. Corn Hydroelectric Station represents a realistic but conservative stability evaluation for the dam structures.

REHABILITATION OF ROADS ACROSS SOFT SOILS

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Abstract

Much of the infrastructure built during the last few decades is reaching the end of its service life and needs rehabilitation. Increased vehicle density, higher axle loads, and the insufficient width of many rural two-lane primary highways (less than the 24-foot pavement width considered desirable), require that the capacity of existing highways be increased through an upgrading program. Modernizing existing narrow highways generally calls for widening, raising of grades, and strengthening of pavement structure. To compound the problem, adverse weather and climatic conditions, and the resulting changes in groundwater regime, have caused slope failure and excessive settlements of foundation soils in numerous areas of the United States.

Rebuilding of highway structures usually poses greater challenges than building new facilities, mainly due to limited realignment options available at problem areas. Highway improvement normally involves raising of grades and additional load application on foundation soils and slopes. This may produce stability problems, especially across soft soils. To reduce the weight of the highway structure at such locations, wood-chips or saw dust have been traditionally used as a replacement for conventional material. Wood is biodegradable and lacks durability. Engineers and researchers have a keen interest in developing materials that are more durable, economical, and lighter in weight to replace conventional materials and enhance the stability of slopes and foundations in problem areas. These apparently contradictory requirements can now be reconciled by the use of rubber-soils.

Various agencies, in the United States and abroad, have employed and evaluated the use of shredded tires as lightweight fill material in a variety of different ways, i.e., soil-chips mixes, layered with soils, or pure chips. In response to a questionnaire survey conducted by the authors, as part of a synthesis study on the "Use of Waste Materials in Highway Construction", some of the state highway agencies (e.g., Minnesota, Oregon, Vermont, and Wisconsin) which experimented with the use of shredded tires in embankments and retaining structures to reduce the weight of fill on weak foundations, reported that the use of rubber-soils as lightweight fill is feasible and quite beneficial. However, information on this application of waste tires is severely lacking. Only a few limited laboratory studies have been reported in the literature.

The authors have conducted a laboratory study to investigate the feasibility of incorporating shredded tires, alone and mixed with soils, in highway structures as lightweight geomaterials. The focus of this research is on determining compaction, compressibility, shear strength and resilient properties of laboratory prepared rubber-soil samples.

Introduction

Scrap tires by the millions are discarded annually in the United States and other developed countries of the world, the bulk of which is currently landfilled or stockpiled. This consumes valuable landfill space, creates a fire hazard, and provides a prolific breeding ground for mosquitos. Efforts to sharply reduce the environmentally and economically costly practice of landfilling have stimulated the pursuit of non-landfill disposal or reuse of waste tires. Several beneficial uses for tires have been proposed in the past and some have been put into practice in various highway and non-highway applications. The use of tire chips as lightweight fill can sharply reduce the tire disposal problem, if they are found technically feasible, environmentally acceptable, and economically beneficial.

The inherent attractive engineering properties of tires have led to their use in a variety of engineering applications. This paper synthesizes the information on the feasibility of using rubber tires as lightweight geomaterial in highway structures. An overview of lightweight materials commonly used for highway embankment fills and other engineering structures, physical and chemical characteristics of tires, and salient aspects of field and laboratory studies in the use of shredded tires as lightweight fill material are presented. In addition, performance, potential environmental impact and constructional aspects of shredded tire embankments are discussed. Finally, a summary of relevant conclusions is presented.

Conventional Lightweight Materials

Lightweight waste materials, such as sawdust, bark, slags, cinders, and ashes, are generally available in abundance and mostly at no cost at the source. These materials have traditionally been used as lightweight fills by the United States highway agencies and may be rationally compared with other waste materials, like tire chips. Sawdust and bark have unit weights ranging from 35 to 64 pcf, are biodegradable, difficult to compact, require treatment to prevent groundwater pollution, need to be encapsuled in soil cover, and undergo significant long term settlement. Salient properties of slags, cinders, and ashes include: dry unit weights ranging from 64 to 100 pcf; may absorb water, resulting in increased density; possess high variability; and their leachates may adversely affect groundwater quality or structures in the vicinity. All these materials been used in the past, although some are more popular than others and some have been only used on an experimental basis or for structures other than highway embankments. The performance and cost differences between the various materials are significant. However, all have compacted densities significantly lower than the unit weight of soils commonly used in embankment construction. Hence their use can substantially reduce the effective weight of the embankment. A questionnaire survey by Holtz (1989) indicated that lightweight fills have been used to some extent by 40% of the United States highway agencies responding to the questionnaire.

Lightweight materials are usually expensive, especially if they are manufactured (e.g., expanded shales and clays, polystyrene, lightweight concrete, etc., see Table 1). Typically, costs range from $50 up to $100 per cubic yard, including transportation (Holtz, 1989). Some waste materials (i.e., sawdust, bark, shells, cinders, slags and ashes, etc.) are almost free at the source and only need to be transported to the site. Thus their cost will depend on the distance between the source of waste material and the site. Lightweight fills have also been found as cost effective alternatives in certain applications in geotechnical engineering (Childs, et al., 1983).

Expanded shale aggregate has been used in lightweight structural concrete and masonry units (Stoll and Holm, 1985). The aggregate is expanded by heating shale in a rotary kiln under carefully controlled conditions at high temperatures (2100°F). The expanded, vitrified mass that results from this process is then screened to produce the desired gradation for a particular application. NCHRP (1971) reported that some expanded shale presents poor freezing resistance and must be kept dry.

Stoll and Holm (1985) conducted large scale triaxial compression tests on specimens of lightweight expanded shales from five different locations in the United States and also performed uniaxial strain tests (consolidation tests) on the aggregate from one of the sites. Their results indicated that the response under triaxial loading was similar to that of many ordinary coarse fill materials; the principal difference is that it weighs roughly half as much as conventional materials. Thus, lightweight aggregates may prove to be useful substitutes for ordinary fill materials when the combination of low weight and substantial shear strength warrants the increased cost. The mechanical properties of the aggregate tend to vary somewhat from source to source, so they should be verified in each instance. NCHRP (1971) states that expanded shale seems to be a favorite lightweight fill material because of its more certain behavior.

Nelson and Allen (1974) reported a successful landslide correction using bark and sawdust in a sidehill embankment. They used a 12 inch gravel base under the pavement section; in addition, an asphalt seal was placed on the exposed slope to retard deterioration and pollution. There have been many other projects that used sawdust and bark as lightweight fill in the Pacific Northwest. Large quantities of sawdust were used in the approach embankments to the Dumbarton Bridge in San Francisco (Holtz, 1989). Edil (1983) reported the use of sawdust, wood chips, and expanded shale in the lower portion of surcharge fills on peat. The lightweight part of the fill was left in place after the surcharge was removed.

Hardcastle and Howard (1991) reported the results of a laboratory study on properties of wood fibers used as lightweight fill in embankments for runways and aprons of Benewah County airport (located near St. Maries, Idaho). The various reported properties of wood fiber are: submerged, dry, and wet unit weights as 5, 14, and 55 pcf, respectively; and ϕ at 5% and 20% strains as 10° and 30°, respectively. The measured settlement after 32 months of an 8 ft. embankment ranged from 0.07 to 0.53 ft, which was twice the predicted value.

Expanded polystyrene (EPS) is considered a superlight material, because it is about 100 times lighter than ordinary fill materials (unit weight as low as 1.25 pcf; Frydenlund and Aaboe, 1988). The material is available in blocks and can be made sufficiently strong to be able to support ordinary highway pavement and traffic loads with tolerable settlements. But it is also an excellent insulator, and there has been some hesitancy among highway departments in the Northern United States to use it within 4 ft. of the pavement surface because of potential differential icing problems. Other problems with EPS include reports of burrowing animals in the material and increases in unit weight because of water absorption (Holtz, 1989).

Conversely, experience with EPS in a number of countries has been very positive. Frydenlund and Aaboe (1988) reported that more than 100 road projects involving the use of expanded polystyrene (EPS) have been successfully completed in Norway with volumes varying from a few hundred to several thousand cubic meters of EPS. Applications include embankments on soft and highly compressible soils, behind bridge abutments, sidehill embankments on unstable slopes and for rapid construction of pedestrian underpasses. In Sweden, more than 20 road embankments have been constructed with EPS (Hartlen, 1985). To prevent the EPS from being dissolved by petrol or other chemicals in case of a spill from an overturned tanker on the road, a 4 to 6 in. reinforced concrete slab is cast on top of the EPS blocks. The concrete slab also contributes to the strength of the pavement structure and reduces the total thickness of pavement material above the EPS blocks. EPS does not decay. The material is not fire resistant, and needs to be properly encapsuled in soil or concrete (Frydenlund and Aaboe, 1988).

Tire Characteristics

Although automobile and truck tires manufactured today are primarily steel-belted radial ply type, other types of tires are available. Some tires are made with fiberglass, aramid, and/or rayon. Most modern tires have a complex composition of natural and synthetic rubbers, chemicals, minerals, and metals. Steel-belted radial ply tires may also contain polyester, steel, or nylon cords. Some

Material	Unit weight (pcf)	Comments
Bark (Pine & Fir)	35-64	Waste material used relatively rarely as it is difficult to compact. The risk of leachates can be reduced by using material initially stored in water and then allowed to air dry for some months. The compacted/loose volume ratio is in the order of 50%. Long term settlement may be around 10% of compacted thickness.
Sawdust (Pine & Fir)	50-64	Waste material commonly used below permanent groundwater level but occasionally employed for embankments with asphalt or geotextile sealed side slopes.
Peat: Air dried Milled Baled Horticultural Compressed bales	 19-32 13 51-64	Found particularly useful in Ireland for repairing existing roads by replacing gravel fill with baled peat.
Fuel ash, slag, cinder, etc.	64-100	Waste materials such as pulverized fuel ash (PFA) are generally placed at least 1 ft above maximum flood level. Such materials may have cementing properties producing a significant increase of strength with time. Some may absorb water (e.g. furnace slag) and increase in density.
Scrap cellular concrete	64	Significant volume decrease during compaction. Excessive compaction reduces material to powder.
Expanded clay or shale (lightweight aggregate)	20-64	Physical properties such as density, resistance and compressibility are appropriate for use as a lightweight fill, although properties are affected by manufacturing process. The material is relatively expensive. A 20 in. soil cover on the slopes and 2 ft. road base thickness are minimal requirements for embankment construction.
Shell (oyster, clam, etc.)	70	Commercially mined or dredged shells are available on Gulf and Atlantic coasts. Size: 0.5 to 3 in. When loosely dumped, shells possess low density and high bearing capacity due to interlock.
Expanded Polystyrene	1.3-6	Superlight material used in Norway, Sweden, U.S.A. and Canada with satisfactory performance and increasing popularity. Thickness varies between 0.5 and 1 m, depending on traffic loads. Very expensive but attractive due to its very low density.
Low-density cellular concrete, Elastizell: Class I Class II Class III Class IV Class V Class VI	 24 30 36 42 50 80	Manufactured with Portland cement, water and a foaming agent ("Elastizell EF"). Six different categories (I to VI) of engineered fill are produced with varying compressive strengths of 10, 40, 80, 120, 160, 300 psi and bearing capacities of 0.7, 2.9, 5.8, 8.6, 11.5 and 21.6 tsf, respectively. The material is cast in-situ and has been used in highway embankments, bridge approaches, foundations, etc. (Elastizell Corp. of America, Ann Arbor, Michigan, 1992)
Tire chips	20-45	Has been widely used in the U.S.A. Benefits include: reduced weight of fill, free draining medium, inexpensive, and recycling of tires. Potential problems include metal and hydrocarbon leachate, high compressibility and fire hazard. Recommended in unsaturated zones of embankments. Requires top and side cap against fire and for confinement (Authors).

Table 1 Lightweight embankment fill materials (adapted from Holtz, 1989; OECD, 1979; Merdes, 1992; Elastizell, 1992; and other sources)

radial tires have a fine carcass wire, whereas bias ply tires do not. Both radial and bias ply tires contain bead wires, which consist of numerous strands of high tensile strength steel. The main constituents of rubber in tires are carbon and oil (hydrocarbons), hence the combustible nature of tires. When tires burn in uncontrolled environments, the black smoke that escapes contains fine particles of carbon. Carbon and hydrogen can make up as much as 96.5 percent of the tire. Rubber tires are designed to withstand the rigors of the environment so that they will have a reasonable useful life on vehicles. Therefore, it is not surprising that discarded tires persist for longer periods. Indeed, it has been estimated that a whole tire requires at least a hundred years to decompose fully (Hofmann, 1974; reported by Cadle and Williams, 1980).

Field Performance of Tire Chips in Minnesota

Various agencies, in the United States and abroad, have evaluated the use of shredded tires as a lightweight material in embankment construction and also for enhancing the stability of slopes in slide areas. The Minnesota Pollution Control Agency (MPCA) has documented over 23 sites (through April, 1992) throughout the state which have used over 80,000 cubic yard of shredded tires (about 2.2 million tires). Over half of these projects are privately owned driveways and roads, 4 are city and township roads, 3 are county roads, and 2 are DNR Forest roads. A few of the projects used shredded tires for purposes other than in road fills. One project in downtown Minneapolis used the lightweight tire shreds as a fill material to support a park and landscaping above an underground parking lot. At another site, tire chips were used as lightweight fill over an existing water main (Lamb, 1992). Six case studies documented by the Minnesota DOT in a recent report are described below (portions excerpted from Lamb, 1992).

Case Study 1: Ramsey County, Minnesota

A stretch of roadway on Ramsey County Road 59 (near St. Paul, Minnesota), which passes over a mucky low lying area with high water table, experienced excessive settlements and in 1990 required reconstruction. An economic and engineering analysis, conducted by TKDA (a consulting engineering firm) and Twin City Testing Corporation (the geotechnical subconsultant), resulted in the selection of shredded waste tires as the design fill material.

The construction commenced in the winter of 1990. The existing material was excavated to a depth of five feet. A geotextile was placed at the bottom and sides of the excavation. Next, wood chips were deposited to a depth of one foot above the water table. About 4,725 cubic yards of shredded tires were then placed on top of wood chips and compacted to a depth of three feet above the original roadway elevation. The 3x3 inch tire shreds were compacted with a dozer. The top geotextile layer was then added and sewn to the initial layer in order to encapsulate the wood chips and tires. A 3 foot layer of granular material, 6-inch base layer, and 5.5 inches of bituminous base and wearing course were placed on a shredded tire fill. The post-construction performance has not yet been reported (Lamb, 1992).

Case Study 2: Benton County, Minnesota

In this case, scrap shredded tires were used as roadway fill across a swamp that is underlain with peat and muck (Mn/DOT, 1990; and Public Works, 1990). The fill is located on County State Aid Highway 21 north of Rice, Minnesota, which is in the north-west corner of Benton County. The original construction across the swamp was stable, but subsequent additions to raise the roadway above the rising water in the swamp overloaded the underlying peat and muck, the consultant (Braun Engineering Testing) reviewed options to correct the soil stability problem. After performing a cost/benefit analysis, it was recommended that shredded tires be used as a lightweight fill material.

Construction began during the fall of 1989. The county excavated the embankment at the distressed portion, installed a geotextile, and then placed shredded tires directly on the geotextile in 2-foot lifts to a height within 3.5 feet of the top of subgrade elevation. After the tire shreds were compacted, an additional layer of geotextile was installed on top of the tires, prior to placing granular backfill. The tire fill supports: about 3.5 feet of clean granular soil cap, a conventional gravel subbase and base, and bituminous surfacing. The compacted shredded tire density is reported as 550 lb/cu yard. About 52,000 tires were used in the 250 foot portion of distressed roadway. Some of the construction specifications included (Public Works, 1990): the largest allowable piece was about 8 inch square or round, and the longest piece allowed was 12 inch long, whichever was less; it was required that the chips be free from any contaminants such as oil, grease, etc., that could leach into groundwater; metal fragments were required to be firmly attached and 98% embedded in the tire sections; all pieces must have at least one sidewall severed. This road has not experienced any significant settlements and the bituminous surface is performing satisfactorily (Lamb, 1992).

Case Study 3: Eden Prairie, Minnesota

In Eden Prairie, Minnesota (near Minneapolis) a road embankment project incorporated shredded tires in order to solve a settlement problem (Lamb, 1992). The original fill, placed over a swamp containing 40 ft. of soft organic soils, failed during construction. Three years after completion, the bed was still settling an average of one foot per year. It was decided to use shredded tires as lightweight fill to correct the subsidence problem. The original fill was excavated to a depth of 10-14 ft and about 4,100 cu yards of shredded tires were then placed in 2-3 ft lifts. The tire shreds were 6-8 inches wide and 12-24 inches long and were compacted by a D-8 dozer to a density of 40-45 pcf. A geotextile was placed on top of the tire shreds and 4 ft layer of common borrow was then placed on the geotextile. After 3 weeks, 12 inches of crushed limestone was graded over the fill material followed by 3.5 inches of bituminous base course. The wearing course was paved the ensuing spring.

Settlement data, obtained from the settlement plates placed both at the bottom and top of the shredded tires, indicate that the fill has performed very well. Over a period of 19 months, the roadway settled an average of 0.9 inches a year, while the subcut (at the bottom of the tires) settled only an average of 0.4 inches per year (Lamb, 1992).

Case Study 4: Prior Lake, Minnesota

In Prior Lake (a suburb of Minneapolis) the new alignment of the intersection of Duluth and Tower Avenues passed over a wetland area with 30 ft. of organic deposits. After analyzing various construction options, it was found beneficial to use shredded tires as a lightweight fill material. A geotextile was placed over the wetland material and then wood chips were compacted to an elevation of one foot above the expected water table level. Approximately three ft of shredded tires (about 9,600 cubic yards) were then graded over the wood chips. The 4 inch tire shreds were easily graded and compacted with dozers and loaders. The tire fill was covered with a 3 ft. of granular fill and base layer. A plate load test (applied directly on top of the shredded tires) indicated that the tire material was very compressible and displayed a very low modulus (Lamb, 1992).

Case Study 5: Lake County, Minnesota

The Lake County Highway Department reconstructed a gravel road using about 3,900 cubic yards of shredded tires on County State Aid Highway 7, near Finland, Minnesota. The original section, built over peat, experienced excessive settlements. After considering the various options, the county decided to construct the road using shredded tires as lightweight fill material. The road was constructed over the existing grade with a 4 ft layer of shredded tires, capped with a layer of geotextile, followed by about 1.5 ft of gravel. The tire shreds were quite large, ranging in size from

4x12 in. to one fourth of a whole tire, and were compacted with a dozer. After two years, the county reports no noticeable settlement of the road section containing tire chips.

Case Study 6: Milaca, Minnesota

The Minnesota Department of Natural Resources reported the use of shredded tires as lightweight fill on a 200 ft section of gravel road. The road, known as Esker Trail, passed over a section of wetland containing unstable peaty soil. The fill section included a layer of geotextile, followed by 3 ft layer (3,000 cubic yards) of shredded tires, and then topped with a second layer of geotextile. This was followed by 1.5 ft of common borrow, and capped with six inches of gravel. It has been reported that post-construction settlements were 40 to 50% less than were expected from mineral fill (Lamb, 1992).

Oregon Slide Correction Project

Based on successful experience of the Minnesota DOT in the use of shredded tires as a lightweight fill in embankment on weak foundation soil, the Oregon DOT also used shredded tires in a slide area on Highway U.S. 42 (Oregon State Route #35, Coos Bay-Roseburg), approximately 25 miles west of Roseburg, Oregon. The slide occurred in a newly-constructed 15 feet high embankment, with a slide block extending 150 feet beyond the toe of the embankment to a small creek running parallel to the highway.

Geotechnical analysis suggested reduction of the embankment weight and the construction of a counterbalance berm between the embankment toe and the creek. The design called for replacement of the existing fill with shredded tires to reduce the weight of embankment. The actual construction involved replacement of 12,800 cu yard of existing soil with 5,800 tons of shredded tires (an estimated 580,000 tires, Read et al, 1991).

A drainage blanket consisting of 12 inches of free-draining rock between two layers of geotextile was placed beneath the shredded tire embankment and the berm in order to prevent the groundwater table from rising within the embankment. Three 10 ft deep French drains were located beneath the blanket to enhance the subsurface drainage. The drainage blanket was required to prevent submergence of tire chips in water.

The embankment construction was completed in two stages to allow traffic on one half of the embankment while the other half was under construction. The shredded tires were brought to the project area from four different vendors, located 150 to 250 miles from the project, using 28 tons "live-bottom" trailers. Dump trucks were employed to deliver the chips to the construction site. A D-8 Dozer was used to spread and compact the chips in 2-3 ft lifts. Each lift was compacted with no less than three coverages in each direction, achieving an in-place density of 45 pcf. The reported density range of loose chips in the haul trailers varied from 24 to 33 pcf, depending on the haulage distance and size of the chips. Post construction density under 3 feet of soil, 23 inches of aggregate base, and 6 inches of asphalt and after 3 months under an average daily traffic (ADT) of 3750 vehicles with 20% trucks, was 52 pcf.

The shredded tire fill was constructed to an elevation 12 inch above the design height to compensate for a 10% anticipated compression (based on in-situ performance of a tire chips embankment constructed in Minnesota; see Geisler, et al, 1989). It was observed that the thickest portion of the shredded-tire fill (approximately 12.5 feet) compressed 13.4% during construction as follows: 16 in. during placement of 3 ft of soil cap; 2 in. during placement of 23 in. of aggregate base; 2 in. during 3 months of traffic and placement of 6 inches of asphalt concrete.

Deflection testing was conducted using ODOT's Falling Weight Deflectometer (FWD). The average deflection of the pavement over the rubber tire fill was approximately 0.020 inch compared to a typical deflection of 0.010 inch normally measured for a similar asphalt- and -aggregate-base pavement constructed over a conventional soil subgrade. Read et al. (1991) concluded that embankment construction using waste shredded chips is a viable technology and can consume large quantities of discarded tires at significant engineering benefits. The economics of using shredded tires in embankment depends on many factors, which vary with the local conditions, including: availability and cost of other lightweight materials; proximity to tire dumps and shredding equipment; and the existence of a state rebate program.

Use of Tire Chips to Cross Boggy Area

The Southeast Chester Refuse Authority in Pennsylvania was confronted with a problem of road construction over soft soil for movement of equipment from the landfill to the storage sheds (Biocycle, 1989). They placed an 18 inch layer of tire chips (2x2 inch) along a 525 feet section of roadway passing over a boggy area, without compaction or any other treatment. It has been reported that the section containing tire chips drains well and provides a good riding surface.

Test Embankment Containing Shredded Tires

The University of Wisconsin-Madison, in cooperation with the Wisconsin DOT, has conducted a limited field experiment to determine the feasibility of incorporating shredded tires in highway embankment (Edil et al., 1990 and Bosscher, et al., 1992). They constructed a 16 feet wide and 6 feet high test embankment consisting of ten different sections, each 20 feet long, using locally available soil and shredded tires in a number of different ways, including: pure tire chips, tire chips mixed with soil, and tire chips layered with soil. They also varied the embankment configuration for different sections of embankment to determine the optimum slope. A geotextile was placed on all sides of tire chips to serve as a separator between the embankment body and the surrounding materials. The embankment was constructed parallel to the access road of a sanitary landfill and exposed to heavy incoming truck traffic.

Compaction was done using a sheepsfoot roller with vibratory capability. Field observations during construction included (Edil, et al., 1990): handling and placement of tire chips was not a problem; a back hoe was found appropriate for spreading the material evenly; tracked equipment could easily maneuver on tire chips; neither vibratory nor static compaction significantly improved compaction of tire chips, however, non-vibratory compaction was found more appropriate; compacted field density varied from 20 to 35 lb/cu ft, depending upon type/size of chips.

Edil, et al. (1990), based on construction and initial post construction evaluations, have reported that construction of embankments using tire chips does not present any unusual problems. Leachate characteristics indicated little or no likelihood that shredded tires would affect groundwater. The main problem is reportedly related to control of compressibility. A two-year monitoring and evaluation of the test embankment supports the use of properly confined tire chips as a lightweight fill in highway, some observations include: after an initial adjustment period, the overall road performance was similar to most gravel roads; embankment sections having 3 of feet soil cap performed better than that having 1 foot of soil; soil and tire chips mixture had similar performance to the pure chip sections with a thicker soil cap, the presence of a thick soil cap reportedly helps reduce plastic deformation; comparatively, the layered section performed worst; leachate analysis indicated that shredded tires show no likelihood of having adverse affects on groundwater quality.

Use of Tire Chips on a New Interstate in Colorado

The Colorado Department of Transportation has recently experimented with the use of shredded tires as lightweight fill material (Lamb, 1992). Shredded tires have been used on a 200 ft portion of Colorado's new Interstate 76, a four-lane highway that will connect west Denver to Nebraska when completed in 1993. More than 400,000 tires chips of about four-inch size have been consumed in a 5 ft fill. The tire embankment has been instrumented for monitoring the long term performance of the fill.

Laboratory Studies

Various databases, including: Compendex Plus (online form of engineering index); NTIS (National Technical Information Service); TRIS (Transportation Research Information System); Enviroline; and Pollution Abstracts, were searched to locate the literature on the subject. Four laboratory studies were identified: 1) a limited laboratory study conducted by the University of Wisconsin-Madison to determine the mechanical properties of rubber and rubber-till mix, and leachate analysis of specimens collected from shredded tires test embankment (Edil, et al., 1990 and Bosscher, et al., 1992); 2) the Minnesota laboratory study on leachates from tire and asphalt materials (MPCA, 1990); laboratory study by University of Maine to determine the properties of tire chips for lightweight fill (Humphrey, et al. 1992 and 1993); and Caltrans study to determine studies are briefly described in succeeding paragraphs.

Wisconsin Study

A limited experimental program was carried out at the University of Wisconsin-Madison to develop quantitative information about the compaction and compression behavior of tire chips, and analysis of leachates from a test embankment made of rubber-soil (Edil, et al., 1990). Their experiment involved placement of rubber chips of different sizes alone and mixed with sand in a 6-in. Proctor mold and then applying load using a disk placed on the tire chips. The load-deformation response of rubber chips indicated that the major compression is irrecoverable; but there is significant rebound upon unloading. The rebound is nearly the same from one cycle to another. It is observed that the slope of the recompression/rebound curve is markedly lower beyond a certain vertical load of about 1000 lbs. Their test results, on specimens of sand/chip ratios varying from 100% sand to 100% chips, indicated that compression increases significantly when tire chips content was increased beyond 30% by weight of sand.

The authors recommend caution in using data reported by Edil, et al. (1990) concerning chips and chip-sand mix, since they conducted tests in a compression mold too small in diameter for the size of chips tested (chip sizes of 1.5 inch and larger were tested in 6 inch Proctor mold). It is likely that greater side frictions are induced in a compression mold incompatible with the sizes of chips tested, which may have deformation response. A careful review of reported data indicates that the reported deformations are significantly lower than those expected under corresponding loads.

Edil et al. (1990) have also reported duplicate EP toxicity and AFS leaching tests performed on tire chip samples by the Wisconsin State Laboratory of Hygiene. The test results indicate that the shredded automobile tire samples show no likelihood of being a hazardous waste. The shredded tires appear to release no base-neutral regulated organics. The tire samples showed detectable, but very low release patterns for all substances and declining concentrations with continued leaching for most substances. By comparison with other wastes for which leach tests and environmental monitoring data are available, tire leachate data indicate little or no likelihood of shredded tires affecting groundwater. Bosscher, et al. (1992) have reported that an overall review of the available leach data and results of the recent leach tests on samples collected from two lysimeters, installed during construction of the test embankment in December 1989, support their initial conclusions.

Minnesota Study on Tire Leachates

The Minnesota Pollution control Agency (MPCA) sponsored a study on the feasibility of using "Waste Tires in Subgrade Road Beds" (MPCA, 1990). Twin City Testing Corporation (TCT) of St. Paul, Minnesota, performed the laboratory study to evaluate the compounds which are produced by the exposure of tires to different leachate environments. They subjected the samples of old tires, new tires, and asphalt to laboratory leachate procedures at different conditions, i.e., at pH 3.5, pH 5.0, approximately neutral pH and 0.9% sodium chloride solution, and pH 8.0. They also conducted field sampling. As a result of elaborate testing and analysis, TCT reached the following conclusions (MPCA, 1990):

1) Metals are leached from tire materials and the constituents of concern are barium, cadmium, chromium, lead, selenium, and zinc.
2) Polynuclear Aromatic Hydrocarbons (PAHs) and Total Petroleum Hydrocarbons are leached from tire materials in highest concentrations under basic conditions.
3) Asphalt may leach higher concentrations of contaminants of concern than tire materials under the same conditions.
4) Drinking Water Recommended Allowable Limits (RALs) may be exceeded under "worst-case" conditions for certain parameters.
5) Co-disposal limits, EP Toxicity limits, and TCLP criteria are generally not exceeded for the parameters of concern.
6) Potential environmental impacts from the use of waste tires can be minimized by placement of tire materials only in the unsaturated zone of the subgrade.

Properties of Tire Chips for Lightweight Fill

Humphrey, et al. (1992 and 1993) have reported the engineering properties of 3-inch size tire chips from three suppliers. Their tests showed that the tire chips are composed of uniformly graded gravel sized particles that absorb only a small amount of water. Their compacted density is 38.6 to 40.1 pcf. The shear strength was measured in a large scale direct shear apparatus. The reported friction angle and cohesion intercept ranged from 19° to 25° and 1.11 to 1.67 psi, respectively. Their compressibility tests showed that tire chips are highly compressible on initial loading but that the compressibility on subsequent loading/unloading cycles is less. The measured horizontal stress indicated that the coefficient of lateral earth pressure at rest varied from 0.26 for tire chips with a large amount of steel belt exposed at the cut edges to 0.47 for tire chips obtained from glass belted tires.

A major concern in using tire chips in embankments are the large settlements the observed in various field (about 10 to 11 in.) and laboratory studies (e.g., Geisler, et al., 1989; Edil, et al., 1990; Lamb, 1992; and Read, et al., 1991). Holtz (1989) comments that no research in the literature discusses settlements of highway embankments. NCHRP (1971) has reported that post-construction settlements during the economic life of a roadway of as much as 1 to 2 ft are generally considered tolerable provided they: 1) are reasonable uniform; 2) do not occur adjacent to a pile-supported structure; and 3) occur slowly over a long period of time. Post-construction settlements of shredded tire embankments can be reduced by: placing a thick soil cap over tires fills, i.e., by increasing confining pressure; and using a rubber-soil mix instead of tire chips alone. The detrimental effects of anticipated excessive settlements can be reduced by using tires under flexible pavements only and letting the tire chips compress under traffic before placing the final surface course.

Another concern in using tires in embankments may be the potentially combustible nature of tires. To reduce the possibility of fire, a protective earth cover may be placed on the top and side slopes of tire embankments. A similar soil cover is recommended for some other lightweight materials, like wood chips, sawdust, slags, ashes, expanded clay or shale, etc. for protection against fire or to prevent leaching of undesirable materials into groundwater. During construction, caution is required to avoid any fires in stockpiled tires or embankment tires that have not yet been capped.

Compacted tire chips (about 2x2 in. nominal size) have permeability values equivalent to typical values for coarse gravel (Bressette, 1984). This property of chips renders them suitable for use in subdrainage as an alternate permeable aggregate. As a highly permeable material, pore pressure developments are prevented in tire fills and backfills. Use of tire chips in alternate layers with non-select fills, like clays, silty clays, etc., will provide a shorter drainage path and thus help accelerate consolidation of the layer.

The use of shredded tires in embankments offers the potential benefit of disposing of large volumes of tires in short sections of highway. For example, the use of an asphalt-rubber pavement overlay utilizes only about 3600 tires per miles of 2-lane road while a mile of 2-lane embankment 20-feet high would utilize about 5 million tires (one tire equals approximately one cubic foot loose bulk density before compaction; Read, et al., 1991).

Present study

A comprehensive work plan based on laboratory testing and evaluations assesses the feasibility of using shredded tires in highway embankments as a lightweight fill. The study primarily focused on determining compaction characteristics, stress-strain-strength behavior and hydraulic properties of compacted rubber-soils (see Table 2). In addition, the study briefly analyzes the environmental impacts of this application of waste tires. The findings of this study provide compressibility and strength parameters for design of embankments incorporating tire chips and for the prediction of post-construction performance and evaluation.

Two types of soils, one from the fine and one from the coarse grained family of soils, were selected and prepared for testing purposes: Ottawa sand - classified as poorly graded sand (SP) according to the Unified Soil Classification System (USCS) and A-3(0) as per AASHTO and Crosby till - classified as sandy silty clay (CL-ML) as per USCS and A-4(0) according to AASHTO.

Shredded tire samples of different sizes and gradations were procured from various tire processing agencies. A 6-inch diameter triaxial cell, a 12-inch diameter compaction/ compression mold, an 8-inch diameter constant head permeameter, and related accessories were designed and custom-made/modified for static and dynamic testing of compacted rubber-soils specimens to determine stress-strain-strength behavior and hydraulic characteristics of rubber-soil mixes. The MTS soil testing system, with loading frame suitably modified to accommodate large size compressibility and shear apparatus, was used to simulate static and dynamic field loading conditions. Necessary modifications were made in the hardware and software of the MTS System to impose the required loading conditions and acquire the data automatically during the testing.

The test data were analyzed and presented in the form of tables/plots (Imtiaz, 1992). Correlations were developed for use in design and performance evaluations of embankments containing tire chips.

Compaction Behavior

During this phase of the research, the testing program was formulated to develop quantitative information about the compaction characteristics of rubber soils and chips alone. The variables considered included: compaction methods, compactive efforts, tire chip sizes, chip/soil ratios, and size of compaction mold. The compaction tests were conducted following methods described in ASTM specifications D 698 (AASHTO: T99-61), D 1557 (AASHTO: T180-61) and D4253. Three different compactive efforts were used, i.e., impact energy equivalent to modified Proctor, standard Proctor, and 50% of standard Proctor method. Tire chips of seven different sizes ranging from sieve No. 4 to 2 inches have been investigated. The soil/chip ratios were varied from pure soil to pure chips (i.e., quantity of rubber chips was varied from 0 to 100% of dry weight of mix).

The following conclusions are drawn, based on a critical analyses of the results obtained from the compaction testing of rubber chips alone and rubber-soils. Vibratory methods of compaction are suitable for rubber-sand. Non-vibratory methods (e.g., Proctor type compaction) are more appropriate for compacting mixes of chips and fine grained soils. Although, a mold six times the maximum size of chips is considered adequate for conducting compaction tests on rubber-soils, it has been found that the size of the mold affects the maximum density of rubber-soils. The small size molds (4 to 6 in.) may yield densities which may be about 10 to 15% lower than those obtained with larger molds (8 to 12 in.) for the same size of chips. The effect of compactive effort on the resulting density of rubber-soils decreases with increasing chip/soil ratios. Only a small effect is observed for an amount of chips greater than 20% of dry weight of mix. Similarly, the density of chips alone is also not much affected by the compactive effort. Only a modest compactive effort is required to achieve the maximum density of chips.

This density is about one third that of conventional soil fills. Density of rubber-soils decreases with increasing chip/soil ratios and the relationship between density versus percent chips is almost linear. Chip density is not very sensitive to the size of chips. However, a trend of increasing density with increasing chip size is found, except in the case of the vibratory compaction method. In this case the maximum density decreases with increasing chip sizes.

Compressibility

Compressibility tests were conducted on tire chips, alone and also mixed with soils, to determine the load-deformation behavior of rubber-soils. A 12-inch diameter compressibility mold was designed and built for testing large size tire chips. The variables considered included: vibratory and impact compaction; compactive efforts - equivalent to modified, standard, 50% of standard, and no compaction; and chip sizes - varying from 0.50 to 2-inch. The samples were subjected to 3 or 4 load/unload cycles to determine the behavior of rubber-soils under repeated loads.

The data obtained were plotted as vertical strain versus logarithm of vertical stress. Based on a critical analysis of the compressibility test results, the following observations were made; the load-deformation response of tire chips indicates that three mechanisms are mainly responsible for total compression of tire chip samples: a) rearrangement and sliding of chips - small compression, occurs mainly during the first cycle, mostly irrecoverable; b) bending and flattening of chips - major portion of compression, mostly recoverable on unloading; and c) elastic deformation of chips - very small compression at high stresses ($\approx$ 20 psi or more), completely recoverable. This indicates that compression of rubber chips can be reduced by increasing confining pressures or filling voids with less compressible material.

The variation in chip sizes had little effect on load-deformation response for higher compactive efforts, i.e., equivalent to modified and standard Proctor tests. However, a trend of higher vertical strains was observed in the case of 0.5-inch chips, compacted using 50% of standard compactive effort.

The increase in compactive effort from standard to modified Proctor had no effect on the compression curves for various chip sizes. However, samples compacted using 50% of standard Proctor effort yielded vertical strains 2% to 4% higher during the first loading cycle than those compacted with standard or modified effort. The uncompacted samples also produced higher strains during the first loading cycle. However, compactive effort had little effect on the load-deformation response of chips during subsequent load/unload cycles.

The curves from rubber-soils with varying chip/mix ratios show that the total compression of samples increases with increasing percent of tire chips, the highest value of compression being for 100% chips. This demonstrates that a blend of rubber-soil provides a mix with lower void ratio, which compresses less than one of pure chips, and will also cause lesser settlement of foundation soil due to reduced weight of fill. About 38% chips by weight of mix is an optimum value for the quantity of chips in a rubber-soil mix, where large settlements are a matter of concern. This chip/soil ratio will yield a compacted dry unit weight of rubber-soil mix which is about two thirds that of soil alone.

Shear Behavior

A number of triaxial compression tests were performed on rubber-soils for determining the feasibility of using tire chips in highway embankments as lightweight geomaterials. Tire chips of different sizes and gradations and two types of soils were tested in the laboratory using a 6-inch diameter triaxial apparatus. The variables considered included: type of soils, methods of sample preparation, size of chips, ratios of tire chips/mix, and confining pressures.

The following salient conclusions are drawn concerning shear behavior of rubber-soils based on a critical evaluation of the test results: unlike soils, the samples of tire chips do not fail by yielding or have a single shear plane, instead the samples exhibit a strain hardening behavior and continuously become stiffer with increased axial straining. The chip specimens at low confining pressures demonstrate symmetrical bulging.

Specimens sheared at high confining pressures compress vertically, with little lateral spreading, and continue to become stiffer even at large strains, when the capacity of the apparatus is reached. Confining pressure is the most important factor affecting the strength of chips. Size of chips and compactive efforts do not significantly affect the shear behavior of tire chips. However, a trend of increasing deviatoric stresses with increase in chip sizes and compactive effort is observed.

The shear behavior of rubber-sand is mainly affected by the level of confining pressures and chip/mix ratios. Higher confining pressures yield higher strength values. The rubber-sand samples exhibit behavior similar to that of sand alone at low chip/soil ratios and similar to that of chips alone at high chip/soil ratios. Compactive effort and chip size has little effect on shear behavior of rubber-sand. Tire chips have a reinforcing effect on rubber-sand mixes. The increase in strength is a maximum at a chip/soil ratio of about 39% at low to medium confining pressures (between 4 and 20 psi), which is considered an optimum ratio for rubber-sands for use as lightweight geomaterials.

Similar to rubber-sands, the rubber-Crosby mixes exhibit stress-strain behavior similar to Crosby till at low chip/mix ratios and like chips alone at higher chip/soil ratios (more than 20% chips by the dry weight of mix). Unlike rubber-sand, tire chips do not have an appreciable reinforcing effect on the behavior of Crosby till. The inclusion of tire chips in Crosby till reduces the deviatoric stress values compared to those of soil alone at corresponding strain levels and increases the strain at failure. The compactive effort and size of chips do not significantly affect the stress-strain behavior of rubber-Crosby.

The failure criterion for rubber-soils needs to be based on allowable strains instead of peak or yielding strength. Strength parameters for rubber-soils for 5, 10, and 15% strain levels have been determined and summarized in Table 2 for tire chips, rubber-sand, and rubber-Crosby, respectively.

Resilient Modulus Testing

A laboratory testing program has been conducted on tire chips of variable sizes and two test soils to determine the resilient characteristics of rubber-soils. The chip and rubber-Crosby samples were prepared using impact, Proctor type compaction and the rubber-sand samples by vibratory method of compaction. The resilient modulus tests were performed on rubber-soil samples with chip/mix ratios varying from 0 to 100%. The following salient conclusions are drawn concerning resilient modulus of rubber-soils: the resilient modulus of soils decreases with increase in chip/mix ratios; this reduction in modulus (greater for rubber-Crosby than rubber-sand) is stress dependent and is substantial, up to 80% or even greater depending on the chip/mix ratios and the state of stress.

Material	Dry Density (pcf)	Strength Intercept c (psi)	ϕ (°)	Hydraulic Conductivity (cm/s)	Compressibility	Reference
Poorly graded clean sand, sand-gravel mix (SP)	100-120	-	37	10-100	-	Hunt (1986)
Medium sand, angular: Loose Dense	- -	- -	32-34 44-46	10^{-1}-10^{-2}	-	Leonards (1962)
Inorganic silt and clay, compacted (CL-ML)	100-120	9.4	32	10^{-4}-10^{-6}	-	Peck et al. (1974)
2-in. tire chips in 6-in. diam. triaxial compression test	38	3.75	21	6-15	-	Bressette (1984)
1-in. chips, Std. Proctor Strain = 10% Strain = 15% Strain = 20%	40	 3.2 4.0 4.8	 14.6 20.3 25.3	0.6	 CR=0.169 RR=0.139 SR=0.108	Authors
Ottawa Sand 1-in. chips 39%, sand 61% Strain = 5% Strain = 10% Strain = 15%	115 89	- 7.3 6.3 5.2	41.4 25.5 34.6 38.1	1.6×10^{-4} 8.7×10^{-3}	CR=0.005 RR=0.003 SR=0.003 CR=0.029 RR=0.019 SR=0.021	Authors
Crosby till, strain = 10% 1-in. chips 40%, till 60% Strain = 5% Strain = 10% Strain = 15%	119 81	10.7 5.6 8.2 9.4	29.4 12.7 21.1 27.0	8.8×10^{-7} 8.8×10^{-3}	CR=0.030 RR=0.007 SR=0.003 CR=0.096 RR=0.056 SR=0.045	Authors

Notes: 1. Compressibility parameters were determined for samples prepared using Standard Proctor compactive effort.
CR = Compression ratio, average slope of ϵ_v vs $\log(\sigma_v)$ in virgin compression zone (σ_v between 4 and 10 psi)
RR = Recompression ratio, average slope of recompression part of curve (σ_v between 4 and 10 psi)
SR = Swelling ratio, slope of rebound of curve (σ_v between 10 and 1 psi)

Table 2 Engineering properties of conventional fills and rubber soils

The resilient modulus data from rubber-soils show a significant scatter, greater than that observed for conventional soils.

Chip size has no effect on the resilient characteristics of rubber-soils. The values of resilient modulus for rubber-soils improve under increased confining pressures. This implies that the use of properly confined rubber-soil fill as lightweight geomaterial is viable. A 3-ft. layer of soil/aggregate as an overburden pressure is considered adequate.

Permeability

Permeability tests have been conducted on tire chips, alone and also mixed with soils, using a large size custom-made constant head permeameter. The tests followed ASTM D-2437 procedures.

The results indicate that the coefficient of permeability for 1-inch size tire chips varies from 0.54 cm/sec to 0.65 cm/sec with compactive effort increasing from the equivalent of modified Proctor to 50% of standard Proctor.

The permeability testing of rubber-soils demonstrate that the coefficient of permeability increases with increase in chip/soil ratios. The permeability of rubber-sand increases from 1.642×10^{4} cm/sec for sand with no chips to 8.696×10^{3} cm/sec for sand with 37.7% chips by weight of mix. However, the rate of increase in permeability values is higher for rubber-Crosby mixes than for rubber-sand with increasing chip/mix ratios. The value of k increases from 8.863×10^{7} cm/sec for Crosby till with no chips to 8.824×10^{3} cm/sec for 40% 1-inch chips by dry weight of mix. This is an increase of about four orders of magnitude, compared to only one order of magnitude increase in k for rubber-sand for similar increase in rubber/mix ratios. The values of permeability for tire chips are equivalent to the typical values for coarse mineral aggregate. These high values of permeability qualify tire chips as a useful fill material in embankments and in drainage layers of leachate collection systems for landfills, temporary/final covers of old landfills, and in drainage layers of pavement systems. However, the use of tire chips in drainage layers of landfill cover and pavements may be deferred until the long-term impacts of leachates from tires is known.

Economic Implications

The cost of using shredded tires in embankments depend on a number of factors that vary with the local conditions, including: cost of chips (primary shreds are generally available free at the source in most of the states at this point in time); distance of shredding facilities from the site and the cost of transportation; cost of placement and compaction; incentives offered by the state in the form of subsidies/rebates, etc.; and the cost of conventional mineral/lightweight aggregates. In Indiana, the major vendor of shredded tire materials is located in East Chicago. Currently, they are willing to provide primary tire shreds without cost. Transportation costs in Indiana vary from $5 to $10/ton for a distance of 100 miles. The exact economic benefits can be determined on a case-by-case basis.

Recommendations

It is evident that the waste tire problem in the United States is of great magnitude and has far reaching environmental and economic implications. It is found, based on a critical analysis of the available options for reuse, recycling, and disposal of scrap tires, that no single option can solve this problem.

The engineering properties of rubber-soils determined as part of this research, namely: index properties, compactibility, compressibility, shear strength, resilient modulus, and permeability, suggest that rubber-soils show significant promise for use in highway embankments. It is found that the use of shredded tires in highway construction offers technical, economic, and environmental benefits under certain conditions. The salient benefits of using tire chips are: reduced weight of fill, increased stability, smaller settlements, and correction or prevention of slides; good drainage medium that avoids the development of pore pressures during loading; reduced backfill pressures on retaining structures; separation to prevent underlying weak/problem soils from mixing with subgrade/base materials; conservation of energy and natural resources; and usage of large quantities of local waste tires, which has a very positive impact on the environment.

There are some potential problems associated with the use of shredded tires in highway embankments, which include: long term impacts of leachates from tires on environments; fire risk; and large compressibility of tire chips. Proper soil cover is required on top and side slopes of shredded tire embankments for safety against fire. During construction, normal caution is required to be observed against fire in stockpiled tires or in embankment tires that have not yet been capped with soil. Potentially large settlements can be reduced by providing a thicker soil cap and using rubber-soil mixtures instead of chips alone. Detrimental effects of post-construction settlements can be reduced by using tires under flexible pavements only and letting the chips settle under traffic for some time before laying a final surface course.

Rubber-soils with chip/mix ratios of 38% (subsequently referred to as the optimum ratio) or less should be used in embankments where large settlements are unacceptable, i.e. near bridge abutments. Rubber-sand at optimum chip/mix ratio possesses excellent engineering properties, including: easy compaction and low dry density; low compressibility; high strength; and excellent drainage characteristics. The free draining characteristics of rubber-sand also reduces the possibility of undesirable leachates from tires, since water does not stagnate in fills. The availability of tire chips in abundance, low cost at the source, and the positive impact of using large quantities of tires are added benefits of using rubber-sand in highway embankments. In summary, tire chips and rubber-sand mixes offers very promising technical, environmental,and economic benefits and their use in highway embankments, above groundwater table, should be promoted.

The use of rubber-Crosby mixes in embankments offer some technical benefits, like low dry density (density at 40% chip/mix ratio is about two thirds that of conventional fill), and good hydraulic characteristics. However, this material has high compressibility, low shear strength, and is difficult to mix/compact in the field. The choice of using a mix of tire chips and fine grained soils should be studied on a case-by-case basis, depending upon site conditions, type and availability of borrow material, and desired engineering characteristics like strength, compressibility, density, permeability, etc.

It is recommended, based on significantly lower values of resilient modulus of rubber-soils compared to conventional subgrade soils, that tire chips or mixtures with soils be used in the subgrade portion of highway embankments and not within three ft. of asphalt pavement surface, to avoid subjecting asphalt pavements to fatigue stresses, which lay cause larger deflections of the pavement surface under repeated traffic loads and thus affects the service life of the pavement.

Further Research on Rubber-Soils

A comprehensive laboratory study that assesses the feasibility of using rubber-soils in loaded and unloaded backfills, in slope stabilization situations and reinforced soil applications of tire chips and rubber-soils in combination with geosynthetics is currently in progress. These study includes: determination of lateral pressure coefficients for rubber-soil fills and computer simulations for the use of rubber soils in embankments on soft soils and wall backfills.

A field study which includes the construction of a test embankment, with adequate monitoring devices, to measure compressibility and leachates from fill has been proposed. The embankment will have three sections containing: 1) tire chips; 2) rubber-sand mix; and 3) chips mixed with locally available fine grained soil. This study will be very helpful in determining long-term performance and development of correlations between laboratory and field parameters.

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A CASE HISTORY OF THREE INFRASTRUCTURE STABILIZATIONS FOR THE ROADS AND HILLSIDES IN GREATER CINCINNATI

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Infrastructure is defined as the basic facilities, services, and installations needed for the functioning of a neighborhood, community or city. Infrastructure includes items such as transportation systems, communication systems, water, sewers, gas and electric services and extends to public institutions including schools, post offices and prisons. A drive through the streets of Greater Cincinnati makes it evident that there has been considerable deterioration of our streets and highways and that there is a need to repair and preserve these elements of the city's infrastructure.

As geotechnical engineers, our services are ever increasing with the demand to provide cost effective, buildable solutions to the complex construction problems associated with the rehabilitation and reconstruction of our infrastructure, especially our streets and highways along the hillsides of Cincinnati.

With the recent growth that Greater Cincinnati has experienced our communities infrastructure is spreading out and is much more utilized. The highways and streets that were constructed over fifty years ago no longer are able to service the increased demands placed upon them. At the same time, Cincinnati's diverse terrain with its rolling upland, hills, valleys, rivers and streams, continues to evolve, moving and creeping slowly over time. Another complicating factor in preserving the integrity of the Cincinnati area is the inherent instability of the hillsides formed on the Kope formation, a clay shale with thin interbeds of limestone. Slippage commonly occurs in the zone between the weathered and unweathered bedrock. Above the bedrock are the overburden clays, silts and some sands of glacial origin which may add to instability.

These unstable hillsides provide for some of our infrastructure's greatest engineering and construction challenges.

An earth retention structure or retaining wall is usually needed to meet the requirements of cutting into these hillsides or for filling over top of a hillside to increase the road size and stop the creep of the hillside. Geotechnical engineers and design/build contractors, therefore, are needed to evaluate, design and create solutions to these complex problems.

Three case histories are presented involving Richard Geottle Inc. Construction Company that illustrate innovative, practical and cost effective solutions to unique infrastructure earth-retention problems.

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Case #1 - The Daylight Building

The Daylight Building is located in a desirable part of Cincinnati that overlooks downtown. Since the office building had insufficient parking for its employees, the vacant lot on the north side of the building was analyzed for its potential use for additional parking.

In their Geotechnical Investigation of 1992, the H.C. Nutting Company described the proposed parking lot property as follows:

"The project site is bounded by Elsinor Avenue, Wareham Drive, and Van Meter Street, and is located along a moderately steep hillside on the west side of Mt. Adams. The project site is characterized by a history of landslides and instability. Around 1916 a landslide occurred below Elsinor Avenue. A retaining wall was constructed in the middle of the hillside in order to stabilize the slide. Further movement of the landslide occurred around 1940. In 1976 a landslide occurred along Wareham Drive. In 1978 the City of Cincinnati constructed a drilled pier wall along a portion of Wareham Drive, just above the proposed parking lot project site.

It must be recognized that this site is characterized by existing and past slope instability. This hillside has always had a low degree of stability . . . . This factor must be recognized and addressed in the planning and construction for the proposed parking lot development project. Unrestrained excavations have the potential to initiate further slope instability".

Richard Goettle, Inc. was initially contacted over ten years ago by the then present owner of the building to discuss possibilities of constructing a retaining wall in conjunction with the parking lot. Richard Goettle, Inc. proposed several options for a wall at the rear of the parking lot, cutting deep into the hillside. The owner at that time was unable to justify the added costs associated with such an undertaking.

The current owner of the building, Hixson, Inc., a well known Cincinnati engineering company, convinced the City of Cincinnati that they would shift their operations and large staff of engineers and architects from Blue Ash to this building within the city limits, provided that the city shared in the costs of construction for the needed parking lot.

The City of Cincinnati agreed and provided a grant to share in the construction costs for the retaining wall and parking lot. The City also agreed to maintain the retaining wall.

Richard Goettle, Inc. was awarded the contract to design and construct the retaining wall. Included in the City's contract, Richard Goettle, Inc. would warranty and guarantee the performance of the wall for five years.

Since the proposed retaining wall was well within the susceptible landslide zone, it was decided that the safest and surest way to construct the wall was to utilize a drilled pier with soldier pile reinforcing and permanent tieback-anchor system. The potential for landslide was minimized as the piles were socketed into rock below the lowest excavation for the parking lot as were the tieback anchors which were socketed into rock well beyond the slip plane failure arc. Richard Goettle, Inc. provided the design for the wall in conjunction with the H.C. Nutting Company's recommendations. H.C. Nutting provided the geotechnical testing, inspection, and monitoring services on the project. Hixson, Inc. provided the facing wall details and overall coordination for the project.

The wall measured 475' in length and rose to a maximum height of 22'. A trapezoidal shaped lateral-earth backwall pressure of 40 times the wall height was used for design in conjunction with the H.C. Nutting design recommendations. This loading was used to calculate tieback anchor loads and for sizing of the structural elements of the wall.

The wall design utilized drilled piers on approximately eight-foot centers. In each drilled pier was placed an HP 12x53 soldier beam. Each pier was filled with lean concrete to fill the void and provide corrosion protection for the steel beam. One row of tieback anchors was installed at an elevation nine feet below the top of the wall. All tiebacks were tested to 133% of their design load of 120 kips. Six tieback anchors were performance an creep tested.

The space between the piers was temporarily protected with wood lagging boards as the excavation progressed. Once the excavation was complete, a permanent facing wall was poured in front of the soldier piles. The wall was secured to the piles through a series of studs along the length of the pile. Positive drainage was accomplished by the use of a four-foot wide drainage mat between each pair of soldier piles and sandwiched between the free draining wood lagging and the concrete facing wall. Weep holes to a collection pipe in front of and below the base of the wall prevented the build-up of hydrostatic pressure behind the wall.

The facing wall is aesthetically pleasing, consisting of rustification grooves and a white Thoroseal coating complete with lighting for evening. The final product looks like a conventional spread footing retaining wall, however, it was constructed at about 70% of the cost of a conventional retaining wall.

The construction for this wall was accomplished from the top down to insure that slippage would not occur during construction. A slight cut was made at the top of the wall which provided a bench for the crane mounted caisson drill which drilled, reinforced and concreted the piers into place. Excavation and wood lagging installation proceeded to approximately two feet below the tieback elevation. Tieback anchors were installed and tested to insure that they would hold the load. Lagging proceeded to the bottom of the wall. Once lagging was complete, the drainage material was installed. The facing wall was gang formed and poured full height in eighty foot sections.

Case #2 - Columbia Parkway

On August 14, 1990, Richard Goettle, Inc. provided a public bid to the Ohio Department of Transportation for a project officially titled "Improving Section HAM-50-26.20, U.S. Route 50 in the City of Cincinnati, Hamilton County, Ohio, in accordance with plans and specifications by grading, draining, paving with asphalt concrete on a bituminous aggregate base, and by constructing a retaining wall." Richard Goettle, Inc. was the low bidder on the project by approximately $2,000,000.00. However, the figure was at almost exactly the Department of Transportation's estimate and Richard Goettle, Inc. was awarded the project.

Columbia Parkway, a part of U.S. Route 50, is a highly traveled road during "Rush Hour" running east-west along the Ohio River connecting portions of the east side of Cincinnati directly to downtown Cincinnati. The road has narrow lanes and many curves, and is slowly creeping toward the river. Portions of Columbis Parkway are being renovated each year. Prior to this award Richard Goettle, Inc.

reconstructed a section of Columbia Parkway in 1976.

The newly awarded project was a section of Columbia Parkway, about 7,350 feet, extending from the intersection of Tusculum Avenue east to Beechmont Avenue. The work included slight realignment and widening of the road, lessening the severity of the curve at Deering and Whitworth Streets (Dead Man's Curve). To accomplish this task it was necessary to install a drilled pier retaining wall along the entire length of the river-side of the road and tieback and concrete face existing portions of the retaining wall along the north-side of the road.

The complete design for the retaining wall was provided by the Ohio Department of Transportation and their consultants. The downhill retaining wall consisted of a total of 979 caissons. The caisson diameters were 30 and 36 inches. The caissons were installed at either 7 or 7.5 feet on center. All caissons were reinforced with rebar cages consisting of #10 and #11 bars or combinations of #10 and #11 bars. Eight to twelve bars were tied and installed full length in each caisson. All caissons were filled with Class "S", 4500 psi, concrete. Caisson details, depending on the location, called for a minimum of five or seven feet embedment into unweathered gray shale. Caisson lengths ranged from fifteen feet to over forty feet.

Between the caissons, 6, 7, and 8 inch thick precast concrete lagging panels were stacked and installed, spanning the distance between shafts. Approximately 72,000 square feet of precast lagging was installed.

Each caisson, with the exception of the ends, required the installation of one or two tieback anchors socketed into the unweathered gray shale. Although, the anchor loads were provided, it was the contractors responsibility to choose the anchor size and determine the required anchor bond length to achieve the design load. A total of approximately 1,104 tieback anchors were installed for this project including tieing back the existing uphill walls.

Construction began in early October 1990 with the maintenance of traffic, shutting down a portion of the road. Construction of the downhill wall was performed by the top-down method to insure that no road slippage would occur during construction. Two lanes of the parkway were closed and traffic rerouted to the remaining lanes. Caissons were installed from the closed traffic lanes using rotary methods and crane mounted caisson drills hanging out over the hill. Once the caissons were installed, the area behind the caissons was excavated to the bottom of lagging depth, and lagging installed. Lagging depths ranged from a few feet to approximately twenty feet. Once the lagging was in place, free draining crushed stone backfill was placed behind the panels.

Tieback anchors were installed and tested by suspending the airtrack drill over the side of the hill using a crane or by constructing a platform from which to work. Once the downhill wall was constructed, the traffic was switched over and anchor installation into the existing walls on the uphill side of the road was undertaken. Of the tieback anchors installed, 526 had design loads less than 100 kips and 578 had design loads greater than 100 kips. Two anchor failure tests were performed for this project; twenty-one creep tests were performed; eighty-one performance tests; and 1,011 proof tests were also conducted. An extensive tieback anchor instrumentation and caisson monitoring pro gram was initiated for this project. Load cells, strain gages and inclinometers were installed at three locations along the wall.

Data from the instrumentation are still being collected as of this date.

Case # 3 - KY 8, Slide 1

In July 1992, Richard Goettle, Inc. was the successful low bidder for the design and construction of a "Permanent Rock Anchored Tieback Wall" with a pressure treated timber lagging face for the Kentucky Department of Transportation. The project was officially titled KY 8 Landslide Correction and Tieback Wall, Slide 1. Richard Goettle, Inc., was one of several pre-qualified contractors for design and construction of tieback walls allowed to bid for this work.

The project is located on the west side of Covington, Kentucky along State Route 8 which runs along the Ohio River. The project is readily visible looking to the right from the I-75 bridge when traveling southbound over the bridge.

According to the KYDOT geotechnical report, this section of KY 8 had experienced stability problems for many years. Their geotechnical branch had made a preliminary investigation in 1980 describing the landslide area as being 1,200 feet long and corresponding roughly to current Sta 9+00 through Sta 21+00. Two distinct failures were defined as "Slide 1" and "Slide 2". After a limited subsurface investigation, the Geotechnical Branch reported that the best long-term stability solution would be to shift the current roadway alignment. It was also reported that any form of correction would cost in excess of $1,000,000.

Another investigationn of the slide area was conducted in 1990 as portions of the roadwaybetween Sta 14+00 and Sta 20+00 (Slide 2) were near total collapse and were threatening to close the road to traffic. The area at Slide 1 showed indications of movement but did not appear to be severe or to interfere with traffic flow. The study resulted in the construction of a permanent tieback wall in the area of Slide 2 (Sta 10+00 to Sta 20+00).

The severity of the failure between Sta 10+00 and Sta 13+00 (Slide 1) increased during 1991 and prompted the district to conduct an additional investigation of the site. The investigation determined that a tieback wall structure would be the most economical solution to the problem. The portion of KY 8 affected by this landslide was about .5 miles west of the intersection of KY 8 and Interstate 75. Beginning near the intersection of Parkway Avenue, the side area extended approximately 460 feet eastward to the existing tieback wall (built to stabilize Slide 2).

The embankment slopes on the north side are generally steep. The river at the closest point is about 120 feet north of road's shoulder and is about 65 feet below the shoulder at normal low pool. The critical area for stability occurred between Sta 11+15 and 12+50. The embankment slope was steep from the shoulder to the toe at the river. This area was an old natural drainageway that had been filled in and the roadway embankment constructed over the deep fill. The subsurface investigation indicated the depth to rock to be the deepest in this area.

Richard Goettle, Inc. designed and sized all of the components of the wall including the permanent rock anchor tiebacks, the soldier piles, the walls, the pressure treated timber lagging, the wall drainage, the hardware andall other incidental items necessary to provide the complete permanent rock anchor tieback wall based on KYDOT recommendations. All components of the wall system were required to be sized using the rectangular lateral earth pressure load corresponding to 0.65(Ka) (γ) (H+S) where (γ) = 125 pcf; (Ka) = 0.49; (S) = 2 ft; and (H) = the wall height in feet. The coefficient of passive earth pressure (Kp) was established at 2.04.

A total of fifty-seven soldier piles consisting of a drilled shaft reinforced with a 14' H-pile or a pair of wide flange sections were installed for this project. Soldier pile shafts were spaced on approximately eight-foot centers. A 30" or 36" caisson was drilled and depending on the cut height, the shafts were reinforced with HP14x73 or a pair of W18x46, 55, or 60 section and filled with concrete. The exposed wall height ranged from 16 to 29 feet and overall caisson lengths ranged from 31 to 55 feet.

Spanning the distance between the soldier piles were face mounted, rough-cut, 6x6 treated southern pine No. 1 dense SR lagging boards.

Two rows of tieback rock anchors were required. Tieback anchor loads ranged from approximately 55 to 166 kips. Tieback anchors were secured to the soldier piles through the use of a wale or installed directly through the pair of beams.

Construction of the wall was performed from the existing Route 8 roadway from the top down. One half of the road was closed to traffic detouring cars through the residential side streets adjacent to the site. Existing trees and brush along the caisson alignment were removed and the downhill slope cleared and grubbed. Soldier piles were drilled into place. A small ramp or bench was excavated on the front side of the wall from which lagging was installed. Lagging and drainage material wasinstalled to approximately two feet below the top row tieback elevation. The top row of tieback anchors was installed.

The first five tieback anchors installed were performance tested. Ten percent of the remaining anchors on the project were performance tested. Two creep tests were performed on this project; one in the top row and one in the bottom row. Once all tiebacks were tested, the excavation, lagging and drainage material installation was continued to approximately two feet below the bottom row of tieback anchors. The bottom row of anchors was installed and tested. The excavation and lagging was brought to final grade. A riprap ditch and drainage pipe was installed in front of the wall. The guardrail was installed the road paved according to KYDOT specifications.

Conclusions

These three case histories demonstrate examples of owners, engineers, contractors and government agencies working together to provide economic solutions to difficult infrastructure rehabilitation and construction problems. Case #1 was not competitively bid, but was negotiated directly with the design/build specialty contractor. Richard Goettle, Inc. provided the building owner and the City of Cincinnati with a least cost, safe, and aesthetically pleasing parking solution so that the full potential of the building could be realized. Case #2 was totally designed by the owner and competitively bid, however, it serves to show that the contractor most familiar with the work and implementing the owners design can provide the "least cost" alternative. Case #3 demonstrates that a pre-qualified design build specialty contractor can provide the construction required to heal our region's ailing infrastructure. All three projects were completed on schedule and without a construction claim.

CASE #1 - The Daylight Building

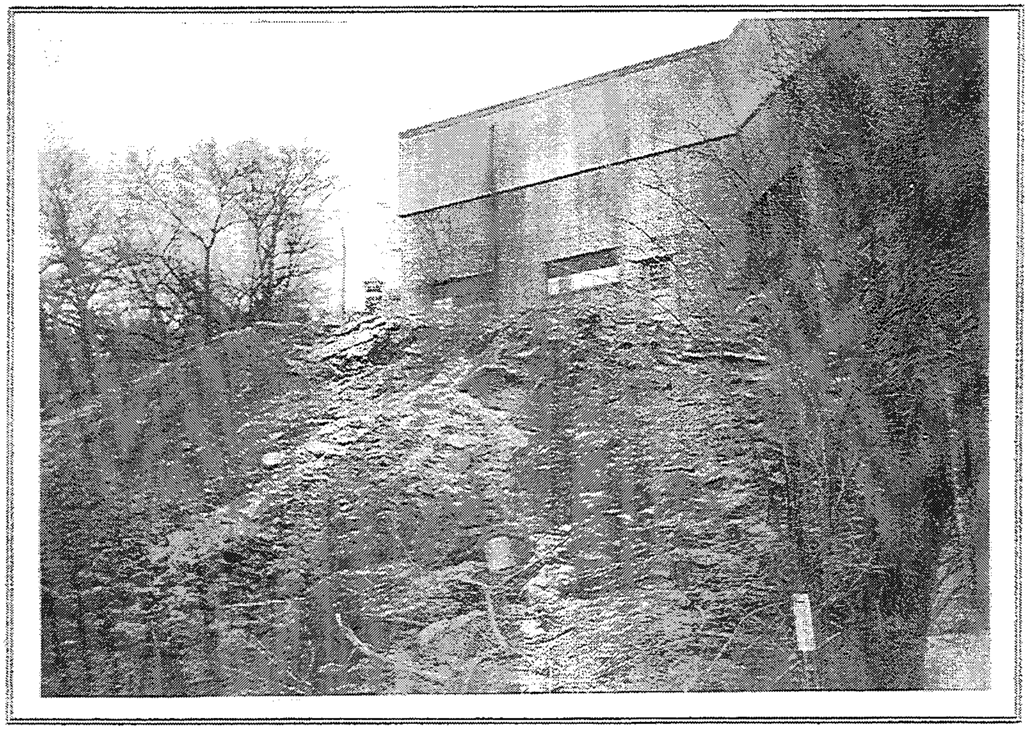

Figure 1.   Existing building hillside

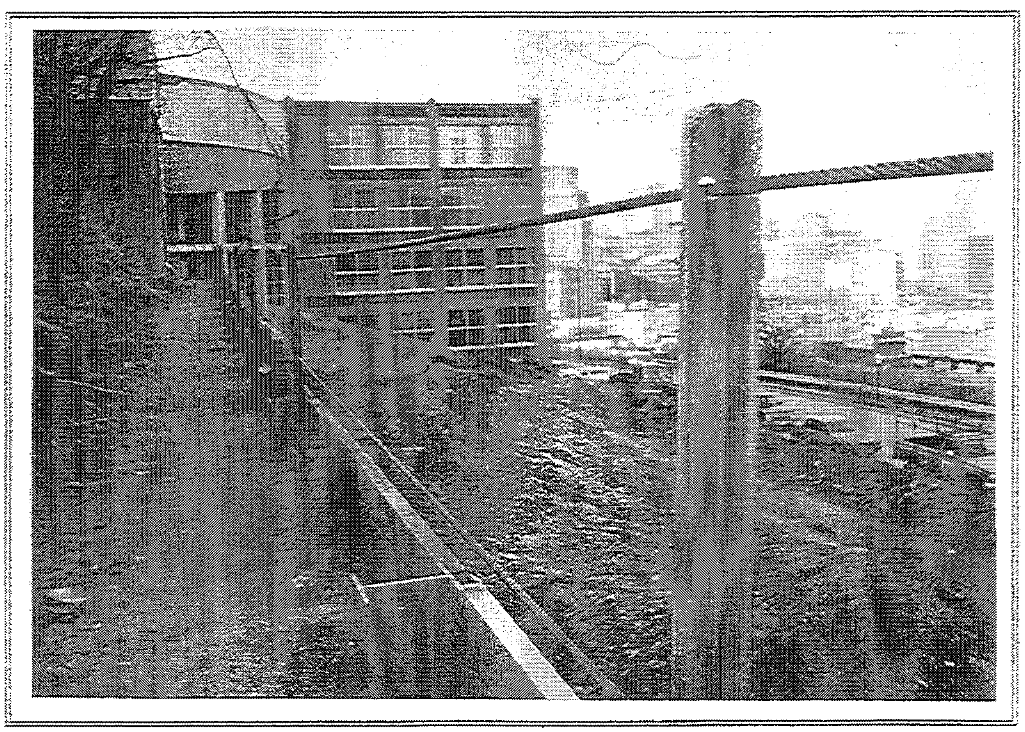

Figure 2.   View behind soldier piles

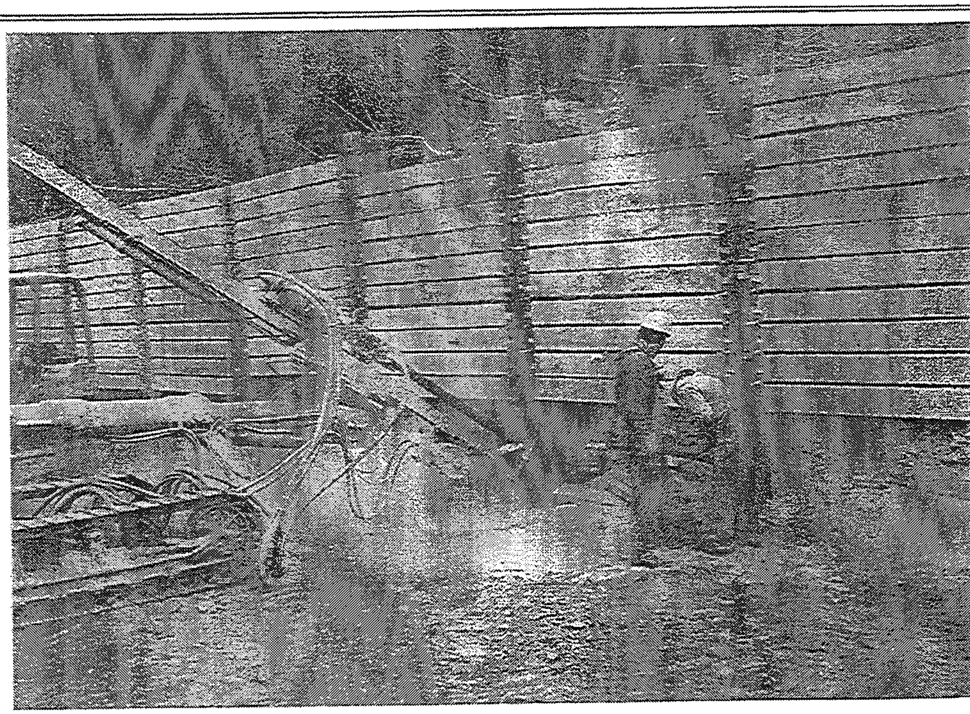

Figure 3. Installation of tieback anchor

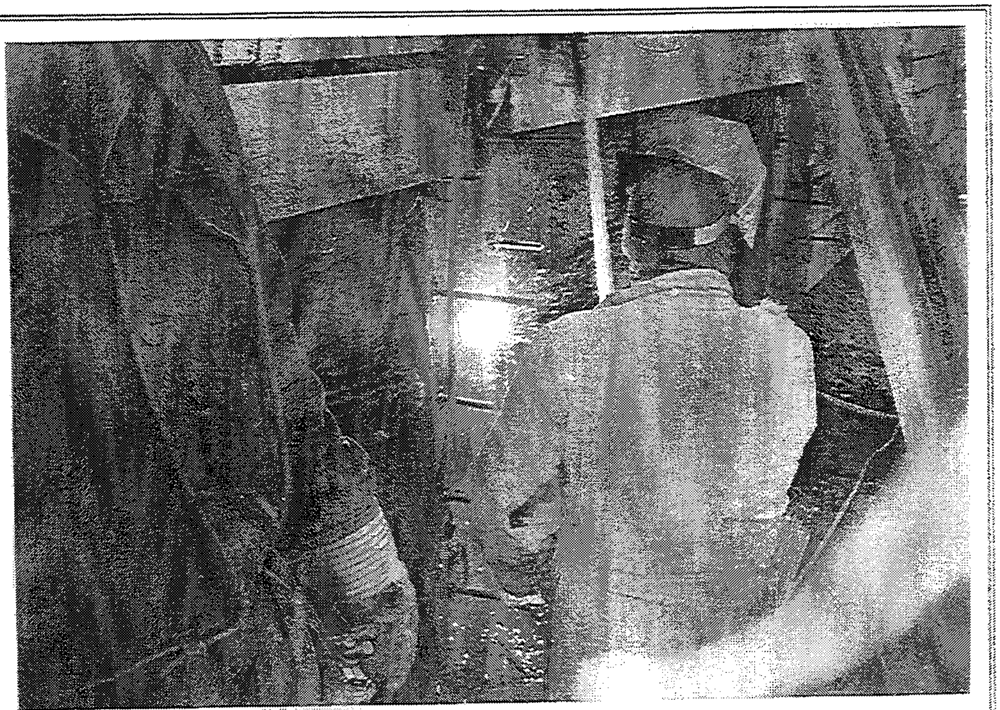

Figure 4. Installation of lagging studs

CASE #1 - The Daylight Building

Figure 5. Wall tied back and lagged to subgrade

Figure 6. Wall to subgrade

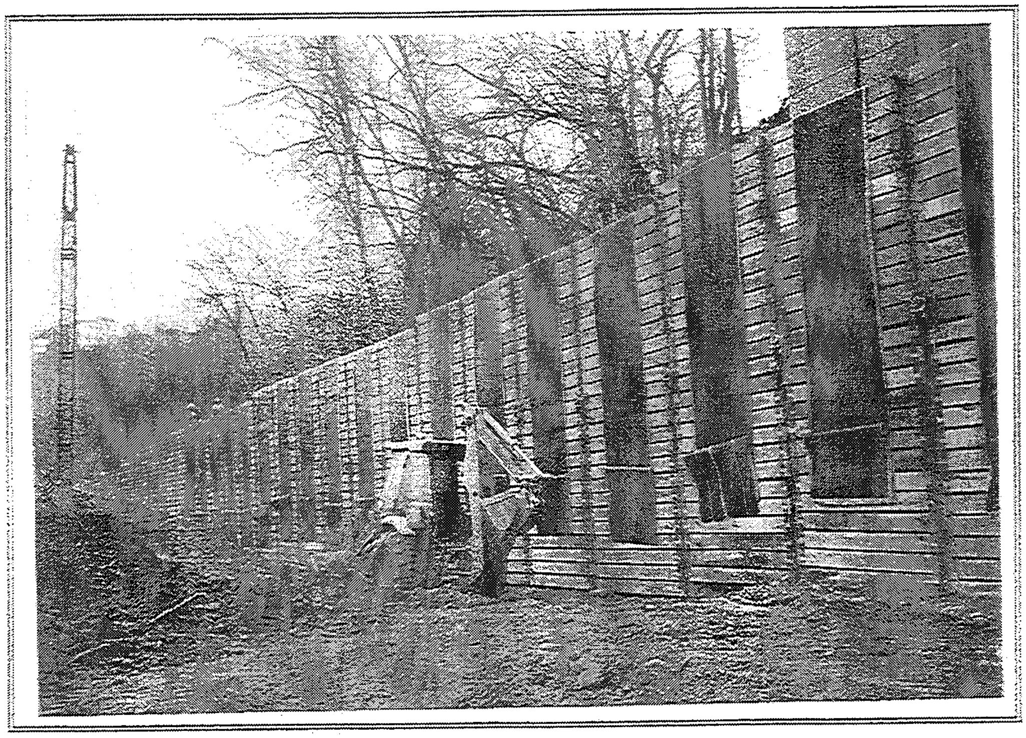

Figure 7. Placement of drainage material

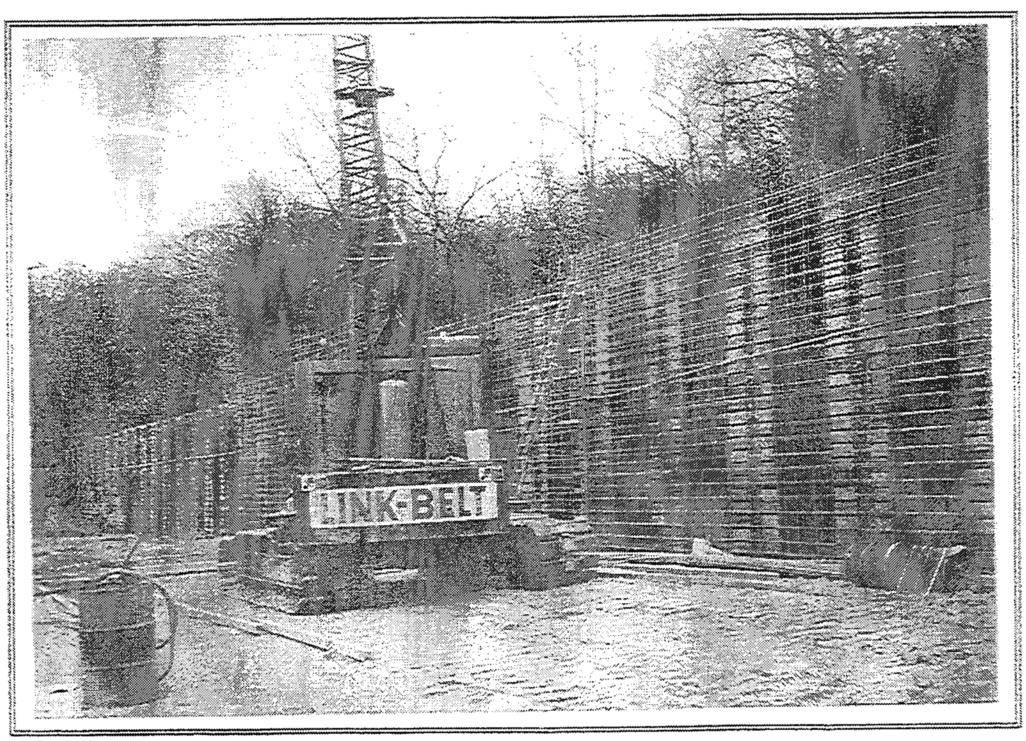

Figure 8. Epoxy coated reinforcing for facing wall

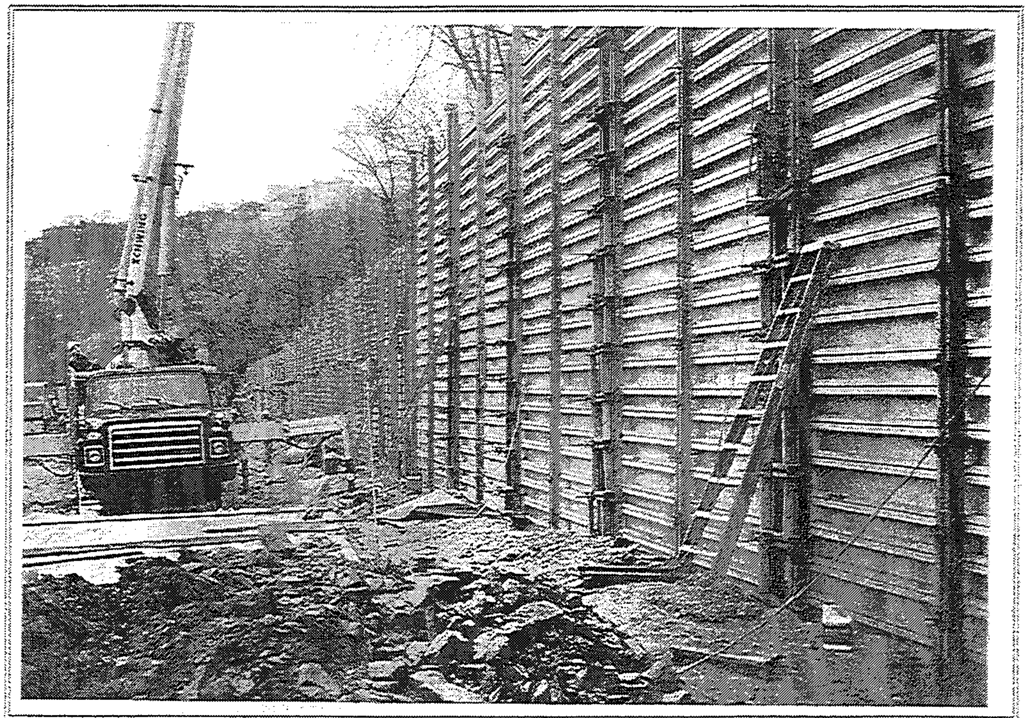

Figure 9. Facing wall gang forms

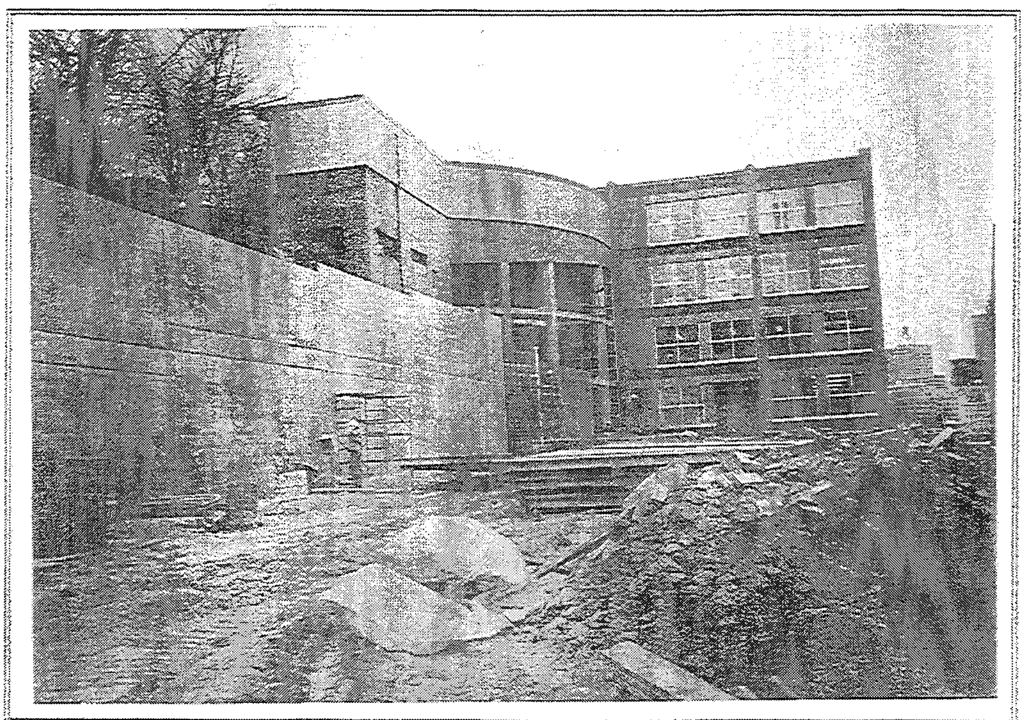

Figure 10. Facing wall

CASE #2 - Columbia Parkway

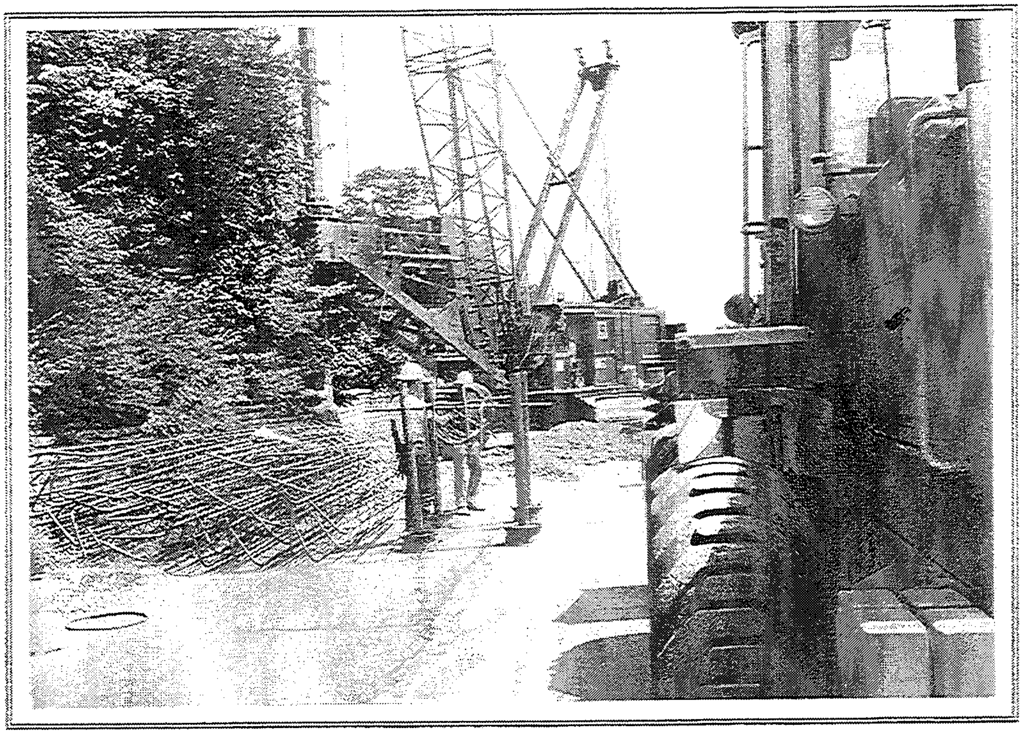

Figure 11. Crane mounted caisson drill and reinforcing cages

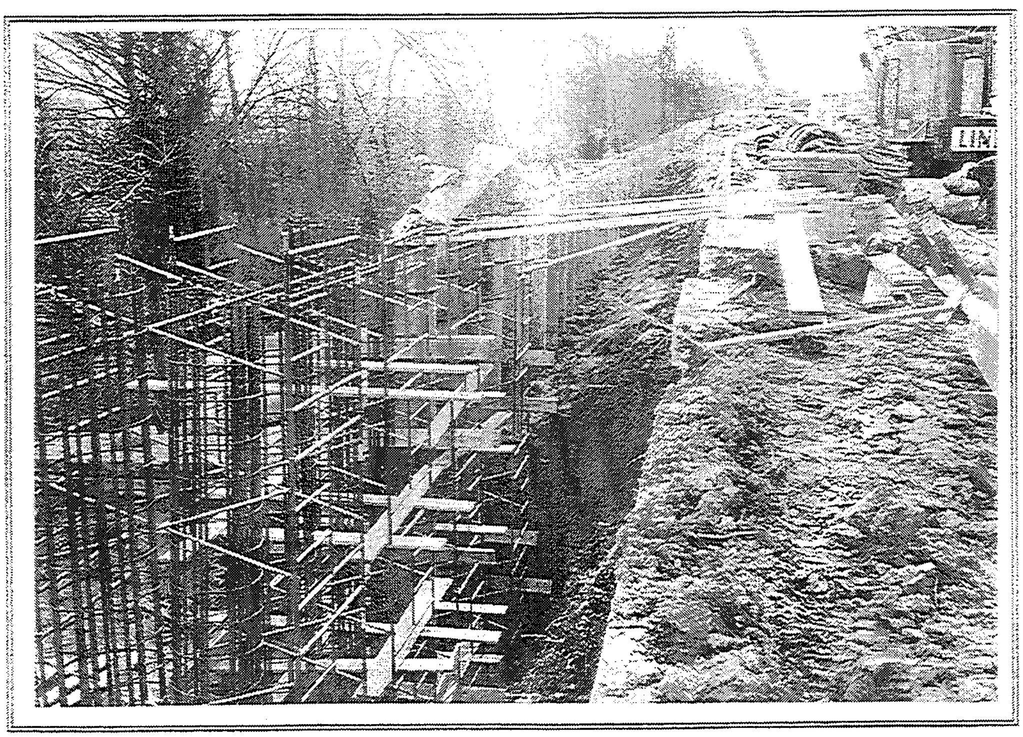

Figure 12. Forming the caisson tops

CASE #2 - Columbia Parkway

Figure 13. Excavation and backfiling

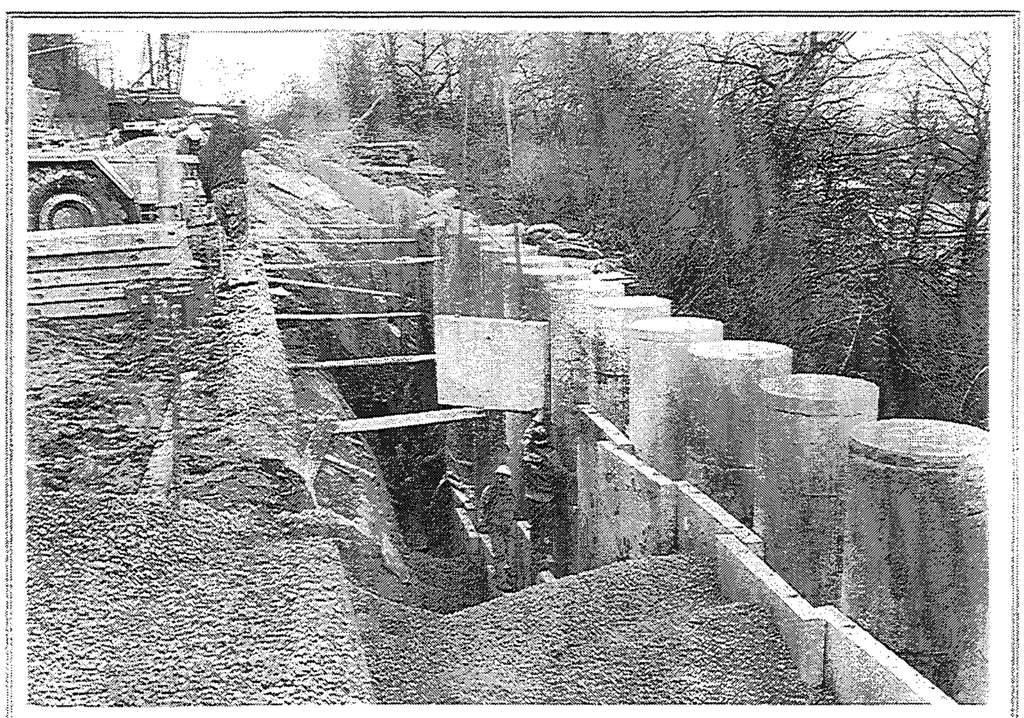

Figure 14. Placement of precast concrete lagging

CASE #2 - Columbia Parkway

Figure 15. Installation of tieback anchors

Figure 16. Installation of tieback anchors

CASE #2 - Columbia Parkway

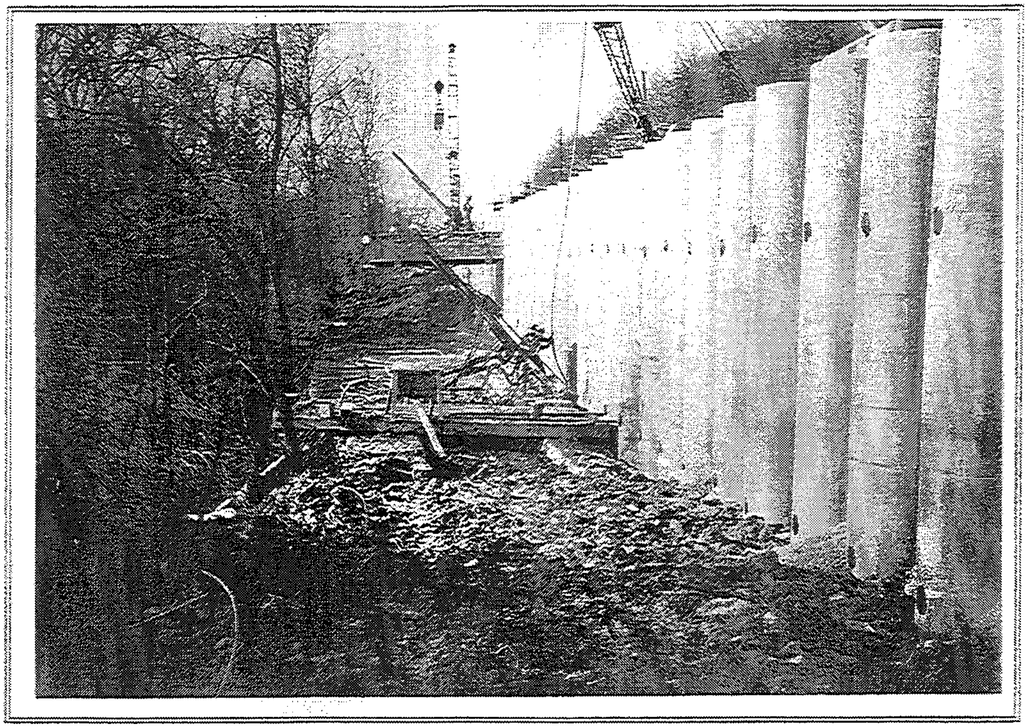

Figure 17. Installation of tieback anchors

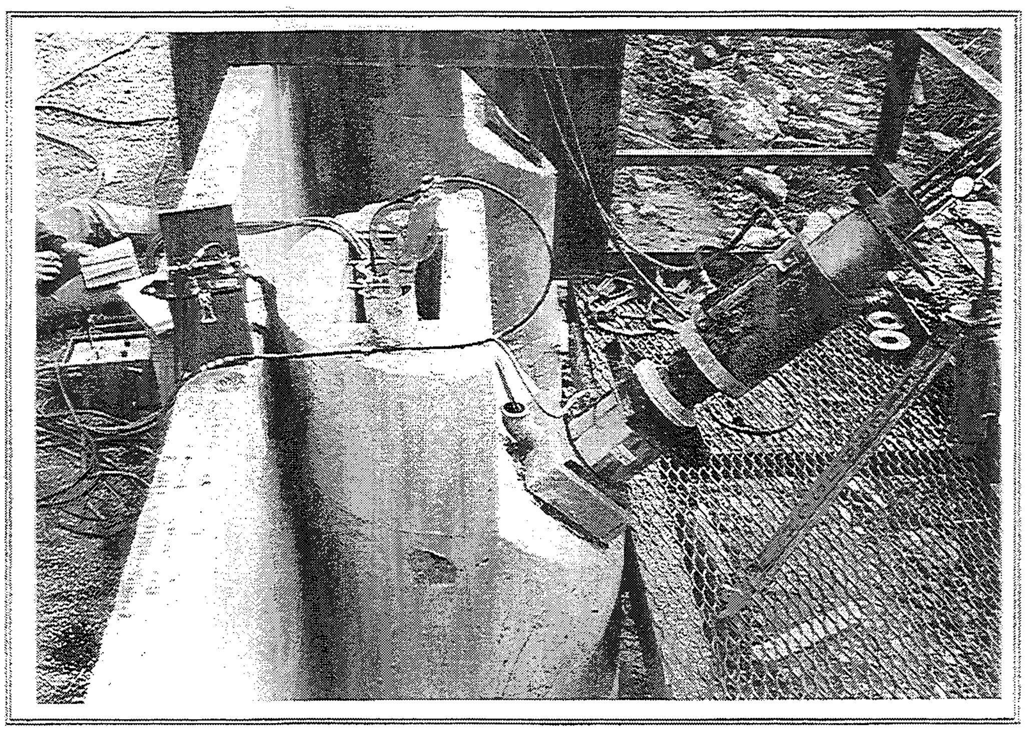

Figure 18. Instrumented anchor load testing

CASE #2 - Columbia Parkway

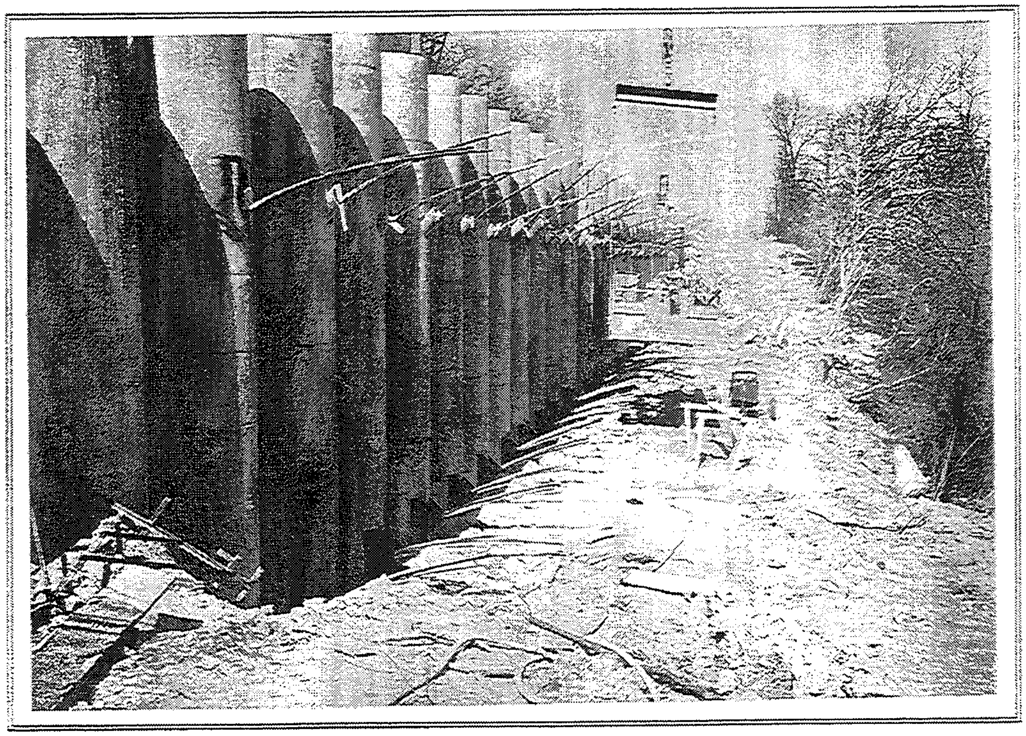

Figure 19. Downhill view of wall

Figure 20. Downhill view of wall

CASE #2 - Columbia Parkway

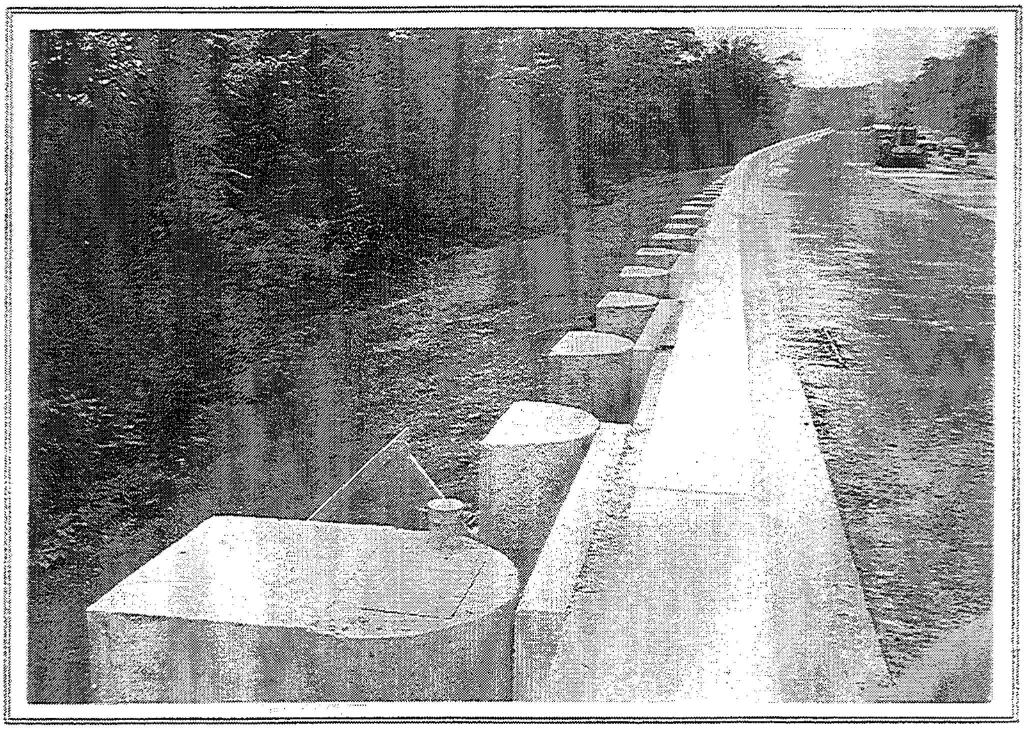

Figure 21. Roaway view of wall

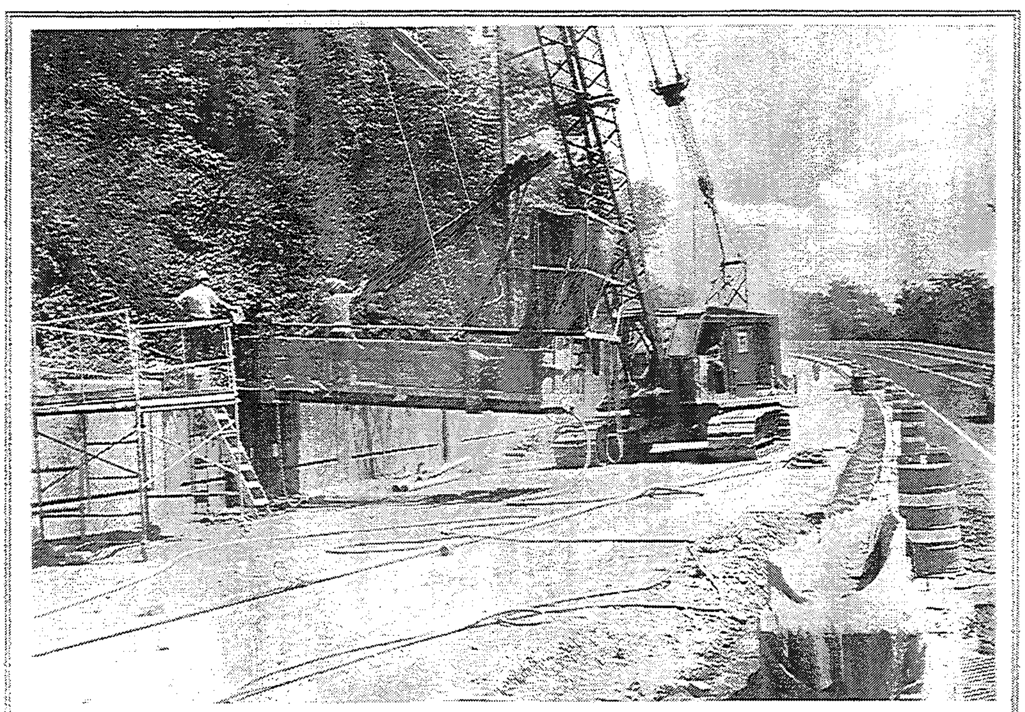

Figure 22. Installation of anchors in existing uphill wall

CASE #2 - Columbia Parkway

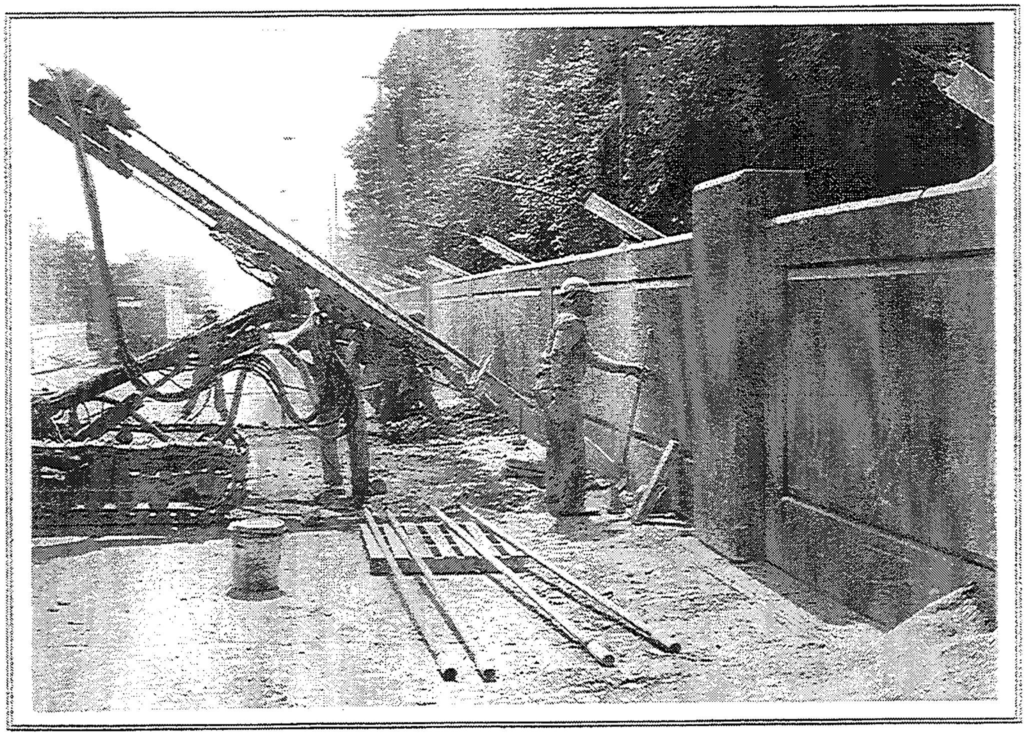

Figure 23. Installation of anchors in existing uphill wall

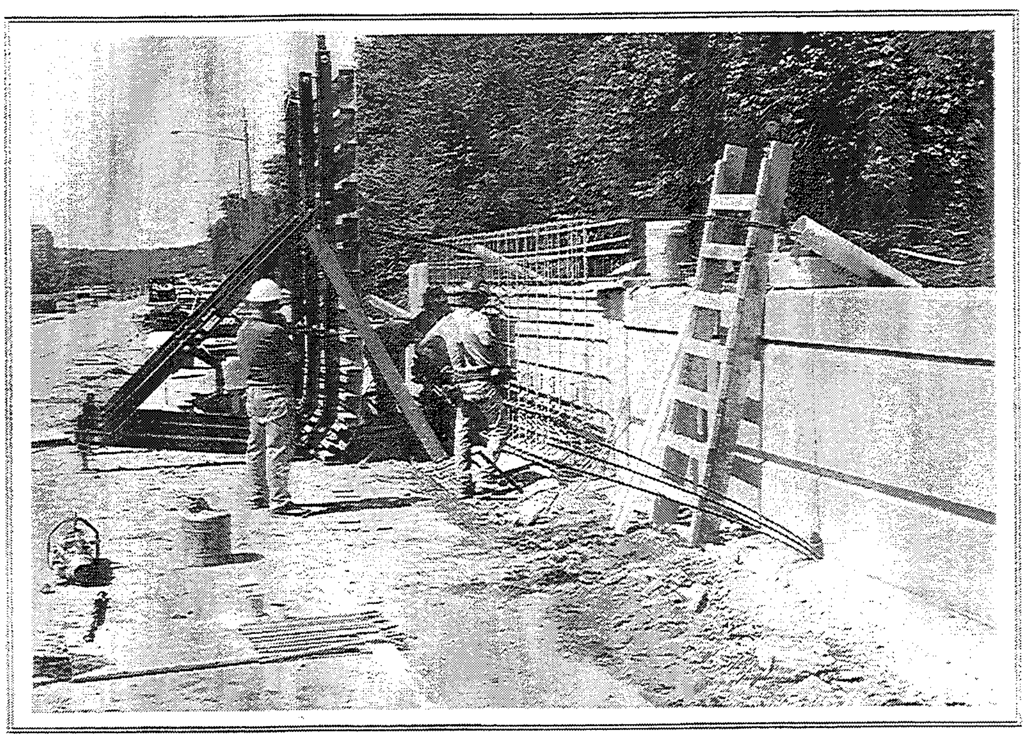

Figure 24. Installation of reinforced facing at uphill wall

CASE #3 - KY 8, Slide 1

Figure 25. Traffic lane used as working bench

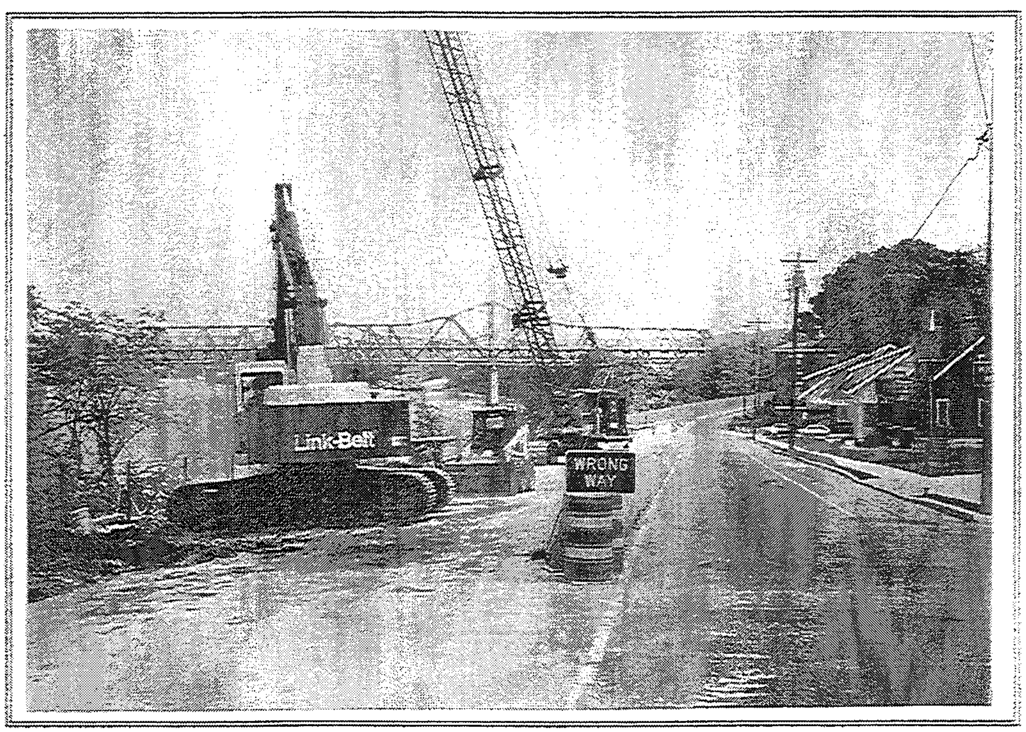

Figure 26. Traffic lane used as working bench

CASE #3 - KY 8, Slide 1

Figure 27. Installation of top row tieback anchors

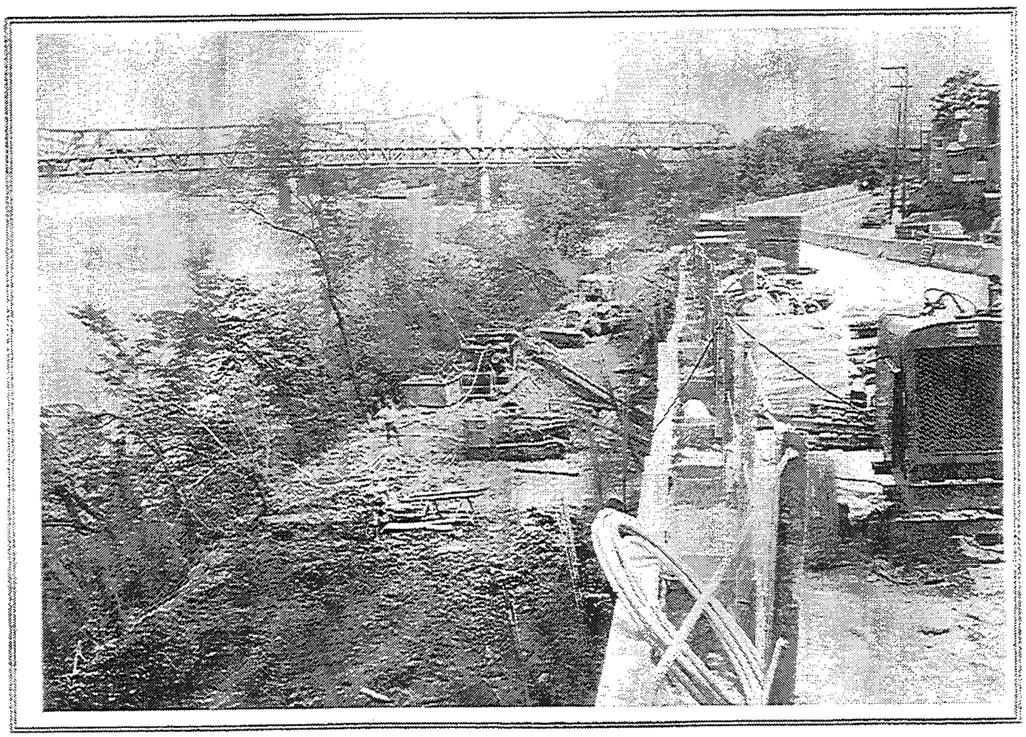

Figure 28. Excavation to subgrade bench

CASE #3 - KY 8, Slide 1

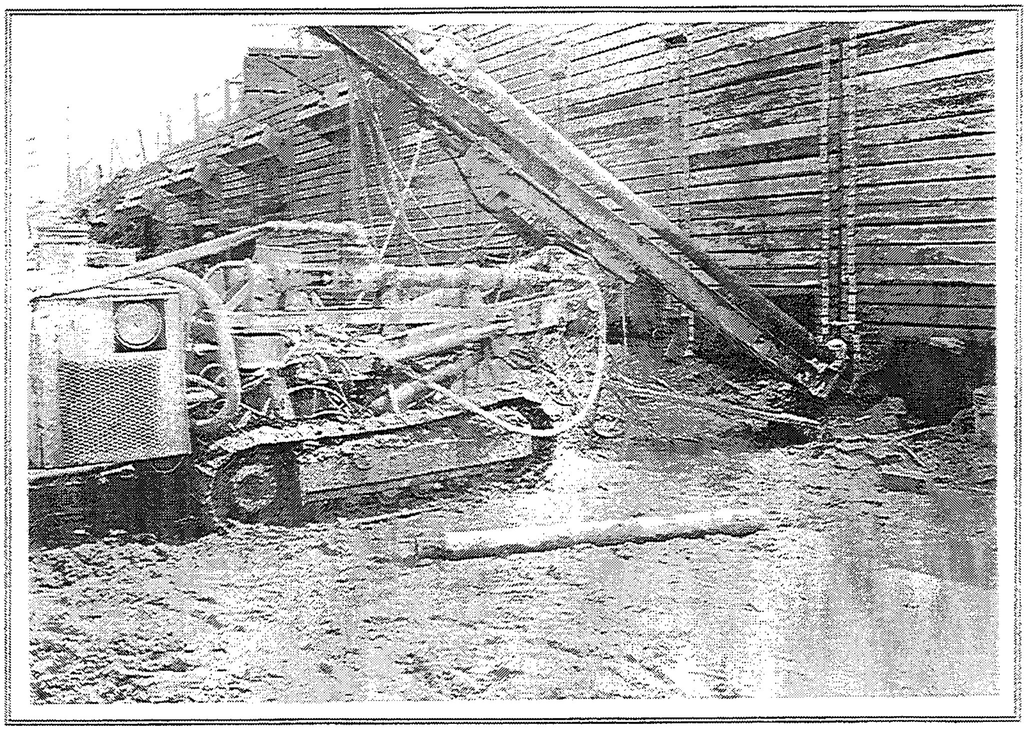

Figure 29. Installation of bottom row tieback anchors

Figure 30. Soldier beams, lagging, studs, and drainage material

CASE #3 - KY 8, Slide 1

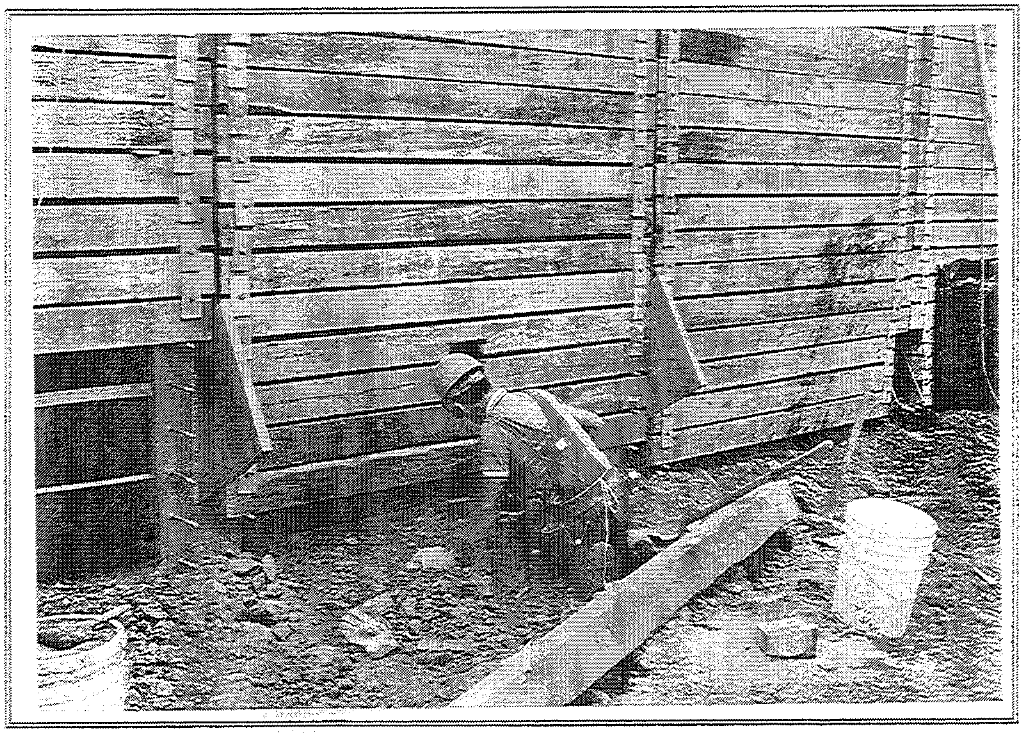

Figure 31. Wedge plates

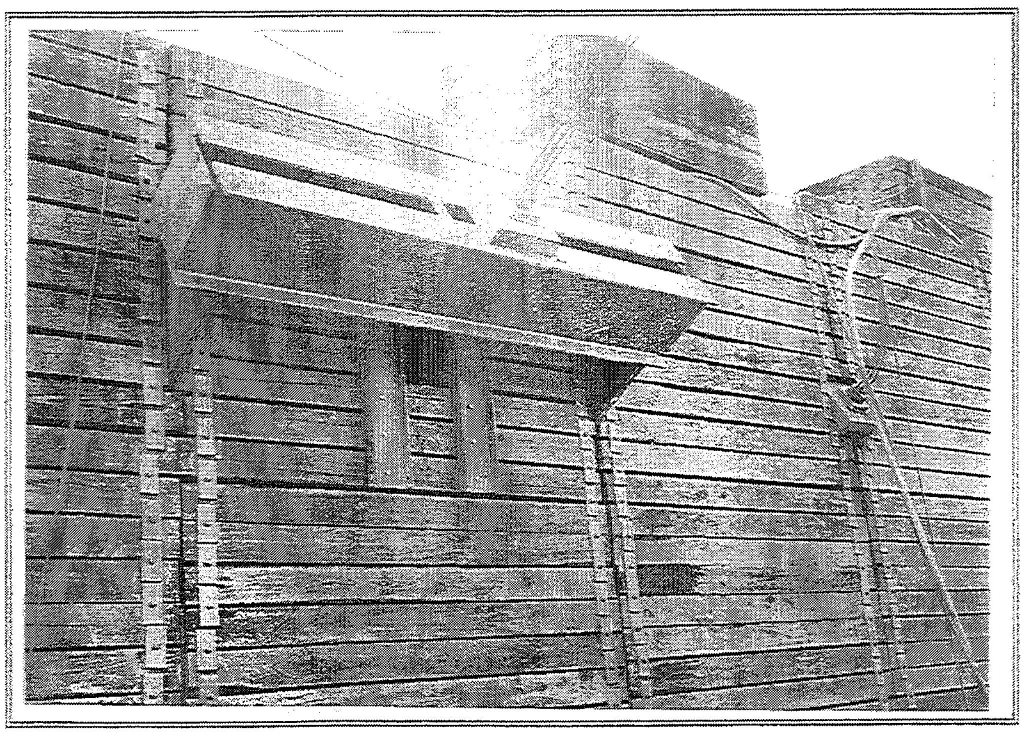

Figure 32. Tieback and wale

CASE #3 - KY 8, Slide 1

Figure 33. Downhill view of wall

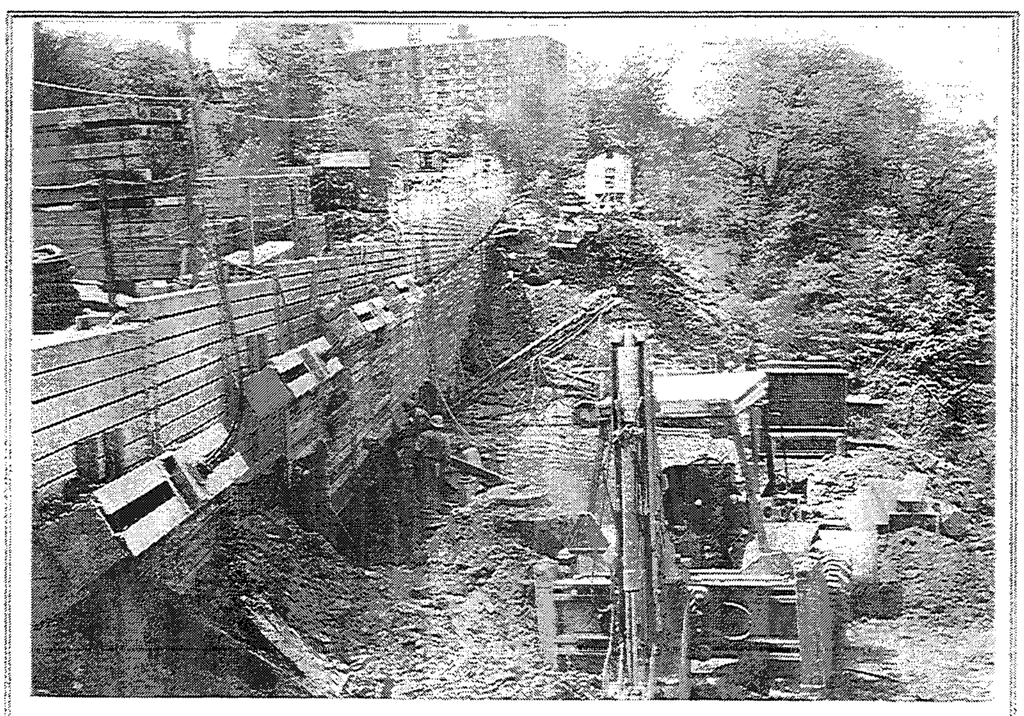

Figure 34. Downhill view of wall

GOETECHNICAL IMPACT ON THE DESIGN OF AN URBAN HIGHWAY RELOCATION

Paul G. Gruner[1]

ABSTRACT

The aging infrastructure of State Route 35 in Dayton, Ohio needed to be upgraded to accommodate increasing traffic flow. The best solution was to relocate it in an area that was tightly constrained. Therefore, all subsurface problem areas had to be handled by the design engineers through special construction techniques and mitigation measures. This paper addresses rock excavation and over-excavation, solid waste landfill excavation, over-excavation of poor soils (A-4b silts), replacement of rock with suitable materials, embankment construction and monitoring in lake areas, removal of lake bottom sediments, construction of bridge foundations in the lake, and construction of drilled shafts through the water.

Project History

The City of Dayton, Ohio, was feeling the strains of the nation's growing problems with aging infrastructure—particularly on one of its heavily traveled thoroughfares, State Route 35. The old route was quickly deteriorating and was no longer capable of handling the increasing load of traffic. With the assistance of Woolpert, the city identified that the best solution for relieving congested corridors and revitalizing neglected urban areas would be to construct the highway, the C.J. McLin, Jr. Parkway, on a new location.

But the design team found that although the solution would work, it required negotiating through some significant constraints: the Veterans Administration (VA) Medical Center, an important social and cultural fixture dating to the 1800s; the GM Inland Plant (Delco); McCalls Manufacturing Plant; Phillips Industries; Lakeside Lake, and a network of churches, residential areas, streets, and utilities.

Consequently, the first step toward designing the highway was conducting an Environmental Impact Statement—a process that was stopped abruptly by objections from the VA Center.

Eventually, the conflicting parties—the Center, the City of Dayton, the State of Ohio, and the Federal Highway Administration —signed a memorandum of understanding in which the Center agreed to the construction under certain conditions.

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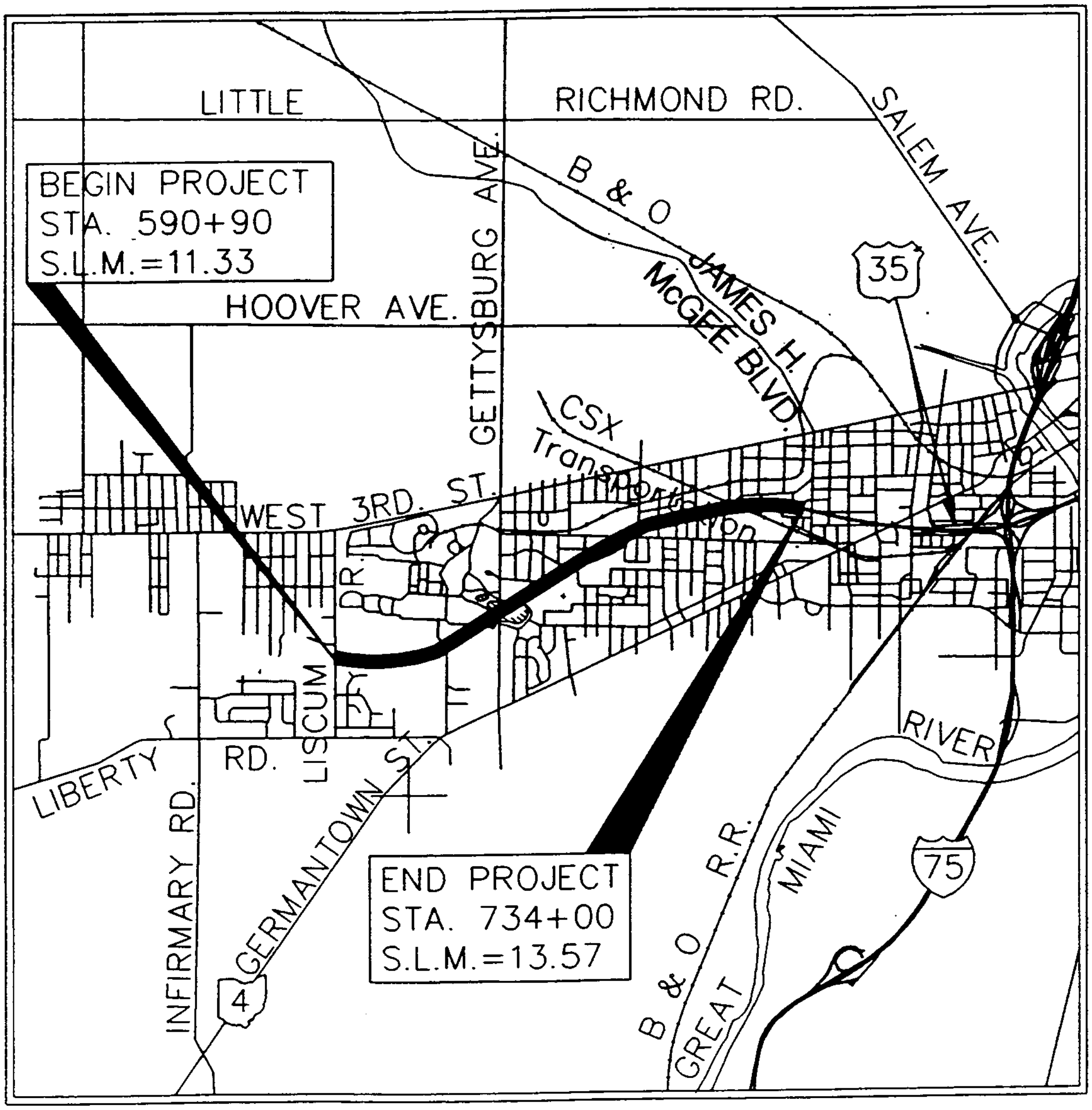
[1] Woolpert, 409 E. Monument, Dayton, Ohio 45402-1261



Location Map of US. 35, C.J. McLin, Jr. Parkway

One of these conditions was that the bridge on US 35 over Gettysburg Avenue and Lakeside Lake, which was between the Center's property and the stable and peaceful Pineview neighborhood, be designed with aesthetic consideration. Woolpert was hired to meet this challenge in 1989 with the $40 million first phase which included relocating US 35 and the bridge.

After months of concept studies, presentations, and meetings with the City of Dayton, the Ohio Department of Transportation, the Federal Highway Administration, and the Veterans Administration in Washington D.C., the design was developed and approved.

But additional challenges faced the team once subsurface problem areas were identified. Given the complicated set of circumstances and constraints, relocating was not an option. Instead, Woolpert design engineers used special construction techniques and mitigation measures. The H.C. Nutting Company was a design partner as a subconsultant providing geotechnical expertise.

Because of the subsurface conditions, the project became more complicated, eventually involving more than 1,000 drawings and 200 cross sections of subsurface detail.

The remainder of this paper will address the numerous subsurface difficulties and how they were handled in the design.

Rock Excavation and Over-Excavation

Bedrock of the Niagara Group —interbedded shale and limestone—underlaid by the Brassfield Group of the Silurian age, consisting mainly of fossiliferous limestone was found in several areas. Special cross sections were developed and excavation was recommended in several primary areas, including the Gettysburg Avenue Ramps A and D and ramp A and C near Western Avenue.

Over-Excavation of Poor Soils

The predominant soil in the upland areas was clayey sand silt, A-4a on the AASHTO Ohio Classification. Varying amounts of clayey silt, A-4b, was overlaid into this soil. A-4b is primarily the topsoil and was excavated in areas where it was unsuitable. Special cross sections were developed for these soils.

Replacement of Soils with Suitable Materials

Because limestone and shale are not appropriate for the top 2 feet of embankments, they were undercut 18 to 24 inches and backfilled with select embankment material. Soils other than topsoil and the fill materials in the landfill are suitable for use as subgrade materials. The best available fill material, clayey sandy silts (A-4a) was utilized for the areas requiring shallow fills.

A-4b silts found within 2 feet of subgrade were undercut and replaced with suitable materials.

Embankment Construction and Monitoring in Lake Areas

The construction of the east embankment for the Gettysburg Avenue and Lakeside Lake bridges partially extended into the lake, which was not drained prior to construction.

During construction, toe stakes are to be set and periodically monitored. On the centerline and edgelines of the embankment, settlement stakes were placed to monitor any settlements.

Removal of Lake Bottom Sediments

The west embankment for the Gettysburg Avenue and Lakeside Lake Bridges was partially constructed in an existing pond, which had to be drained and mucked of all soft lakebottom sediments.

In addition, between 2.5 and 5 feet of soft sediments had to removed from Lakeside Lake. Special cross-sections were developed for this area as well.

Solid Waste Landfill

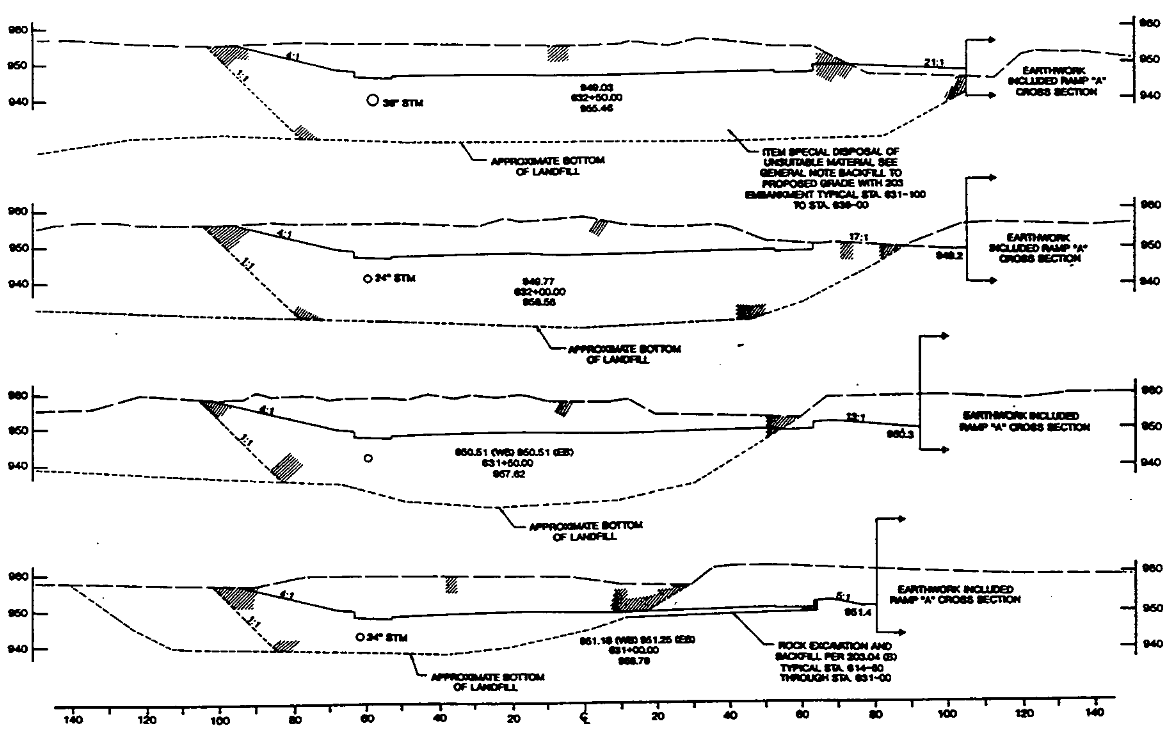
A closed, solid waste landfill lay in the path of the proposed highway. In addition to rock, the landfill contained cinders, wood, glass and rubber waste and an assortment of biodegradable debris ranging from paper to cloth.

The landfill issue was particularly challenging because of potential environmental concerns impacted the level and type of ground modifications required. Woolpert used Westinghouse Environmental and Geotechnical Services, Inc. to provide sediment and groundwater sampling in the landfill area. Results indicated there should be no problems with disposing of excavated soil and all toxins were at acceptable levels.

The fill material was underlaid by either shale and limestone bedrock or overburden soils consistent with those in other sections. Special cross sections were also required for this area. Trash within the entire project construction limits was completely removed and replaced with embankment.

STATION	LANDFILL AREAS		LANDFILL VOLUMES	
	CUT	FILL	CUT	FILL
631+00	2053	0	5250	0
631+50	3617	0	7284	0
632+00	4250	0	8312	0
632+50	4727	0	20846	0

STATION	ROCK AREAS		ROCK VOLUMES	
	CUT	FILL	CUT	FILL
630+50	179	0	231	0
631+00	71	0	231	0



Cross Section Drawing of Solid Waste Landfill

Construction of Bridge Foundations in Lake

Crushed rock was used to construct the portion of the embankment below water level. The granular fill was pushed into the lake to form a working plane on which the rest of the embankment could be constructed.

Construction of Drilled Shafts Through the Water

The piers for the bridges are founded on drilled shafts extending to rock which is 30 to 40 feet below the lake surface. The top 10 to 15 feet of these shafts are in the lake water and the remainder through silt sediments. In order to construct the drilled shaft, earth cofferdams were placed in the lake and then subsequently removed.

Conclusion

With careful attention to the subsurface issues, this project design was successfully completed. In fact, the project won the *Outstanding Engineering Achievement Award* of the Ohio Society of Professional Engineers.

The reasons for its success were skilled and experienced design engineers, effective negotiating between political entities and the community, and perhaps most important and effective and interactive relationship between the design engineers and the subsurface/soils engineers.

SOIL-STRUCTURE INTERACTION RELATIONSHIP FOR BURIED CONDUITS: MODULUS OF SOIL REACTION

Mark T. Bowers*

ABSTRACT

The author has conducted a number of studies for the Metropolitan Sewer District of Hamilton County, Ohio. These studies have centered on the rehabilitation of existing sewer lines via trenchless technology. A key parameter in the design of new liners for these conduits is the modulus of soil reaction. It has been determined that the modulus of soil reaction is not an inherent soil property but is in fact highly dependent on the soil-structure interaction. The most important properties of the soil-structure system have been found to be the degree of compaction of the backfill, the depth of cover, the span of the conduit, the soil modulus, the flexural rigidity of the conduit wall, Poisson's ratio for the backfill, the deflection of the conduit, and the unit weight of the backfill. The paper provides a review of the historical development of the modulus of soil reaction and examines values of the modulus for various soil types, depths of cover, and relative compaction of backfill.

INTRODUCTION

The author has been involved with the Metropolitan Sewer District of Hamilton County, Ohio for some years as a consultant on remedial actions for deteriorating sewers, be they replacements or relinings. The design work with relining materials has included epoxy-resins, other resin-impregnated felts, fiberglass, PVC, polyethylene, and other materials. The main question revolves about how thick the relining material has to be to carry the soil pressure and induced stresses from surface applied loads. A critical factor in the design equations is that of E', often called the modulus of soil reaction. A short review of the development of the modified Iowa formula for the analysis of flexible pipe is in order.

Spangler (1941) noted why flexible pipes perform well when buried despite their low stiffness compared to rigid pipes. He attributed this ability of flexible pipes to support vertical soil loads to two principles (Moser, 1990):

a) the redistribution of loads around the pipe, and

b) the passive pressures induced as the sides

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of the pipe move outward against the surrounding soil

Spangler incorporated the effects of the soil surrounding the pipe as they affected the pipe's deflection. His analysis led to the derivation of the Iowa formula as follows:

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 e r^4}$$

where ΔX = horizontal deflection or change in diameter, in.

D_L = deflection lag factor, usually taken as 1.0 if the prism load is used for design; otherwise 1.5

K = bedding constant (a function of the bedding angle, see Moser, 1990)

W_c = Marston load per unit length of pipe, lb/in.

r = mean radius of the pipe, in.

E = modulus of elasticity of the pipe material, psi

I = moment of inertia of the pipe wall per unit length, in^4/in.

e = modulus of passive resistance of side fill per unit length, lb/in^2 per in.

Watkins and Spangler (1958) examined the Iowa formula dimensionally and determined that the forementioned modulus of passive resistance of the sidefill (e) was not a true soil property as its dimensions are not those of modulus. Watkins defined a new soil parameter called E' or modulus of soil reaction as

$$E' = er$$

This called for a change in the Iowa formula as follows:

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3}$$

This formula is known as the modified Iowa formula. The question still remains as to what value should the modulus of soil reaction take.

As concerns infrastructure and the relining of sewers, a number of proprietary methods have been developed, each with its own programs based either on trench methods (the modified Iowa formula) or trenchless methods. In either case a value of E' must be considered in the programs made available to this author by various vendors. Moser (1990) states:

> The most useful method has involved the measurement of deflections of a buried pipe for which installation conditions are known, followed by a back calculation through the Iowa formula to determine the effective value of E'. This requires assumed values for the load, the bedding factor, and the deflection lag factor. Inconsistent assumptions have led to a variation in reported values of E'.

This author conducted a literature review concerning E' for the Metropolitan Sewer District (MSD) of Hamilton County, Ohio in May 1989. In that study the literature of the past 35 years was reviewed. Some 93 pertinent articles were found. A brief review of that history may be helpful and is presented in the following section.

HISTORICAL DEVELOPMENT: MODULUS OF SOIL REACTION

Spangler and Donovan (1957) indicated the usefulness of the Iowa deflection formula was hampered by a lack of knowledge concerning the modulus of passive resistance (e). It is remembered that the modulus of passive resistance was related by Watkins to the modulus of soil reaction E' by

$$E' = er$$

The Research Council on Pipeline Crossings of Railroads and Highways (1964) indicated that the Iowa formula "has been limited as a design tool because of inadequate means of determining E'." That Council presented a table for values of E' in units of psi as a function of soil density and conduit deflection (see Table 1).

Table 1: E' Values as a Function of Percent Soil Backfill Compaction Vs. Percent Pipe Deflection (after Research Council on Pipeline Crossings of Railroads and Highways, 1964)

Deflection of Conduit %	% Compaction (ASTM D698) Sand 100	Sand 90	Sand 80	Clay 100	Clay 90	Clay 80
1	4400	2800	1450	4020	2630	1240
2	3560	2090	1080	2500	1720	870
3	3150	1750	900	1860	1300	700
4	2900	1550	800	1520	1070	610
5	2700	1400	720	1300	920	540

This article discussed the development and use of an apparatus known as the "Modpares" device which was an acronym for <u>mod</u>ulus of <u>pa</u>ssive <u>res</u>istance. Watkins and Nielson, the principal writers of the article, admitted the following:

> ...there is insufficient evidence to establish a rational relationship between E' as determined on the Modpares device and the E' that must be used in the Iowa formula.

Furthermore, they reported that "before the Modpares method can be used to predict a soil modulus E', it must be correlated with actual field measurements." Krizek et al. (1971) indicate that to that date no such correlation had been made. To the knowledge of this author the correlation was never made as no further reference was made to the Modpares device in literature. Moser (1990) noted "many research efforts have been attempted to measure E' without success."

Nielson (1967) showed a relationship between pressure, deflection, the modulus of elasticity, and the modulus of soil reaction. He showed that the modulus of soil reaction could be determined from a triaxial shear test since E' is a function of Poisson's ratio and the modulus of elasticity. This is impractical for MSD's use in the remedial work of miles of deteriorated sewer.

Nielson et al. (1969) attempted to correlate the modulus of soil reaction to the results of CBR tests, Hveem's stabilometer test, and standard soil properties such as density, compaction, moisture content, and plasticity index. For example, they give the following equation using the CBR number as the basis:

$$E' = 312\ CBR$$

The following summary was presented by Nielson et al.:

> ...through a knowledge of the compacted dry density, a modulus of soil reaction can be predicted for use in conjunction with the Iowa formula to provide a reliable computation of the culvert deformation without recourse to either an arbitrary selection of E' or special soil testing. Only the dry density value...is required.

Table 2 is a summary of their recommendations. The AASHTO T180 compaction criterion was arbitrarily chosen. The authors indicate that because there is relatively little difference between the plotted curves of 3 and 5%, it seems reasonable to use an average value. They further caution:

> ...these values should not be used unless more accurate means of analysis are not available, and then only with the knowledge that error as much as 100 percent may exist.

Table 2: Suggested Design Values of Modulus of Soil Reaction as a Function of Percent Compaction (T180) (modified from Nielson et al. 1969)

% Compaction (T180)	E' (psi) @ 3% deflection	E' (psi) @ 5% deflection
75	500	400
80	1550	1300
85	2050	1800
90	2500	2300
95	5500	5000
98	15000	14000

Soil types were not specified for this table. Nielson et al. found little correlation of E' to water content or the plasticity index.

Krizek et al. (1971) prepared a state-of-the-art volume covering previous and on-going research, design procedures, construction practices, and field performance related to both rigid and flexible pipe culverts. They emphasized the following:

> ...a great deal of engineering judgment is involved in applying the Marston-Spangler and ring compression theories as a basis for designing buried conduits.

This statement reflects the strongly empirical nature of the design procedures developed by Marston, Spangler, Watkins, and White and Layer. The paper gives correlations to the Modpares device, relations to the Hveem stabilometer test (which was correlated to the Modpares device values), correlations to CBR values, relations to vertical pressure through various dry densities, and a relationship between percent compaction and the modulus of soil reaction.

Howard (1972) indicated that in its early use a constant E' value of 700 psi was advised for all pipe loads for all soils when the backfill was placed at 90% of its maximum dry density. He noted that "Spangler now [1972] recommends that E' values be selected based on experience and judgment." Howard conducted laboratory tests on flexible pipe using a lean clay as backfill. Using the soil pressure at the horizontal pipe diameter in conjunction with the horizontal deflection, E' for the 90% backfill tests averaged 544 psi and E' for the 100% backfill averaged 1208 psi.

Linger (1972) wrote an excellent history of the development of the soil-structure interaction problem. He noted that "...current design practice is based largely on work conducted in the 1920's and 1930's, and despite the success of these practices, they are empirical in nature and depend heavily on experience and engineering judgment."

Krizek and Kay (1972) wrote that "the characterization of the soil is probably the most important consideration in the soil-conduit system, and, more specifically, the modulus of the soil is probably the single most important parameter that affects the response of the system." It is further pointed out that "...there is considerable support for the position that one can make an engineering estimate that is as good as or better than any value that can be experimentally determined...."

Nielson (1972) pointed out that the pressure distribution on a conduit for a low-density soil is nearly uniform. This pressure distribution is adequate for low-density soils that have a high value of Poisson's ratio. However, for high density granular soils with a low value of Poisson's ratio, the pressure distributions appear to change somewhat. Spangler did most of his work on low density clay; his pressure distribution is essentially that obtained from elastic theory. Nielson notes:

> ...if the modulus of soil reaction E' is determined by using the calculated values from the theory of elasticity for pressure and deflection at the horizontal diameter, the difference in pressure distribution makes the Iowa formula predict too much deflection for soils with a high value of constrained modulus and low value of Poisson's ratio.
>
> ...because the modulus of soil reaction is directly associated with Spangler's pressure distribution, any question concerning the validity of the pressure distribution on the pipe will also be directly applicable to the modulus of soil reaction.

Nielson concluded that an average value of modulus of soil reaction could be written as

$$E' = 0.8\ M_s$$

where M_s is the constrained modulus of elasticity.

In a related work by Allgood and Takahashi (1972) it is noted that the constrained modulus of elasticity M_s "may vary from essentially zero for saturated clays to several hundred thousand for granular materials under high stresses." They caution that one needs to be aware of the variability in M_s. Even in replicate tests in a laboratory where care was taken to replicate placement of dry sand in a bin using the sand-fall method, "variation of M_s from the mean was +/-

20%." A greater variation must be expected in field installations.

Nielson and Statish (1972) presented E' values [the work of Hsieh, 1968] for soil mixes as a function of dry density as determined by AASHTO T180 and T99. This information is presented in Table 3.

Table 3: Modulus of Soil Reaction Values E' (psi) as a Function of Dry Density (modified from Nielson and Statish 1972)

Gradation	Dry Unit Weight (pcf)			E' @5% Deflection
	T180	T99	γ_{dry}	(psi)
Sand	118.2	112.3	112.3*	2105
			115.25**	14042
			95.5***	1474
Mix 1	134.5	132.1	132.1	14287
			133.3	15419
			112.3	1711
Mix 2	131.2	127.6	127.6	12313
			129.4	14389
			108.4	1866
Mix 3	132.5	128.0	128.0	16954
			130.25	24157
			108	1883
Mix 4	126.7	117.5	117.5	1828
			122.1	10994
			98.1	815
Mix 5	133.0	123.9	123.9	3776
			128.45	9735
			105.0	1212
Mix 6	130.5	121.8	121.8	3071
			126.15	5273
			103.3	984

Notes: T180 is equivalent to ASTM D1557
* = dry density for T99 (ASTM D698)
** = dry density for (T180 + T99)/2
*** = dry density for (0.85) of T99

The work of Parmalee and Corotis (1972) tends to negate the relationship between density and modulus which was developed by previous authors cited. Parmalee and Corotis noted the following:

> The Iowa deflection formula is based on the assumption that the supporting strength of buried corrugated metal pipe installations arises through the lateral reactive soil pressures induced at the sides of the pipe. Since its introduction more than 30 years ago, the Iowa deflection formula has served as a basic criterion for the design of buried flexible pipe systems. The formula is based on an assumed distribution of loading around the pipe, and contains three parameters that are empirical in nature [K, D_L, and E']. The present study examines the significance and possible variation of these parameters in the deflection equation. Limited field studies have indicated that realistic variations in these parameters can lead to an almost threefold change in design requirements. One of the most influential parameters in the formula is the modulus E'. Values of E' as determined from the measured response of full-scale installations exhibit a thirtyfold variation. Yet these data form the basis for establishing recommended values of E' for design use. Extrapolation of the observed deflections from field tests shows that in most cases use of the suggested design criteria yields unconservative fill heights. The present study shows no strong correlation between E' and percentage of standard Proctor density for the soil adjacent to the pipe, the pipe diameter, or the ratio of fill height to pipe diameter. A statistical analysis compares observed values of E' with common probability distributions, and a log-normal distribution is fitted. Probabilities associated with various ranges of E' confirm that there is no rational basis for recommending a design value that is much greater than the median, or central, value.

Spangler, in a discussion of this paper, indicated that he was "in complete agreement with the authors' conclusion that there is insufficient knowledge available at this time [1972] to enable a designer to select realistic and economical values of the needed parameters." He went on note that at the time the formula was originally developed, experimental evidence indicated the important influence of the modulus of soil reaction, but that the range of experiments was quite narrow. Since that early work, Spangler said, "applications of the equation to field settings had revealed a range in E' from 234 to 8000 psi." Spangler concluded that "a massive program of field measurements to establish a working body of data was in order."

Howard and Selander (1974) noted that "a particular soil at a given density gives a unique E' value for that soil regardless of the pipe diameter. The soil modulus E' has not yet [1974] been related to a laboratory test and must be considered a semiempirical factor that is based on experience and judgment."

Parmelee and Corotis (1974) reported that values of E' range from a maximum of approximately 11000 psi to a minimum of -500 psi. They indicated that extreme caution should be exercised in assigning values to E'. The thrust of the article is an evaluation of the approaches researchers have taken to evaluate E' through finite element models and other elastic analyses, correlations with laboratory tests, use of test cells, rearrangement of the Iowa deflection formula, and the monitoring of quantities which are simpler to measure. They conclude that the soil-structure interaction problem is a non-linear phenomenon and that the exact solution to the problem is only obtained through full-scale field installations.

Howard (1977) summarized seven years of research at the Bureau of Reclamation concerning the modulus of soil reaction. He presents a table (see Table 4 below) which considers soil type-pipe bedding material versus degree of compaction of the bedding. Data from over 100 field tests were used in developing the table. Limitations to the use of this table are clearly enumerated in his article.

ASTM D3839-79 "Underground Installation of Flexible Reinforced Thermosetting Resin Pipe and Reinforced Plastic Mortar Pipe" indicates that the modulus of soil reaction is a function of soil type, degree of compaction, and moisture content. Recommended values of E' for various soil and compaction conditions are given in the appendix to the standard. The values in the table are those suggested by Howard (1977).

Table 4: Bureau of Reclamation Values of E' for Iowa Formula (for initial flexible pipe deflection) (after Howard, 1977)

Soil type-pipe bedding material (Unified Classification System*)	E' for degree of compaction of bedding, lb/in²			
	Dumped	Slight, < 85% proctor, < 40% relative density	Moderate, 85%–95% proctor, 40%–70% relative density	High, >95% proctor, > 70% relative density
Fine-grained soils (LL > 50)† Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent soils engineer; Otherwise use E' = 0			
Fine-grained soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1000
Fine-grained soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles; Coarse-grained soils with fines GM, GC, SM, SC contains more than 12% fines	100	400	1000	2000
Coarse-grained soils with little or no fines GW, GP, SW, SP‡ contains less than 12% fines	200	1000	2000	3000
Crushed rock	1000	3000	3000	3000
Accuracy in terms of percentage deflection§	± 2	± 2	± 1	± 0.5

*ASTM Designation D2487, USBR Designation E-3
†LL = liquid Limit
‡Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC)
§For ± 1% accurncy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

Rogers (1987) reiterated the conclusions drawn by Linger (1972) fifteen years earlier when he stated:

> ...the type of soil used to surround the pipes was found to have a considerable influence on pipe behavior. Various uncompacted granular materials provided good support, whereas uncompacted silty clay and silty sand did not. Compaction of the surrounding soil had a variable influence wholly dependent on soil type. Silty clay performed well only when thoroughly compacted in thin layers. Light compaction of a broadly-graded granular soil improved performance slightly, whereas uniform gravel was generally considered to be unresponsive to compaction.

Jeyapalan (1987) presented the following table (see Table 5) as part of a seminar on underground pipelines.

Table 5: Suggested Values of E' as a Function of Soil Type, Degree of Compaction, and Depth of Cover (after Jeyapalan, 1987)

Soil Type	Depth of Cover (ft)	E' (psi) for Given Relative Compaction (% T99, ASTM D698)			
		85%	90%	95%	100%
fine-grained	0-5	500	700	1000	1500
soils w/<25%	5-10	600	1000	1400	2000
sand content	10-15	700	1200	1600	2300
(CL,ML,Cl-ML)	15-20	800	1300	1800	2600
coarse-grained	0-5	600	1000	1200	1900
soils w/fines	5-10	900	1400	1800	2700
(SM,SC)	10-15	1100	1700	2300	3300
	15-20	1300	2000	2700	3800
coarse-grained	0-5	700	1000	1600	2500
soils with	5-10	1000	1500	2200	3300
little or no	10-15	1050	1600	2400	3600
fines	15-20	1100	1700	2500	3800
(SP,SW,SP,GW)					

It is noted that Jeyapalan's values are slightly higher than Howard's (1977) for the coarse-grained materials. The major difference lies in the values for the fine-grained soils with low plasticity. Given adequate cover (minimum 4 feet) Jeyapalan's values are about two and one-half times those of Howard (1977). This author believes it appropriate that no other author has listed E' values for fine-grained materials of high plasticity. Such soils should not be used as backfill. It is acknowledged that soil profiles in the Cincinnati area may contain such soils (CH). Nevertheless, the state-of-the-art is such that trench widths should have a width so as to provide proper support to the pipes through the use of select backfill, regardless of the nature of the natural profile.

Mandich (1991, 1993) discussed the influence of the method of evaluation of deflection in rehabilitative pipe work involving relining of existing sewers. He noted that "engineering computations, such as those based on the trench formulas, which calculate pressure of the soil and the friction resistance along the lines of the trench, are not applicable to rehabilitation of a pipeline by the trenchless method." He examines the stability of the soil surrounding the pipe in terms of zones of dilatancy, zones of compression, and zones of shearing, located above and below the pipe. In his approach there is "less need for non-scientific factors such as enhancement factors and empirically derived factors." Trenchless methods result in "lining system deflections due to buckling and joint deflections that are drastically reduced from those calculated by trench methods." This approach deserves further investigation.

CONCLUSIONS

It has been determined that the modulus of soil reaction is not an inherent soil property but is in fact highly dependent on the soil-structure interaction. The most important properties of the soil-structure system have been found to be the degree of compaction of the backfill, the depth of cover, the span of the conduit, the soil modulus, the flexural rigidity of the conduit wall, Poisson's ratio for the backfill, the deflection of the conduit, and the unit weight of the backfill. A number of tables of E' have been presented in the permanent literature. Over the years these values have not changed greatly, but less conservatism seems to be evident in the recent work of Jeyapalan (1987) for example. This seems in line with the thoughts of Krizek et al. (1971) when they stated:

> In view of the scarcity of failures attributable to design shortcomings and the fact that failures do not normally entail loss of life, it appears that current safety factors can be reduced; however, the application of current procedures and criteria render it extremely difficult to quantify the degree of present conservatism.

It should be understood that experience and engineering judgment will not be eliminated from this process. It is the opinion of this author that more work is required to substantiate the use of various E' values when used in conjunction with the relining of existing sewers by various methods. The reason for this is that the liner is installed in an environment in which most of the deflection of the existing pipe has already occurred. The original pipe will normally carry most of the earth loads. Soil loads on the liner are possible when severe line damage has resulted in large missing sections of the original conduit or there has been a general deterioration of the conduit for any of several reasons. A shift in thinking from trench methodologies to trenchless methods of checking deflections given a certain thickness of liner is in order where the relining of sewers is concerned.

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HEMLOCK TUNNEL REHABILITATION

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ABSTRACT

The City of Rochester Water Bureau owns and operates the Hemlock Water Tunnel, which conveys approximately 45 million gallons per day of potable water from the Bureau's upland reservoirs to its distribution system in Rochester, New York, approximately 25 miles to the north. The 12,000-ft. long, 6-ft. horseshoe-shaped tunnel was constructed in 1894 and was designed to function as a gravity conduit. In 1989 the Water Bureau began designing a water treatment plant at the upland reservoirs, which required modifying the tunnel operation from one of gravity flow to one of a low pressure conduit. This paper describes the investigations undertaken to evaluate the existing tunnel lining and assess the structural capacity as a pressurized conduit. Critical issues that were addressed included soil and bedrock loads, internal and external hydrostatic pressures, hydraulic capacity, infiltration and exfiltration tolerances, and potential presence of naturally occurring gases in the bedrock. Remediation alternatives included modifying the existing brick lining or installing a new internal liners of steel, ductile iron, high-density polyethylene, or fiberglass pipe. A review of the evaluation process is provided along with a description of the contracting methods and construction procedures used to install the fiberglass composite pipe lining that was selected.

Background

The Hemlock Tunnel is located in the Finger Lakes Region of Western New York State in the Town of Livonia. The tunnel was constructed in 1893 and 1894 by the City of Rochester to provide municipal water from the Hemlock Lake Reservoir to the City, which is located approximately 25 miles north.

The reservoir had been completed in 1874 and had supplied water through an initial cast iron conduit. The tunnel was added when a second parallel conduit was constructed.

The reservoir was formed by damming the northern end of one of the naturally occurring narrow, glacially-formed lakes that are commonly referred to as New York State's Finger Lakes. Hemlock Lake is one of the western most of these lakes and is approximately 7.3 miles in length. The water elevation is approximately 905 ft. above sea level, which is about 400 ft. higher than average ground surface in Rochester making a gravity water supply system technically feasible and desirable.

The City was able to secure the properties forming the watershed around both Hemlock Lake and the adjacent Canadice Lake, which provided an excellent water supply to the City for over 100 years.

The tunnel consists of a 12,000-ft. long masonry structure extending north from the end of Hemlock Lake, through an area of high ground forming the glacial valley wall. The tunnel terminates at an overflow structure along the original lake outlet, which also forms the beginning of a twin steel and cast iron conduit system that carries water the remaining 23 miles to the City of Rochester.

The tunnel is horseshoe-shaped with a height and width of approximately 6 ft. The masonry lining consists of an 8-inch thick brick interior, backfilled with varying thicknesses of mortar and rubble. Approximately 60 percent of the tunnel length was constructed in rock. The remainder was built using cut and cover techniques in soil overburden. A tunnel plan and profile is shown in Figure 1.

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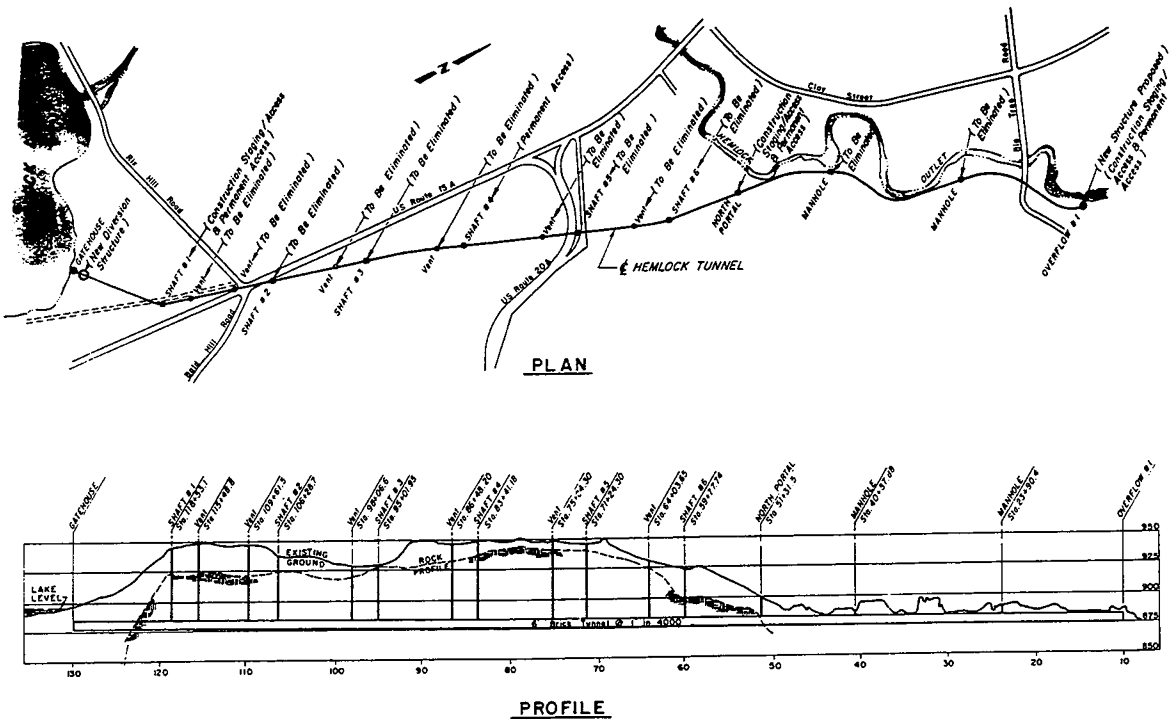

Figure 1 - Hemlock Tunnel plan alignment and profile.

Since its original construction, the tunnel has functioned as a gravity conduit, typically carrying water at approximately 90 percent of its maximum capacity of 48 million gallons per day (mgd). The City's records indicate that very little maintenance has been required to keep the tunnel functional throughout its life. The last major inspection and repair effort took place in 1966 and consisted of cleaning and repointing and caulking open mortar joints in the tunnel crown and sidewalls. Subsequent inspections in 1984 and 1989 showed the tunnel to be in generally excellent condition, with only minor areas indicating some level of distress.

Gradually, the City's water requirements changed and modifications and improvements to the system were made to meet their needs. By 1989, the demand during summer months was higher than the system was capable of transmitting under gravity flow. Despite intense watershed management, the City also occasionally had trouble satisfying State Health Department requirements, primarily with respect to turbidity levels when lake levels were low or during storm events.

It became evident that a water treatment facility was required to meet the City's long-term water needs. Once plans for the new plant were underway, it was also apparent that the tunnel should be evaluated to upgrade its condition to allow maximum use of the water resource and to meet the demands of transmitting treated potable water. There was the need to improve the tunnel water tightness to prevent infiltration of untreated groundwater and a desire to switch from gravity to a pressurized flow condition.

In late 1989, the City retained the design team of STV/Seelye Stevenson Value & Knecht and Haley & Aldrich, Inc. to investigate the tunnel and develop contract documents to modify and upgrade the tunnel lining to meet the new requirements. It would be necessary for the team to have documents ready and permits in place such that the lining repairs could be completed within a narrow window between the fall of 1990 and spring of 1991, during which time the City's water needs could be met by temporarily pumping around the tunnel through an existing bypass conduit. It was critical that the tunnel be back in service in time for the peak summer demand.

Tunnel Investigations

SSVK and H&A developed an investigative program that would allow evaluations of the tunnel lining condition during a brief two-week period in September 1989. With the exception of this one period, no access to the tunnel would be available throughout the remainder of the design process.

The next opportunity to remove the tunnel from service would be in the fall of 1990, during which time potential contractors would be invited to visit the tunnel in preparation for bidding the planned lining repairs.

The SSVK/H&A team conducted detailed tunnel mapping and surveying to document existing conditions and verify record drawing information. In addition, a program of non-destructive testing using ground penetrating radar (GPR) and shallow seismic refraction survey techniques was completed to assess the structural properties of the existing brick lining. The non-destructive testing was performed by Weston Geophysical Corporation of Westboro, Massachusetts.

Core drilling was also performed at several locations from within the tunnel to physically inspect conditions behind the tunnel lining and correlate those observations with the non-destructive test results. The recovered core samples of brick and mortar were also sent to a materials testing facility to determine compressive strength and modulus for use in subsequent lining analyses.

Later, after the tunnel was put back in service, a program of test borings and monitoring wells was completed to verify subsurface conditions along the tunnel alignment and at key structure locations.

Subsurface Conditions

As indicated in Figure 1, approximately 7,300 ft. of the Hemlock Tunnel was constructed through bedrock. The rock consists of a sequence of platy, fissile, black to dark gray shales of Upper Devonian age, known locally as the West River Shale. Above the bedrock are soil deposits comprised largely of glacial till with some alluvial and colluvial deposits depending on proximity to streams and the slopes forming the valley sidewalls.

The northern portion of the tunnel, constructed originally by cut-and-cover, is located primarily within glacial till soils that are dense to very dense, well-bonded sands and silty sands with varying quantities of gravel, clay, cobbles, and boulders forming the overall matrix. The materials directly in contact with the brick tunnel lining consisted of those same soils placed as backfill when the conduit had been completed.

Groundwater levels along the tunnel alignment varied depending on location, but generally ranged from approximately 6 to 21 ft. above the tunnel crown. In some areas, groundwater levels were judged to have been lowered by the draining effects of the of the tunnel. Water levels were influenced by recharge from surface and subsurface drainage down into the valley from the east. Within bedrock,the groundwater also contained low levels of hydrogen sulfide picked up while leaching through the shale deposits.

Tunnel Lining Conditions

The existing tunnel lining in the Hemlock Water Tunnel consisted of multiple courses of brick laid up in a horseshoe shape approximately 6 ft. high and 6 ft. wide at the springline, as shown in Figure 2. Record drawings indicated that the lining consisted of two brick courses in rock tunnel sections and three courses in soil tunnel or cut-and-cover sections.

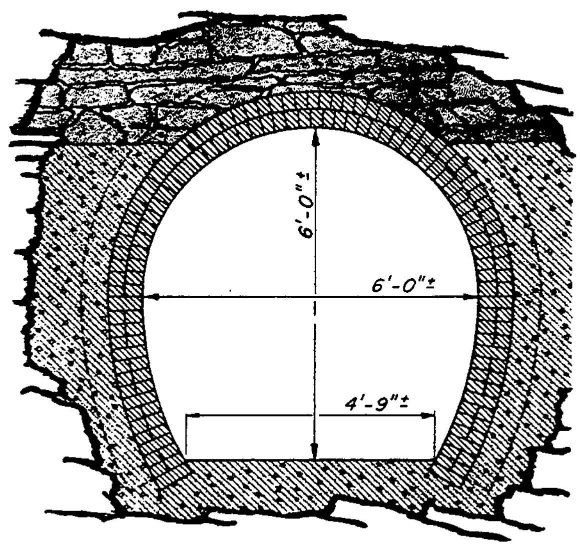

Figure 2 - Typical tunnel lining cross section.

The tunnel invert was essentially flat in the rock tunnel sections and was formed of mortar or concrete. A curved brick invert was present in the soil sections and for a short distance upstream and downstream of each shaft location.

The brick lining had performed very well over the 100-year operating life of the tunnel. Evidence of the repairs made during the 1966 effort were readily apparent and only limited indications of subsequent deterioration in the past 20 years was detectable during initial walk-throughs.

To better define lining conditions, the SSVK/H&A team conducted a series of investigations including detailed tunnel lining mapping, ground penetrating radar surveys, seismic refraction profiling, and limited core sampling.

The tunnel lining mapping was conducted to develop a detailed inventory of cracks, openings, groundwater infiltrations, and other such observations that would be helpful in assessing the tunnel lining condition and could be used to evaluate the lining structural capacity. The information collected was summarized graphically on mapping log forms and plotted on graphic projections representing the entire surface of the tunnel lining.

The ground penetrating radar surveys (GPR) were performed by moving a 500-MHz antenna along the tunnel crown and sidewalls. The continuous radar scans provided information that could be summarized graphically along with mapping data on the same tunnel overlay. Anomalies in tunnel backfill conditions were identified and grouped into four general categories:

1. Areas of water saturated backfill conditions;

2. Areas of wet or decoupled backfill conditions;

3. Areas of increased backfill thickness; and

4. Areas of potential voids or large backfill particles.

These observations alone did not reveal a great deal about the lining condition. However, when used in conjunction with the tunnel mapping data, they were helpful in identifying potential trouble spots or unusual conditions.

At the completion of the GPR surveys, limited seismic profiling surveys were performed at selected locations to supplement the GPR data and acquire specific physical properties for use in subsequent lining evaluations. Profiling was conducted in short reaches at locations where both relatively good lining conditions were identified and at locations where problems were suspected.

Finally, 4-in. diameter thin-wall core samples were taken through the tunnel lining at a number of discrete locations where both normal and suspect backfill conditions were anticipated. The core holes provided opportunity to visually inspect the character of the brick lining/back grout/bedrock interface to determine the effectiveness of the original grouting effort and the present condition of the lining/bedrock contact surface.

The GPR and seismic profiling surveys indicated potential anomalous conditions over approximately 40 percent of the tunnel length. As might be expected, most of the questionable backfill and backgrout conditions were identified within the rock tunnel sections. However, most of the cracking and measurable lining deformations were within the soil tunnel and cut-and-cover sections where external loads were likely to be higher and ground stiffness lower.

The core sampling results showed that the GPR and seismic profiling techniques were good indicators of potential problems much of the time, but could not be relied upon to pinpoint lining deficiencies in all cases. The core samples also indicated that the brick masonry was in exceptionally good condition at most locations. Many of the samples collected were able to be maintained intact such that the brick/mortar matrix could be tested for combined compressive strength and modulus. Measured compressive strengths ranged from approximately 2600 psi to over 7500 psi for five core samples tested. Modulus values were measured at between 6000 and 7000 ksi.

<u>Lining Evaluations</u>

Tunnel lining evaluations were performed to assess the capacity of the existing brick and mortar lining to withstand anticipated future tunnel loadings, including external loads from soil, bedrock, and hydrostatic pressures, as well as new internal pressures ranging from approximately 13 psi under normal operating conditions and up to 30 psi under certain surcharge situations.

External loads were generally expected to remain the same as experienced during the tunnel's life with the exception that external hydrostatic pressures were expected to increase if the tunnel were modified to eliminate groundwater infiltration.

In general, three potential lining modification scenarios were considered:

1. The existing brick lining in rock tunnel sections improved to eliminate infiltration by repairing sections and performing additional contact grouting;

2. Supplementing the existing brick lining in both rock and soil tunnel sections by adding shotcrete to the inside lining surface and contact grouting; and

3. Installing a new circular lining within the existing tunnel, consisting of either steel or fiberglass composite pipe.

Combinations of these various methods were also evaluated to identify the most cost-effective solution.

It was determined relatively early in the investigation that the existing brick lining would not be adequate to resist internal pressures along most of the soil tunnel and cut-and-cover sections without some internal modification, due to insufficient soil cover.

To assess whether modifications to the brick lining would be appropriate, the SSVK/H&A team conducted detailed structural evaluations of the various lining configurations using a two-dimensional computerized finite element method. The tunnel geometry was modelled using a continuous framework of linear elements. Elements were also included to model the surrounding soil and rock masses, with varying properties to account for the range of backfill conditions observed during the field investigations. The analyses were also conducted under a variety of external and internal loading conditions to cover each identified lining configuration/load scenario.

In general, the lining analyses showed that the brick lining was not appropriate to resist the internal pressures within the rock or soil tunnel section due to inadequate tensile strength and the inability to reliably eliminate infiltration.

However, a shotcrete reinforced lining was shown to be adequate for the entire rock tunnel length, provided additional backgrouting was performed. A nominal 4-in thick shotcrete layer, reinforced with steel fibers, in combination with the existing brick lining was determined to have sufficient capacity to resist each of the anticipated loading conditions.

The shotcrete material would be relatively easy to apply and would provide a high degree of flexibility at existing structures and shaft locations. The final lining would also result in less tunnel capacity reduction than other alternatives such as the slip-lining option. However, the shotcrete modification represented some potential difficulties with respect to quality control and necessitated a thorough contact grouting effort behind the existing brick lining to achieve the required ground interaction. That grouting effort was extremely difficult to quantify and was expected to represent a substantial portion of the lining repair costs.

In addition, it was determined that the fiber-reinforced shotcrete lining would not have adequate tensile capacity to resist the internal pressures under surcharge conditions in certain portions of the soil tunnel section. Therefore, the shotcrete tunnel lining would need to be supplemented with some other method, such as slip-lining, in up to approximately one fourth of the tunnel length.

Other concerns such as potential cracking and subsequent infiltration, along with possible corrosion of reinforcing fibers in the lining weighed against the shotcrete lining repair alternative.

Analyses were performed for slip-lining repairs assuming both a welded steel pipe and a composite fiberglass pipe within the existing tunnel lining. The pipe materials were evaluated for essentially the same loading conditions as the shotcrete lining alternative except that the circular pipes received the benefit of a uniform initial grout pressure to minimize uneven loading conditions. A steel lining thickness of 3/8-inch was determined to be appropriate for the rock tunnel sections and a 7/16-inch thickness was recommended for soil tunnel lengths.

Alternatively, a composite fiberglass pipe was also determined to be acceptable with two grades specified for the rock and soil tunnel sections. Pipe with a stiffness rating of 46 psi was required for the rock tunnel and 72 psi for the soil tunnels. The pipe considered for the project consisted of the type manufactured by Hobas, which involves a proprietary method of spin casting the fiberglass material against an external mandrel.

In both cases, the pipe could be supplied in a nominal 60-in. inside diameter that would fit inside the existing tunnel lining as shown in Figure 3. The tunnel cross-sectional area would be reduced substantially. However, the reduced frictional losses of the manufactured pipe materials allowed the desired tunnel capacity to be maintained.

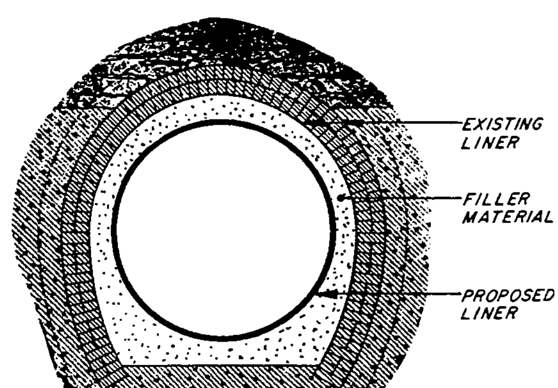

Figure 3 - Proposed slip-lined tunnel repair.

The use of slip-lined pipe materials presented some increased difficulties at shaft and structure locations, but the improved hydraulic characteristics and minimal on-site quality control requirements compared to the shotcrete alternative were judged to outweigh the increased material and installation costs.

Ultimately, the SSVK/H&A team and the City determined that the contract documents should be prepared requiring a slip-lined installation of either welded steel or Hobas fiberglass pipe.

Lining Repair Construction

In August 1990, the City awarded a contract to Hall Contracting Corporation of Charlotte, NC to perform the planned tunnel remediation. Hall had won the award with a bid of approximately $4.9 million dollars to install a slip-lined composite fiberglass pipe inner lining. The pipe used for the project was manufactured by Price Brothers under a licensing agreement with Hobas.

The pipe was all manufactured in Green Cove Springs, FL and was delivered to the project site by truck in lengths varying from 2 ft. to 20 ft.

In general, the project work consisted of the following major components:

1. Implementing bypass operations to divert water around the tunnel;

2. Dewatering and accessing the tunnel at three locations: the Entrance Structure near the planned treatment plant, at the North Portal where the original rock tunnel and cut-and-cover sections met; and at the Overflow Structure where the tunnel connected to the two pressure conduits;

3. Performing limited repairs to the existing brick lining and conducting contact grouting to cutoff major water inflows;

4. Installing the Hobas pipe, backgrouting the annular space between the pipe and tunnel lining, and installing new connections at both ends of the tunnel; and

5. Removing the existing access shafts and vents and constructing new permanent access points at the three work sites.

Actual work on the tunnel repairs began in September 1990 and continued essentially without interruption until the tunnel was put back into service in June 1991.

After limited repairs were made at specified locations within the tunnel and preparations were made at the access points, the contractor began installing pipe sections at the Overflow Structure. Later, installation was begun at the Entrance Structure, with the North Portal being the last location utilized.

The contractor installed all the pipe on the project using one crew working shifts from 8 to 12 hours per day.

To properly position the pipe sections, the contractor first installed hardwood guide rails on the tunnel invert. Blocking was also provided at critical points around the pipe perimeter at the joints as each pipe length was installed to prevent movements during backgrouting.

The pipe sections were lowered into the tunnel at the access points by conventional backhoes or cranes, as shown in Figure 4.

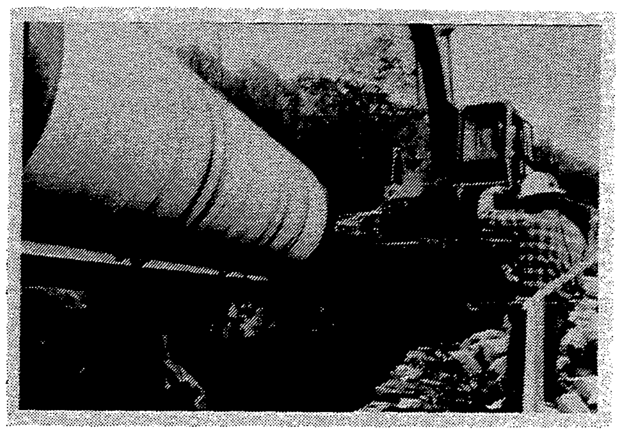

Figure 4 - Lowering pipe sections at the Overflow Structure.

The pipe lengths were then transported to the required destination along the tunnel using modified hand-operated fork lifts as shown in Figures 5 and 6. The longest pipe lengths weighed approximately 10,000 lbs.

The pipe laying schedule was coordinated so as to minimize transport around eight existing curves and to maximize the use of long pipe lengths. The tightest curve in the tunnel had a radius of approximately 76 ft., which was the only curve that pipe was not transported through.

The contractor was able to negotiate 20-ft. pipe sections through all the curves with radii of 200 ft. or greater.

In general, the longest transport length for any pipe section was approximately 6000 ft. or half the total tunnel length.

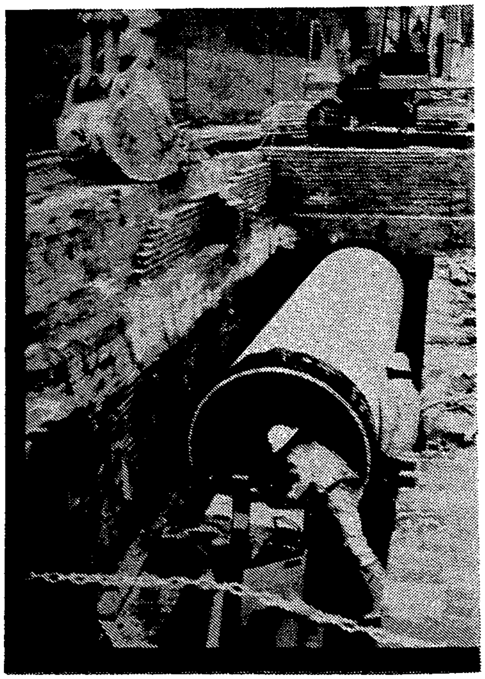

Figure 5 - Positioning pipe sections on fork lifts for transport.

Once in position, the pipe joints were pushed home using a system of jacks. The joints were designed as bell and spigot connections with a neoprene gasket and were rated at 30 psi internal pressure.

After the joints were made, each was pressure-tested using an internal packer system as shown in Figure 7. The tests were performed by applying pressure in increments up to a maximum of 30 psi and held for two minutes. Tests were judged as passing if no pressure was lost through the test period.

Special Dresser coupling connections were required at each end of the pipe where the tunnel joined the outfall from the future water treatment plant and the pressure conduits.

In total, 638 individual pipe lengths totaling 12,200 ft. were placed in the tunnel. The last section was installed on in April 1991, eight months after the start of the work. The maximum placement rate during the project was 320 ft. in one day.

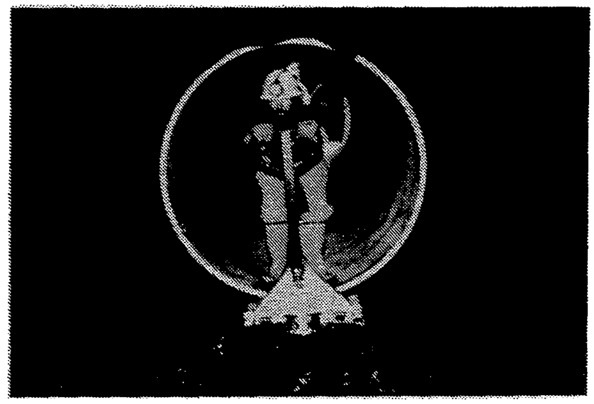

Figure 6 - Pipe lengths being negotiated through the tunnel with fork lifts.

Backgrouting around the Hobas pipe was performed by Dowell Schlumberger working as a subcontractor to Hall Contracting. The grouting was performed using equipment adapted from the oil and gas well industry.

Grout was injected from the three access points and existing vent shafts along the alignment up to a maximum distance of approximately 650 ft.

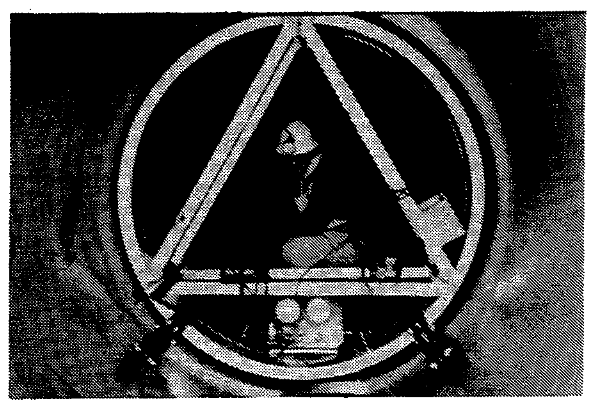

Figure 7 - Pressure testing joints to 30 psi.

The grouting was performed in four stages, working from pipe invert upward to the crown, to avoid floating pipe sections and causing undue stress on pipe joints and blocking.

The grout mix consisted of Type I cement and water at a weight ratio of approximately 1.6 cement to 1

CONTEMPORARY PRACTICE IN THE STABILIZATION OF CONCRETE DAMS BY POST-TENSIONED ROCK ANCHORS

Dr. Donald A. Bruce*

ABSTRACT

Permanent post-tensioned rock anchors have been used for over twenty years in America to stabilize existing concrete dams and their appurtenant structures. This paper provides a state of practice review focusing particularly on construction, corrosion protection and performance. Aspects of design are also addressed. Two areas requiring national attention, namely attitudes towards corrosion protection, and long term performance monitoring, are highlighted.

INTRODUCTION

Permanent post-tensioned rock anchors have been used in America for over twenty years to help existing concrete dams meet contemporary safety standards. Anchors have been used in dam raising operations where they have proved more economical in resisting the increased overturning movements than the placement of additional concrete mass. However, their most common usage has resulted from dam safety re-analyses, based on the new criteria relating to P.M.F. (Probable Maximum Flood) and M.C.E. (Maximum Credible Earthquake): designs of dams constructed in the first half of this century are often found to be deficient and owners are obliged by law to take appropriate remedial action.

Common applications of anchors therefore include providing

- resistance to overturning
- resistance to sliding, and
- resistance to seismic effects.

However, in the United States alone, one can also cite their use in a range of ancillary applications, including:

- stabilization of rock abutments
- combatting the effects of alkali/aggregate reaction
- security of tunnel portals and open cuts
- stabilization of excavations for plunge pools and spillways, and
- stabilization of lock structures against lateral and vertical forces.

Such dam repairs are conducted throughout the country and extend from private, utility-owned structures through those owned by bodies such as the Tennessee Valley Authority, to the great structures under the aegis of the Federal Government. As the average age of these dams continues to increase, and our ability to monitor and analyze them improves, so we may expect the use of permanent post-tensioned anchors to continue to rise.

At this juncture in the United States, we have attained an admirable level of general competency in anchor technology, although there remain a certain number of details of a practical and philosophical nature where we differ from practice in other countries. Indeed, one of the most fundamental differences is that we have no national standard or code for rock and soil anchors. The Recommendations of the Post-Tensioning Institute (PTI) (1) come closest, but these are often altered and "improved" upon by individual specification writers, or are, unfortunately, ignored completely

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and highly "original" project specifications substituted. As a consequence, certain key issues are simply not addressed in a uniform manner, being viewed in a very parochial way, depending on the personal experience and agenda of the engineer involved.

The PTI Recommendations are currently under review with the goal to create a national document which will, where judged appropriate, reflect the best of international experience. As background to some of the issues which will be addressed in the field of rock anchors, this paper provides a brief state of practice review. It focuses on the more contentious or less well understood issues in construction and performance especially.

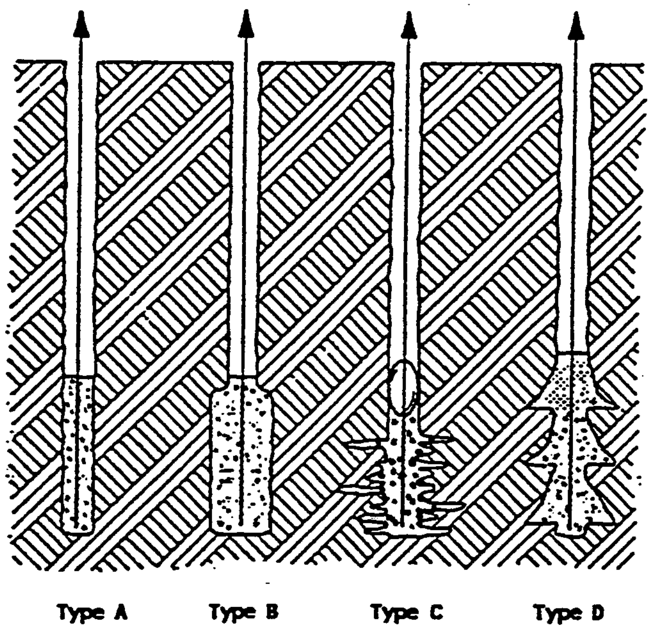
COMMENTS ON BASIC DESIGN PROCEDURES

It would seem that the basic design methods remain largely as researched initially by Littlejohn and Bruce (2), and summarized more recently by Hanna (3) and Xanthakos (4). For example, the overall resistance to pullout, by general rock mass failure, is calculated using simple assumptions on the geometry of the rock mass conceptually engaged, and the weight of rock in that mass. Certain designers, armed with reliable data on rock mass structure and strength parameters have optimized designs, and safely shortened the fixed anchor embedment length accordingly. Most still acknowledge the real or potential presence of less competent rock for the uppermost 10 feet and so permit bond zones to be commenced only below such elevations in supposedly fresher, better quality material.

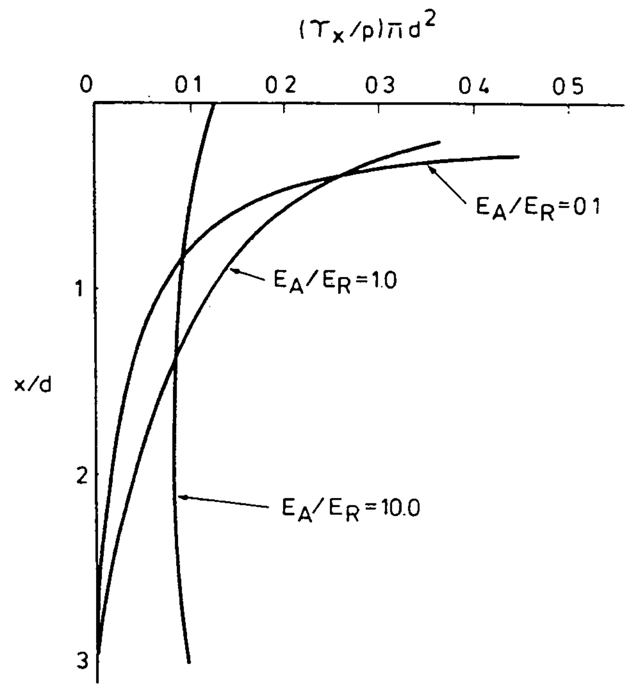
Regarding the design of the bond zone itself, rock anchors for dams invariably fall into Littlejohn's (5) Type 'A': straight shafted with gravity pressure grouting (Figure 1). The choice of appropriate rock-grout bond values is traditionally based empirically on the unconfined compressive strength of the rock, or on the results of past successful applications which is valid only as long as any variations due to construction method are accommodated. More engineers are becoming aware that the actual bond is not evenly distributed over the whole rock-grout interface but most do not appear to take this into account at the design stage. The most enlightened designers are, however, insisting on special pre-production test programs to verify bond values, and time related performance (6, 7, 8, 9). Such programs have again confirmed clearly the mathematical and laboratory theories of load transfer mechanisms, and the relation of bond stress distribution to the elastic modulus of the confining rock mass. In most rock conditions, and specifically where the ratio of the grout modulus to the rock modulus is less than 1, then the load is transferred from tendon to rock only in the upper 5-10 feet of the bond zone: the remainder of the bond zone is in effect, the physical safety factor (Figure 2) (10). The rigid application of "average" bond values may, however, lead to the calculation of extraordinarily and needlessly long bond zones.

Careful analysis of the elastic component of total tendon extensions during performance testing of production anchors also confirms this phenomenon, further discussed below.

Computers have proved to be invaluable in analyzing structures to determine the amount of additional post-tensioning force and its optimal points of application (4). They have also speeded the calculation of anchor lengths and geometries, but based always on the basic traditional assumptions of load transfer mechanisms. They appear



<u>Figure 1.</u> Main types of cement grouted anchors (Littlejohn, 1990)
Type A: straight shaft, gravity grouted
Type B: pressure grouted <u>during</u> installation
Type C: pressure grouted via a sleeved pipe <u>after</u> initial installation grout has set
Type D: underreamed, gravity grouted



<u>Figure 2.</u> Variation of sheer stress with depth along the rock/grout interface of anchor (10).

not to have fostered new methods of anchor design per se.

ASPECTS OF CONSTRUCTION

DRILLING

There has always been concern about the potential the drilling operation may have for damaging the structure. In earlier days, diamond drilling was common as it was considered that this highspeed, low torque method would induce minimum vibrations or flushing pressure surges, and would also drill through steel embedded in the concrete. These advantages, however, are invariably offset by cost, and by technical drawbacks including a restriction to smaller diameter holes, and the provision of a very smooth borehole wall, not conducive to high bond development.

Contractors involved on larger anchor projects then adopted rotary drilling methods involving the high torque, high thrust machines otherwise used in water well drilling. Such rotary methods typically provide relatively low penetration rates in all except the softer, argillaceous geologies, and holes can have substantial deviations, given the principle of the drilling action. In addition, the drilling rigs tend to be larger, often truck mounted, and so frequently difficult to move and position on dams with restricted access.

The use of percussion drilling techniques was often discouraged, and is still prohibited in certain areas. Although top drive percussion is rare in such works, given its restricted depth, diameter and linearity potentials, down-the-hole rotary percussion has always been favored in certain quarters for such work, and its popularity is rapidly growing.

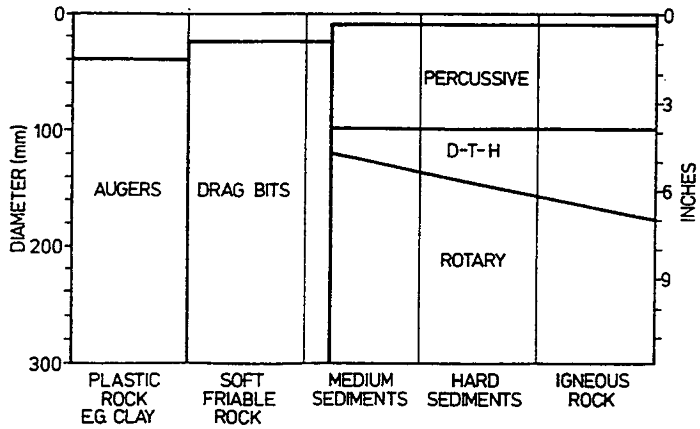
A compact rotary head, and a mast system capable of even moderate pull up and thrust are adequate to move and rotate a drill string. The percussive energy is provided by a down-the-hole hammer, located immediately above the drill bit, and powered by compressed air. This rotary percussive method has been proved to be the fastest, cheapest and straightest way of drilling holes of diameters 4 inches or more through rock and concrete to depths of over 300 feet (11).

Figure 3 (12) provides a useful guide to the initial selection of drilling method. Most recently, the work conducted at Stewart Mountain Dam, AZ (13) provided an excellent opportunity to demonstrate the advantages. This structure is a double curvature arch, built in the early 1930's and previously considered a "delicate" structure (Figure 4) due to its being suspect to seismic excitation:

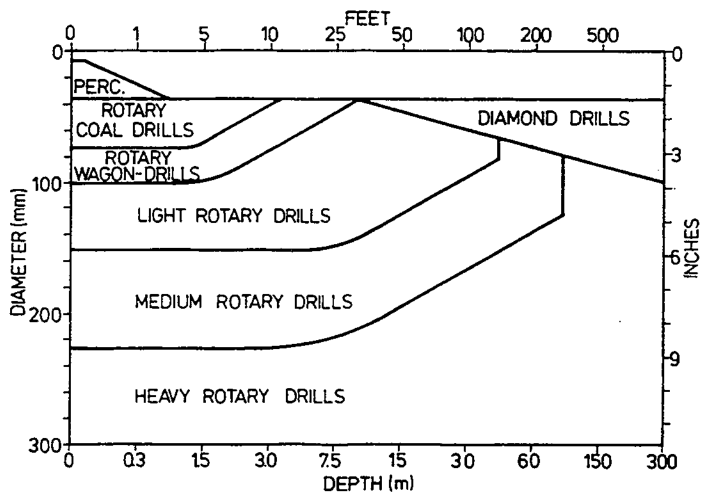
- 10 inch diameter holes drilled to over 260 feet with deviations generally less than 1 in 200;
- penetration rates of over 60 feet/hour recorded in concrete and granite;
- drill masts could be set up in very restricted access areas to accuracies measured in minutes, in both inclination and bearing;
- the effect of the compressed air flush on lift joints was minimal.
- the impact of the hammer vibration on the structure was minimal (Figure 5).

Thus, although current practice features a variety of drilling methods, there is no doubt that down-the-hole drilling is becoming the most popular and accepted choice, and the results from Stewart Mountain Dam underline firmly this shift of opinion.

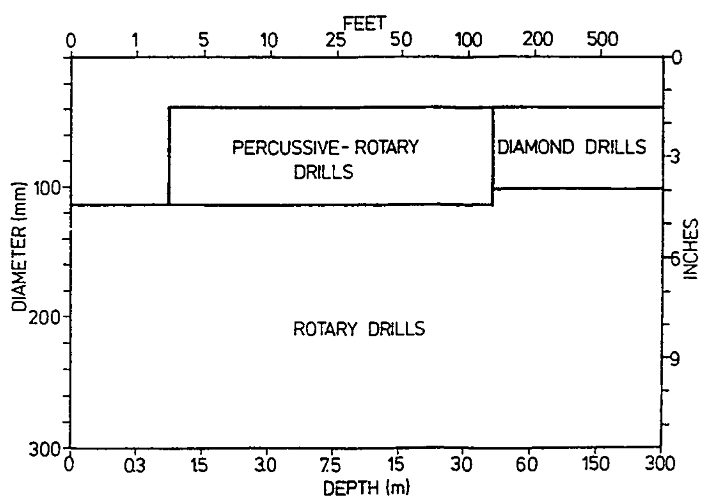
During drilling, and later construction steps, the impact of these activities on existing drainage arrangements must be



Preferred methods of drilling different classes of rock and at different hole diameters. Depth of hole generalised



Preferred methods in soft friable rocks



Preferred methods in variable strata

Figure 3. Guidelines for drilling equipment and method selection (12).

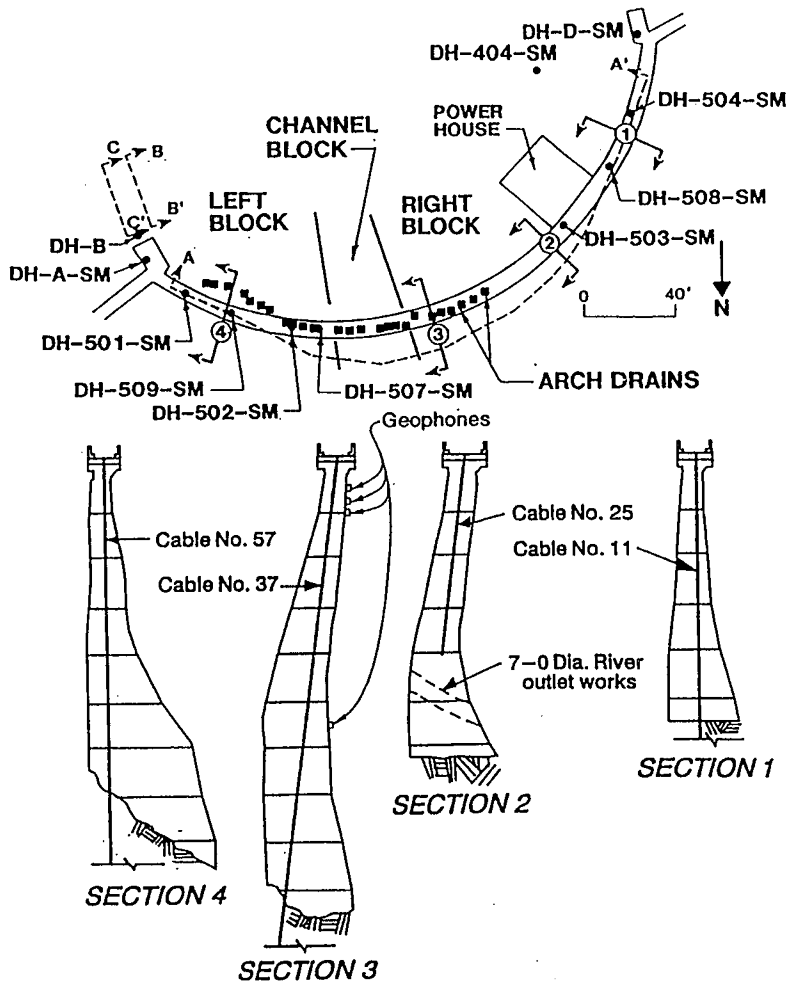
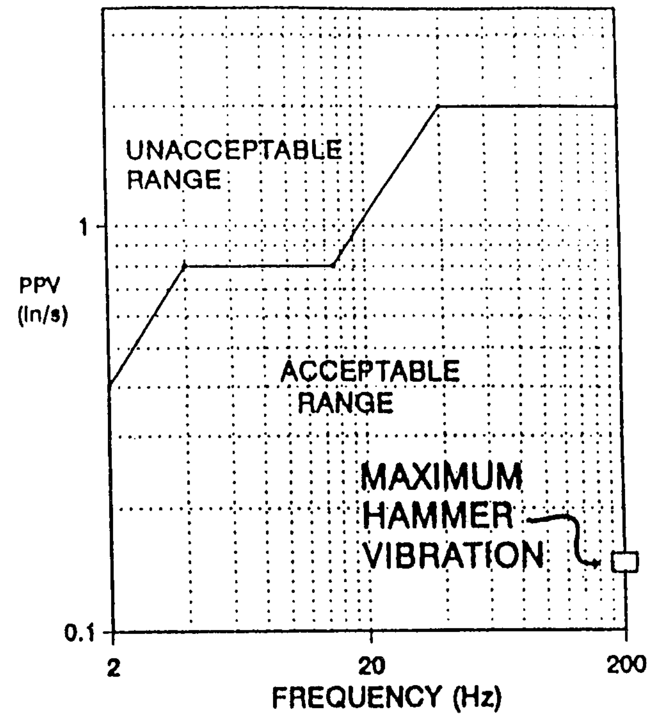


Figure 4. Typical sections through the arch, showing inclination of anchorages, positions of toe drains, and locations of geophones, Stewart Mountain Dam.



Elevation of Geophone (ft)	Approx. Drill Bit Elevation at Max. Peak Vector Sum (ft)	Maximum Peak Vector Sum (in/sec)	Approx. Distance from Geophone to Hole (ft)
1521.12	1521	0.039	5.0
1517.14	1454	0.017	5.0
1512.94	1581	0.066	5.0
1423.15	1433	0.147	8.0

Figure 5. Data from geophone monitoring during down-the-hole drilling through concrete. Stewart Mountain Dam. (Reclamation Acceptability Criterion).

monitored. Figure 6 illustrates how, by using careful techniques and methods, anchors can be installed in even the most sensitive of dam/foundation systems without lasting influence on the pre-existing drainage provisions.

HOLE DEVIATION AND MEASUREMENT

Acceptable tolerances are specified for each project, and reflect the geometry of the dam-anchor system and the criticality of the structural assumptions. As tabulated by Bruce (11), these tolerances have typically ranged from 1 in 60 to 1 in 240, with most being around 1 in 100. Hole straightness is less frequently addressed, although it is wise to consider the possibility of the tendon free length being in contact with the borehole wall during stressing and to generate appropriate straightness criteria reflecting both hole and tendon geometry.

Hole deviations have traditionally been measured after drilling, using various types of inclinometer/gyroscope instruments. These have had various drawbacks, including accuracy, sensitivity, and the time needed to process and analyze the data. The US Bureau of Reclamation has developed an extremely accurate method based on optical principles, but this can operate, economically, only in completed holes, and, practically, only in dry holes.

In the unique case of Stewart Mountain Dam, where hole positions had to be identified at 10 to 20 foot intervals during the drilling of each hole, to provide early warning of the need to correct possible deviations, a rate gyro inclinometer (EC) was adapted from the oil exploration industry (Figure 7). This device allowed fast and easily interpreted data to be made available at the drill site, to an accuracy of 1 in 400. These extraordinary advantages are, of course, reflected in the price - a factor which rules it out of common practice.

WATER TESTING

It is common practice to subject part of each hole at least to a permeability test after drilling. Should the hole, or section of the hole, accept more water than a criterion states, then it is pregrouted and sealed with a neat cement grout. Such pregrouting is often required in advance in holes which intersect large water bearing fissures at the concrete-rock contact. In such circumstances, bulking agents (such as sand), or flow control additives (such as sodium silicate) are added to help resist washout of the grout prior to its setting. Alternatively, some type of hydrophyllic chemical grout may be used. This is a common problem in many older dams built on "horizontal" argillaceous sediments, or in karstic limestone terrains. Equally, holes which interconnect during drilling must be routinely pregrouted and redrilled.

Water tightness criteria are typically of the form "0.001 gallons/inch diameter/foot/minute at an excess pressure of 5 psi". As pointed out by Littlejohn (14) though, this is not an altogether logical approach: for example, once the hole is filled with water, the outflow reflects the fissure characteristics, not the borehole diameter. In addition, holes may be water permeable, but not grout permeable, and, as the whole point of the exercise is to assure that no anchor grout subsequently escapes from the borehole, the relationship between fissure geometry and cement particle size is critical.

Littlejohn therefore recommends that pregrouting be carried out only at stage permeabilities of 10 Lugeons or more. This is equivalant to a flow of about 0.4 gal/minute at

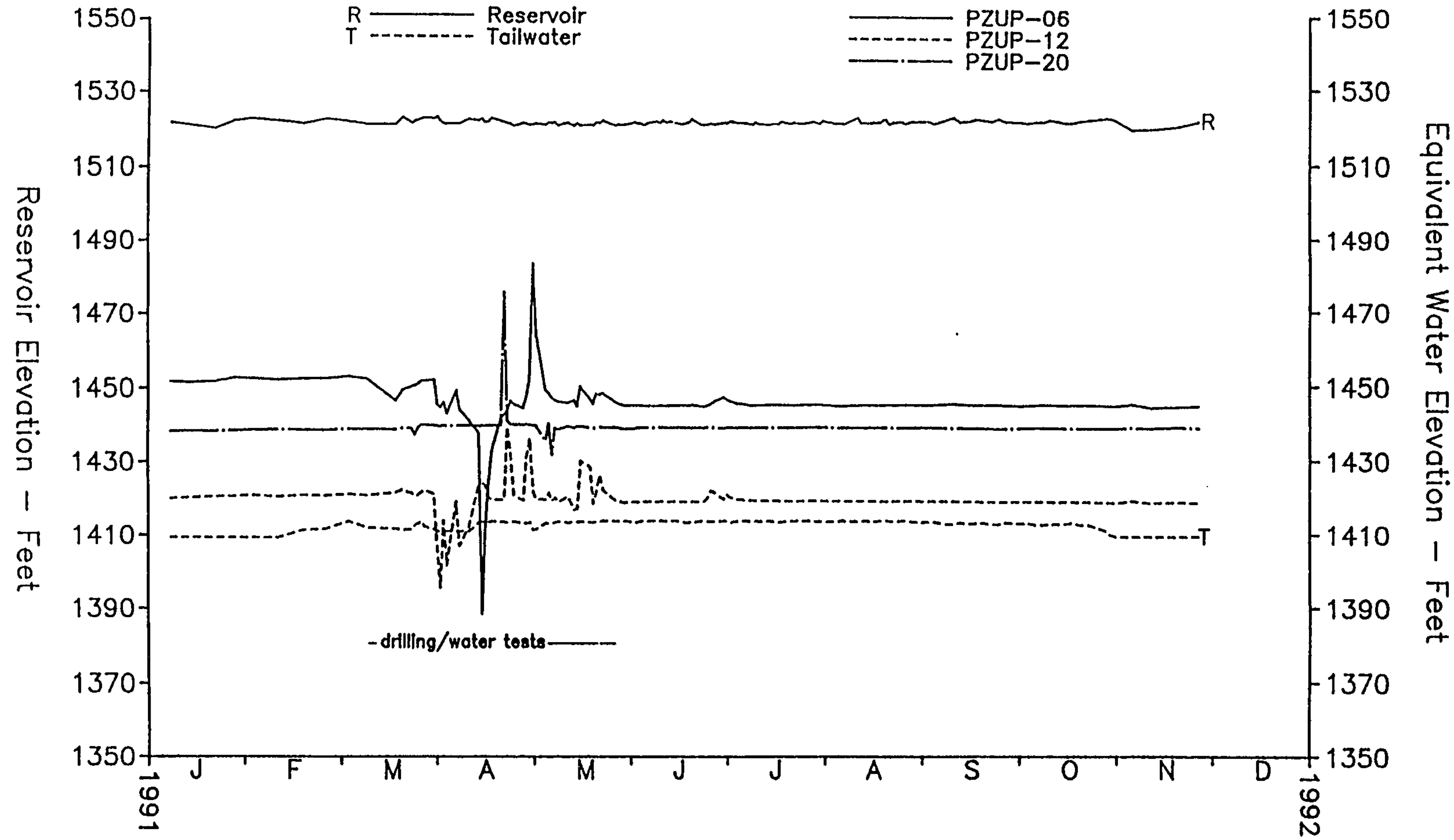


Figure 6. Arch drain piezometer records during anchor construction, Stewart Mountain Dam.

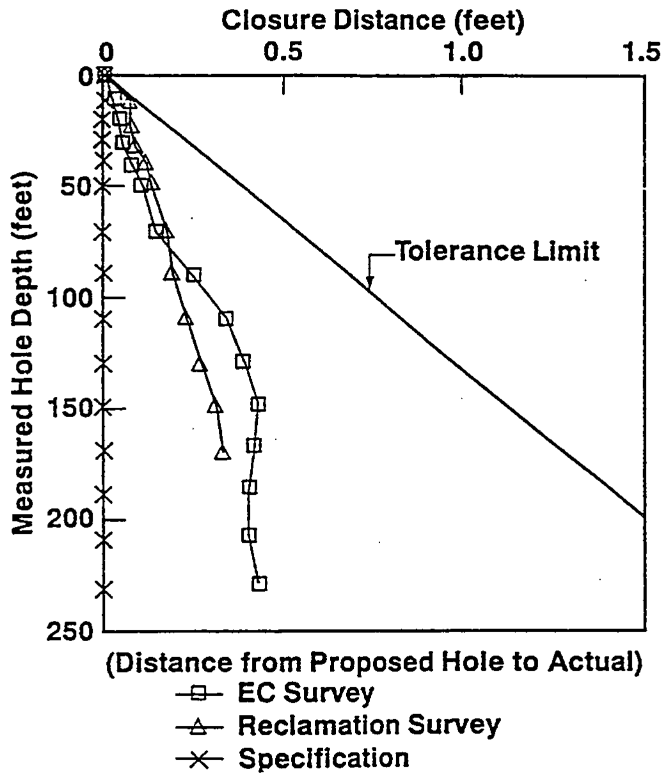


Figure 7. Typical hole deviation data, as monitored in Hole 37, Stewart Mountain Dam.

an excess head of 15 psi, and so can be two or three times more generous than the criterion quoted above, depending on hole diameter and assumed stage length. However, since U.S. practice in tendon protection against corrosion is weaker (below), then this extra emphasis on borehole water tightness is not necessarily wasteful. Any hole encountering artesian pressure is usually pregrouted, regardless of the magnitude of inflow. After pregrouting, redrilling is usually accomplished by rotary drilling within 12-24 hours, using air or water flush.

GROUTING

High speed, high shear cement grout mixers are now widely used. These ensure uniform and intimate mixing of the cement particles and the water. This efficiency permits the preferred lower water content grouts (w/c = 0.40 to 0.45 by weight) to be used, leading directly therefore to higher and earlier strengths and reduced bleed potential (2-4% acceptable) without the need for additives (Figure 8). Type I/II cement is most common, with Type III restricted to cases where unusually high early strength is required, such as in the case of a short, preproduction test program.

Although some specifications call for the use of special additives to meet various goals, there is no doubt that neat grouts, properly mixed and placed are nearly always adequate. The most notable exception is when grouting anchors in high temperatures or where long pumping distances are unavoidable. Here, plasticizer/retarding agents, in small amounts, have proved useful in the mixing and injection phases without causing any long term strength problems. On the other hand, additives that cause expansion by producing gas are now discredited for a variety of

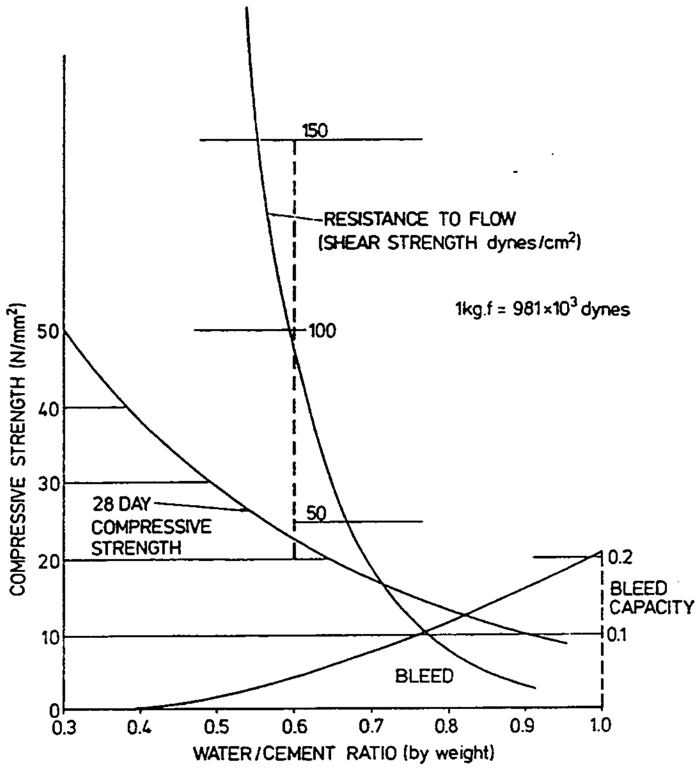


Figure 8. Effect of water content on grout properties
Note: 1 N/mm^2 ≈ 145 psi.

reasons including grout consistency and corrosion potential. Likewise, gelling or thixotropic additives are also avoided, partly due to the extreme sensitivity of the grout properties to their concentration, and partly due to their presence compromising bond development.

Regarding quality control and assurance, cement is usually delivered and measured by the bag, and water by calibrated tank, or by water meter. Quality assurance is still mainly provided, retrospectively, by crushing cubes, the conventional 28 day strength target being 3000 psi. More recently, attention is being paid to testing the fluid properties of the grout also, and the flow cone (fluidity) Baroid Mud Balance (specific gravity and hence w/c ratio), and measuring cylinder (bleed potential) are becoming commonly specified controls.

Special measures are often specified for grouting in especially cold or hot conditions. However, it is most common to simply avoid such conditions by appropriate scheduling of the work.

TENDON ASSEMBLY, INSTALLATION, AND GROUTING

Bar tendons tend to be restricted to shorter anchors (say 50 feet) and lower capacities (say about 40 tons). Most commonly, multistrand tendons are used, and the trend is towards high capacity and considerable length: tendons of 58 strands over 300 feet long were installed recently at Lake Lynn Dam, PA (6, 9).

Tendons are commonly factory assembled, and delivered to site in coils about 8-10 feet in diameter. On certain occasions, they have been placed in their holes by helicopter, but most commonly this is achieved by using mechanical uncoilers, or simply by long mast crane. All specifications call for "controlled" tendon installation.

The component strand is typically 0.6 inch diameter with low relaxation properties. Spacer/centralizer units are specified in the bond zone at regular intervals (usually around 10 feet), with intermediate steel bands to provide a "noded" or rippled effect. These should guarantee a minimum interstrand spacing of 1/4 inch, and a minimum outer grout coverage of 1/2 inch. Spacers in the free length are less common, and more widely separated. Practical and theoretical considerations limit the amount of borehole that can be occupied by the strand to less than 15% of its volume. Tremie tubes are attached during initial fabrication and are most usually located centrally within the tendon. Nose cones are added to minimize the risk of tendon or hole damage during installation.

There are still differences in opinion regarding the acceptability of the strand surface condition. At one extreme are inspectors who will tolerate no rust on the surface: this zeal is misguided, as it is well known that the presence of a light, non flaky corrosion will actually enhance grout/steel bond development. Equally, the presence of rust states that no other surface coating is present, in the form of grease, lubricant or other oils resulting from the manufacturing process.

Grouting is either conducted in one operation (i.e., bond length and suitably decoupled free length, followed by stressing), or two operations (i.e., grout bond length, stress, then grout free length). This is a project specific decision, with the engineer compromising the advantages and problems of each method to optimize the performance. Two-stage grouting, for example, does clarify the stressing analysis, but also makes the grouting operation more complex to control.

CORROSION AND CORROSION PROTECTION

Virtually every rock anchor installed in a dam is regarded as permanent, to conceptually function throughout the lifetime of that structure. Corrosion protection is therefore a vital and integral part of anchor design and construction.

On the global stage, it is perhaps only in this aspect that U.S. practice is perceived as being deficient, even though considerable advances have been made in the last few years following the works of FIP (15) and Littlejohn (5) in particular. The major point of difference between U.S. and foreign practice is in the concept of double corrosion protection. Foreign engineers, following their national codes, do not regard cement grout as an acceptable barrier to corrosion, in that it carries the potential for microfissuring under load. This can be as much as 1/10 inch wide at 4 inch centers (16). An acceptable barrier is one which can be inspected prior to installation. Therefore, a tendon incorporating a plastic sheath, and grouted in place with a normal cement grout is regarded as a singly protected tendon overseas, but a doubly protected tendon in the U.S. The least protected part of the tendon defines the class of protection.

American engineers may argue, with a certain justification, that most dams are founded on "good", impermeable rock which is then further grouted, if necessary, prior to anchor installation. In short, the real danger of water percolating through possible microfissures in both rock mass and grout - and then finding a flaw in the plastic protection is generally regarded as a tolerable risk.

Within the last few years, attitudes toward long multistrand tendon protection have, nevertheless observed the following progression:

a) bare strand in bond zone, individual sheaths on the free length steel;
b) as a) except for a full length, outer "group" sheath of corrugated plastic (polypropylene or polyethylene);
c) epoxy coated strand (and two phase grouting);
d) epoxy coated strand, with individual sheaths in the free length, permitting one phase of grouting.

It should be noted, however, that there remain real and potential difficulties when using epoxy coated strand: for example, when the epoxy protection also occupies the space around the center ("king") wire of a seven-wire strand, load losses due to creep can be surprisingly high at steel stresses 70% GUTS and over.

In the current absence of a national policy towards corrosion protection, individual owners are responsible for specifying the degree of hole corrosion protection they want to pay for. In contrast, the need to efficiently protect the top anchor hardware - typically more at risk to atmospheric corrosion and mechanical damage - is more widely understood, and so more consistently effected. Indeed, there is a growing trend to not use the conventional top anchor hardware: after primary grouting and stressing, secondary grouting is conducted. However, in this case, the upper 20 feet or so of the free length is left uncoated and so the strand is bonded via the grout to the dam over this length. When the grout has set, the temporary top anchor is removed and the strands cropped off level with the dam crest (17).

STRESSING AND TESTING

The PTI Recommendations (1986) form the most common basis for conducting both the routine Proof Tests, and the more onerous Performance Tests. Load-extension data are recorded on the first load cycle, which often generates more anomalous information than if data were recorded only on the second cycle, after certain permanent movements had been eliminated (e.g., bedding in of head plate). Experience with long multistrand tendons (17, 6, 18) has led to the setting of the Alignment Load (AL) on individual strands using a monojack. In this way, AL, usually about 5% of the Design Working Load (DWL) is precisely placed on each strand: subsequent multijack loading is therefore conducted in the knowledge that each strand is accepting equal load and so no unforeseen overstressing will occur.

At DWL, tendon stresses are typically 50-60% GUTS while at Test Load (TL) tendon stresses over 80% GUTS are prohibited. Test safety factors are therefore at least 1.33, although rarely over 1.50.

The analysis of stressing data is also conducted according to PTI Recommendations (1986) and acceptability gauged by the relation of actual extensions to "control envelopes" generated by theoretical extensions of acceptable free lengths. On sites with very high quality rock, and where by far the greatest component of total tendon extension will be purely elastic, it is prudent to monitor wedge pull-in to further refine the apparent permanent movement component. This pull-in may be as much as one third of one inch at 80% GUTS. As an additional refinement, jack and structural movements may be monitored, but this is rare except in the case of thin, delicate structures (13).

As an extra aid to analyzing stressing data, it is becoming more common to cycle the load back to AL, after TL has been achieved, prior to raising it again to the final lock off load (typically 5-15% over DWL). This extra cycle provides a means of easily partitioning the elastic and permanent components of total tendon extension at TL. Analysis of the former, by reference to the relationship

$$\text{Extension} \propto \frac{\text{Load x Length}}{\text{Area x Elastic Modulus}}$$

will permit the amount of apparent tendon debonding to be calculated. This is extremely useful in evaluating basic anchor performance.

LONG TERM PERFORMANCE AND SPECIAL TESTING

In common with the rest of the world, few data are published on the long term performance of anchors in service. In the vast majority of cases, top anchors are concreted in, after final stressing, and are therefore unaccessible. In other cases, restressable heads or load cells have been incorporated, but the data, if monitored, are used for internal purposes, and never considered sufficiently interesting for publication. Likewise, structural monitoring of anchored dams is often conducted, but again rarely published. One may conclude, however, that no significant long term problems have been noted, with the load losses being, predictably and wholly, due to natural relaxation of the tendon. Against this silent background, the data from Stewart Mountain (13) are particularly useful, especially the confirmation that the gradual and uniform application of prestress along the crest causes no differential strains between adjacent construction blocks.

One encouraging trend is the willingness of more enlightened owners and consultants to sanction preproduction tests in advance of the main works. For example, the test at Lake Lynn Dam (7, 9) was conducted to establish ultimate grout/rock bond stresses, and to research time-dependent performance in a compressible, creep-susceptible sedimentary sequence. The latter data in particular, proved of great value in understanding otherwise unexpected phenomena during the stressing of the subsequent production anchors, and so defused a potentially confrontational situation. The tests done at Stewart Mountain Dam (8) contributed directly to that particular job's requirements, but also to the technology at large, so fundamental were their scope.

One hopes that similar tests will be encouraged - and the results published - in upcoming dam repairs of similar type.

FINAL REMARKS

Prestressed rock anchors have become a popular and reliable solution to many of the structural problems inherent in older concrete dams. In the United States, the scale and complexity of these problems has fostered the skill and experience of the dam remediation community to achieve an excellent international reputation. However, certain aspects such as attitudes towards corrosion protection and long term performance monitoring need still to be addressed in a more systematic fashion. These are challenges facing all of us involved in the technology, but through the spirit of partnering we have grounds for optimism that these challenges will be fruitfully fulfilled.

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THE USE OF JET GROUTING FOR UNDERPINNING AND TEMPORARY EXCAVATION SUPPORT OF A HISTORIC BUILDING

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ABSTRACT

Tuskegee University was founded in 1881 by George Washington Carver. Built on this campus in 1901, Dorothy Hall is a brick structure which in 1992 needed renovation. Moreover, the construction of an adjacent parking garage was also proposed. An excavation approximately fifteen feet deep was required along the northeast wing of the building. Subsurface investigation revealed loose sands and a high ground water level at the site. An in-situ ground modification process, jet grouting, was selected for both underpinning and temporary excavation support. Jet grouting involves the use of high pressure jets of air, water and grout to mix in-place soil with cement grout to create soilcrete. During the initial installation of the jet grout columns differential movements greater than anticipated were observed. Because of these movements, another ground modification process, compaction grouting, was used to confine the soils adjacent to the building so that jet grouting could continue. Subsequently, additional significant movements were not observed. This paper presents a case history of the temporary excavation support and underpinning of Dorothy Hall.

INTRODUCTION

Tuskegee University, formerly Tuskegee Institute, was founded by George Washington Carver in 1881. Dorothy Hall was constructed in 1901 under the direction of its architect, Robert R. Taylor. Tuskegee University's campus became a historic landmark in 1965. In 1974, Dorothy Hall, as part of the historic district, was identified as a National Historic Site by the National Park Service.

As Tuskegee University continues to upgrade its facilities, engineers have been faced with unique challenges associated with the restoration of a number of historic structures. In 1992, as part of this restoration process, renovation of Dorothy Hall was proposed along with the construction of an adjacent parking garage.

Before the recent construction and remodelling, the building contained "Women's Industries," and the

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University's guesthouse. After the current construction is completed, Dorothy Hall will house the Tuskegee University Kellogg Conference Center for continuing education and an expanded guesthouse.

Dorothy Hall is of brick construction, the bricks having been made by the Tuskegee staff and students. The three-story building is laidout in an "H" pattern and was founded on shallow, narrow, brick and mortar footings. During the renovation, an excavation approximately 15 feet deep was required along the northeast wing of Dorothy Hall. Subsurface investigation revealed loose sands and a high ground water table at the site. In addition, this portion of the building was very fragile with visible structural distress and a history of foundation movements. Because the structure could not withstand appreciable differential movements, an underpinning system was required that would minimize movements during load transfer. An in-situ ground modification process, jet grouting, was selected for both underpinning and temporary excavation support of the northeast wing of Dorothy Hall.

Jet Grouting. Jet grouting involves the use of high pressure jets to mix air, water and cement grout with in-place soils to create soilcrete. The resulting mass can be installed in a variety of geometries. Jet grouting can be used in a wide range of soils, from clays to clean sands and gravels.

There are three different types of jet grouting systems currently in use: single, double and triple-rod systems. The single and double rod systems employ a highly pressurized grout slurry to cut and mix the soil in-situ; these two systems are preferred for sands and gravels. The high percentage of soil remaining after jetting provides aggregate for the soilcrete. The double-rod system's efficiency is enhanced by the addition of an air sheath around the jet nozzle to facilitate cutting.

The triple-rod system differs in the use of a high velocity water jet sheathed in a cone of air to more effectively cut the soil and simultaneously tremie inject an enriched cement slurry to replace the soil. When silts and clays are present in the soil to be jetted, the triple-rod system is the preferred system due to its ability to exhaust the fine grained soils from the ground and replace them with a rich cement grout. For all three systems, the slow and uniform lifting of the drill rod typically creates a relatively uniform column of soilcrete.

Compaction Grouting. Compaction grouting is the injection of low-slump sand/cement grout (a slump generally less than two inches), under pressure to compact adjacent soils through displacement. The technique was originally developed in California during the 1940's to level and compact beneath homes. The use of compaction grouting was later expanded during the construction of the Baltimore subway system in the late 1970's. During soft ground tunnelling, compaction grout was injected above the advancing tunnel to densify soil loosened during excavation, thereby avoiding settlement of overlying structures.

SYSTEM DESIGN

Subsurface Conditions. A geotechnical study for the site had been performed during the fall of 1991. As part of the study, one boring and a test pit had been made in the immediate vicinity of the proposed excavation. The boring was advanced approximately 25 feet through predominately sandy soil prior to termination in a hard silt/clay layer. The density of the sandy soil generally increased with depth. However, two intervals of "Weight of Tools" (i.e. very loose) material were encountered. Ground water was also encountered at a depth of about eight feet below the ground surface.

Proposed Underpinning/Excavation Support Geometry. Figure 1, on the following page, is a plan view which shows the outline of the northeast wing of Dorothy Hall and the location of the individual soilcrete columns. The support system was formed in-situ by overlapping three to 3.5 feet diameter columns of soilcrete. The 28-day compressive strength of the soilcrete was specified as 500 pounds per square inch (psi). The soilcrete wall shown was used for both underpinning of the footings and temporary excavation support. The individual soilcrete columns shown in the interior of the building were used for underpinning of internal walls and columns.

The wall was designed to resist overturning, sliding and shearing forces; bearing capacity and settlements were examined for the wall and for the underpinning columns. Overturning of the exposed wall and settlement during load transfer were expected to be the most critical conditions. To resist overturning, battered columns of soilcrete were proposed along the main face. Figure 2 shows a profile of the battered wall section. Approximately one centimeter of settlement was expected for the wall, and about two centimeters were expected for the underpinning columns.

Monitoring Plan. Crack monitors, or "tell-tales" were installed on existing structural cracks prior to the start of construction. In addition, fifteen vertical movement monitoring points were installed on the building. The locations of the fifteen points are also shown on Figure 1. Monitoring was to be performed twice daily during construction (a fax machine was set up on site, and the monitoring results sent to all concerned parties).

CONSTRUCTION ACTIVITIES

Photograph 1 shows the site prior to the start of jet grouting. During site preparation the upper interval of soil was removed to the level of the working grade. At the completion of this excavation, the ground water level was found to be slightly lower than the level found during exploration.

During installation of the test columns, and during drilling operations to install the jet grout columns, the underlying layer of hard silt/clay which had been previously detected during exploration was found to be an average of about three feet higher than assumed during the design. Moreover, based on the test section, the average diameter of the soilcrete in this interval was greatly reduced and highly irregular. Figure 3 shows an idealization of the reduced section. As a result, any soilcrete mass below the sand-clay interface was neglected in the design relative to stability of the wall. To compensate for this, the batter angle of the columns was increased to provide additional mass at the base of the wall.

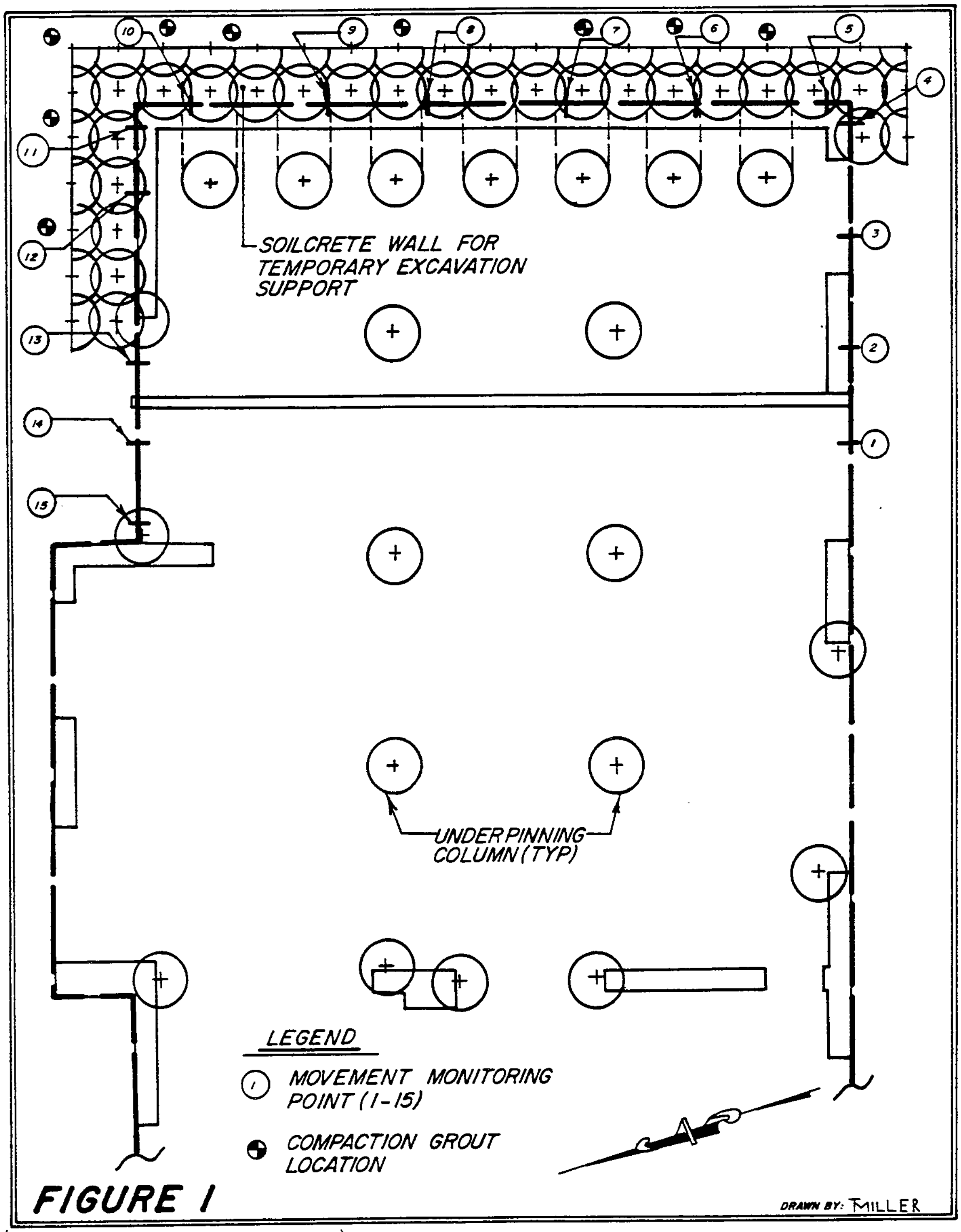

Figure 1 - Plan view of the northeast wing of Dorothy Hall.

Jet grouting activities began on 31 July with the installation of two columns beneath the existing wall. Settlements larger than anticipated were observed during installation of the first column. Figure 4 shows the vertical movement monitoring versus time during construction. Two additional columns within the wall were installed each day on 3, 4, and 5 August with no indication of the rate of settlement slowing. Settlements at this point were approaching four centimeters at the northeast corner of the structure. During this time, the use of jet grouting was being reevaluated, as were other methods which could be used to mitigate future movements.

The sequence of installation for adjacent soilcrete columns is very important when attempting to minimize structural movements. Even the installation of a single 3.5 feet diameter column assumes the surrounding soil can arch around the weak zone created by the wet grout. In addition, it is typically assumed that the structure being underpinned can span the width of one column where a temporary loss of support occurs. The excessive movements occurred at Dorothy Hall because the soils surrounding the work area were extremely loose/soft and could not effectively arch, and Dorothy Hall itself, was too fragile to span any significant distance. Therefore, compaction grouting was selected as a mitigative measure to densify and confine the surrounding soils.

The grout mix for the compaction grout consisted of fly ash and sand with no cement. The intent was to place compaction grout that could be later jetted. Compaction grouting was performed at nine locations around the outer perimeter of proposed wall as shown on Figure 1. Approximately 6.9 cubic meters of grout was injected into the first four compaction grout holes on 11 August. About 3.8 cubic meters of grout was injected in three holes on 12 August, and an additional 3.8 cubic meters of grout was injected in two holes on 14 August. Jet grouting operations were resumed on 15 August, and were completed on 29 August. Subsequent to the compaction grout injections only an additional 11 millimeters of settlement were recorded.

The 28-day compressive strength of soilcrete samples was found to average about 1800 psi and vary widely. The minimum 28-day strength for all tested samples was 710 psi, about 40 percent greater than the design value of 500 psi.

Photograph 2 shows the excavated face of the soilcrete wall supporting the northeast wing of Dorothy Hall.

CONCLUSIONS

Despite the problems encountered during construction, the project was completed successfully. Lessons learned from the experience include:

1) Intervals of loose/soft soil are zones of weakness into which grout can move.

2) A fresh jet grout column is a hole, albeit filled with wet grout.

3) The largest potential for movements when using jet grouting techniques to underpin structures and provide excavation support is not necessarily during final

excavation, but can occur during column installation. Therefore, proper sequencing of column installation is critical to reducing movements.

4) Subsurface conditions can change with time, and are often highly variable even over short distances.

5) There is no substitute for timely and accurate monitoring data during construction.

6) Teamwork and good communication between parties are essential when troubleshooting problems encountered during construction.

The underpinning of Dorothy Hall was a challenging engineering project which highlighted the difficulties associated with the restoration of aging infrastructure.

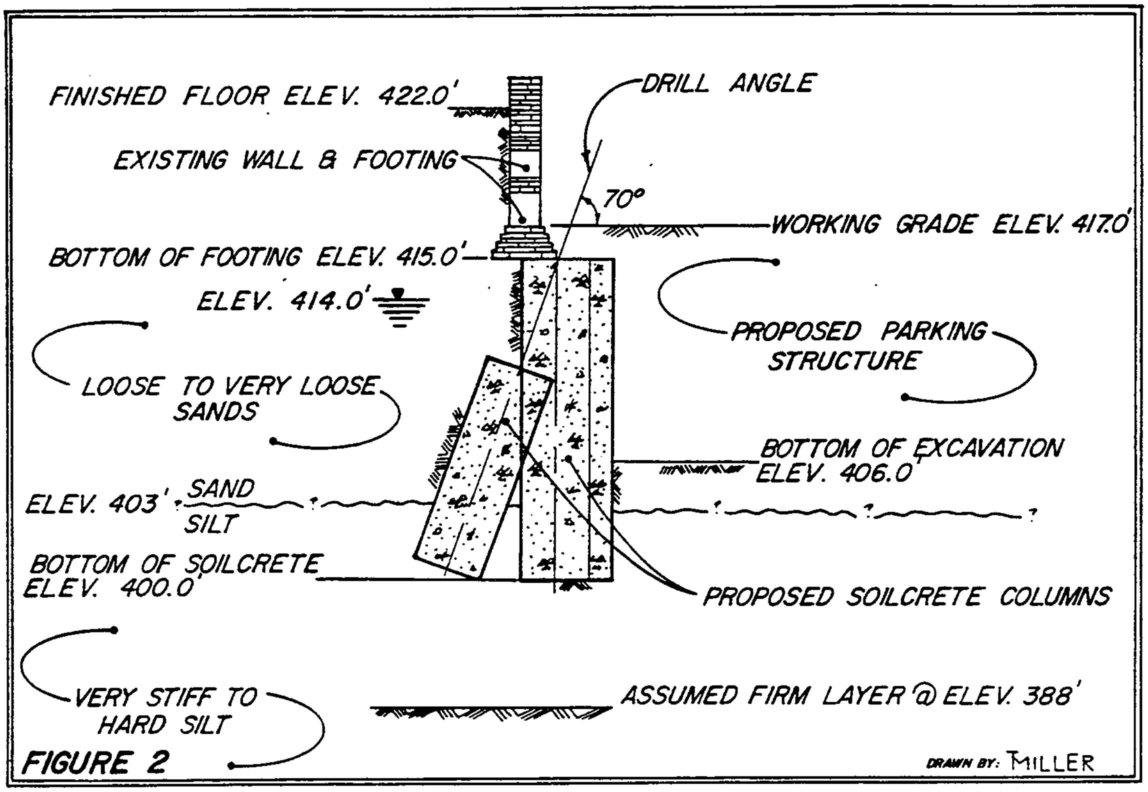

Figure 2 - Initial cross-section of the battered soilcrete wall.

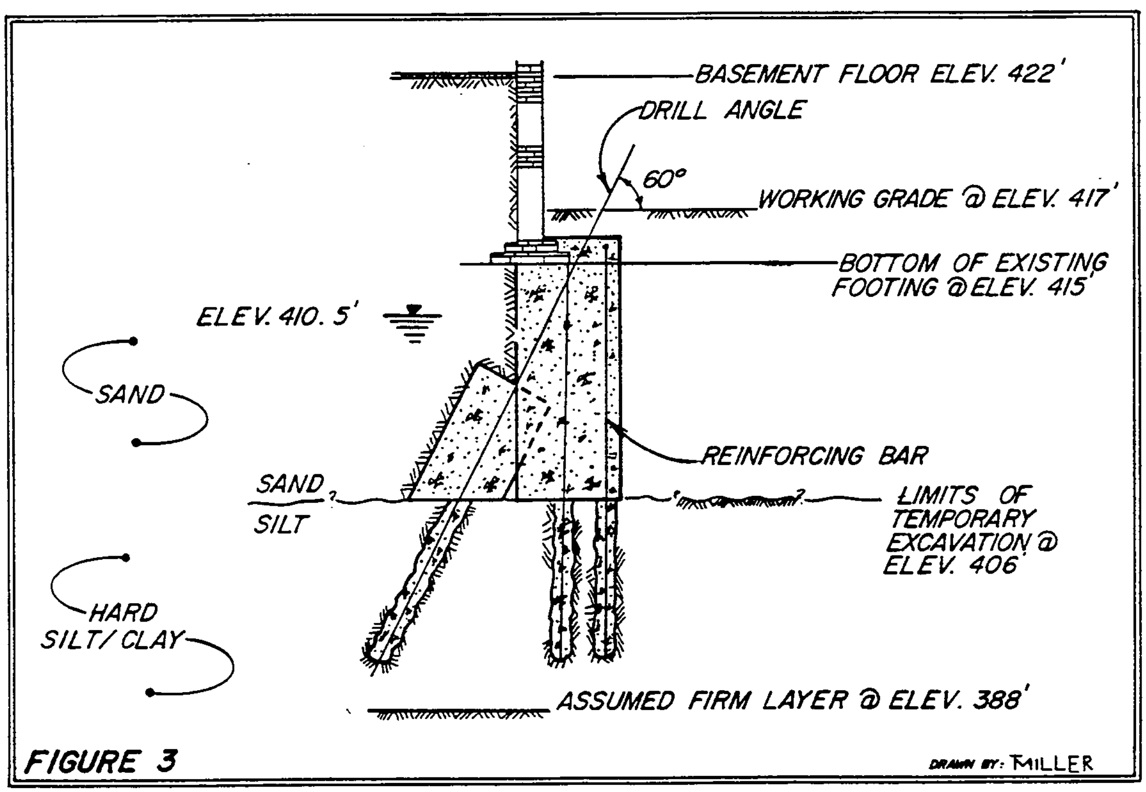

Figure 3 - Modified cross-section of the battered soilcrete wall.

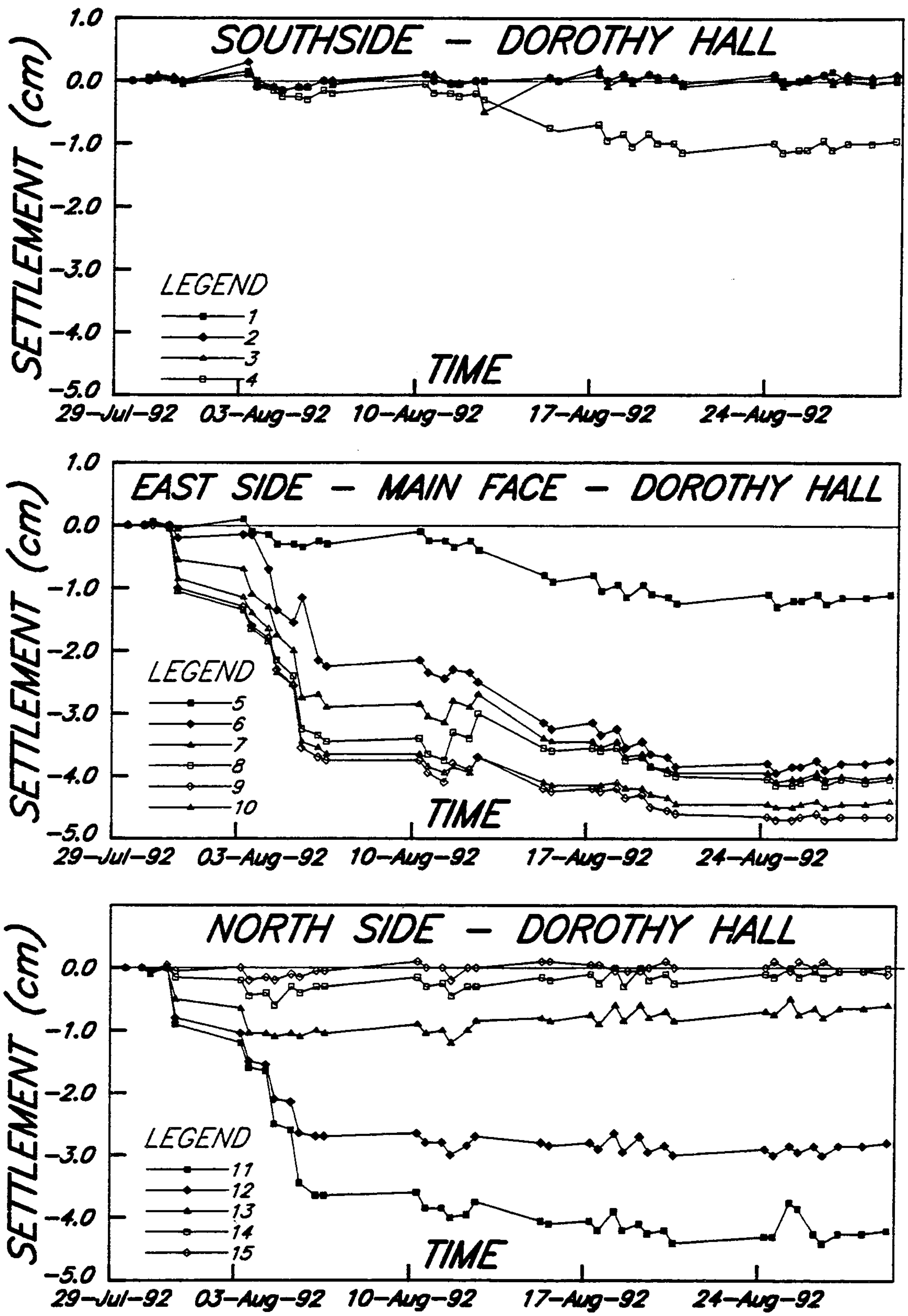

Figure 4 - Vertical movement monitoring results versus time.

Photograph 1 - The northeast wing of Dorothy Hall prior to the start of jet grouting.

Photograph 2 - Excavation completed, the soilcrete wall is exposed.

GEOTECHNICAL ASPECTS OF BRIDGE RECONSTRUCTION

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ABSTRACT

Despite economic and budget constraints, there has been a growing awareness of the necessity for infrastructure rehabilitation and reconstruction. This need has become more critical in recent years due to the aging of existing structures and increased complexity of new structures. Past experience has shown that the concept of "deferred maintenance and reconstruction" is both wasteful and potentially hazardous to the public safety.

Bridge rehabilitation or reconstruction is generally comprised of modifications, additions or total replacement of the existing structure. Some of the geotechnical aspects of bridge reconstruction may include:

- Review of available geotechnical data, subsurface soil investigation and related laboratory tests.
- Define geological profiles along the bridge alignment and at other critical structural components.
- Establish design parameters for geotechnical analysis and structural study.
- Evaluation and possible reuse of existing foundations.
- Strengthening, reinforcing or replacing the existing foundations.
- Selection of optimum foundation system for bridge replacement.
- Design and protection of adjacent structures.
- Minimize vibration effect on nearby structures during construction.
- Foundations for integral abutments.

This paper describes case histories in the New York City area which briefly illustrate the geotechnical aspects of some recent bridge reconstruction.

INTRODUCTION

The Bureau of Bridges plays a vital role in the New York City Department of Transportation's (NYC DOT)mission to ensure the safe and efficient movement of people and goods throughout the City of New York. The bridge structures under the Bureau's responsibility are essential to the vast and diverse network of roadways throughout the city. The City which is famous for its Brooklyn, Manhattan, Williamsburg and Queensboro bridges spanning the East River, has 10 percent of its bridges cross over water. The majority of the structures comprise of overpasses, elevated highways and pedestrian walkways spanning the city's labyrinth system of railroad and subway tracks, the arterial highways and local streets. The intensive program of reconstruction undertaken by the NYC DOT will ensure the continued usage of these critical links in the City's infrastructure well into the 21st century.

GEOLOGY

Geology plays an important role in the planning and construction of structures. What lies beneath the surface of the ground is of vital importance to the designer and builder. New York City's five boroughs cover three physiographic units: the Coastal Plain, New England Upland and Triassic Lowland. They contain nine different foundation rock types and dozens of soils as shown in figure-1. The foundation bearing material available range from high-strength gneissiod or granite to soluble marble to soft-sensitive, low shear strength, and high moisture content, organic

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silty clay having very limited bearing capacity. Each rock and soil type has its own engineering characteristics and, in addition, local problems exist within each major type.

- Replacement of the superstructure
- Widening the superstructure.
- Upgrading the load carrying capacity of the superstructure

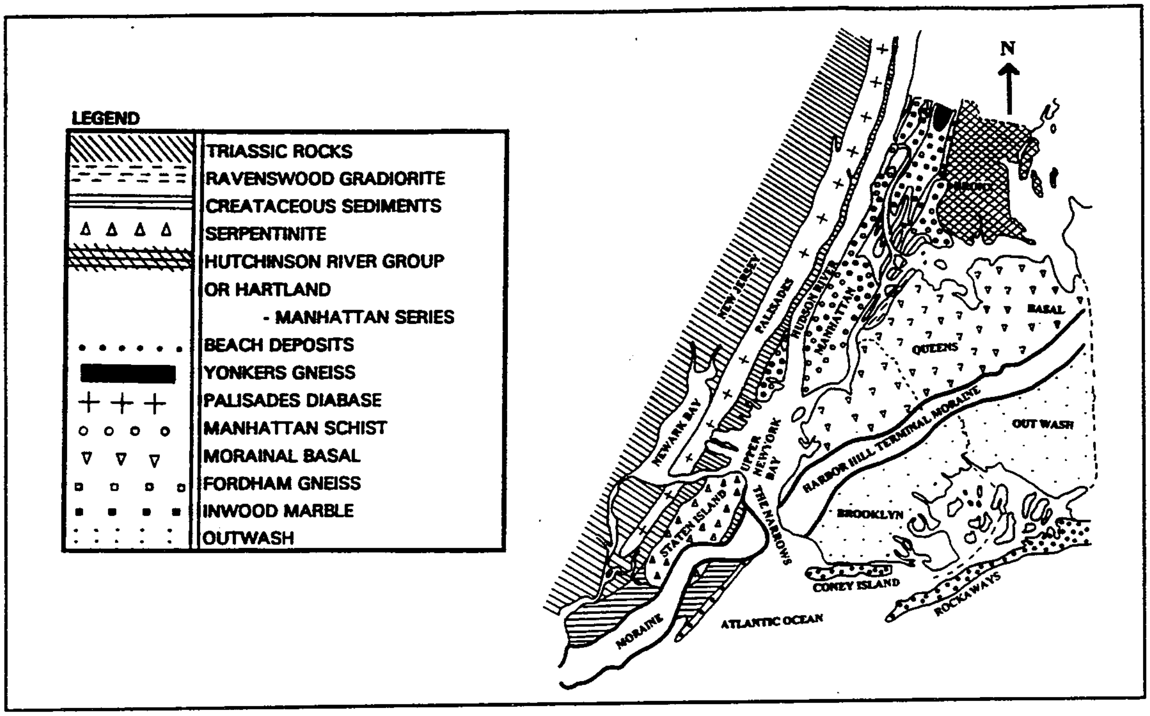

Figure - 1: Geological Map of New York City

The complexity of the geology and potential foundation problems require that the design and construction of foundations must conform to the City's comprehensive building code. The bridge structures in New York City are also to conform with the requirements of "Standard Specifications of Highway Bridges" prepared by The New York State Department of Transportation. The codes have classified soil and rock into various categories based on their geological history and engineering characteristics. For any structure, <u>accurate soil and rock classification by the geotechnical engineer is essential for foundation design.</u>

CASE HISTORY NO 1: USE OF EXISTING FOUNDATIONS

Evaluations of existing foundations are required when investigating the alternatives proposed for reconstruction of a bridge. The existing foundations could be reused in the following cases:

Often, original bridge drawings and pile installation records are no longer available thereby, complicating the evaluation of existing foundations.

On a bridge project located in the Borough of Staten Island, one of the reconstruction alternatives called for replacement of entire structure except for its foundation piles. The bridge was constructed in 1932 and comprised of five spans supported by two spill-through abutments and four reinforced concrete frame bents. The existing piles to be retained for reuse are precast concrete piles. However, no data was available regarding subsurface conditions, the length, driving records or capacity of the piles. It was determined that the following tasks should be performed to obtain the relevant data and to assess the adequacy of existing pile foundations.

1. Calculate the structural loads imposed on the piles by the existing and proposed structure and determine the percentage of change in loading experienced by the piles.

2. Define the subsurface conditions along the bridge alignment.
3. Estimate the length of the piles so that their geotechnical and structural capacity can be approximated.
4. Review and assess the bridge inspection findings for any deficiency or distress observed which could be attributed to the existing substructure.
5. Check the applicable codes to ensure that the use of existing foundations meets the specified criteria. Additional analysis may be required to satisfy the applicable seismic requirements of the codes.
6. Analyze the information gathered in the above items and determine the need for pile testing and strengthening the existing foundations.
7. Estimate the cost savings in using existing foundations. This item perhaps may become a critical factor in the decision process.
8. Review the overall advantages and disadvantages of using existing or new foundations on the project as a whole.

Results of the analysis performed in accordance with the criteria established above are as follows:

It was determined that the piles will be subjected up to additional ten percent of the existing load. If the scheme selected imposes load on the pile in excess of ten percent of the existing load, the piles are recommended to be tested to confirm their capacity and additional piles be added, if required. However if the additional load on the pile is less than ten percent of the existing load, the need for load tests is evaluated based on available subsurface and pile data and past performance of the substructure.

The subsurface conditions beneath the structure were explored by taking borings which are depicted in the form of geologic profile in figure-2. The profile indicates that the top of competent soil starts at relatively shallow depth (approximately elevation - 30) which seems to suggest that the pile tips may be in this stratum.

Methods to determine the pile lengths beneath the existing structure were explored. It appears that there are non-destructive methods available which can estimate the pile lengths. The reliability of these methods and suitability at this site require further study.

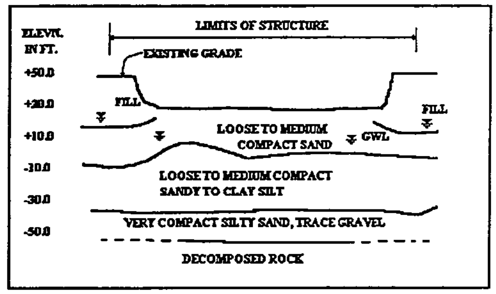

Figure - 2: Geological Profile

As indicated above, although the proposed scheme will load the piles between 8 to 10 percent above the existing loads, NYC DOT's consultant recommended that the representative piles should be load tested prior to its use to eliminate uncertainties in the existing pile design and its installation.

There are static and dynamic methods available to test the existing piles. The use of dynamic measurements to predict pile ultimate capacity appears to be the preferred alternative to static load tests, due to the following reasons:

1. In situations where there is a relaxation or rebound, the ultimate bearing capacity of the piles can best be evaluated by dynamic method of analysis for the various degree of relaxation.
2. Dynamic testing can be used to test many piles in one day, whereas the conventional method of static load test can obtain information for only one pile after a test period of 1 to 2 days.
3. Dynamic analysis can be utilized in situations where different pile sizes were used to carry different design loads.

<u>CASE HISTORY NO 2: MINIMIZE VIBRATION EFFECTS TO THE ADJACENT STRUCTURE</u>

Construction in a high density urban area like New York City presents unique foundation problems due to the proximity of adjacent structures, underground utilities, subway systems etc. and protection of these structures are essential to safely carry out the new construction. Generally, the support for heavy structures like bridges require the use of piles supported foundations. Pile driving generates waves that propagate through the strata and can have detrimental effects on cohesionless soils that support structures. Various criteria have been proposed to describe the safe limits of vibrations for structures and soils, and the criterion most generally accepted is peak particle velocity. Frequently, a maximum peak particle velocity of 2 inches per second is specified as a criterion to prevent structural damage. Four inches per second is the threshold of actual damage. However, densification of loose cohesionless soils by pile driving is often more damaging than vibrations transmitted to the structure itself. Settlement from pile driving in loose-to-medium compact, narrowly graded sands can result from peak particle velocity much less than the 2 inch per second. There are sites where peak particle velocity as low as 0.1 to 0.2 inches per second measured on the ground surface accompanied significant settlement. The alternate method for installing deep foundations that eliminates vibration problems is to select drilled-in foundations. The use of drilled shafts (also known as bored piles, drilled piers, and caissons) can normally eliminate possible harmful vibrations during construction.

In Brooklyn, a vehicular bridge scheduled to be replaced exists adjacent to an elevated structure owned by Transit Authority (TA). The TA structure supports four tracks and a passenger station. It was constructed in 1930's adjacent to and over the existing vehicular bridge. The proposed vehicular bridge will be constructed in close proximity to TA structure. The consultant for the NYC DOT recommended the use of high capacity drilled-in caissons and low capacity minipiles for this structure in order to minimize the effect of vibration on the nearby elevated TA structure. The subsurface conditions in the vicinity of piers for this project are shown in figure-3.

The soil conditions at the site are primarily the result of glacial activity. The overall site stratigraphy may be briefly described as a sequence of dense granular, glacial soils over bedrock, overlain by recent organic silts, clays, and fill. It is evident from the geological profile that short driven piles could support the pier loads. However, the consultant recommended drilled shafts socketed into rock to minimize the effects of vibrations on the foundations of the adjacent structure. Drilled shafts have found increasing acceptance in the engineering community. The design of drilled shaft is intimately tied to construction procedure and the quality of inspection.

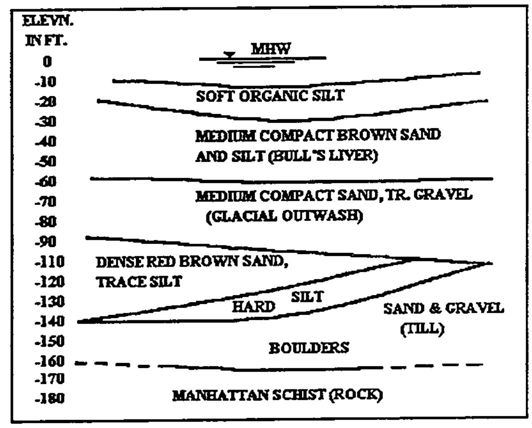

Figure - 3: Geological Profile

The mini piles proposed for the project are small diameter, low to medium capacity, drilled and grouted piles. These piles appear to be ideal for bridge foundations with difficult access, restricted vertical clearance, poor ground conditions, or sensitive surroundings. Due to their diverse and flexible capabilities, the use of mini piles has significantly increased in recent years.

INTEGRAL ABUTMENTS

A conventional abutment is the substructure which supports the end of a single span or the extreme end of a multispan superstructure. In addition, the abutment serves as a retaining wall and controls the soil adjacent to the superstructure at the end of the bridge. The abutments are classified according to height (Stub or Semi-stub), shape (straight or L shaped, shoulder or cellular) and structural behavior (Cantilever, Gravity, Counterforted).

An Integral Abutment is a straight, stub abutment cast monolithically with supporting piles and the deck. The integral abutment bridge concept is based on the theory that due to the flexibility of the piling, thermal stresses are transferred to the substructure by way of a rigid connection between the superstructure and substructure. The piles of an integral abutment are positioned in a single row to facilitate flexibility. While Steel H piles are preferred due to their flexibility, an alternate pile type can be incorporated into this scheme. The piles are installed by inserting into preaugured holes which extend approximately eight feet below the bottom of the stem of the abutment. The sand filled preaugured holes allow the flexibility required in the piles to accommodate the movement expected in the structure due to thermal changes.

Prior to abutment construction, the piles must be connected to the stringers supporting the deck slab. The approach slab rests on the abutment backwall and dowels provide for continuity of the superstructure with approach slab. Finally, the deck, abutment stem and approach slab are cast, thus creating a jointless bridge.

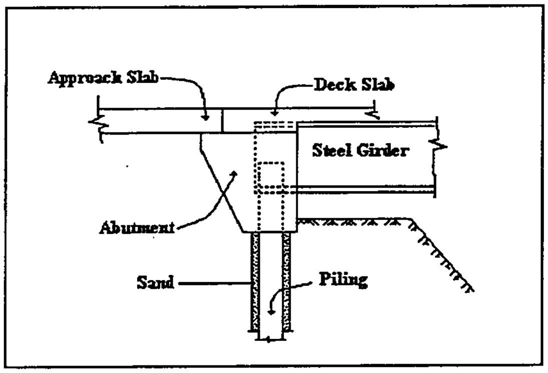

Figure - 4: Integral Abutment

A bridge with conventional abutments has joints between its deck and the substructure. Although several types of bridge deck joints are commercially available, they are costly to construct and maintain. The deck joints are the major source of deterioration of all bridge components adjacent to and under these joints. Any design which can eliminate the deck joints is looked upon favorably by the NYC DOT.

The NYC DOT plans to use bridges with integral abutments at several locations in Brooklyn for bridges carrying streets over subway. The existing bridges at these locations are four or six span concrete arch frame. The expected benefits of this scheme are:

1. The new integral abutments, being of shallower depth, require less excavation, thus minimizing adverse effect on the adjacent structures.

2. Generally, less number of piles are required for integral abutments resulting in overall cost reduction.

3. The single span construction eliminates the need for intermediate piers which improves the substandard horizontal clearance to the subway below.

4. The integral abutment will be constructed behind the existing Abutment. The scheme requires partial removal of existing abutment. The vital control cables belonging to TA for transit operations are located in the ducts housed within the existing abutment. As these control cables will not be disturbed during construction, the scheme will cause minimum disturbance to transit operations.

5. Staged construction is accomplished more effectively.

CONCLUSIONS

The intent of this paper has been to share some of the substructure related experiences on bridge rehabilitation projects in an urban setting. The cost of replacing bridge can be significantly reduced if the substructure, particularly the foundations, of the original bridge can be reused. Perhaps, the reuse of old bridge foundations will increase in future as more bridges are replaced and funding gets tighter. Reliable testing methods and definite guidelines for reusing existing bridge foundations will be required. Drilled-in foundations and integral abutments are useful foundation systems to protect nearby structures from construction associated vibrations and to facilitate construction in congested environment.

ACKNOWLEDGMENT

The authors thank Mr. Peter Pizzuco, P.E.; First Assistant Commissioner for Design, Bureau of Bridges, NYC DOT for providing useful suggestions during the preparation of this paper.

APPENDIX I

APPENDIX II

APPENDIX - II

PAST PROCEEDINGS OF OHIO RIVER VALLEY SOILS SEMINAR

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

ORVSS	XVII	NATURAL SLOPE STABILITY AND INSTRUMENTATION, October 17, 1986, Clarksville, Indiana
ORVSS	XVIII	LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION, November 6, 1987, Fort Mitchell, Kentucky
ORVSS	XIX	CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES, October 21, 1988, Lexington, Kentucky
ORVSS	XX	CONSTRUCTION IN AND ON ROCK, October 27, 1989, Louisville, Kentucky
ORVSS	XXI	ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING, October 26, 1990, Cincinnati, Ohio
ORVSS	XXII	DESIGN AND CONSTRUCTION WITH GEOSYNTHETICS, October 18, 1991, Lexington, Kentucky
ORVSS	XXIII	IN SITU SOIL MODIFICATION, October 16, 1992, Louisville, Kentucky