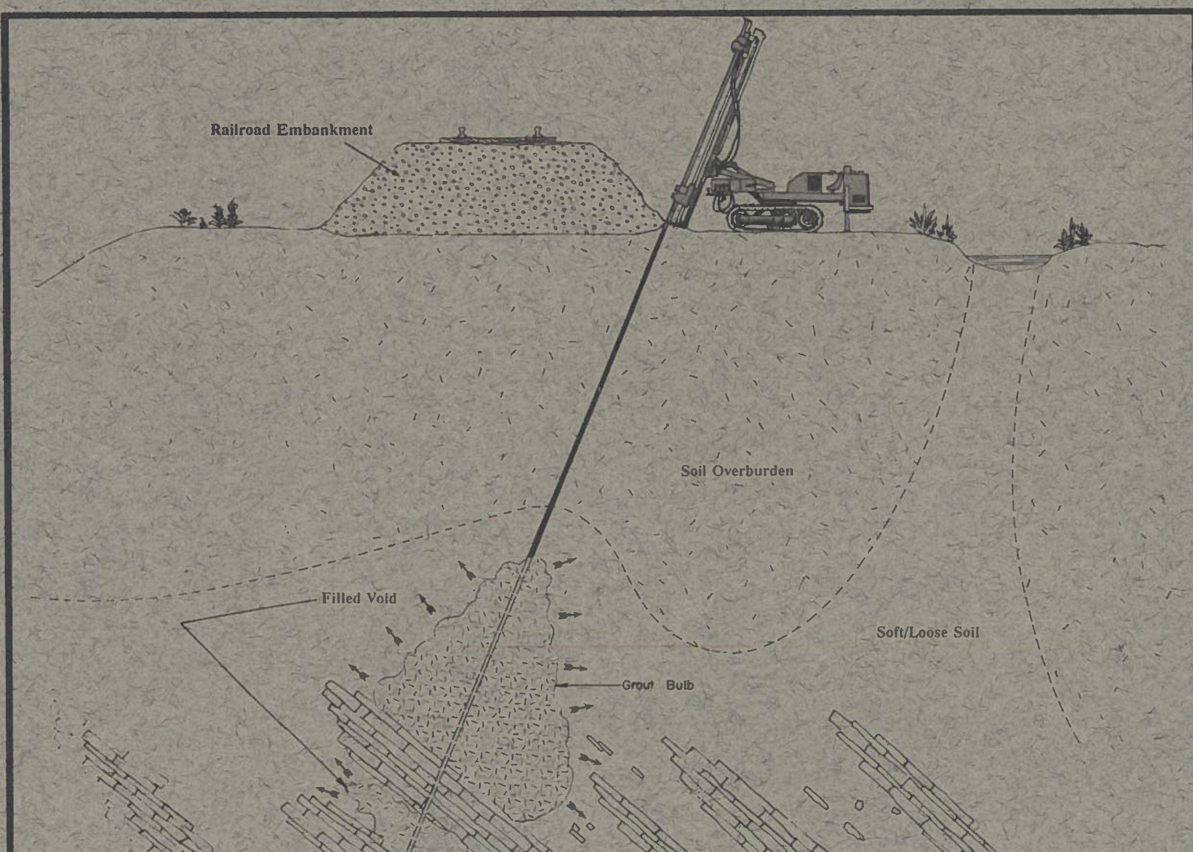
ORS XXIII

IN-SITU SOIL MODIFICATION



Louisville, Kentucky
October 16, 1992

ORVSS XXIII AGENDA
October 16, 1992

8:50 am	**Welcome and opening remarks:** Keith D. Coombs

MORNING SESSION
Daryl J. Greer, Moderator

9:00 am	**KEYNOTE ADDRESS: In-Situ Ground Modification for New and Remediation Construction,** by Joseph P. Welsh
10:00 am	**Construction of an Apartment Complex Over a Reclaimed Quarry: Arbors of Watermark, Columbus, Ohio,** by Stephen C. Pasternack and Daniel G. Longo
10:30 am	**BREAK AND EXHIBITORS DISPLAYS**
10:50 am	**Foundation Stabilization for Salt River Bridge, Fort Knox, Kentucky,** by John P. Jent
11:20 am	**Sand Drain Induced Consolidation of a Peat,** by E. Gregory McNulty
11:50 am	**Soil Mixing for Soil Improvement - Two Case Studies,** by Christopher R. Ryan
12:20 pm	**LUNCH AND EXHIBITORS DISPLAYS**
1:50 pm	**AFTERNOON ANNOUNCEMENTS**

AFTERNOON SESSION
Ted Vogelpohl, Moderator

2:00 pm	**KEYNOTE ADDRESS: New Horizons in Ground Anchorages, Pinpiles, and Cement Grouting,** by Donald A. Bruce
3:00 pm	**Foundation Stabilization System Using Jet Grouting Techniques,** by George K. Burke and Gary T. Brill
3:30 pm	**BREAK AND EXHIBITORS DISPLAYS**
3:50 pm	**Application of Lime - Fly Ash Injection in Runway Rehabilitation,** by Donald V. Manley and Arthur D. Pengelly
4:20 pm	**The Use of Compaction Grouting to Remediate a Railroad Embankment in a Karst Environment,** by Gary T.Brill and James Drew Hussin
4:50 pm	**CLOSING COMMENTS**
5:00 pm	**SOCIAL HOUR AND LIGHT DINNER:** ASCE Student Chapter University of Louisville

PROCEEDINGS OF THE TWENTY-THIRD
OHIO RIVER VALLEY SOILS SEMINAR

IN-SITU SOIL MODIFICATION

October 16, 1992
Holiday Inn - Hurstbourne
Louisville, Kentucky

Sponsored by

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"IN-SITU GROUND MODIFICATION FOR NEW AND REMEDIATION CONSTRUCTION"

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ABSTRACT

Although Ground Modification is a relatively new field of geotechnical engineering and construction, it is experiencing rapid growth due to its capabilities of economically solving problems in new, remedial and remediation construction. Technologies of Ground Modification are also emerging to solve problems in environmental applications. The paper will update the status of some of these Ground Modification techniques.

INTRODUCTION

An effort will be made to reflect on where we were, where we are, and predictions as to where we will be in the field of Ground Modification. Due to the space limitations, frequent references are cited so that interested professionals can delve into areas of interest when the need arises. From a historical view point on Soil Improvement and/or Ground Modification, it is suggested to read the following:

a) "Soil Improvement - History, Capabilities and Outlook", a report by the Committee on Placement and Improvement of Soils of the ASCE's Geotechnical Engineering Division published in February, 1978, based on presentations made at the ASCE Annual Convention in Philadelphia in the Fall of 1976 coinciding with the Nation's Bicentennial. This publication vividly shows how little soil improvement work had been accomplished up to that time; but, made the prediction that the projected growth in this area would be high, but tentative.

b) "Soil Improvement - State of the Art Report" by J. K. Mitchell, as published in the Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering held in Stockholm, Sweden in 1981 was a masterpiece in summing up the soil improvement industry just over ten (10) years ago.

c) "Construction Considerations for Ground Modification" by J.P. Welsh from the International Conference on Deep Foundations, Beijing, China, September 1986. This publication presented a table entitled "Assessment of Ground Modification Construction in the United States" and suggests that there are six (6) basic methods of Ground Modification;

adhesion, densification, reinforcement, excavation-replacement, physical chemical alterations, and biological transformation. The former four (4) consist of mainly maturing methods, while the latter two (2) consist of methods that are emerging and will be developed primarily to solve in-situ environmental problems.

d) "Soil Improvement - A Ten Year Update", the Proceedings of a Symposium sponsored by the Committee on Placement and Improvement of Soils, Geotechnical Engineering Division, ASCE, April 1987, which give a committee report on the major areas of Soil Improvement: In-Situ Reinforcement, Densification, Reinforcement of Constructed Earth and Chemical Admixtures and Miscellaneous Methods. This publication also included ten (10) major case histories of Soil Improvement that ensued between 1976 and 1986. This publication made no predictions as to possible future growth and gave little indication as to projected new development, particularly in the area of Environmental Geotechnology.

All of these publications are focused on the design, construction, and evaluation of Ground Modification techniques for construction applications, where the driving force is benefit to cost ratio. Consulting Geotechnical Engineers who have elected to add environmental applications to their repertoire, soon find that over fifty percent (50%) of their billings are in this specialty area. Future articles on Ground Modification will emphasize the emerging role of the techniques in Environmental Geotechnical Engineering where the driving force is regulatory. The ASCE Geotechnical Engineering Division's Specialty Conference "Grouting, Soil Improvement and Geosynthetics" held in New Orleans, Louisiana, February 1992 had, as one (1) of the four (4) keynote lectures, "The Role of Soil Modification in Environmental Engineering Applications" by Professor James K. Mitchell and W. Van Court, University of California, Berkeley.

The remainder of this paper will focus on new developments in specific methods of Ground Modification for new, remedial and remediation construction and environmental applications and some areas of research needs.

Dynamic Deep Compaction (DDC)

Dynamic Deep Compaction can be defined as a densification of soil deposits by means of repeatedly dropping a heavy weight onto the ground surface (Figure 1). Commercially emerging in the United States in the early 1970's, this system has made rapid progress due to its relatively economical cost. Lukas' 1986 Federal Highway publication, "Dynamic Compaction for Highway Construction" is the major guideline for this technique. It covers the advantages and disadvantages of this method of densification, suitability of deposits for Dynamic Compaction, gives anticipated degree of improvement, applied energy requirement, methods to monitor the improvement, contracts,

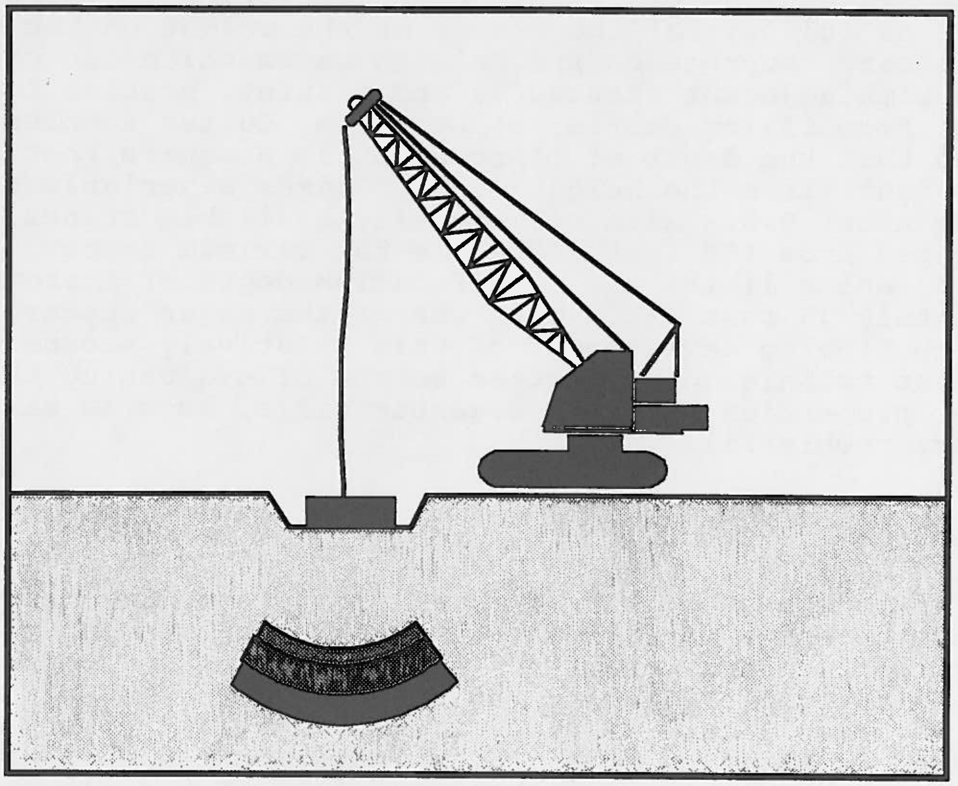

Figure 1: Dynamic Deep Compaction (DDC)

specifications, etc. Since this publication, a new utilization is the densification of sanitary landfills to obtain additional capacity out of landfills to extend the life of the landfills and give more uniformity to prevent differential settlement for the closure cap. In certain situations, the additional capacity obtained from DDC more than paid for the densification program. Galante (1991), reported on a trial at a landfill near Philadelphia, PA, that DDC produced a gain in capacity of about 8% over traditional compaction methods. The author noted that, if DDC was a scheduled part of the landfill operation, an air space gain of 15% would be anticipated. Dynamic Deep Compaction is currently applicable to solve the following problems:

1. Reduction of foundation settlement due to loose or low density soils
2. Protecting soil from liquefaction in seismic zones
3. Permitting construction on fills
4. Densifying sanitary landfills for highway construction
5. Densifying mine spoils
6. Pre-treatment of potentially collapsible soils
7. Increasing the capacity of sanitary landfills

The major limitations of Dynamic Deep Compaction in improving soils are as follow: a) the energy of the weight on the ground creates shear, compression and Raleigh waves which can cause problems with adjacent structures and a safety problem to personnel from flying debris; b) Leonards, Cutter & Holtz (1980) indicated that the depth of improvement is a square root function of the weight times the height of drop times a variable factor averaging about 0.5. With commercially available cranes, twenty tons dropped from 100 feet (30 m) is the maximum energy available, which limits the densification depth of improvement to approximately 35 feet (10.7 m). One of the major research needs for the continuing development of this relatively economical soil improvement technique is a better method of evaluating the before and after properties of non-homogenous fills, such as sanitary landfills, rubble fills, etc.

Vibroflotation

Depth vibrators have been used for over 50 years to improve loose, non-cohesive sands. The utilization of a depth vibrator for densification and compaction of loose sands was developed in Germany in the mid 1930's and was introduced to the United States in Cape May, New Jersey in 1948, Brown (1977). The vibrations are induced by rotating eccentric weights mounted on a shaft and driven by a motor located in the upper part of the vibratory casing. The soils are subject to accelerations of up to 3 g's, which causes liquefaction in the loose sands and improvement in the relative density as high as 85%. Sand can be fed down around the vibrator to make up for the densification of the in-place sand, or an entire site can be dropped in elevation. The typical vibrator is between 10-15 ft long (3 m - 4.5 m) and weighs approximately seven (7) tons (1,814 kg). Depending upon the nature of the soil, the diameter of influence of the vibrator is up to 14 ft (4.5 m); however, depending upon the degree of relative density improvement required, the normal probes are spaced on a triangular pattern 6-12 ft (1.85 - 3.7 m) on center. Utilizing this technique, sands have been improved to depths of 115 ft (35 m). The major limitation of the vibroflotation system is the grain size of the soil; only sands can be densified and the efficiency of the technique decreases as the cohesion increases. The normal limitation is a maximum of twelve percent (12%) silt and clay, with the maximum clay fraction being less than three percent (3%).

Stone Columns (Vibro Replacement, Vibro Displacement)

In order to expand the capabilities of the vibratory probe in improving the finer grained soils, the stone column concept was developed in the late 1950's. Where stratas of cohesive soils were encountered, these soils were removed by the jetting action of the vibratory probe and replaced by stone as the backfill material in lieu of sand. Thus, the columns of stone replaced the cohesive soils and any granular soil in the stratification were also densified (Barksdale & Bachus 1983). The next major development was the dry vibro displacement technique; instead of jetting the vibrator in place and feeding the stone around the perimeter of the vibrator, the stone was installed to the tip of the vibrator through an auxiliary pipe attached to the vibrator. The soil is displaced rather than replaced, and normally smaller diameter elements are formed by the vibro displacement system. However, the lack of jetting water has minimized the fines laden water causing an environmental problem. The first use of the vibro-displacement method in the United States was for the Steel Creek Dam Foundation at the Savannah River Plant in South Carolina (Keller, Castro, and Rogers, 1987, and Dobson, 1987).

The ASTM publication "Deep Foundation Improvement: Design, Construction and Testing" contains twenty two papers, the majority of which deal with current technology in stone columns and are the proceedings from a Symposium sponsored by ASTM Committee D-18 on Soil and Rock presented in Las Vegas, Nevada, January 25, 1990. This publication should be a part of every geotechnical engineer's library.

Juan Baez, under the tutelage of Geoffrey R. Martin, Professor, University of Southern California, is investigating how the vibrator causes liquefaction in the ground and if the subsequent densification will prevent liquefaction in case the earthquakes of design occurs.

Mitchell and Wentz (1991) University of California, Berkeley, studied the performance of improved ground during the Loma Prieta Earthquake. They reported on twelve (12) sites where seven (7) types of soil improvement had been utilized, including stone columns, and concluded that none of the sites had experienced damage or distress as a result of the Loma Prieta Earthquake of October 17, 1989. They cautioned that the Loma Prieta Earthquake was of only moderate intensity and short duration with the average site studied experiencing ground acceleration of only about twenty five percent to seventy five percent (25%-75%) of the design earthquake values, and the durations were very short compared to predicted values. As with most projects in the United States, each site was originally designed to prevent liquefaction in case of a design earthquake, but no instrumentation was installed at the sites to monitor the earthquakes and its specific effects. Obviously, much more study is needed in liquefaction prevention; particularly as codes are becoming more stringent and being updated for liquefaction, and we are learning more and more about the differences in the earthquake types from the east and west coasts, and the central United States.

Grouting

As indicated by Figure 2, there are five (5) basic types of grouting. Slurry, Compaction and Chemical grouting are matured systems in the United States, with Jet and Fracture grouting emerging technologies.

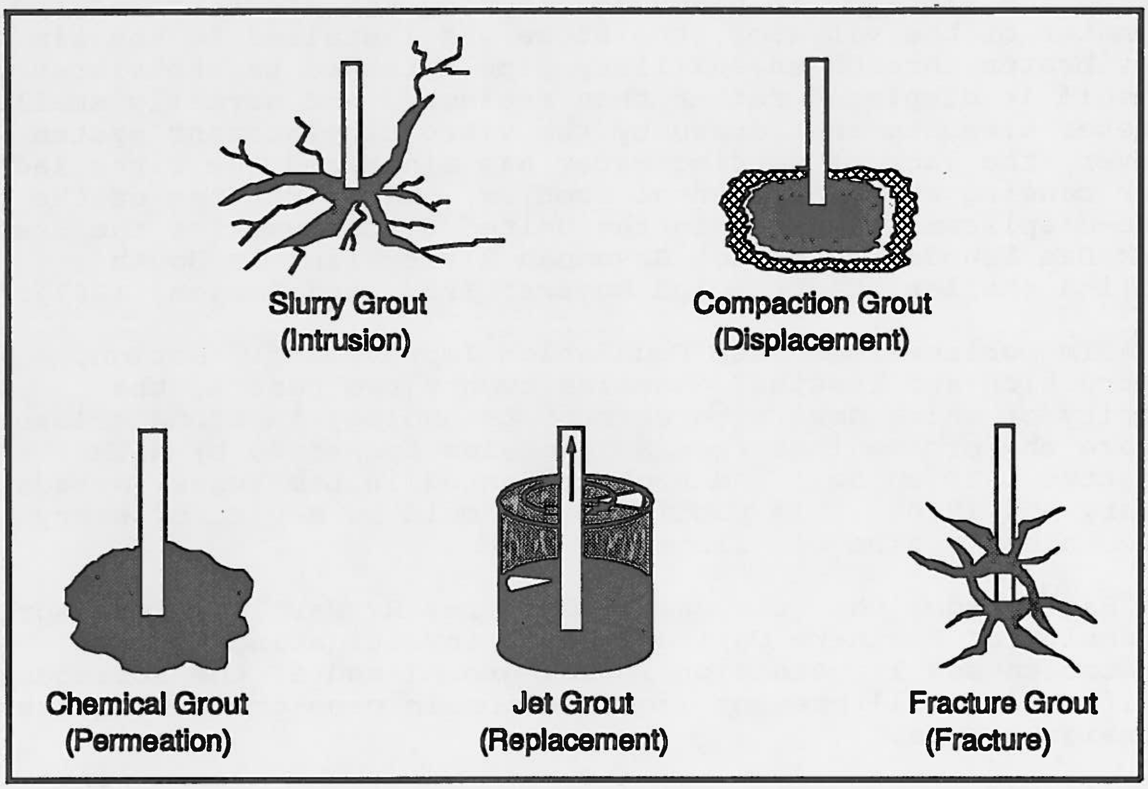

Figure 2: Five Basic Types of Grouting

A brief discussion on each is as follows:

1. *Slurry Grouting*: The development of the fine grind cement, Zebovitz, et al, (1989), and Moller, et al (1984) and the utilization of computers for dam grouting , Davidson, (1984) are the latest advancements in Slurry grouting. New publications are: Holsby's "Construction and Design of Cement Grouting" (1990), and Weaver's "Dam Foundation Grouting" (1991). Both are "must" volumes for any professional involved with dam grouting. A patented process of injecting waste into waste and controlling the end product, (Chesner & Welsh, 1992) may be one of the emerging technologies of the 90's.

2. *Compaction Grouting*: Compaction Grouting was defined by the American Society of Civil Engineers Geotechnical Engineering Division's Committee on Grouting (1980) as follows: "Compaction Grout - Grout injected with less than 1 inch (25 mm) slump. Normally a soil-cement with sufficient silt sizes to provide plasticity together with sufficient sand sizes to develop internal friction. The grout generally does not enter soil pores, but remains in a homogenous mass that gives controlled displacement to compact loose soils, gives controlled displacement for lifting of structures or both." Compaction Grouting was originally developed in the western portion of the United States as a soil improvement

technique to offset the lack of control of slurry grouting in solving settlement problems under existing structures. By trial and error, it was found that decreasing the slump of the grout resulted in the less fluid slurry grout remaining as a mass around the grout pipe tip, thus giving better control to the densification of the ground which was the cause of the settlement (Graf, 1969) (Warner, 1982). The next major development was to use compaction grouting to prevent settlement caused by soft ground tunneling operations (Baker, 1983). The Seattle Subway project used non-cementatious grout for the first time to allow for re-injection, if any additional settlement some time after tunneling occurred. Compaction Grouting has been effectively designed and used for new and remedial construction to prevent liquefaction and differential settlement, either singularly or with other ground modification techniques (Baker, 1985) (Henry, 1987) (Welsh, 1988) (Schmertmann, et al, 1986) (Mitchell & Welsh, 1989) (Salley, et al, 1987).

3. *Chemical Grouting*: Chemical Grouting is any mixture of materials used for grouting purposes in which all elements of the system are pure solutions (no particles in suspension)" (ASCE Grouting Committee 1980).

Chemical grouts are used for two basic purposes, structural and water control. Structural chemical grouting is designed to give cohesion to sand, in effect make sand into sandstone. Water control chemical grouting is designed to form a water barrier in granular soil formations or seal off water infiltration in existing structures.

Chemical grouting is the most researched form of grouting. It has been utilized on all the major subway systems in the United States, with the most challenging recent project being its use on the Los Angeles Subway. Horizontal chemical grouting was performed for a 300 ft length, full face for the twin 20 ft bores beneath ten (10) major lanes of Interstate 10. As no significant settlement took place during the soft ground tunneling operations, this project verified the technical and economical utilization of chemical grouting (Gularte, et al, 1991). Before the concrete liner was installed, a major fire took place in one of the tunnels destroying the lagging boards and rib supports and only the chemically grouted soil was effectively supporting the 20 ft diameter tunnel excavation; thus a further benefit of the chemical grouting, which was not considered in the design process, was realized.

Ten (10) years ago, I had the pleasure of addressing the Thirteenth Ohio River Valley Soil Seminar giving a paper entitled "Chemical Grouting Utilized for Underpinning and Water Control for a Pit Installation" (Welsh, et al, 1982). Recently, at the City of Cincinnati's Water Treatment Plant, there was another use of chemical grouting that I'd like to discuss. Two 78 in diameter water tunnels were required to be installed beneath an existing potable water filtration building which had been constructed on loose, clean sand. Thirty (30) horizontal grout injection holes, ranging from 68 to 96 ft long, were drilled through the wooden lagging supporting the excavation. The holes were arrayed in four rows on five ft centers and checked for accuracy with a special electronic alignment tool. Approximately 105,000 gallons of Geloc-4 sodium silicate grout were pumped through sleeve port pipes installed in the drilled holes, creating a grout zone of approximately 64,400 cu ft. All grouting was completed prior to beginning the tunneling. With the chemical grout holding the tunnels' face, the tunnel excavations were completed within a few weeks. The filtration building was constantly monitored during the tunneling operation, but virtually no indication of settlement or voids was detected.

4. ***Jet Grouting***: This system was introduced to the United States in the 1980's based on 1970 developments in Japan and further refinements from Europe. There are three (3) primary jet grouting systems; single, double, and triple rod, and each has advantages and limitations (Figure 3). The major limitations of the single and double systems are that the existing soil is a major part of the end product and minimizes the design ability of the total project. The triple rod system excavates the majority of the soil by high pressure water encapsulated with air and the cavity created being instantly filled with the pre-designed, cementatious material. This system has shown significant advantages, particularly for underpinning and excavation support (Burke, et al, 1989) (Burke & Welsh, 1991).

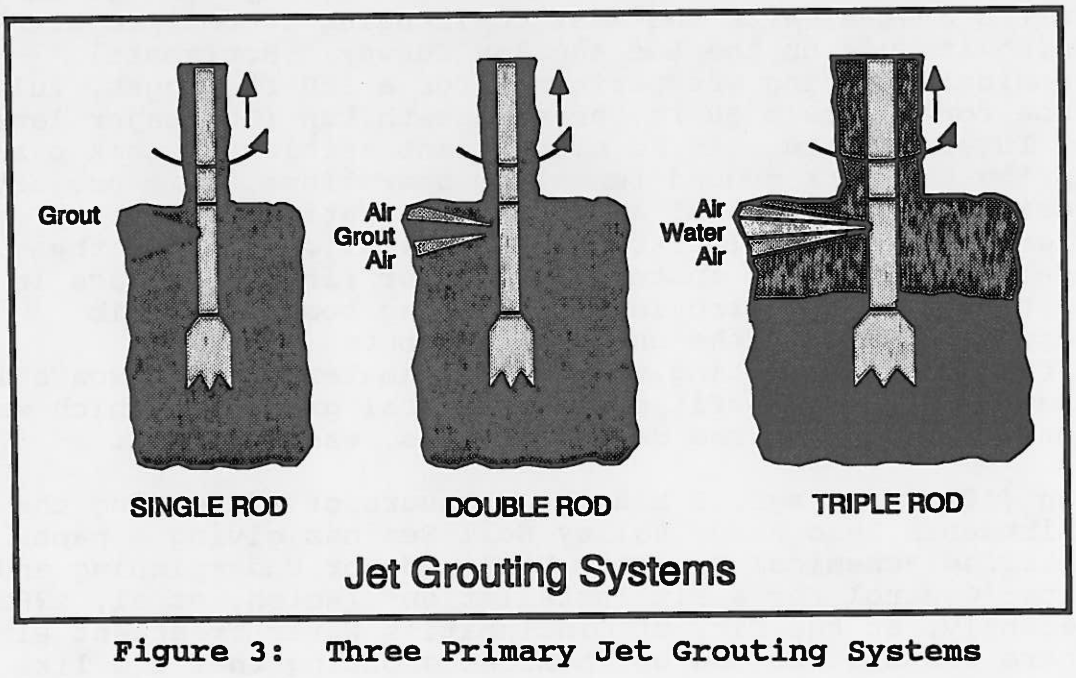

Figure 3: Three Primary Jet Grouting Systems

Jet Grouting has also been utilized to form bottom seals for deep excavation and to seal off abandoned wells to prevent spread of contaminants to deep aquifers.

An in-situ soil washing test program was performed for the first time in Hamburg, Germany. The jet grouting technique was used to remove contaminated soil from underneath existing structures without causing settlement. Removal of the contaminants from the soil was then accomplished on the surface and the cleaned soils, with cement added, were re-injected into the ground (Grisham & Sonderman, 1990).

5. *Fracture Grouting*: This overseas development is just being introduced into the United States and will be another tool in our arsenal to prevent and rectify settlement problems.

Miscellaneous Ground Modification Techniques

The following are some other Ground Modification techniques for remediation of contaminated soils and rock that are emerging and that need better geotechnical engineering, research, development, testing procedures, cost, etc.:

a) In-situ Vitrification
b) Soil Vapor Extraction
c) Electro-Kinetics
d) In-situ Solidification, Stabilization & Fixation
e) Freezing
f) Bioremediation
g) Pump & Treat
h) Surfactant Flushing

SUMMARY

New utilizations and better methods of QA/QC and design are being developed for the mature methods of Ground Modification; while the emerging technology, driven by regulations, has techniques developing so rapidly that it is difficult to keep pace as to which techniques should be considered for which environmental problem.

The four (4) major areas of utilization of Ground Modification technology in the future will be:

1) To solve site specific problems for new, remedial, and remediation construction

2) To improve the ground to prevent liquefaction

3) To combine two or more methods of Ground Modification to solve site specific problems

4) To use current and new methods for site remediation projects

CONCLUSIONS

As Civil Engineers practicing in the geotechnical engineering field, we are experiencing an ever growing challenge to keep abreast of more and more technology in the standard practice of geotechnical engineering. In the emerging area of remediation of contaminated soils, if we do not keep current, we are going to find ourselves playing a minor role, dictated to by scientists as to how to remediate problems in soils, our chosen profession of engineering. I predict that the next ASCE publication on Soil Improvement will be largely dedicated to environmental cleanup techniques.

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CONSTRUCTION OF AN APARTMENT COMPLEX OVER RECLAIMED QUARRY: ARBORS OF WATERMARK COLUMBUS, OHIO

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ABSTRACT

The project consists of a 35-acre apartment site and is located adjacent to the Scioto River near the central downtown area of Columbus, Ohio. Sand and gravel deposits were mined on this site from the 1930's to the early 1960's, creating a large lake. The wasted fines from aggregate grading concrete batch plant operations were flumed back into the excavated lake filling the western portion. From 1977 to 1986, the site was filled with miscellaneous material to approximately 20 feet above the top of the flumed material. The fill consisted of soil and building rubble materials received from downtown building demolition and excavations.

Final development of this site included the construction of sixteen 2 and 3 story apartment buildings. This project was particularly challenging both from a technical and geotechnical field management viewpoint. This paper discusses both the technical foundation considerations and also the project management involved with the successful development of a very difficult site.

SITE HISTORY - ORIGINAL OWNERS

Depositional History

The site is located south of and adjacent to Watermark Drive in Columbus, Ohio. This general area has been developed with a combination of office and apartment buildings. The site of the apartment complex was originally a mineral aggregate quarry. After the completion of the sand and gravel mining, the waste products of the aggregate processing operation and later from an on-site concrete plan (fine sand, silt, and silty clay) were hydraulically flumed back into the excavation. The coarser material settled out closer to the flume outlets in the northeastern part of the site, with a correspondingly higher percentage of soft fines deposited in the south and west areas of the parcel (see Figure 1). The resulting non-uniform deposits consisted of alternately interbedded layers of soft silty clay, clayey silt, and loose sand and silt overlying the natural dense sand and gravel formation. To complicate the difficult subsurface conditions, the site was filled from elevation 713 to the 100-year flood elevation of 732 with uncontrolled soil fill and concrete construction rubble. Earthwork for the apartment development began in early January, 1990. Figures 2 and 3 show aerial photographs of the site taken in 1955, 1960, 1972, and 1989.

Early Geotechnical Investigations and Recommendations

The time-line information shown in Figure 4 relates the involvement over the life of the site development of the owners (initial and final), the two geotechnical engineering consultants, BBC&M Engineering Inc. (BBC&M) and Geotechnical Consultants, Inc. (GCI), and the schedule of site filling. This information is useful in understanding the depositional development and the formulation of the final geotechnical recommendations for site improvement. From the time that fill placement began at the water elevation of 713 in 1977 to the final construction elevation of 734 achieved in 1990, two owners and two geotechnical consultants were involved with the project. The proposed development of the subject site changed from industrial, to commercial, and finally multi-family residential over this time period. The initial owner was the aggregate company that performed the original mining on the site. Several preliminary investigations were performed in 1976 and 1983 by BBC&M over the entire mined area including the areas to the north of the apartment site to establish the potential for development. Based on these investigations, three distinct areas and foundation approaches were defined by the initial consultant. These included: 1) an Upland Area; 2) a Marginal Area; and 3) a Lowland Area (see Figure 1).

The Upland Area was represented by land north of an abandoned railroad spur line which formerly ran through the site. This area was not part of the apartment complex development and was not quarried extensively because of the presence of facilities for the former sand and gravel operation and concrete plant. Portions of the area were excavated or leveled by filling, but the fills appeared to consist of clean, suitable soils which were either placed in a controlled fashion or had been consolidated by large stockpiles of aggregate that were at one time stored at the site. Natural soils in the area consist

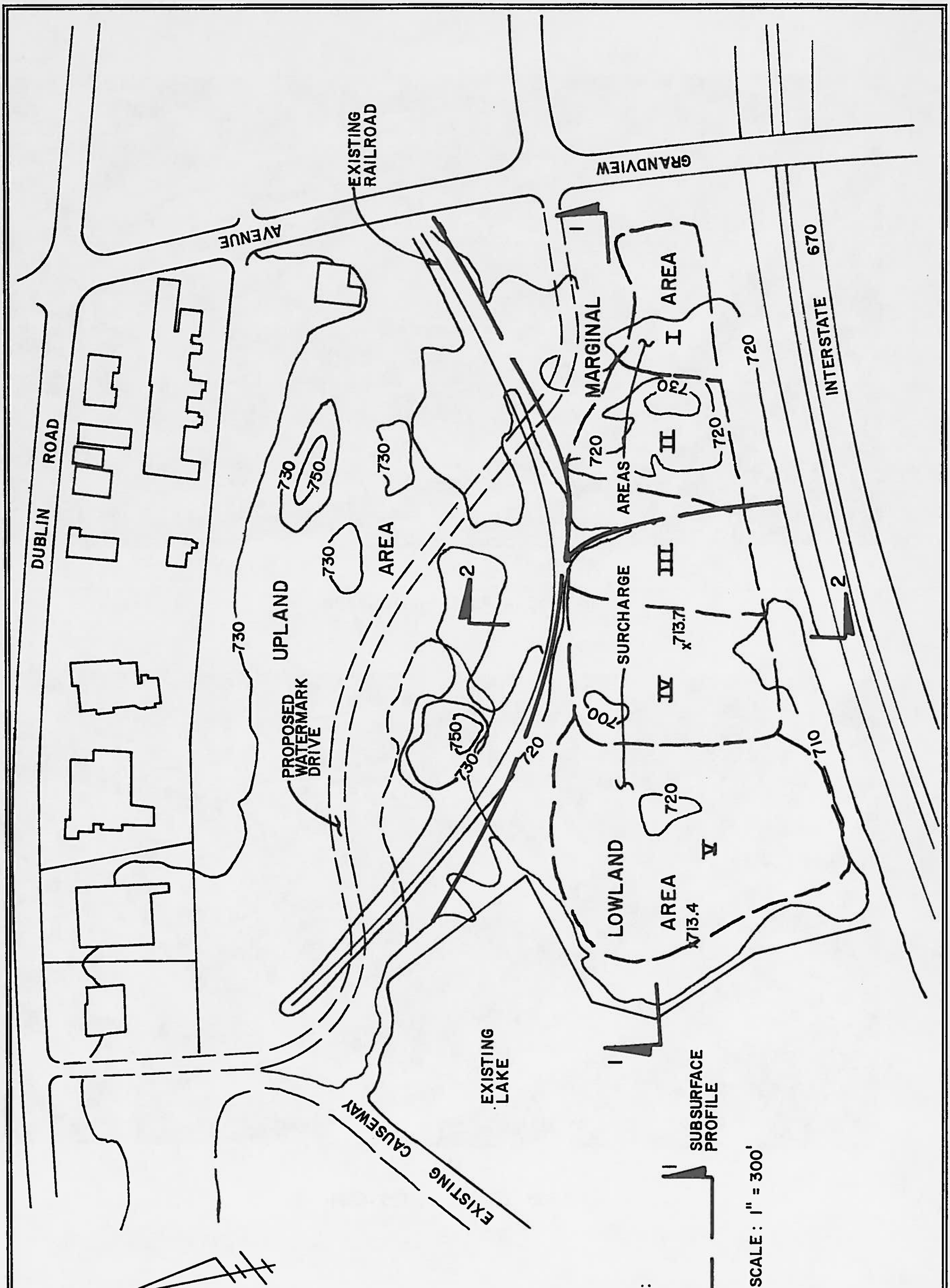

EXISTING RAILROAD
GRANDVIEW AVENUE
DUBLIN ROAD
UPLAND AREA
MARGINAL AREA
INTERSTATE 670
SURCHARGE AREAS
LOWLAND AREA
PROPOSED WATERMARK DRIVE
EXISTING LAKE
EXISTING CAUSEWAY
SUBSURFACE PROFILE
KEY:
SCALE: 1" = 300'
N
I
II
III
IV
V
730
750
720
700
710
713.7
713.4
1
2

Figure 1.

(a)

1955 AEREAL PHOTOGRAPH

(b)

1960 AEREAL PHOTOGRAPH

FIGURE 2

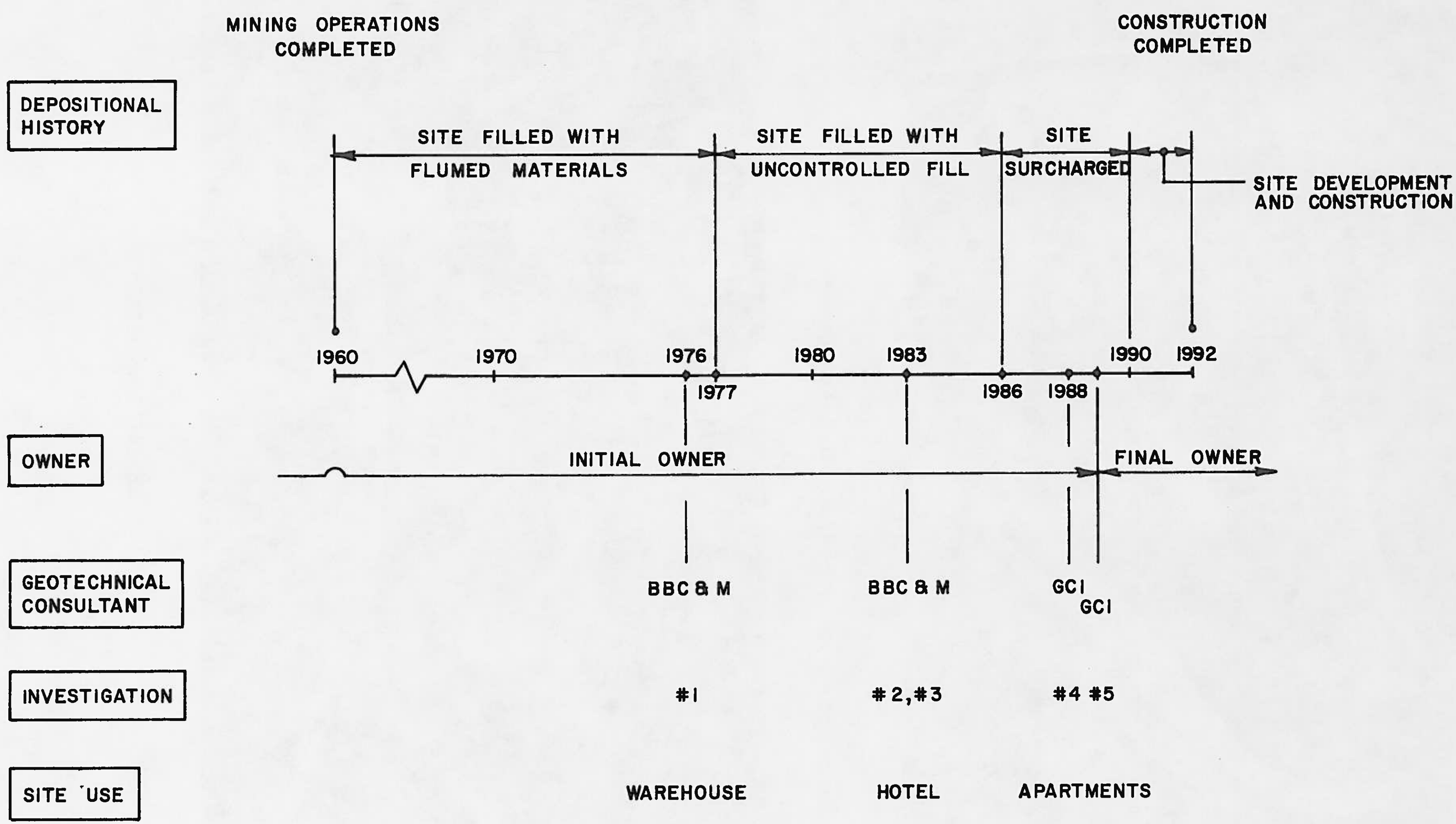
(a)

1972 AEREAL PHOTOGRAPH

(b)

1989 AEREAL PHOTOGRAPH

FIGURE 3



DEPOSITIONAL HISTORY
MINING OPERATIONS COMPLETED
CONSTRUCTION COMPLETED
SITE FILLED WITH FLUMED MATERIALS
SITE FILLED WITH UNCONTROLLED FILL
SITE SURCHARGED
SITE DEVELOPMENT AND CONSTRUCTION
1960
1970
1976
1977
1980
1983
1986
1988
1990
1992
OWNER
INITIAL OWNER
FINAL OWNER
GEOTECHNICAL CONSULTANT
BBC & M
BBC & M
GCI
GCI
INVESTIGATION
#1
#2,#3
#4 #5
SITE USE
WAREHOUSE
HOTEL
APARTMENTS

Figure 4.

generally of zones of strong cohesive glacial till and deposits of relatively dense granular materials.

The Lowland Area included all of the land which existed close to the normal water surface of the nearby lake, which includes the western half of the subject apartment site. This area was quarried extensively for sand and gravel deposits to elevations as low as 670, and was then filled to existing grade by flumed waste material. The flumed material consisted of fine sand and silt in a very-loose condition and of deposits of silty clay in a very-soft condition. These materials were highly compressible and possessed practically no shear strength. Underlying most of the lowland area fill material was dense natural sand and gravel over medium-hard to hard gray limestone.

The Marginal Area was defined as the area below the Upland and to the east of the Lowland Areas. The subsurface conditions in this area varied considerably and consisted of one or more of the following materials: 1) mostly natural soil; 2) relatively strong fill over natural soil; 3) both thick and thin deposits of flumed material; and, 4) different types and consistencies of flumed fill.

Results of the preliminary geotechnical reports included recommendations for the construction of a main roadway; filling to the required flood plain elevation of 732; roads and pavements; and structure foundations. Recommendations were made that the roadway be located north of the southern boundary of the Upland Area and be constructed with dredged granular material, creating a lake inlet in the subject site. The consultant strongly recommended that additional fill in the Marginal and Lowland Areas be placed in a controlled manner and that the rate of placement be monitored in the lowland area to prevent shear failure of the alluvial deposited flume silt.

Shallow foundations were considered feasible for light to moderately-heavy structures in the Upland Area with some minor anticipated site preparation required, while extended foundations to bedrock were anticipated for structures within the subject site (both Marginal and Lowland Areas).

Early Site Development

In 1983, a roadway was constructed along the northern edge of the subject parcel. An inlet was formed through the excavation of sand and gravel used as fill in the roadway construction (see Figures 1 and 3b). From 1977 to 1986, considerable uncontrolled fill was placed on this site. This fill consisted of random soil and construction rubble (referred to in this paper as 'mechanical fill'). In 1986, the initial owner requested recommendations from the soils consultant for site preparation with consideration to the recently placed 20-plus feet of uncontrolled fill. No intended use had been established at the time of this request. It was recommended that the central portion of the site be surcharged with 10 feet of soil fill, beginning on the east end and progressing to the west. The consultant monitored the settlement of the mechanical fill and underlying flumed material with settlement platforms, and advised the owner when surcharge material could be advanced in position. Recorded settlements resulting from the surcharge loading ranged from 4 to 8 inches. The time required for the 10 foot surcharges to complete primary consolidation ranged from 10 days on the east end of the site over the flumed granular deposits to over 4 months on the south and west areas overlying the flumed cohesive materials. The locations of the surcharged areas are shown in Figure 1.

Until 1987, the specific scope of development remained undefined and the preliminary recommendations for fill placement provided by the geotechnical consultant were generally not followed. A concrete rubble dike was placed along the western shoreline area as an attempt to consolidate and strengthen the flumed material prior to filling the interior of the site. The 20 feet of uncontrolled fill in the interior of the site was placed in a manner which consisted of dumped piles of material which were leveled with a dozer prior to the next lift of dump piles. During conversations with the earthwork contractor that leveled the dumped material, it was revealed that mudwaves occurred in the flumed soils located in the southwest portion of the site due to the rapid placement of fill. The 10 foot high surcharge placed across the site resulted in reasonable settlements, but there were concerns that the site conditions were still unfavorable due to methods and makeup of the 10 to 20 feet of uncontrolled fill. In 1987, the initial owner was taken over by another company (still referred to as the initial owner) which resulted in a change in geotechnical consultants from BBC&M to GCI.

SITE HISTORY - NEW OWNERS

In the fall, 1988, a developer (final owner) interested in purchasing the property proposed an apartment complex with specific building sizes and locations, and contracted GCI to perform a preliminary subsurface investigation for the eastern half of the site. Preliminary recommendations included the use of shallow dynamic compaction and shallow foundations for the eastern portion of the site and extended foundations with structural floors for the western portion of the site. Due to the difficulty of the site soil conditions, the developer hired two geotechnical consultants from outside the area to perform a technical peer review of GCI's preliminary recommendations. Consultant #1 expressed grave concern about placing any structures on the site, regardless of the type of site treatment, foundation, or location within the surcharged areas, due to the potential for long term settlement from the migration fill material into voids located within the mechanical and rubble dike fill. Consultant #2 indicated that neither dynamic compaction nor extended foundations were necessary since the site had been sufficiently surcharged. Both consultants agreed that, due to the highly variable subsurface conditions, an extensive final subsurface investigation including at least 2 borings per building should be performed, of which GCI concurred.

Final Subsurface Investigation

Even with the large variance of technical opinion between the two peer reviewers with regards to the development approach presented by GCI for this site, the developer purchased the property for development, and expressed confidence in the soil consultant and authorized a final subsurface investigation of the site.

Both standard penetration borings and Dutch Cone penetration testing were performed for the final subsurface investigation. General subsurface profiles based on sections shown in Figure 1 are presented in Figures 5 and 6.

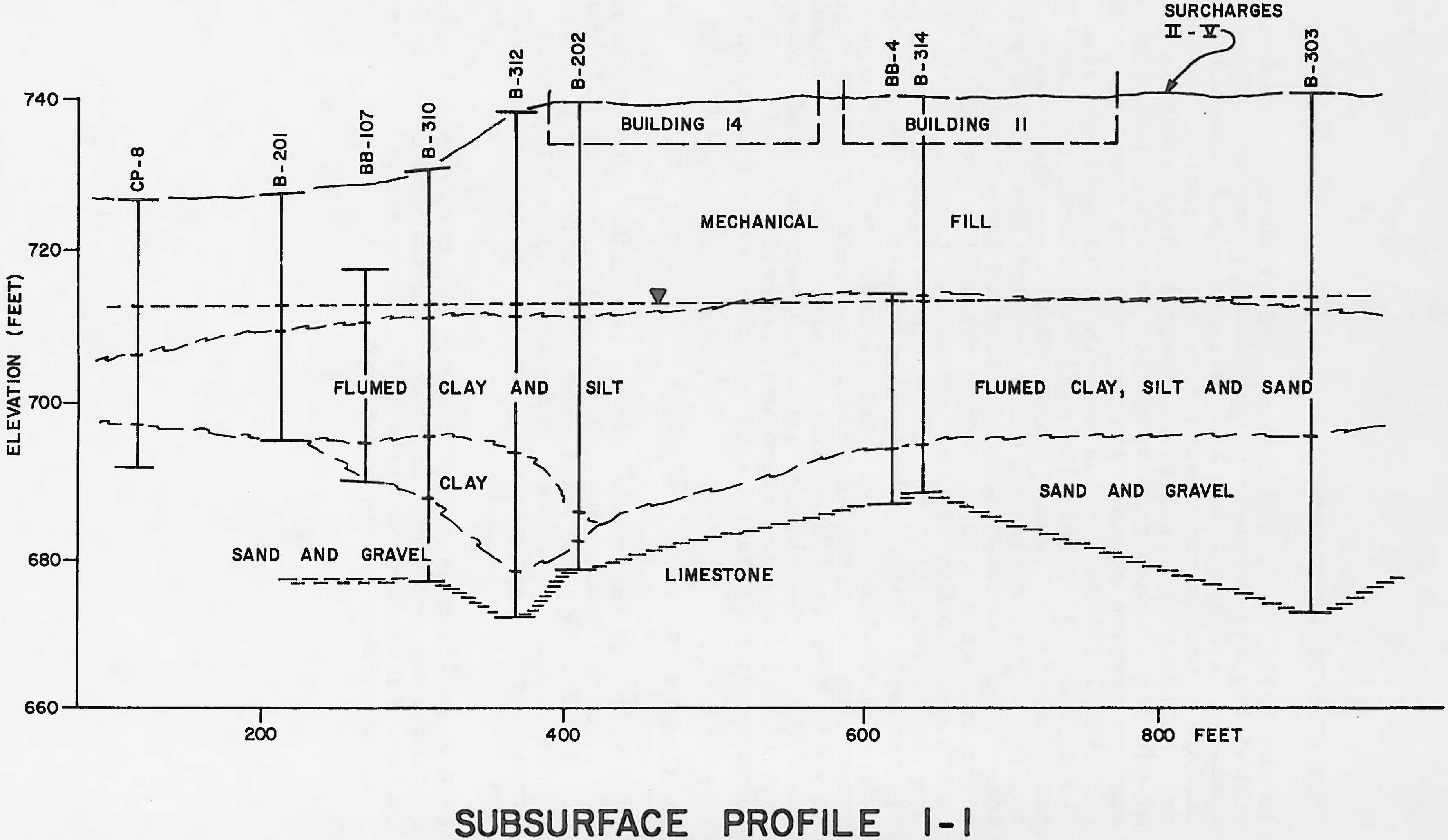
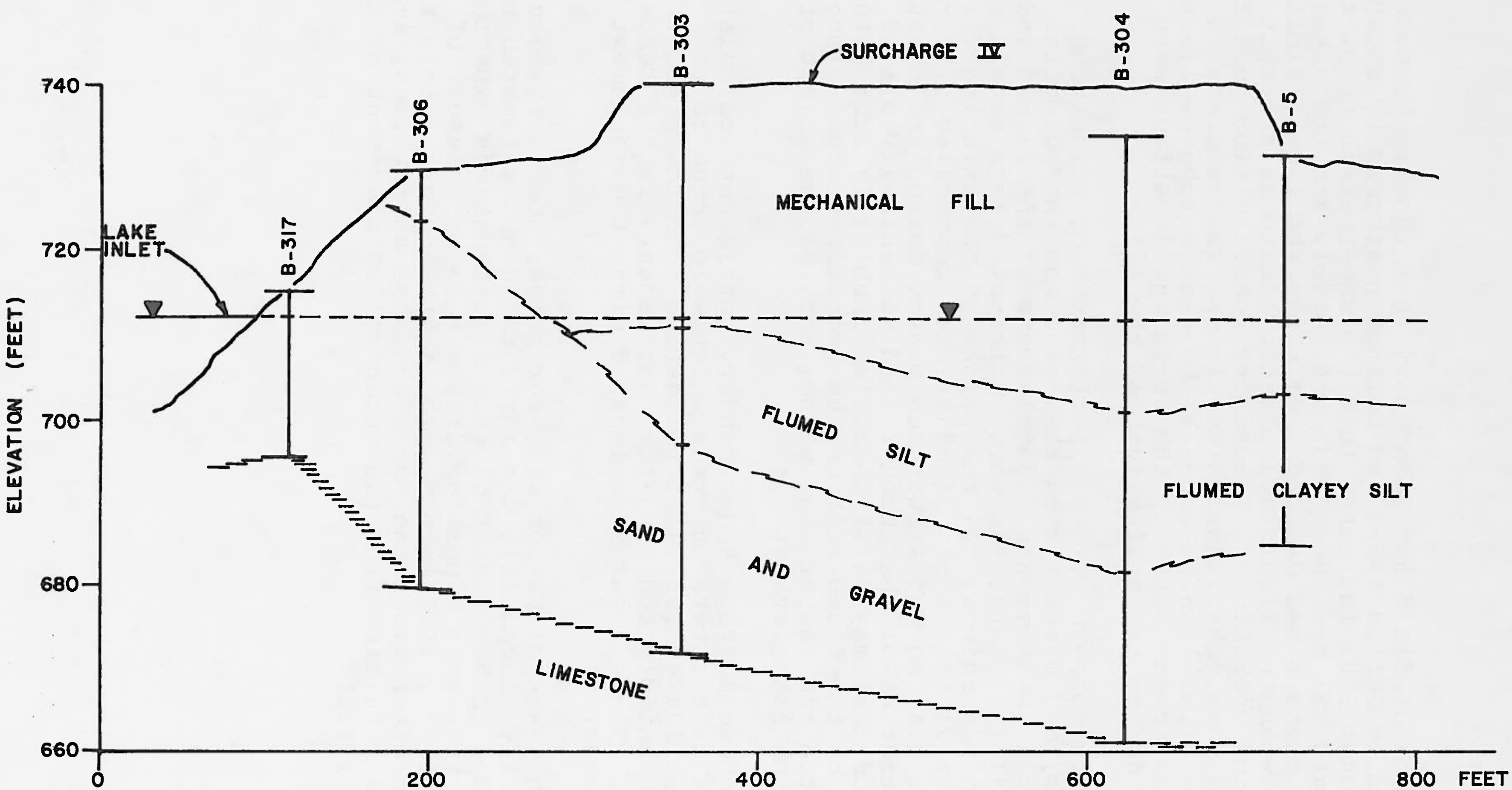

SURCHARGES
II - V
CP-8
B-201
BB-107
B-310
B-312
B-202
BB-4
B-314
B-303
BUILDING 14
BUILDING 11
MECHANICAL
FILL
FLUMED CLAY AND SILT
FLUMED CLAY, SILT AND SAND
CLAY
SAND AND GRAVEL
SAND AND GRAVEL
LIMESTONE
ELEVATION (FEET)
740
720
700
680
660
200
400
600
800 FEET
SUBSURFACE PROFILE 1-1

Figure 5.



SUBSURFACE PROFILE 2-2

Figure 6.

Comparisons of field and laboratory test results between borings performed by GCI in 1989, and borings performed by BBC&M from 1976 through 1983 indicated that the shear strength of the flumed material had increased considerably and the natural moisture content had dropped from above the liquid limit to mid-range between the liquid and plastic limits as a result of the surcharging operations. Laboratory consolidation test results indicated that consolidation settlement was complete in the flumed material located beneath the past surcharge areas, and that areas to the south of the surcharge locations were still settling due to the self-weight of the fill.

The primary concern with the performance of structures, utilities, and pavements on this site was the potential for differential settlements. Settlements of the flumed and mechanical fill materials were anticipated as a result of the following mechanisms: 1) the weight of the existing fill and surcharge; 2) the weight of additional controlled fill required for final grading and structures placed during and construction; 3) infiltration of the uncontrolled mechanically placed fill into voids that may exist in buried rubble; 4) from lateral movement of the flume silt in the unconfined areas along the south ditch and western shoreline; and, 5) the effect of rising and falling lake levels.

Primary consolidation and secondary settlements calculated for the flume silt strata in areas which were found to be underconsolidated (still consolidating from the present loads) indicated anticipated long-term settlements in the range of 3 to 6 inches. These areas were located along the southwest border of the property.

Even with the application of a surcharge, some settlement of the mechanically placed fill was anticipated and estimated to range from 1 to 3 inches. Most of this adjustment was expected to occur in the western portion of the site as a result of infiltration of fill into the voids of concrete rubble fill placed in dikes along the north and west shore lines, and from any voids resulting from the uncontrolled placement of the mechanical fill.

The lateral movement of the flumed material along the western shoreline was controlled through the placement of a 4 horizontal to 1 vertical slope. This option was selected by the final owner in lieu of monitoring the lateral movement during the construction of a steeper slope.

The effect of the rising and falling water table was a consideration especially due to the sensitive nature of the underlying flumed material. This lake is used as flood storage for the nearby Scioto River, which elevations are controlled both upstream and downstream by a dam and weir, respectively.

FOUNDATION RECOMMENDATIONS

Two types of foundation systems were recommended for this site, namely: 1) System 1 - a shallow foundation system; and 2) System 2 - an extended foundation system. System 1 consisted of ground modification using shallow dynamic compaction (SDC) in the building pad areas in conjunction with a stiffened mat-type foundation which would provide a more even distribution of stresses and would minimize the effects of differential settlement. The SDC program consisted of dropping a 12 ton free-falling weight, 3 to 5 blows per location on a 10 foot square grid pattern, from heights ranging from 15 to 20 feet. The purpose of the SDC was to provide a "proofrolling" of the mechanically placed fill, and ultimately reduce the potential differential settlements from this strata. The heights were limited in order to minimize the buildup of excess pore pressures in the underlying flumed material so that shear failure of the underlying flumed materials would not occur. The SDC was performed within the building pad and perimeter area extending 10 feet beyond the building footprint. A post-tensioned inverted concrete "waffle" slab was selected by the structural engineer for use as the foundation component.

System 2 consisted of driven pile foundations extended through the mechanical fill and flumed material bearing on the underlying dense sand and gravel or limestone strata. Drilled piers were ruled out as an option due to the existing rubble and high groundwater conditions. Structural grade beams and structural floors were recommended for use with this system.

It was anticipated that the construction rubble would present problems during the installation of driven piles. Once the piles penetrated the top 20 to 25 feet of the mechanically placed fill, no unusual problems were expected in driving the remaining lengths of the pile.

Reinforced concrete piles were originally recommended for use at this site due to their high concrete compressive strength (6,000 psi) their potential to push rubble aside during driving, and the ability to check their structural integrity after they were installed. The pile contractor offered an alternative of 7-inch-diameter closed-end driven pipe piles filled with concrete. Capacity of these piles including the negative skin friction were within acceptable limits, although concern was expressed regarding the ability of these piles to penetrate the top 25 feet of mechanical fill without being damaged. It was initially estimated that up to 30 percent of the piles may be damaged and require offset piles be driven.

A Pile Driving Analyzer (PDA) was used at three locations across the site to determine the driven capacity and to provide a correlation of the blow count to the pile capacity. Hammer efficiency was also determined with this method. This information along with the known depth to bedrock, enabled the soil technician to establish when the pile capacity was obtained and determine any change in hammer efficiency during driving.

The foundation recommendations based on the final site plan layout included that System 1 be used in buildings 1, 2, 8, 10, 14, 15, 16, 17, and the north clubhouse. It was recommended that System 2 be used in buildings 3, 4, 5, 6, 7, 9, 11, 12, 13, and the southwest pool and southwest clubhouse. Figure 7 shows the final building and site layout.

Utility Considerations

Due to anticipated differential settlements, special details for the utility lines crossing from the surcharged to the non-surcharged areas were considered. Final design included the use of river crossing pipe with high allowable joint rotation for water lines and flexible joint connections for building tie-ins. Additional undercuts and bedding requirements were used in the placement of sanitary and storm sewers.

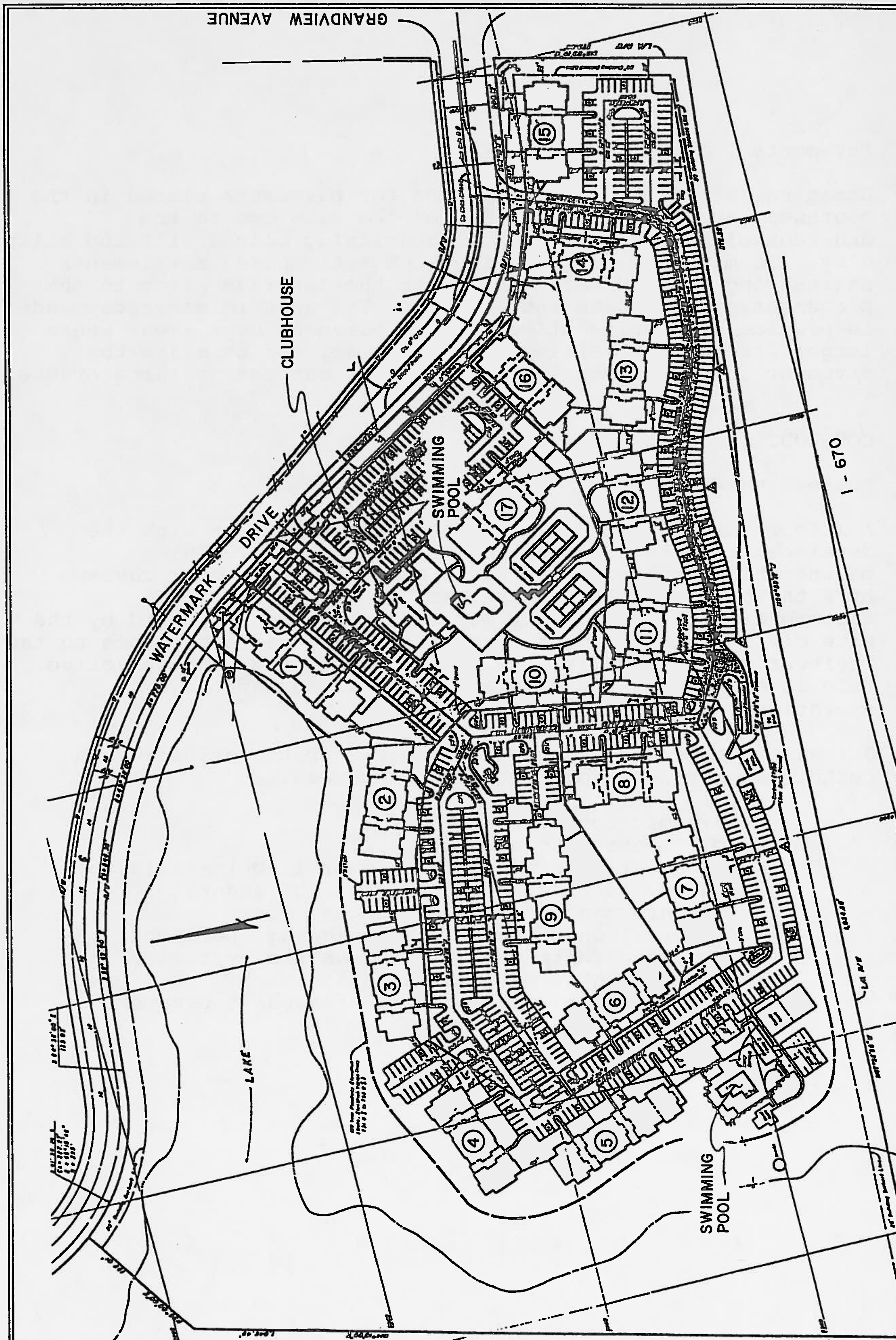

GRANDVIEW AVENUE
CLUBHOUSE
SWIMMING POOL
WATERMARK DRIVE
I - 670
LAKE
SWIMMING POOL

Figure 7.

Pavements

Damaging settlements were expected for pavements placed in the southwest and southern portions of the site due to the underconsolidated nature of the underlying flumed silt and silty clay. To minimize cracking from the anticipated settlements, engineering geogrid was placed over the subgrade prior to the placement of aggregate and asphalt. The geogrid was recommended to provide a bridging effect of the pavement over areas where large differential settlements may occur, and to allow the pavement to settle more as a unit, thus decreasing large cracks.

CONSTRUCTION

Project Management

A life insurance company became an equity partner with the developer for the construction of this project. This partnership resulted in an additional technical peer review by a third geotechnical consultant of the foundation recommendations provided by GCI and site plans prepared by the site Civil Engineer. The general approach was acceptable to the reviewer. Additionally, the insurance company took an active role in monitoring the day to day activities of the site operations.

GCI was retained by the general contractor to perform field technician services for the following areas:

- dynamic compaction
- placement of controlled fill
- observation of water and sewer line installation
- review of steel reinforcement for general conformance to the structural plans
- observation of the post-tensioning operations
- concrete testing of the foundation systems
- monitoring of pile installation
- proofrolling and placement of asphalt pavement

FINAL SITE PREPARATION

Dynamic compaction and general earthwork were started on January 2, 1990, which is a very difficult time of year to begin earthwork in Columbus. Frozen soils occurred to a depth of 5 inches, and were stripped and removed from the building pad areas prior to dynamic compaction and fill placement. Due to wet surface conditions, mud and rock materials in some building areas were projected approximately 100 feet upon weight impact during the dynamic compaction operations. The site pads were further undercut to remove wet materials. No. 2 crushed limestone was used to mix the saturated crater soils to create a stable building pad area. Due to the variation of the fill materials, compaction testing could not be performed, and compaction quality control was based on the visual proofrolling of each lift. If yielding material was detected, then additional stone was added and mixed until the area became stabilized. On the average, the settlement induced from the dynamic compaction ranged from 6 to 12 inches. By mid-January, dynamic compaction, pile driving, and construction of the compacted fill in buildings 1 and 2 were underway. Lights were brought onto the site so that night work could be performed.

The structural engineer's office was in Atlanta, Georgia. Correspondence with the structural and geotechnical engineer was performed on a weekly and sometimes daily basis depending on problems which occurred during pile driving. Damaged piles often required offset piles to be driven. As it turned out, less than 5 percent of the piles were damaged. The closed end driven pipe piles worked well in that inspection of the driven condition was performed by dropping a light from the ground surface down the center of the pile.

Additional problems occurred during the placement of the steel reinforcement in the buildings with the System 2 foundations. Difficulties arose when it was determined that the concrete contractor was inexperienced in the placement of complex steel reinforcement. An interesting interaction between the consultant/contractor/owner occurred, resulting in a new contractor brought to the site to complete the System 2 foundations.

Buildings 1 and 2 were located along the southern edge of the Upland Area and were overlying some fill over naturally occurring silty sand. The proposed grades required that a sidehill fill be placed in the existing inlet area. As a result, the lakeside (inward) of the buildings would bear on 20 feet of controlled fill, and the outward side would bear on approximately 5 feet of controlled fill.

Fill placement in these areas were complicated by two issues: 1) the existence of buried topsoil at the base of the slope; and 2) a buried concrete mass was encountered. The concern with the presence of the topsoil in Building 2 involved the slope stability of the placed fill. A failure plane would likely pass through the horizontal layer of topsoil where the resisting shear strength was very low. For this reason, the topsoil was completely removed in the Building 2 area. During this excavation, a mass of concrete over 200 feet long, 5 to 15 feet wide, and 1 to 12 feet thick was encountered. This area was the location where concrete trucks had emptied their excess loads along the edge of the railroad embankment into the lake during concrete operations which existed after the quarry operations had ended. Since the top of the concrete mass was 5 to 10 feet below the finished floor elevation, the fill above the concrete in the area of the building was removed and replaced with a controlled fill material, with the concrete mass remaining in-place.

Suitable material for the construction of the sidehill fill was not available on-site. Since both stability and settlement was of concern, a properly controlled fill was required. At an ideal site, a homogeneous soil placed in 8 inch lifts with proper moisture control and compaction would be used. Needless to say, Watermark was not an ideal site, and the surcharge material available as fill was not a homogeneous soil. Segregation of acceptable site material from unacceptable material was not possible.

Two approaches were offered regarding the fill in Buildings 1 and 2. The first was to import off-site borrow fill soil which had the proper moisture content and density characteristics for use as a controlled fill. This approach would provide a consistent fill and minimize large potential settlements. A second approach was to use a blended mix of the miscellaneous surcharge material with a 40 to 50 percent mix of 2 inch crushed limestone aggregate. The purpose of this approach was to construct a fill that will act primarily like an aggregate fill with the surcharge materials used to fill the voids of the aggregate. Alternating lifts of 6 inches of aggregate and 4 to 6 inches of surcharge fill were placed, mixed and compacted. The compactive effort of each lift proceeded until no pumping was detected. Soft areas required additional amounts of No. 2 stone, mixing, and compaction.

LESSONS LEARNED

The following are items that were learned and should be considered when dealing with the site preparation of a project with difficult subsurface conditions:

1. When consultants are changed midway into a difficult project, find out why. It is important that all geotechnical reports and information be available to the new consultant for review prior to accepting a new project which is difficult and has the potential for future problems. In this particular situation, it was clear that the reason for the change was not technical, but was primarily a result of a change in management of the original owner;

2. When dealing with an uneducated client in the development of a difficult site, assume the role of a teacher. After the change in management of the initial owner, the person in contact with the geotechnical consultant was also the person in charge of marketing this property. This type of person wants to hear only "good" things about the site conditions in order to make the site look attractive. Fortunately, the developer had a civil engineering background and understood the risks, requirements, and benefits of the site;

3. Clearly define all risks to the owners. Then redefine them, summarized in writing, to all parties. For projects having difficult subsurface conditions, insist on working and meeting directly with the owner (instead of through an architect or engineer), and have the owner retain the geotechnical consultant directly, if possible;

4. If the client lacks complete confidence in your recommendations, or brings up the issue of technical peer review, treat it as a positive approach to the total site solution. In this case, technical reviews strengthened the preliminary recommendations as well as supported the scope of final subsurface investigation required for this type of site.

5. Be aggressive in pursuing a continuity of the geotechnical services between the design and construction phases. On-site representation of the design geotechnical consultant during the construction of this project enabled daily engineering decisions to be made based on detailed knowledge of the site conditions. Additionally, consultant's recommendations which were not being followed were immediately brought to the attention of the general contractor and owner before problems could snowball.

6. Make the field technician reports an engineered product. Comprehensive field reporting and daily interaction with the geotechnical project engineer are essential for the successful completion of a difficult earthwork and foundation project. Daily reports which included the basic WHO, WHAT, WHERE, WHEN, WHY, AND HOW were a necessity for this project, since they were sent to and read by the architect, developer, insurance company, reviewing architect, and general contractor. Issues reported in daily reports ranging from days to months old were constantly discussed at project meetings.

7. Communicate all problems to the client in a prompt and immediate manner, followed by a written summary of your conversation. On a site where many activities are occurring simultaneously, old information is often costly information.

8. The risk to a geotechnical consultant should be proportionate to his benefit. Any site is buildable provided the owner has enough money, and the clear understanding and acceptance of the risks that come along with the benefits in the development of a difficult site.

ACKNOWLEDGEMENTS

The authors appreciate the plan drawings provided by EMH&T Engineering used in the preparation of some of the figures for this paper. The first author also thanks his wife, Rose, for her patience and understanding of the late evenings at the office during the preparation of this paper.

FOUNDATION STABILIZATION FOR SALT RIVER BRIDGE
FORT KNOX, KENTUCKY

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U.S. Army Corps of Engineers

ABSTRACT

A 90-foot high bridge was constructed across the Salt River in the period June 1987 to December 1988 to provide an all-weather crossing for tanks and other heavy vehicles to the northern section of the Fort Know Military Reservation. The crossing had to accommodate river level fluctuations of up to 60-feet. Existing foundation soils at the bridge site consisted of soft, highly compressible, silts and silty clays. The bridge design was a 5-span, post-tensioned concrete, segmental I-beam bridge supported on end-bearing H-piles. The bridge itself was the only one of its type in Kentucky at the time it was built and had the longest span (160-feet) of its kind in the United States. Large fills were placed on each abutment to utilize materials excavated for the access road and to reduce bridge length. Installation of 30-40 feet deep stone columns on the riverbanks, immediately adjacent to the river, stabilized the abutment fills against foundation slope failure. Subsequent insertion of wick drains further back from the river into the soft foundation soils underlying the 70 to 90 feet high, rock-fill abutments dramatically reduced the necessary settlement time from 15-years to a total of 6-months. This drastic reduction in construction time provided expedited access to the north portion of Fort Knox and significant reduction in costs.

BACKGROUND

An access road to and a bridge across the Salt River at Fort Knox, Kentucky was required to connect the larger, developed south portion of the reservation with the smaller, undeveloped area north of the river, see Plate 1. A fixed amount of funds was programmed and appropriated for this purpose. A contract was awarded for preparation of Plans and Specifications to American Engineering Company of Lexington, Kentucky; who in turn contracted with Rhodes and Associates, also of Lexington, to conduct a geotechnical investigation. This investigation revealed soft, compressible soils along the riverbanks at the proposed bridge site. They recommended installation of wick drains in the foundation to speed foundation settlement, in conjunction with staged construction (7 10-feet lifts) to achieve stability of the embankments at the bridge abutments. This procedure was considered by the Corps of Engineers Project Manager too uncertain with respect to both cost and time.

The Geotechnical Branch of the Louisville District Corps of Engineers was asked to confirm the weak foundation conditions and to develop alternative foundation treatments which would allow expedited construction. To these ends, a very limited supplemental investigation (2 borings) was conducted and confirmed the weak nature of the bridge abutment foundations.

FOUNDATION CONDITIONS

Based on the limited number of borings, the foundation at the bridge site was taken as medium stiff to stiff brown and gray alluvial silty clay, with silt lenses, from the surface, approximate elevation 420, down to elevation 360, underlain by silty, sandy gravel extending down to about elevation 330, where weathered shale was encountered, see Plate 3. Placement of the rock-fill embankments with no foundation preparation would have resulted in instability (Short Term Factor-of-Safety equal to 0.85) and significant settlement (38-inches over a period of 13 years). The bridge piers were to be supported on end-bearing H-piling, and thus, foundation settlement would have to be essentially complete to prevent damaging downdrag on the piling. Design strengths of the materials involved are shown on Plate 4. Strengths of the limestone rock-fill and stone column stone are thought to be slightly conservative. The design undrained shear strength of the foundation clay is thought to be accurate. Finally, the foundation clay was assumed uniform over its entire depth.

FOUNDATION STABILIZATION DESIGN

Stabilization of the foundation was comprised of four components, see Plates 2, 5, and 6.

1. The clay foundation was undercut as much as possible, down to river level, approximate elevation 390, to be replaced with rock-fill. This measure improved the short term factor-of-safety to about 1.0 and reduced the calculated foundation settlement to about 18-inches over 3-years.
2. The second element was to improve the strength of the remaining foundation clay with installation of stone columns, see Photograph 1, placed in an equilateral triangular pattern;

   42-inch diameters on 7-feet centers on the south embankment
   (Stone area = 23%, Clay area = 77%), and
   42-inch diameters on 6-feet centers on the north embankment
   (Stone area = 31%, Clay area = 69%).

   Design of the stone columns was performed according to (Bachus and Barksdale, 1983), increased the short term factor-of-safety to about 1.1, and provided foundation drainage.
3. The third stabilization measure was to place wick or strip drains (Koerner, 1986) in the foundation clay landward of the stone columns to vastly expedite foundation settlement within these areas, see Photograph 2.
4. The last item was to install instrumentation, see Plates 6 and 7, to monitor foundation stability, piezometric levels, and settlement. Analyzes of this data aimed at controlling the pace of embankment construction so that adequate stability was always maintained, and at determining when little, additional settlement would occur so that H-piling to actually support the bridge, could be driven.

The stabilization design thus assured that with the stone columns and wick drains in place, relatively rapid consolidation of the foundation clay occurred, on the order of 6-months. Assuming consolidation of the clay within the stone column area alone (undrained to drained strengths), the factor-of-safety increased from 1.1 to 1.3. With full consolidation of the clay within both the stone column and wick areas, the factor-of-safety increased from 1.1 to 1.7.

CONSTRUCTION

Prime contractor for the project was E.H. Hughes Company of Louisville, Kentucky. Contract was awarded for $8,722,000, which included a 2-mile long access road, as well as the bridge.

Stone columns and wick drains were installed by GKN Hayward Baker of Tampa, Florida, with only minor modifications during the following periods:

May 1987; North and south abutments undercut from about elevation 420 down to 390 (by Hughes)
July 31- August 11, 1987; 245 stone columns installed in south abutment
August 25-26, 1987; 265 wick drains installed in south abutment

August 12 - September 3, 1987; 380 stone columns installed in north abutment
September 12-15, 1987; 585 wicks installed in north abutment

South Bank

Placement of the rock-fill on the south abutment began on September 18, 1987. On September 22, a small crack was observed adjacent to the river. Rock-fill placement was abruptly halted on September 25. By September 28, the crack had lengthened and widened substantially. Visual observation of the failed portion of the embankment relative to the adjacent natural riverbank revealed that THE SOUTH EMBANKMENT HAD FAILED ALONG A PREVIOUSLY FAILED SCARP, RIGHT AT THE RIVER'S EDGE. The bank was unloaded adjacent to the river and two additional borings were drilled in this area. Both borings showed a zone of very soft river clay (muck) with N-values less than 6, underlain by stiffer clays with N-values in the 10-13 range. The extent of the very soft clay noted in the borings was consistent with the observed surface cracking and depth at which shear movement was detected at the two inclinometers. Back-calculation along the failure plane for instability indicated an undrained shear strength of about 350 PSF. Utilizing this value of undrained cohesion for the foundation clay from the edge of the stone column improved foundation to the river, a remedial design was expeditiously designed, see Plate 8, which included:

Removing a substantial portion of the river bank and replacing it with dumped rock-fill (shear key), and
Moving the embankment back away from the river about 20-feet.

The shear key on the south abutment was constructed during the period Oct 19 to November 10, 1987, at a cost of $120,000. Placement of rock-fill on the south abutment subsequently resumed on November 21, 1987 and was substantially completed on January 21, 1988 (58 days) with no additional problems. Due to approximately two months of lost time and to the inclement weather conditions, the contractor elected to place KDOT #57 stone in the embankment earth cores (through which piling was to be driven) in-lieu of silty clay. No problems were encountered in subsequently driving the H-piling through the stone.

North Bank

On the north side, the design already called for the embankment to be set back about 20-feet from the riverbank. However, two additional borings were drilled to investigate conditions beneath the embankment toe. Based on these borings, an undrained shear strength of 300 PSF above elevation 370 and an undrained shear strength of 1000 PSF below elevation 370 down to the stratum of sand and gravel were utilized for the foundation clay riverward of the stone column improved area. Using these parameters, a rock-fill trench, see Plate 9, was designed which supported the embankment toe until sufficient consolidation occurred to improve the foundation strength. The rock-fill shear trench on the north abutment was installed during the period November 4 -20, 1987, at a cost of $60,000. Rock-fill placement on the north abutment was begun on December 9 and was substantially completed by May 13, 1988 (155 days) with no additional problems.

Bridge Erection

Unforeseen problems were encountered in lifting and placing the very heavy concrete bridge girders (7-1/2 feet deep x 80 to 105-feet long), performed by Javier Steel Corporation, Louisville, Kentucky. The original intention was to float in a 400-ton ringer crane to lift all girders into position. The 1988 drought, however, lowered water levels to the point that the single heavy crane could not be floated in to the site. Instead, two 250-ton Manitowoc 4100 crawler cranes were positioned, one on each side of the river, see Photograph 3. On the south abutment, the rock-fill shear key was adequate to support the heavy crane. On the north abutment, a crane support system had to be constructed at the riverbank to allow the crane to safely reach the massive concrete beams carried to the site on barges. This support system was comprised of driven H-piling, welded cross-members and heavy timber matting.

Instrumentaton

As mentioned above, instrumentation was provided to monitor conditions during construction. The amount of instrumentation had been scaled back to reduce costs. The following instrumentation was provided at each of the two abutments and was considered to be the absolute minimum:

- 4 Well-point piezometers
- 2 Closed, air-actuated (Terra Tec) piezometers
- 2 Settlement plates
- 2 Surface monuments
- 2 Inclinometers.

The locations of these instruments are shown on Plates 6 and 7. Representative plots of data from these devices are shown on Plates 10, 11, and 12. The following general trends were observed in the instrumentation:

Piezometers-(Excellent reliability and accuracy, good agreement between contractor QA and Corps readings, relatively easy to read)

- The behavior of the well-point and closed, air-actuated piezometers was very similar,
- The piezometric levels at the instruments reacted very quickly and consistently with river level fluctuations,
- Negligible pore pressure increases due to embankment construction were noted.

Settlement Plates and Surface Monuments- (Fair reliability and accuracy, relocation and possible movement of reference points made readings difficult, only contractor QA read, high degree of surveying precision required)

On the south embankment, with embankment placed from elevation 388 to 398 (when embankment placement was suspended), the vast majority of foundation settlement due to placement of 10-feet of the embankment occurred in 30 days and thus acted as a field consolidation test),

The settlement plates and surface monuments were in consistent agreement and reacted to the embankment placement,

The settlement plates and surface monuments rose slightly when the river rose,

Total settlement on the south embankment was about 3-inches (15-inches predicted),

Total settlement on the north embankment was about 6-inches (18-inches predicted).

Inclinometers- (Excellent reliability and accuracy, good agreement between contractor QA and Corps readings, relatively easy to read)

Vast majority of lateral movement occurred in the foundation clay, approximate elevation 355 to 390,

At the south abutment, about 2 inches of lateral movement toward the river occurred,

At the north abutment, about 4 inches of lateral movement toward the river occurred,

By plotting the lateral movement at a fixed elevation vs time, could use as a measure of time rate of consolidation.

CONCLUSIONS

Based on the overall performance of the project and the instrumentation, the following conclusions can be drawn:

- -The combination of natural sand lenses, stone columns and wick drains made the foundation essentially free-draining,
- -There was essentially no pore water pressure build-up in the foundation due to embankment construction,
- -The vertical and lateral movements in the foundation occurred under short term conditions,
- -The stone columns were absolutely essential in providing increased foundation strength, especially during construction, and providing foundation drainage, see Plate 13,
- -The wick drains allowed rapid consolidation of the foundation clays, see Plate 14,
- -Negligible settlement within the rock-fill occurred,
- -The limited amount of instrumentation provided consistent and timely monitoring of the performance of the embankments. It should be noted that by monitoring this instrumentation, it was possible to allow placement of the embankments faster than planned and thus to make up for the delays resulting from the required foundation modifications (shear key on the south and shear trench on the north).

LESSONS LEARNED

Needed a more complete geological reconnaissance of the area, including more borings in the immediate river bank area,
Needed more flexibility in design of the bridge-
Bridge length and costs were set early in the project and could not be increased to accommodate poorer than anticipated foundation conditions,
Stone columns/wick drains were well-suited for the foundation stabilization,
Instrumentation was invaluable in monitoring foundation performance to allow rapid solutions to unforeseen problems, and
Excellent communication between the contractors and Corps elements facilitated rapid resolution of problems.

ACKNOWLEDGEMENTS

KDOT-Division of Materials, Geotechnical Section
Shared their experience (successes and failures) with stone columns and wick drains,
Corps of Engineers, Ft. Knox Area Office (Construction)
Kept in very close coordination with Louisville Geotechnical Branch (Design)
E.H. Hughes-
Patience while corrective foundation modifications were developed.
Close coordination with them and their QA contractor, Rhodes and Assoc.
Dr. R. D. Barksdale, Consultations on feasibility and design of stone columns

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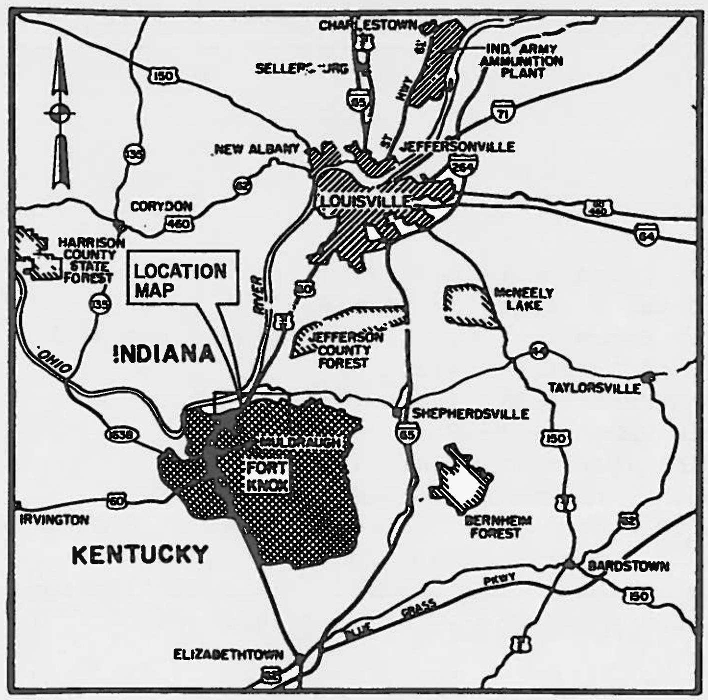
PLATES

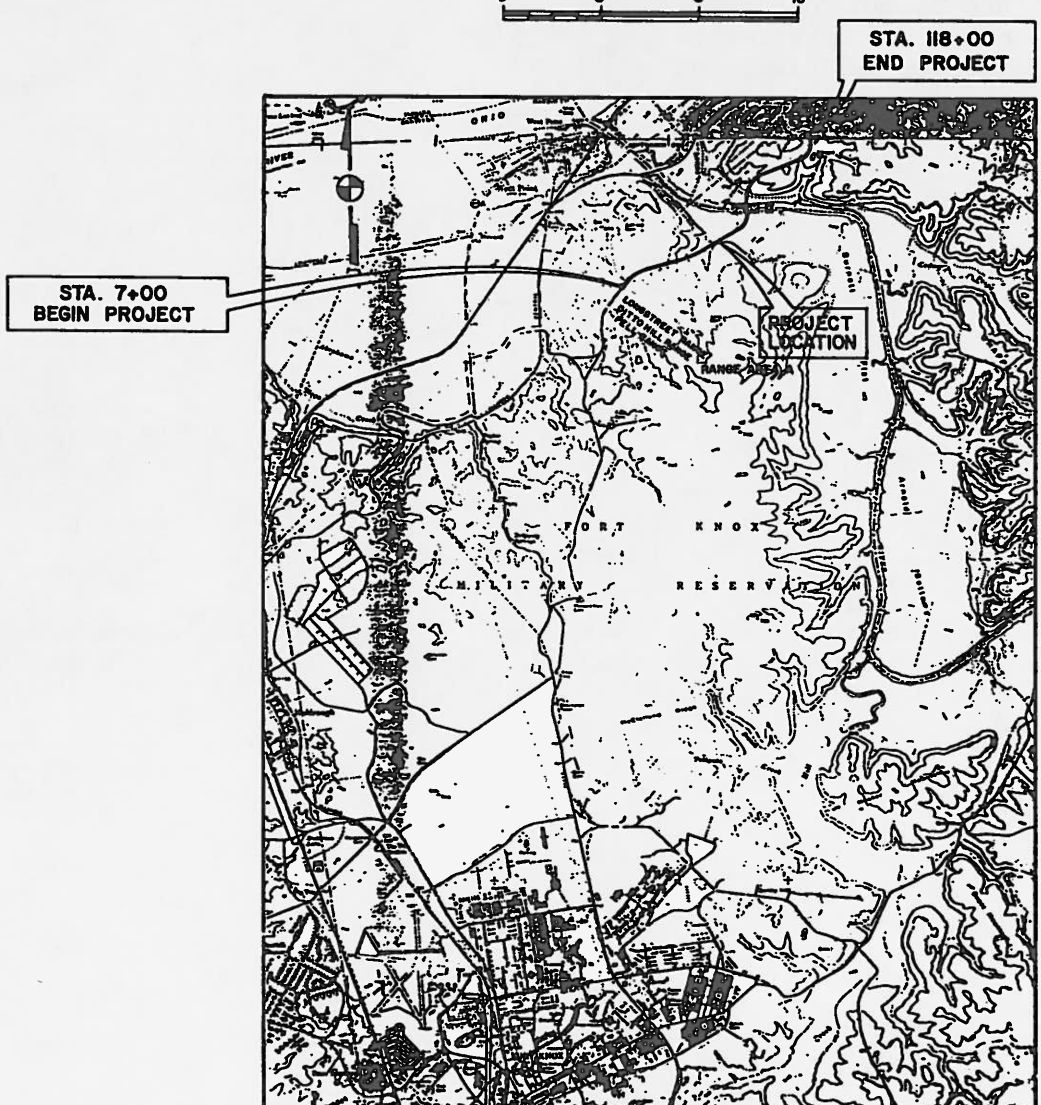
PHOTOGRAPHS



VICINITY MAP

SCALE: 1 IN. = 8 MI.

8 0 8 16



LOCATION MAP

SCALE: 1 IN. = 4000 FT.

4000' 0 4000' 8000'

PLATE 1
Vicinity Map

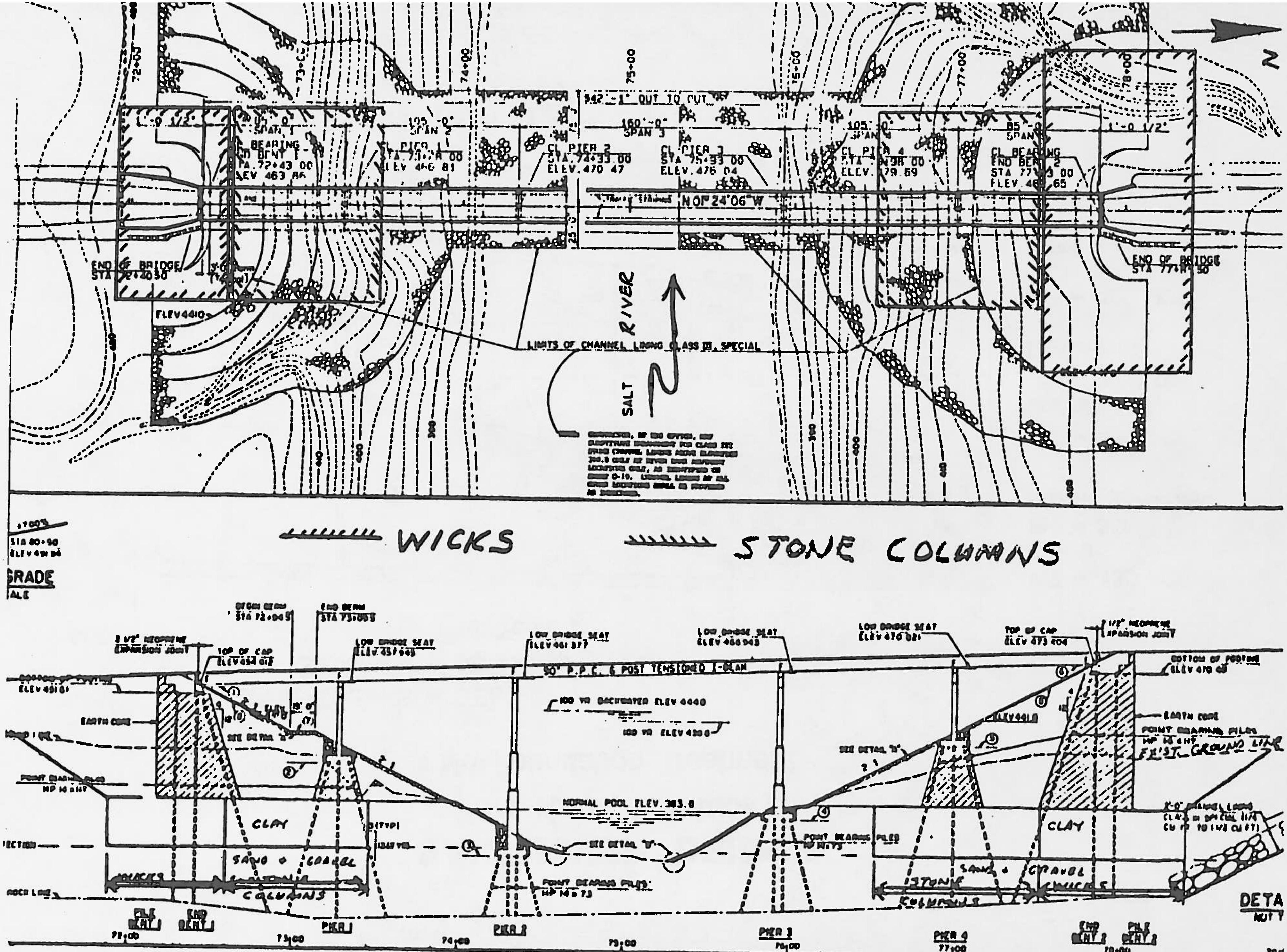
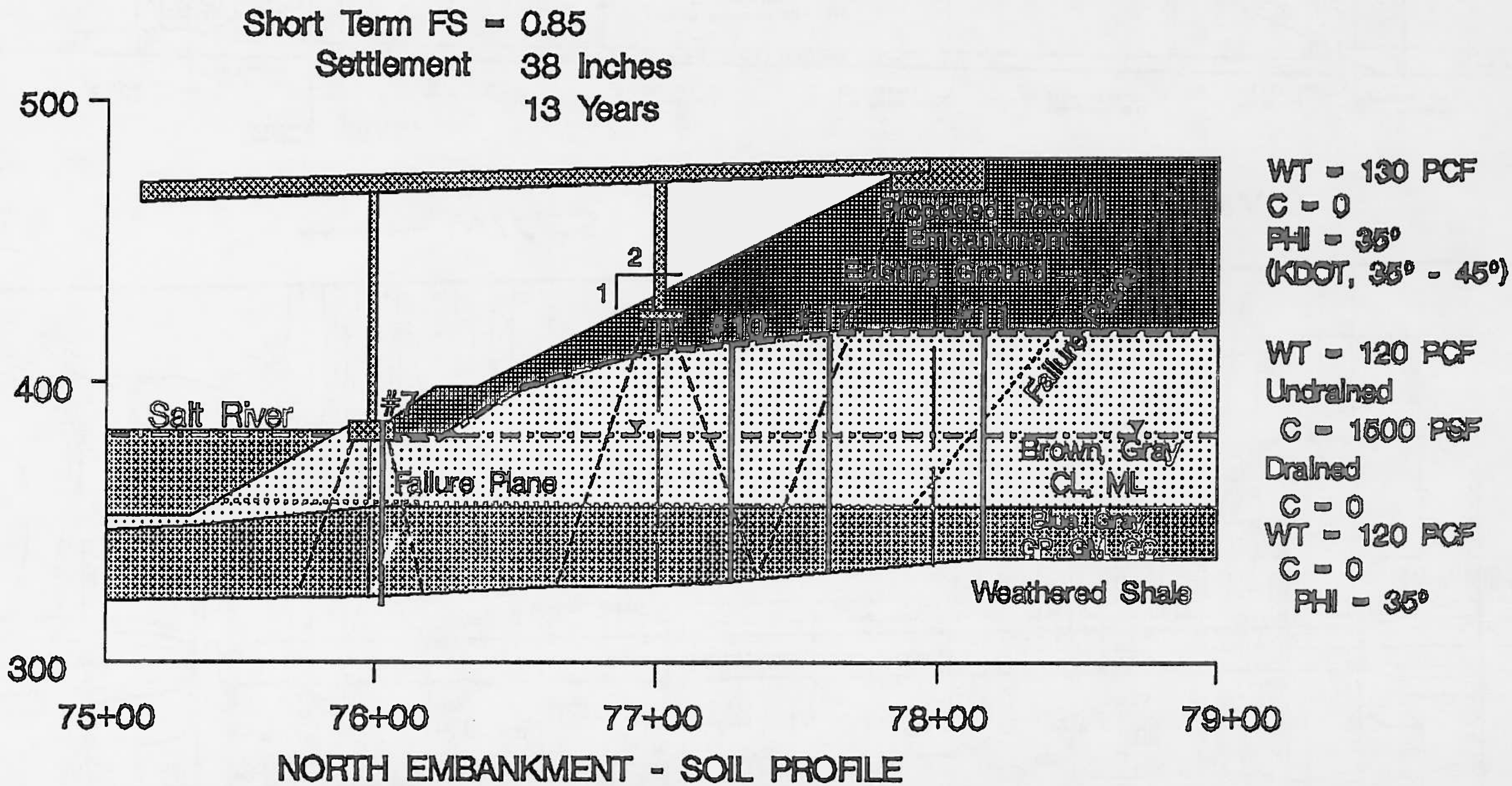


PLATE 2
Site Plan

SALT RIVER BRIDGE
Fort Knox, Kentucky
No Foundation Treatment



Salt River Bridge

Material Strength Properties (Both Banks)

Material	Source	Selected Value
Foundation "Clay"	A/E, COE Lab Test	Short Term c=0.75TSF d=0 Long Term c=0 d=30"
Limestone Rockfill	Kentucky Dot	35
Stone Columns Stone	FHA Design Manual 40-45	40

SALT RIVER BRIDGE

Fort Knox, Kentucky

North Embankment
With Stone Columns & Wicks

Short Term FS = 1.1
Settlement 18 Inches
6 Months

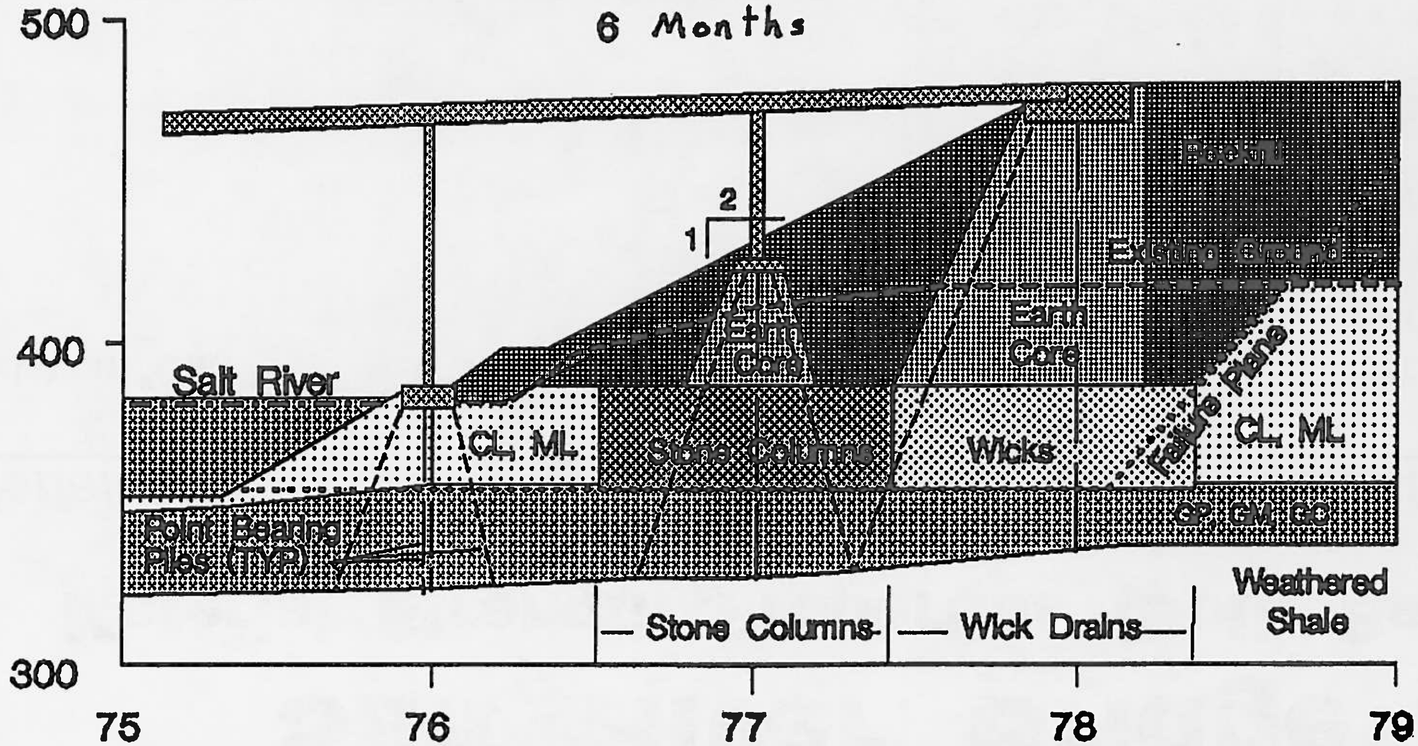


PLATE 5

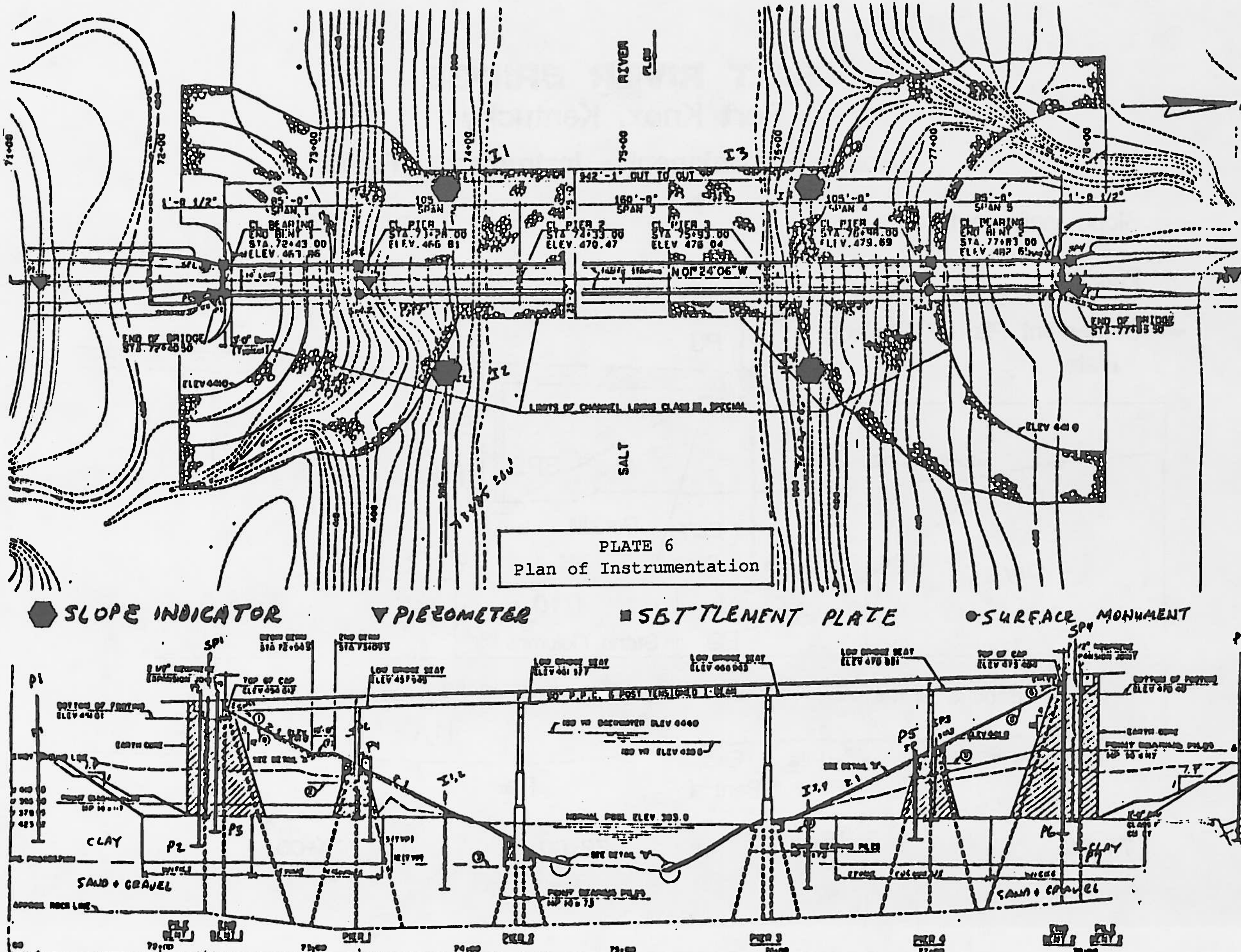


PLATE 6
Plan of Instrumentation

SALT RIVER BRIDGE
Fort Knox, Kentucky
South Embankment - Instrumental Profile

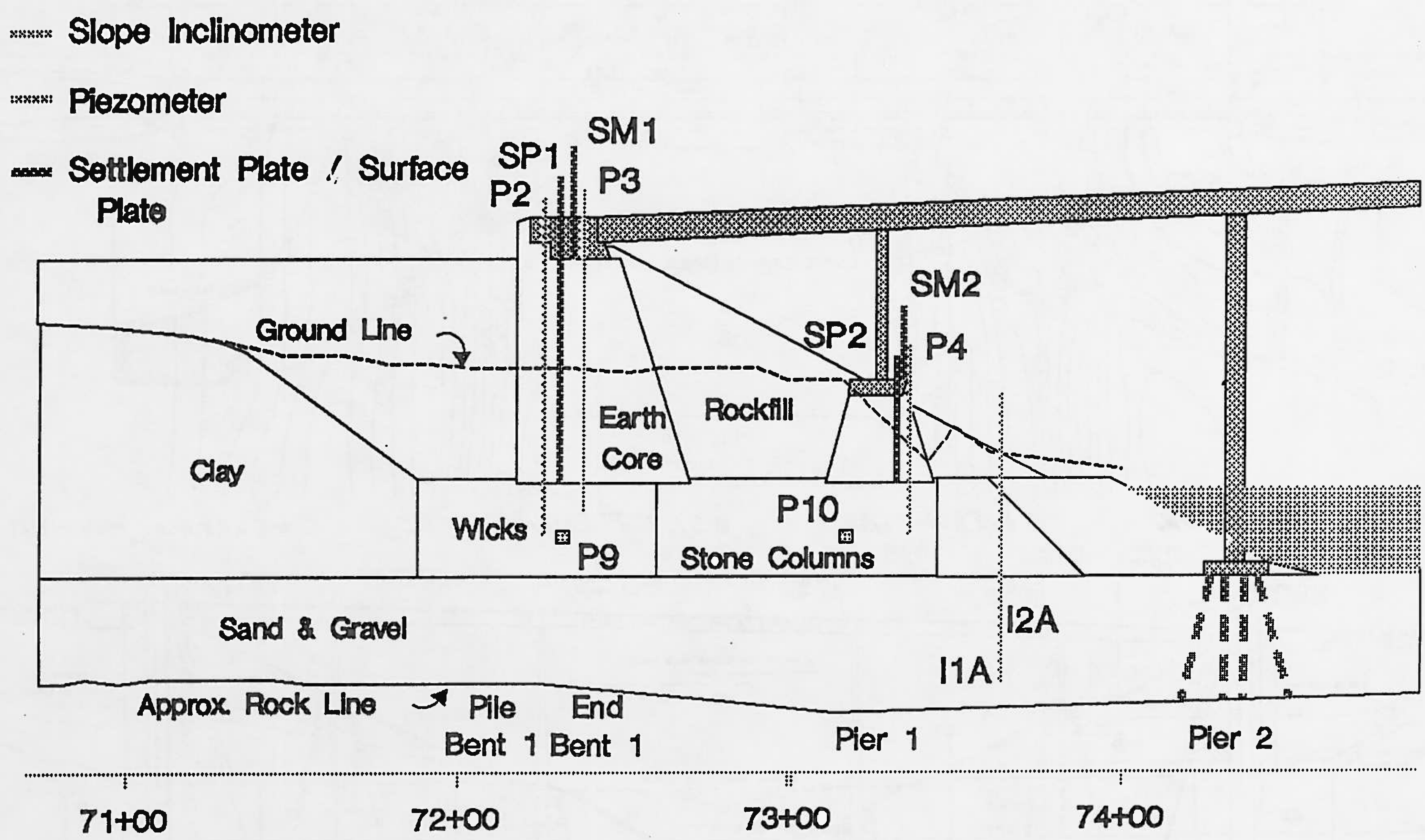


PLATE 7

SALT RIVER BRIDGE
Fort Knox, Kentucky
South Embankment
Shear Key

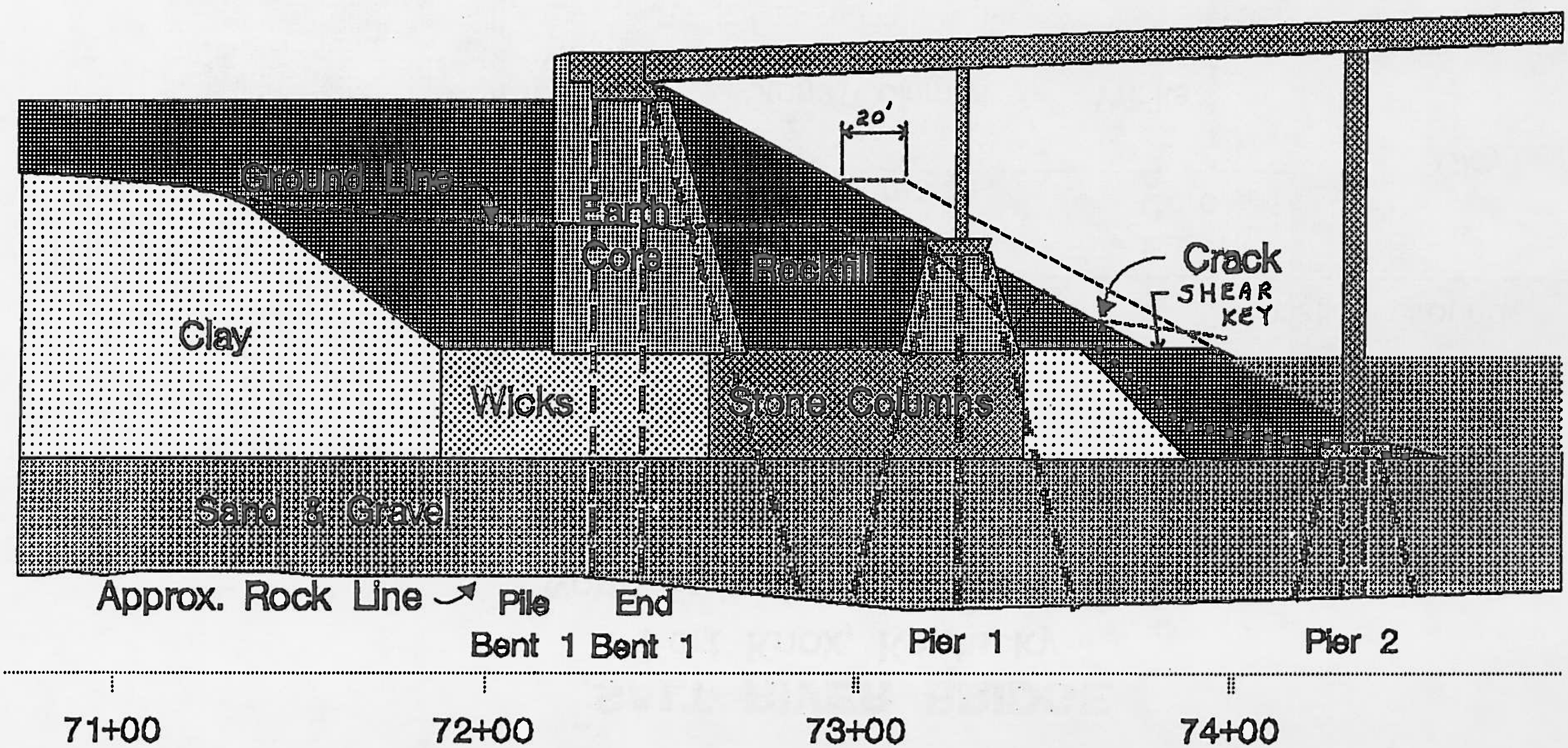


PLATE 8

SALT RIVER BRIDGE
Fort Knox, Kentucky
North Embankment - Shear Trench

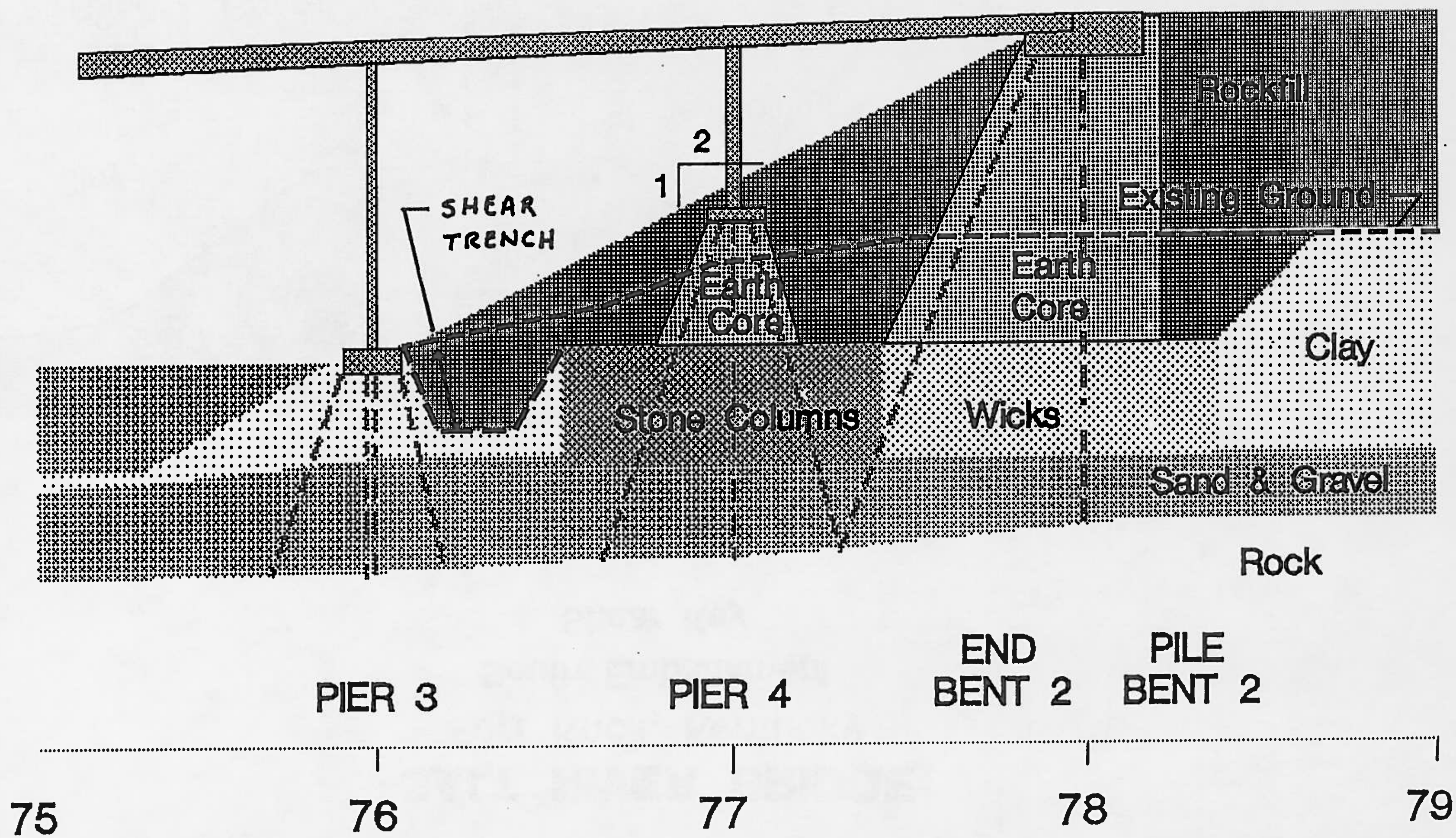


PLATE 9

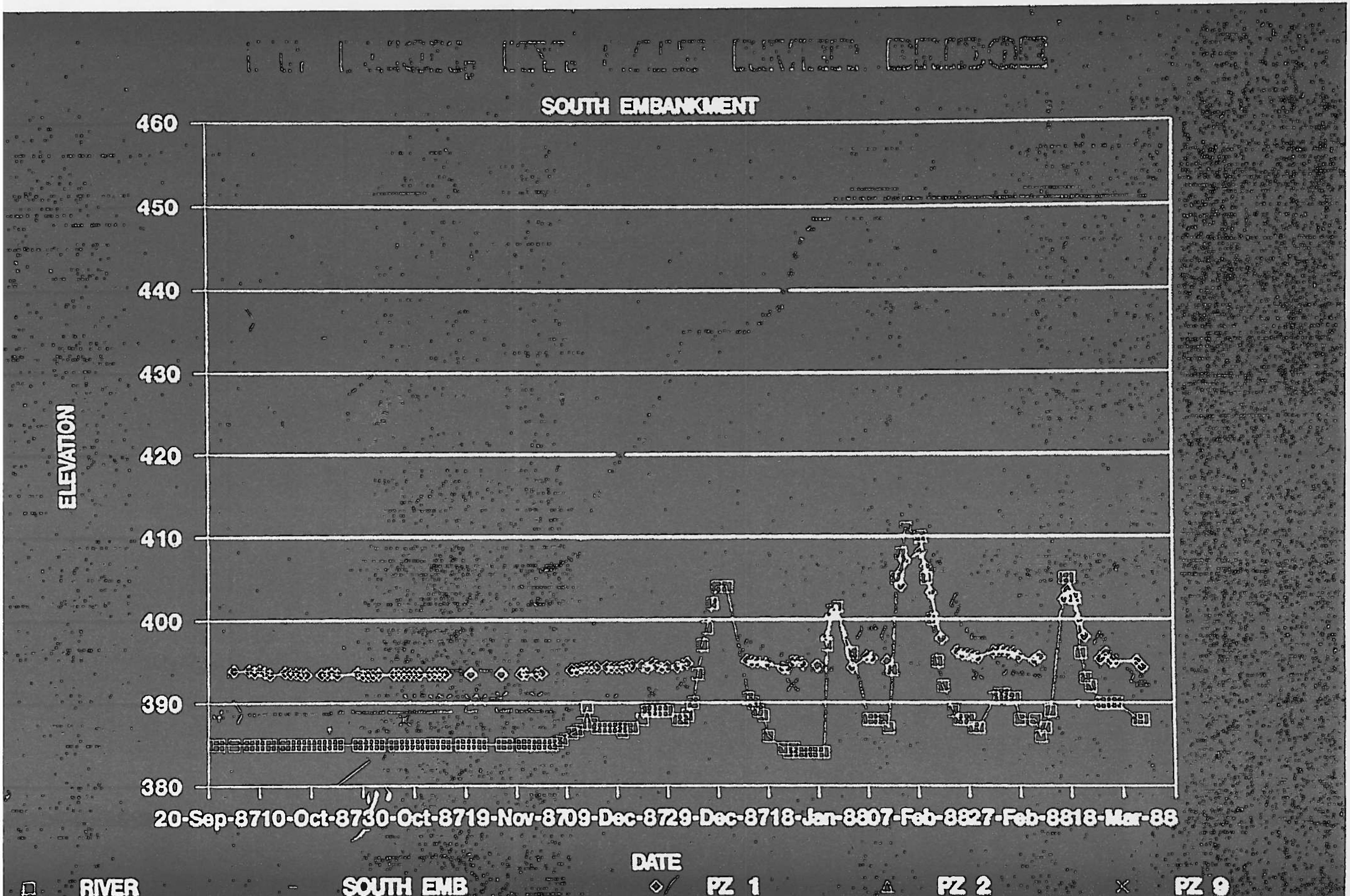


PLATE 10
South Embankment
Piezometer Plots

Ft. Knox, Ky. Salt River Bridge
North Bank
Settlement Plates & Surface Monuments

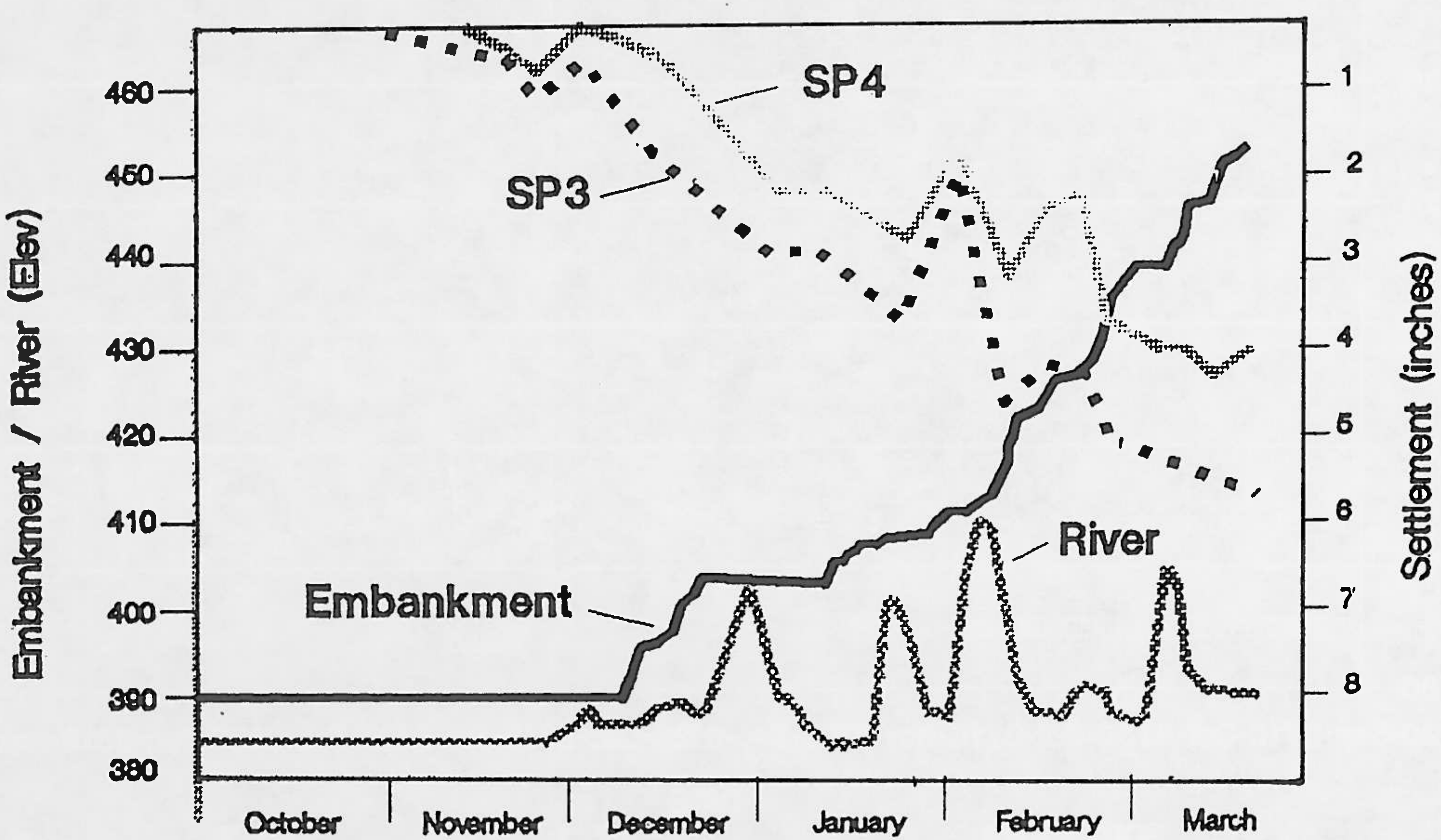


PLATE 11
North Embankment
Settlement Plate & Surface Monument Plots

PROJECT: SALT RIVER BRIDGE

HOLE NO. I4AA

Acum-Acuminitial

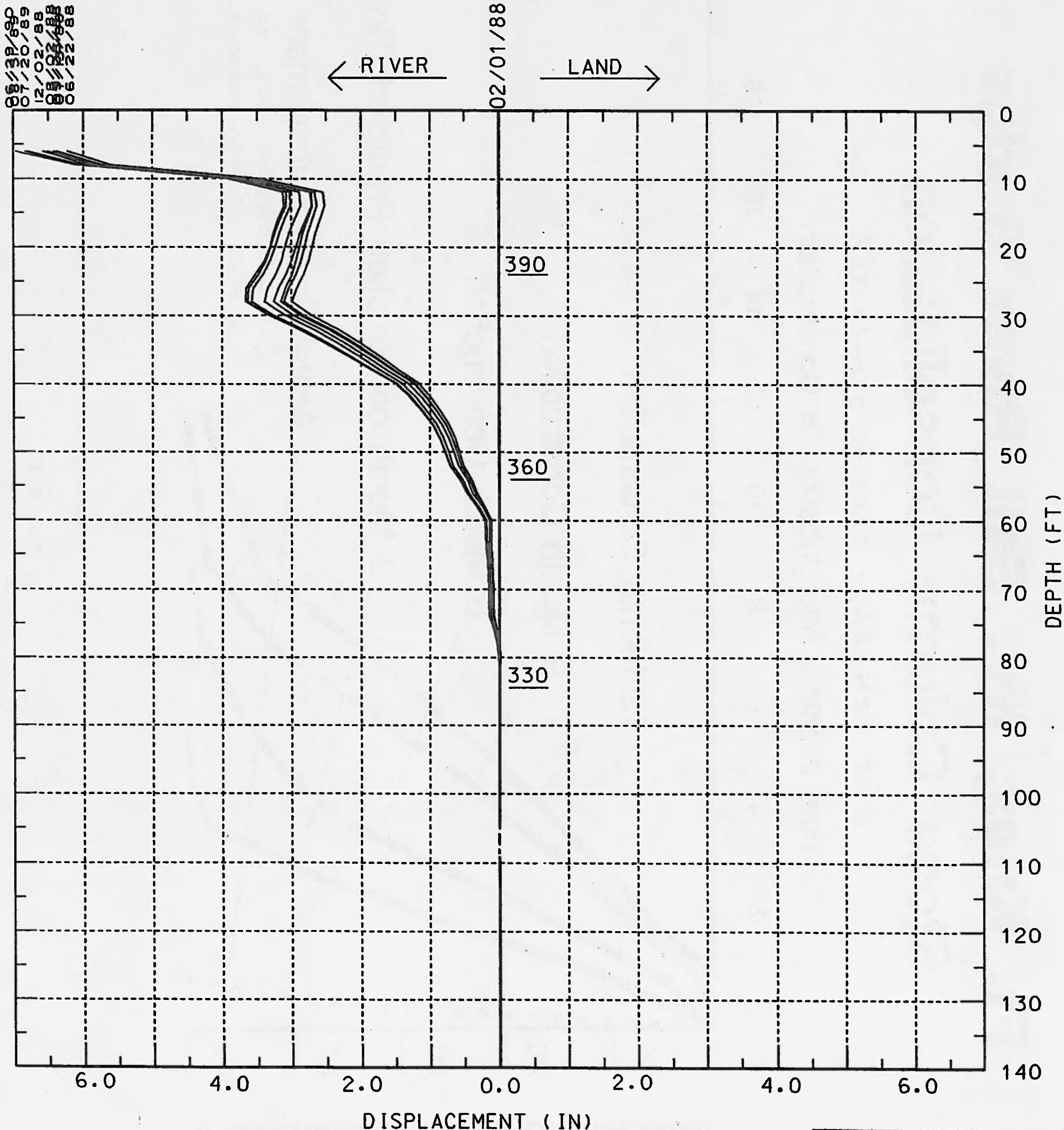


PLATE 12
North Embankment
Inclinometer Plots

Ft. Knox, Ky. Salt River Bridge
Stone Column Consolidation

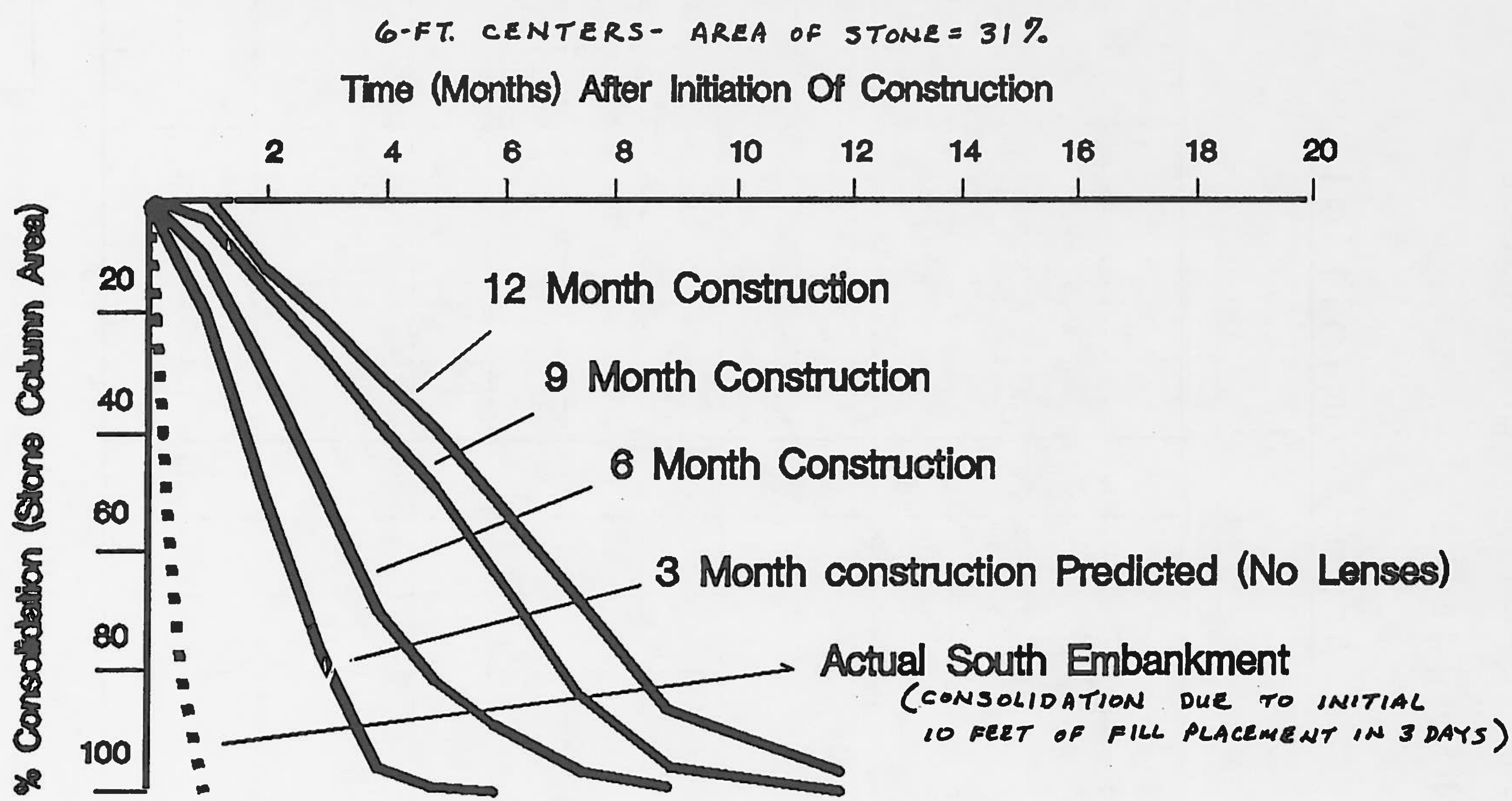
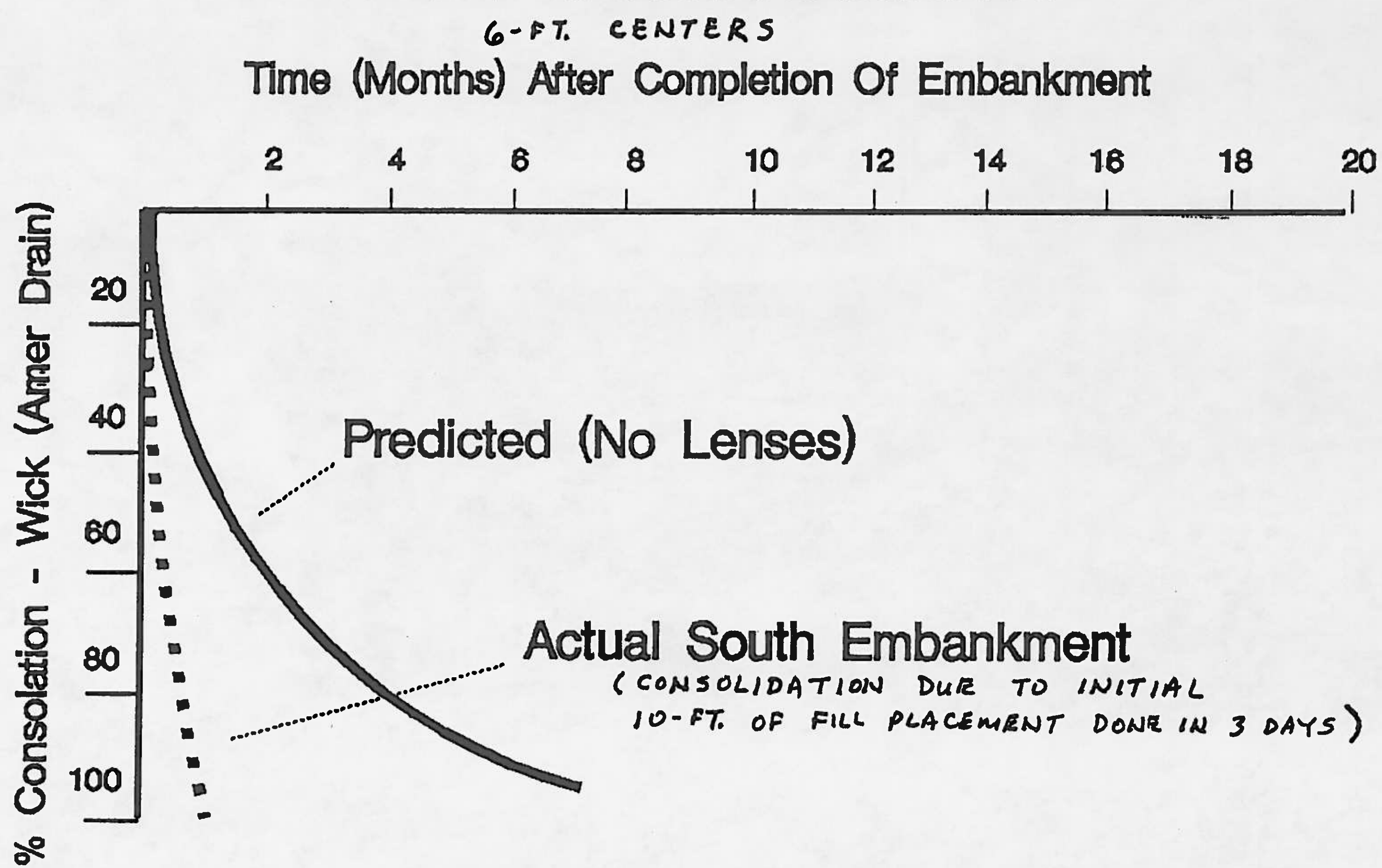
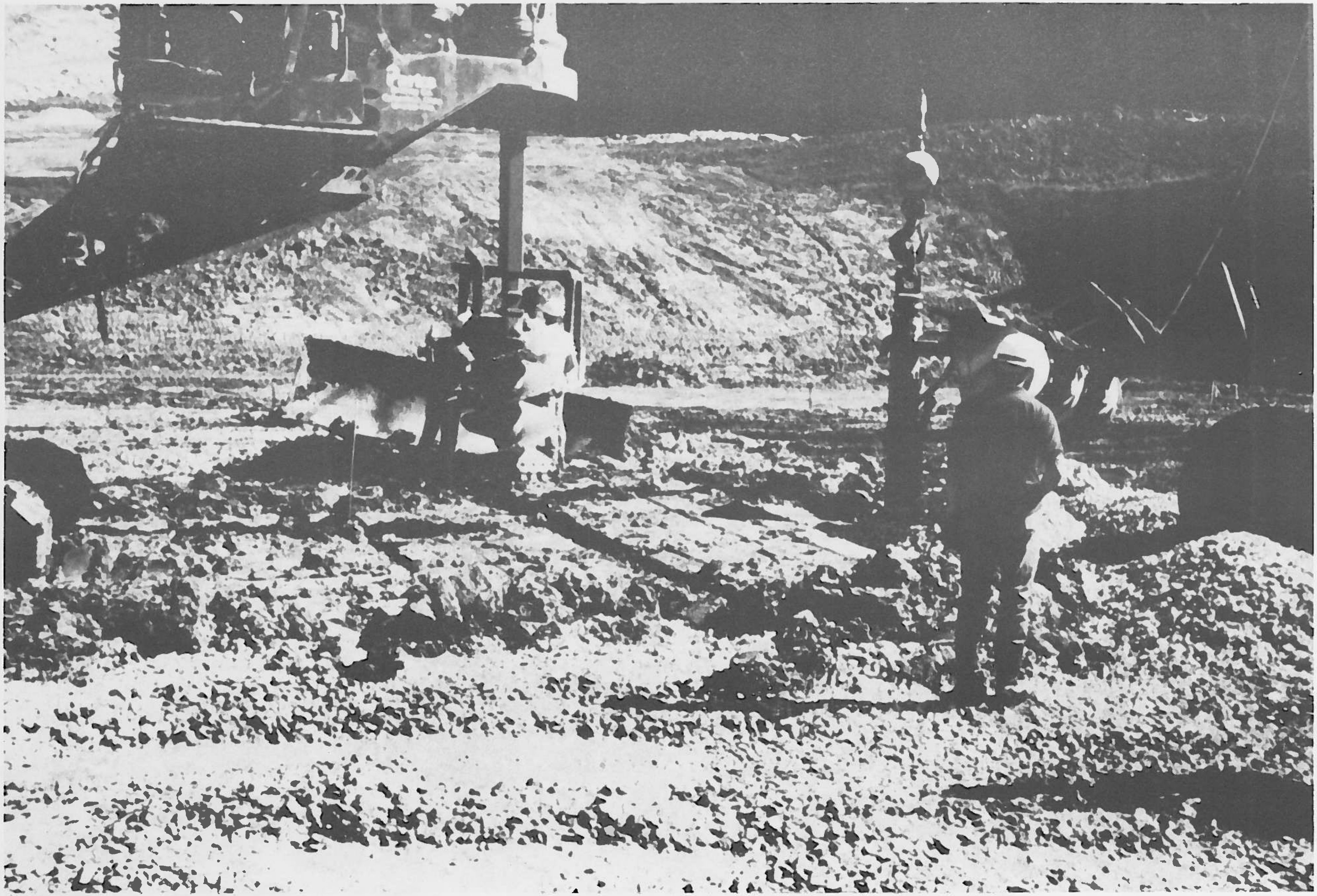


PLATE 13

Ft. Knox, Ky. Salt River Bridge
Wick Consolidation





33"DIA AUGER

PHOTOGRAPH 1
Stone Column Construction

VIBRATOR

PHOTOGRAPH 2
Wick Drain Installation

PHOTOGRAPH 3
Bridge Girder Placement

SAND DRAIN INDUCED CONSOLIDATION OF A PEAT

E. Gregory McNulty, PhD, PE
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Lexington, Kentucky

ABSTRACT

Several investigators have suggested that sand drains cannot accelerate the consolidation of peat because of the predominance of creep effects. In addition, some have claimed that radial decreases in hydraulic conductivity due to increases in effective stress will greatly retard consolidation with sand drains. The author compares predictions from a nonlinear, large-strain finite element analysis with field measurements. This paper shows that accounting for field disturbance caused by construction activities correctly predicts field settlements. In addition, these analyses show that ignoring a radial variation in properties results in a small error in predicted rates of settlement.

INTRODUCTION

Several investigators (Lake, 1961 and 1963; Casagrande and Poulos, 1969) have concluded that sand drains fail to accelerate the settlements of a peat deposit. In particular, one investigation (Lake, 1961) concluded that drainage distance had no effect on the rate of settlement. This conclusion implies that creep, not dissipation of excess pore pressures, controls the consolidation of peats. In addition, another investigation (Barron, 1978) postulated that radial changes in hydraulic conductivity due to changes in effective stress would greatly affect sand drain performance. This paper shows that dissipation of excess pore pressures controls the consolidation of a peat when one accounts for field disturbance caused by construction activities. Furthermore, this paper shows that radial decreases in the hydraulic conductivity near the sand drains only slow the rate of settlement slightly. The following sections discuss a case history involving detailed field measurements, comprehensive laboratory testing, and analysis by a finite element code.

FIELD MEASUREMENTS

In 1974, the widening of the Eastern New Hampshire Turnpike of I-95 provided an excellent opportunity to investigate the efficacy of drains in peat. Construction included about 244 m (800 ft.) of roadway built through a tidal marsh of peats, organic silts, and clays near Hampton-Hampton Falls in the area of Taylor River. The site consisted mostly of tidal marsh whose ground elevation ranged from about -1m (msl) in the tidal backwater to 1.5m (msl) to the toe of the existing embankment. Before placement of pavement, construction of high embankments of about 5m served as surcharge with sand drains to accelerate settlements. To avoid slope stability failures of the embankment during construction, the New Hampshire Department of Public Works and Highways installed settlement platforms, piezometers, slope indicators and alignment stakes.

The most critical section involved soils between Stations 5913 m (194+00 ft) and 5944 m (195+00 ft). Figure 1 gives an approximate profile view of the soil stratigraphy and the final embankment at Station 5928 m (194+50 ft). Table 1 summarizes the subsurface stratigraphy before start of construction.

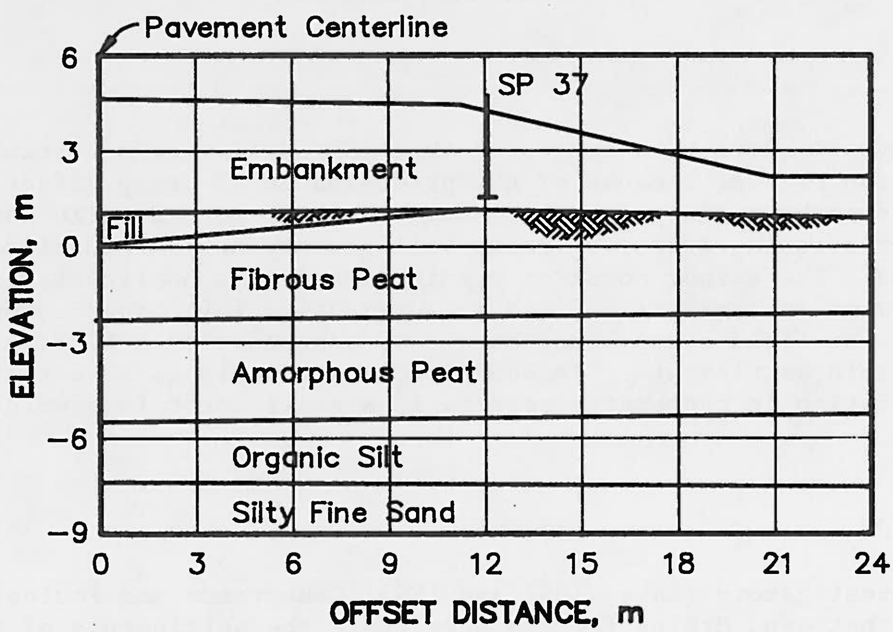

Figure 1. Profile view of embankment and underlying stratigraphy near Station 5928 m (194+50 ft) at Taylor River.

Table 1. Subsurface Stratigraphy Before Start of Construction

Layer	Amorphous Peat	Thickness m	Total Unit Weight kN m^{-3}
1	Fibrous Peat	2.9	102.1
2	Amorphous Peat	3.4	103.7
3	Organic Silt	1.7	142.9
4	Sand	---	-----

Because of the gradual transition from one peat to another, uncertainty exists in the layer thicknesses shown in Figure 1. Unpublished analyses subsequent to those reported earlier (McNulty, 1982) suggest that the numerical solution has only a small sensitivity to the individual peat layer thicknesses.

Figure 1 shows the cross-section at the location of Settlement Platform (SP) 37 where readings started on June 22, 1974. Subsequent discussion will refer to June 22, 1974 as Construction Day 22. A slope inclinometer at R29.1m

(R95.5 ft) and alignment stakes at R48.8m (R160 ft) allowed measurement of horizontal movements during construction of the embankment. Construction of a layer of clean sand started at Day 20 to serve as a drainage layer for the sand drains. Mud waves were observed as bulldozers pushed the sand into place (Recker, 1982). By Day 46, lateral movements about 2.1m (7 ft) and ultimately reaches 2.4m (8 ft) shortly after installation of sand drains around Day 50. A jetting tool installed the sand drains into 7.9m (26 ft) of compressible soil. Each drain had a radius of 0.15m (0.5 ft) in a triangular pattern with a 1.2m (4 ft) spacing. The water table lay at the original ground surface. Throughout construction of the 5m high embankment, the alignment stakes showed no further lateral movements. Installation of slope inclinometers at Day 90 resulted in no slope inclinometer data during movement of the alignment stakes. Nevertheless, the settlements observed at SP 37 probably relate directly to the mass movements induced by the placement of the sand blanket by the bulldozers. Analyses to be discussed later in this paper suggest that observed settlements occurred at stresses too low to be associated with subsidence due to dewatering or dissipation of pore pressures during consolidation.

Construction of the main embankment started on Day 87 and continued until Day 157 when the elevation reached about 5 m. At Day 340, bulldozers cut the embankment back to an elevation of about 3.8 m for construction of the pavement.

LABORATORY DATA

Laboratory data included index tests, incremental and radial inflow consolidation tests. Table 2 (adapted from McNulty, 1982) summarizes ranges in index properties for the compressible soils.

Table 2. Soil Properties

Soil Type	Total Unit Weight kN m-3	Natural Water Content %	Liquid Limit %	Plastic Limit %
Fibrous Peat	92.7 - 113	200 - 875	325 - 650	100 - 375
Amorphous Peat	102 - 121	125 - 375	150 - 300	50 - 150
Organic Silt	132 - 157	45 - 175	45 - 135	30 - 60

While Table 2 gives high natural water contents and Atterberg limits, other investigators (Casagrande & Poulos, 1969; Murray 1971) have found similarly high values.

Conventional incremental tests were performed (Haley & Aldrich, 1974) on 6.35 cm (2.5 in) diameter by 1.9 cm to 2.54 cm (0.75 to 1.00 in) thick undisturbed specimens taken from 7.62 cm (3 in) Shelby tubes. Three inflow radial and 2 vertical flow tests were performed (Trautwein, 1980) on undisturbed specimens of fibrous peat taken from 7.62 cm (3 in) Shelby tubes. Free strain theory was used to reduce data for the radial inflow tests. No completely remolded specimens underwent laboratory tests.

Figure 2 gives the consolidation coefficients for the fibrous peat. Figure 2 shows that the radial coefficients typically exceeded the vertical by about factor of 10. Space limitations prevent the showing of the vertical consolidation coefficients for the amorphous peat and organic silt. These data and recommendations to use the upper range of the scatter bands are discussed elsewhere (McNulty 1982, Figure 7.4b and c). Because the fibrous peat had C_r/C_v ratio equal to 10, it was assumed that C_r/C_v also equaled 10 for the amorphous peat and organic silt.

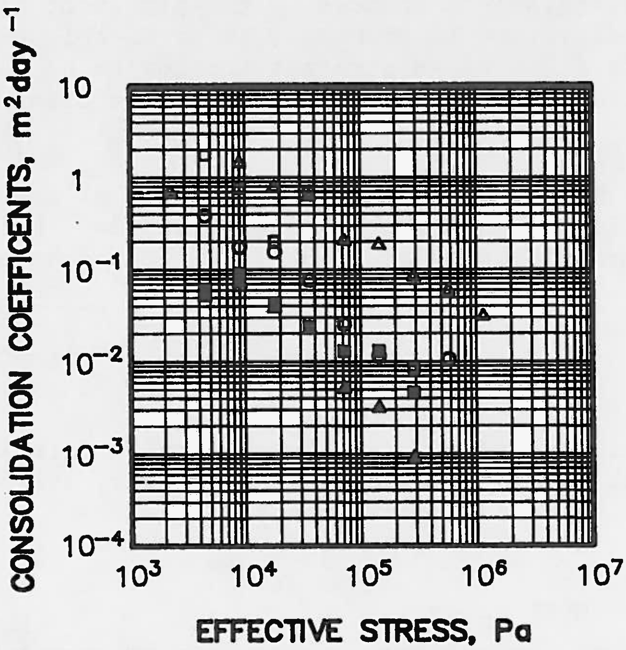

Figure 2. Consolidation coefficients for fibrous peat: radial (open symbols) and vertical (solid symbols).

Figure 3 gives the ranges in stress-strain behavior for the peats and organic silt. As the next section will discuss, calibration of predicted against observed settlements yielded the disturbed curves.

In a finite element analysis involving layered systems one must convert the consolidation coefficients to the fundamental parameters of hydraulic conductivity, k, and specific storage, S_s. The vertical coefficient of consolidation C_v equals $k/(\gamma_w m_v) = k/S_s$, where m_v equals the soil compressibility and γ_w equals the unit weight of water. The use of consolidation coefficients to satisfy continuity of flow between soils of different compressibilities results in an incorrect ratio in the hydraulic gradients at the layer interfaces (McNulty, 1982, Pg. 40). Instead, one can calculate the compressibility, m_v, from the stress-strain curves in Figure 3.

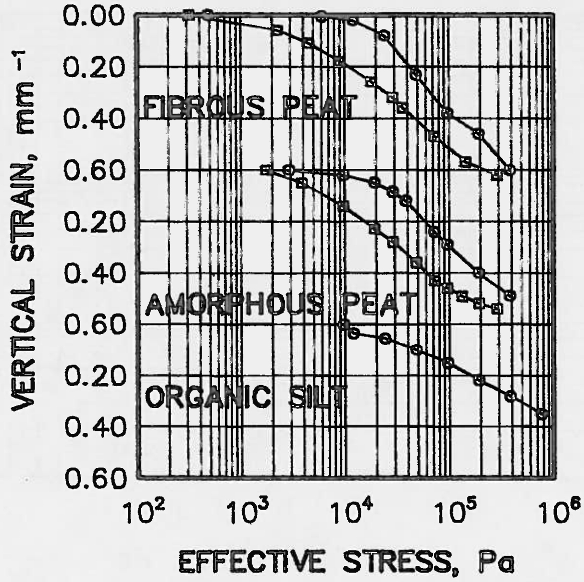

Figure 3. Ranges in stress-strain curves for laboratory tests where upper curves (open circles) represent undisturbed and lower curves (open squares) represent disturbed soils.

Then one calculates the hydraulic conductivity as a function of vertical strain from the corresponding consolidation coefficients. A regression fit of the resultant k versus strain data with a low degree polynomial allows transformation of k to a function of effective stress. Mapping of strains to the stress-strain curves transforms k to a function of effective stress. Given the new smoothed relationship of k with effective stress, one can then calculate specific storage from k and C_v: $S_s = k/C_v$. Figure 4 shows the variation of hydraulic conductivity and specific storage with effective stress for the fibrous peat. Note that the hydraulic conductivity decreases smoothly as the effective stress increases. In contrast, the specific storage shows radical changes with effective stress. Typically, S_s will increase an order of magnitude or more as the effective stress exceeds the preconsolidation stress.

TIME-SETTLEMENT ANALYSES

The code SUBFE, an extension of the one developed earlier (McNulty, 1982), predicted settlements versus times at SP 37 using laboratory and calibrated (backed-in) soil properties. The code allows nonlinear variations in the specific storage and hydraulic conductivity through input of the coefficients of consolidation and hydraulic conductivity with effective stress. Finite element shape functions interpolate the state variable excess pore pressure at each gauss point from the element nodes. SUBFE assigns different soil properties to each gauss point to allow a pointwise radial and vertical variation of hydraulic properties with effective stress. An Updated Langrangian formulation (Belytschko, 1983) continuously adjusts the finite

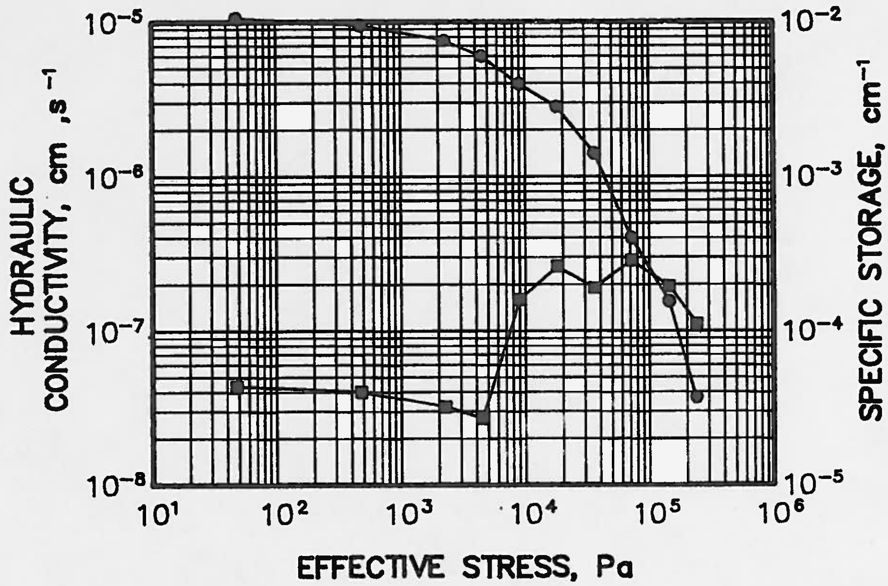

Figure 4. Variation of hydraulic conductivity (solid circles) and specific storage (solid squares) of fibrous peat with effective stress.

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element mesh to account for large one-dimensional vertical strains calculated with small strain incremental theory. In addition, the code represents any external loadings as equivalent fill elevation with time. Furthermore, SUBFE accounts for settlement dependent submergence effects on applied load due to changes in the elevation of the water table. The code allows for changes in boundary conditions at any time, including the installation of sand drains or recovery wells at any time. This code has been verified with closed-form solutions, a nonlinear finite difference algorithm, and published results for a nonlinear, large-strain theory (McNulty, 1982, Chapter 5). Also, partial validation of the code with independent experimental data from a large-strain consolidation test has been accomplished (McNulty, 1982, Chapter 5).

Analyses with Laboratory Soil Properties

Figure 5 compares the measured settlement with those from finite element analyses based on "undisturbed" laboratory properties for cases of no drains and drains installed. Sand drains appear to have substantially reduced the consolidation times. However, predicted settlements with drains underpredicted the rate of settlement before drain installation and overpredicted the rate of settlement after drain installation. Before installation of the sand drains, measured settlements occurred at stresses too low to be associated with subsidence due to dewatering. In addition, the alignment stake data strongly suggest that mass flow caused these movements. Consequently, predicted initial settlements should include settlements caused by mass flow to allow proper consideration of settlements caused by subsidence due to dewatering or consolidation due to dissipation of excess pore water pressures.

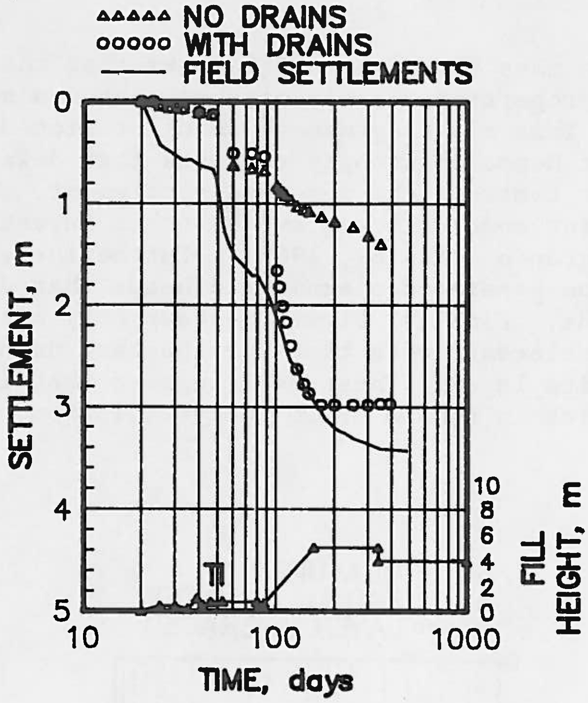

Figure 5. Finite element prediction based on "undisturbed" laboratory properties assuming no drains installed (open triangles), drains installed at Day 50 (open circles), and measured field settlements (curves without symbols).

However, even without adjusting the predicted settlements for mass movements, Figure 55 shows that the predicted settlement rates exceed the measured rates after drain installation. Disturbance caused by mass movements and jetting of the sand drains may explain the slower than predicted time rate of settlement in the field. Because of such disturbance, the soils in the field probably incurred corresponding reductions in the hydraulic conductivity. Therefore, the hydraulic properties measured in the laboratory probably exceed those controlling settlement in the field.

Analyses with Calibrated Soil Properties

Trial and error adjustment of soil properties can produce calibrated soil properties that can predict measured settlements. Because the number of unknowns exceeds the knowns in our problem, this inverse formulation will not yield a unique solution. However, such calibrated data can confirm the physical effects of soil disturbance in the field. Figure 2 gives the disturbed stress-strain curves derived for the peat materials used in the final calibration analyses. Another reference (McNulty, 1982, Figures 7.8 and 7.9) gives the related data on hydraulic conductivity and specific storage. Figure 6 shows the finite element predictions based on "calibrated" or "backed-in" soil properties for two cases: one with radial variation of properties and another with soil properties averaged over the entire layer. Note that the settlements predicted before drain installation now include those associated with mass flow. Figure 6 shows that the prediction using a radial variation of properties closely matches measured settlements after sand drain installation. This close agreement among predicted and measured settlements in a peat deposit strongly suggests that dewatering caused by consolidation of peat controls the observed settlement. This result strongly contradicts the earlier conclusions drawn by other investigators (Lake, 1961; Lake, 1963; and Casagrande & Poulos, 1969). Furthermore, Figure 7 confirms that fill construction generated piezometric heads that directly correspond to the applied fill loads. Finally, Figure 6 shows only a slight increase in rate of predicted settlements with time for the case using soil properties averaged over an entire layer. These data suggest that ignoring the radial variation of properties in a sand drain analyses introduces only a small error.

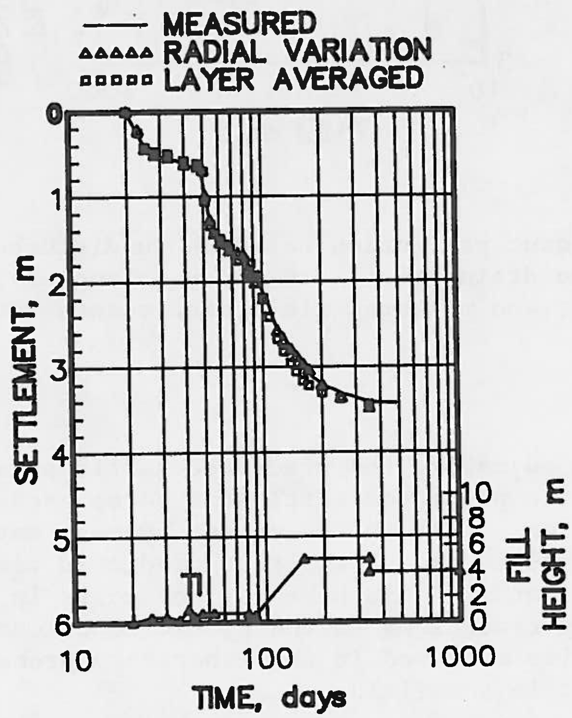

Figure 6. Finite element predictions based on "calibrated" (or backed-in) properties with radial variations of soil properties (open triangles) and averaged layer properties (open squares) as compared to measured field settlements.

ACKNOWLEDGEMENTS The National Science Foundation supported this research through Grant CME-7918195. Also, the New Hampshire Department of Public Works and the firm of Haley and Aldrich, Inc., provided some of the necessary data. Eric Schieve at the University of Texas performed the radial flow laboratory tests. J.T. Collins and S.J. Trautwein of the University of Texas performed the field sampling.

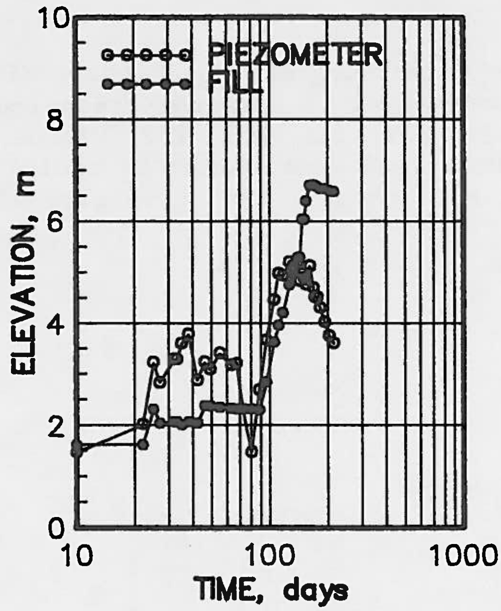

Figure 7. Comparison of measured piezometric response to applied fill load with time.

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SOIL MIXING FOR SOIL IMPROVEMENT
TWO CASE STUDIES

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INTRODUCTION

Although the process of soil mixing originated in the United States in the 1950's, its major development has occurred over the last twenty years in Japan. To date, while there have been thousands of projects performed in Japan using some form of soil mixing, it is only recently that it has found a wide application on sites in the United States. Significantly, it was in the United States that its potential use for remediation of hazardous waste sites was first recognized and implemented. In North America to date, it has been used for foundation block stabilization, retaining walls, cutoff walls systems and the fixation/solidification of contaminated soils, with over forty contracts completed.

Soil mixing is divided into two categories, Shallow Soil Mixing (SSM) and Deep Soil Mixing (DSM) with, as the names imply, different depth capabilities.

SHALLOW SOIL MIXING

The process uses a crane-mounted drill attachment which turns a single-shaft, large diameter auger head (normally 6 ft to 12 ft) which consists of two or more cutting edges and mixing blades. As the auger head is advanced into the soil, grout is pumped through the hollow kelly bar and injected into the soil at the pilot bit. The cutting edges and mixing blades blend the soil and grout with a shearing action. When the design depth is reached, the auger head is normally raised to expose the mixing blades at the surface and then allowed to readvance to the bottom to ensure complete mixing.

The mixing head can also be enclosed in a bottom-open cylinder to allow for closed system mixing of waste and powdered reagents. The dry treatment chemicals are then transferred pneumatically, with the system designed to control dust and potential airborne contaminants.

A total of 110,000 cy of hydrocarbon contaminated sludge has recently been stabilized using closed system mixing at the Amoco Refinery in Whiting, Indiana, using dry cement as the reagent (Ryan and Jasperse, 1989).

Even though the auger is driven by a drill platform producing more than 300,000 ft lbs of torque the large diameter of the augers limits the effective depth of treatment for SSM to around 40 ft.

DEEP SOIL MIXING

For depths greater than this upto around 100 ft or for harder soils, the sister technique, Deep Soil Mixing (DSM), is used. The DSM rig is similar to the SSM machine except that upto four hydraulically driven 3 ft diameter auger/mixing shafts are used to limit the torque requirement. The rig is illustrated in Figure 1.

Grout flow to each of the augers is controlled by electronic flow meters which, depending on drill rates, send a predetermined amount of grout per foot of drilling depth to be mixed in the column.

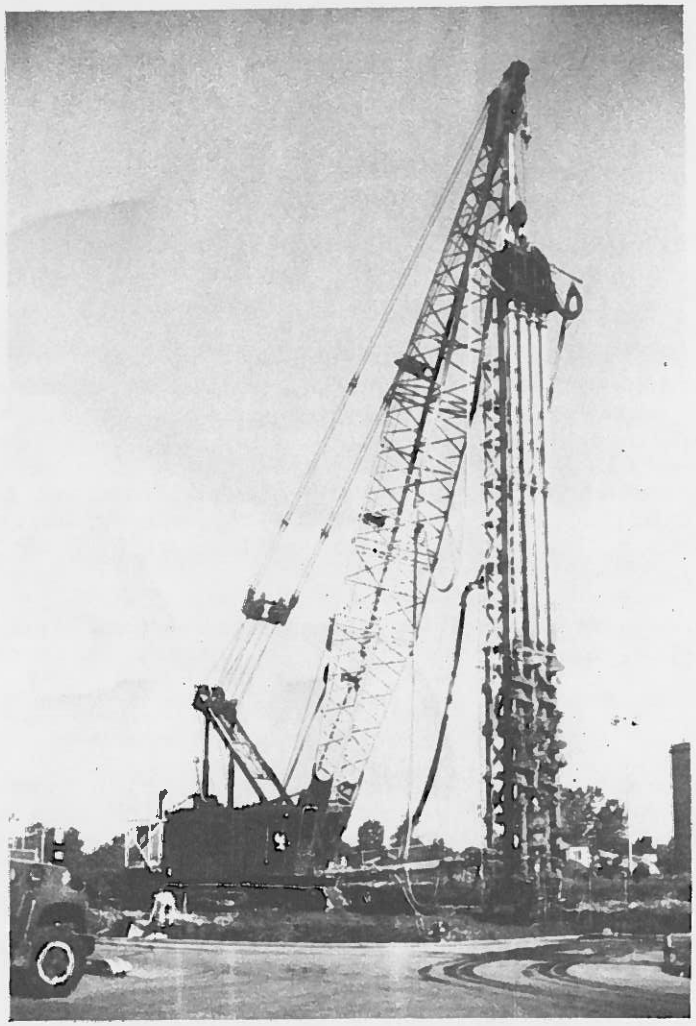

Figure 1. DSM Rig

For support of deep excavations, H-beams are dropped and vibrated into the DSM soilcrete columns immediately after mixing, to provide a structural system. This type of DSM wall has major advantages over other shoring systems, especially when a high groundwater table is present. This approach was used successfully recently, to shore the 40 ft deep excavation required for a new 60 mgd wastewater pumping station in San Mateo, California (Reams, Glover and Reardon, 1992).

DSM has many applications; but for large volume shallow applications DSM is often not economical, as was the situation for the two case studies to be described in this paper. SSM was the preferred option on both sites, one application involved site remediation while the other was for conventional geotechnical construction. Both projects demonstrate very effectively the adaptability of the process to a wide range of site conditions.

CASE HISTORY ONE
Crofton, British Columbia

Project Background

In 1990-91 a coastal pulp and paper mill, located at Crofton on the southeast coast of Vancouver Island, British Columbia, installed secondary effluent treatment facilities including two large spill tanks.

The two concrete tanks had diameters of 270 ft and 180 ft and a height of 35 ft and were to be located on an area of land previously reclaimed from the sea by uncontrolled dumping of sand and silt spoil in the 1950's.

Ground Conditions

The site generally consisted of pit-run sand and gravel surface deposits, over desiccated, very stiff, becoming loose and soft sand and silt fill. The average thickness of the fill crust was 6 ft overlying typically 12 ft of loose deposits. The fill was underlain by about 4 ft of medium dense beach sand overlying very dense, overconsolidated silt, which is an inter-till deposit. The groundwater was typically 10 ft below the site surface. A typical section through the site is shown on Figure 2.

The fill materials are characterized by the following gradation limits:

	% Passing U.S. Standard Sieve Size	
	#40	#200
Range	47 - 100	7 - 60
Average	85	30

The variation of the fill strength is represented by the penetration profile envelope shown on Figure 3.

Design Considerations

The mill is located in an area of potentially high seismic activity (Puget Sound Seismic Zone) which has a maximum magnitude of 7.5 (Richter scale). The design seismic horizontal peak firm ground acceleration (PGA) is 0.26 g at the National Building Code of Canada (NBCC) minimum risk level of 10% probability of exceedance in 50 years (475 yr return period). The new facilities are designed to survive the NBCC event but not the lower risk, high magnitude events considered possible in the region.

The liquefaction potential was evaluated by comparing the cyclic shear stress ratio values, based on the design PGA with the values required to cause liquefaction, for a Magnitude 7.5 earthquake, as proposed by Seed. The analyses indicated a zone of loose fill between 5 ft and 10 ft thick could liquefy under the design event.

A fundamental element of the design approach was the involvement of the owner and his civil consultant in reviewing the foundation alternatives with respect to their hazard reduction potential, relative to their cost. The interaction between the project group produced the concept of partial foundation treatment. The partial treatment approach required positive support to the tank walls but allowed the tank floor to settle.

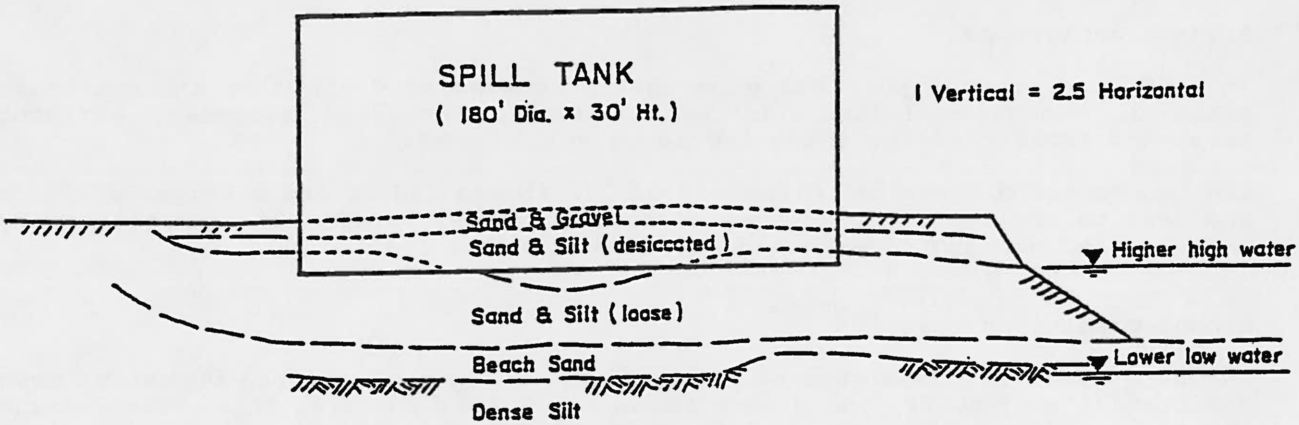

Figure 2. Typical Section Through Site

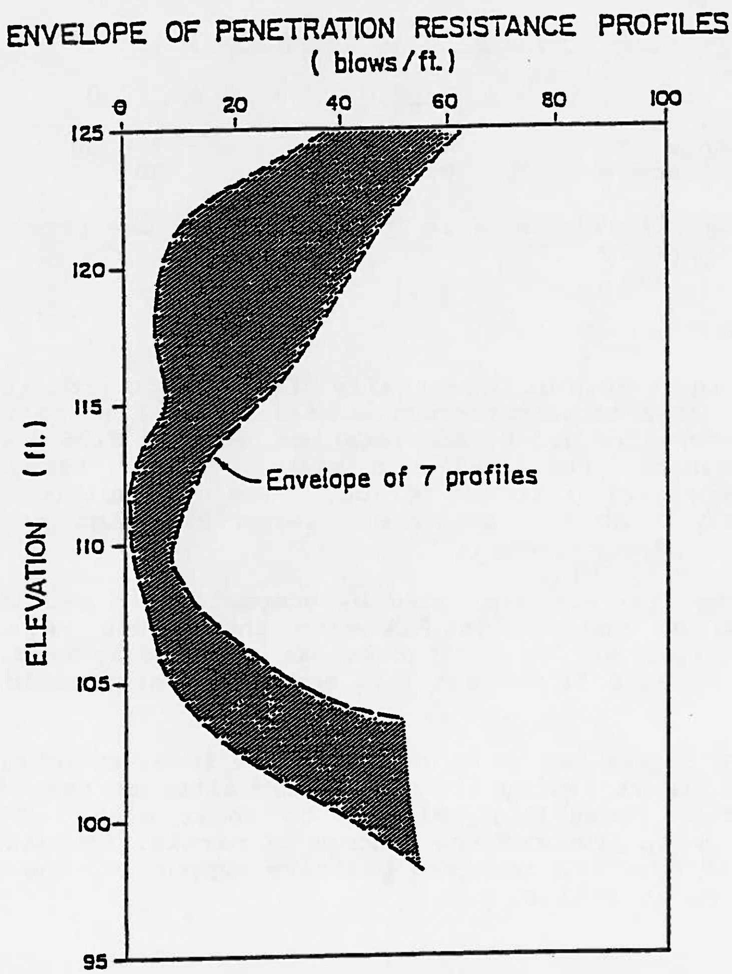

Figure 3. Penetration Resistance Profile

Foundation Alternatives

Several foundation alternatives were initially reviewed for their static condition benefits and included the following groups:

- preload
- excavate and replace
- ground improvement
- piles.

After consideration of the seismic loads and cost factors, the ground improvement alternate was adopted. The final selection of the specific ground improvement technique involved consideration of the following methods:

- dynamic compaction columns
- jet grouting
- soil mixing
- vibro-mortar columns.

Bids were accepted from specialist geotechnical contractors based on a performance specification and was not method specific.

The large diameter soil mixing technique (soilcrete columns) offered by Geo-Con was reviewed and adopted for the tank wall support. This was the first time that this method of support had been used in North America.

Soilcrete Column Design

The 12 ft diameter soilcrete columns were designed as a continuous tangential ring beneath the tank walls. The large and small tanks were supported on 71 and 45 columns, respectively. Eighteen additional columns were used to support some ancillary tanks and also to provide an arched "dam" connecting the two rings of tank columns, thereby providing ground restraint to facilities located between the two tanks.

The column 28-day compressive strength of 125 psi was specified to provide adequate strength for the tank wall foundation maximum bearing pressure of 9 ksf. The column strength was based on an empirical ratio between compressive and shear strengths of 0.3. As noted below, a substantial increase in strength occurs following the 28-day period.

The column and foundation geometry was determined from stability analyses under static (full tank) and dynamic (almost empty tank) loadings. Normal bearing capacity, force and moment equilibrium analyses were carried out for the static case. In the seismic case, similar analyses were carried out but assuming the contained soil acted as a heavy fluid with an acceleration component (0.13 g), which was also applied to the soilcrete column. Substantial loss of support was assumed for the external grade, together with the same acceleration component. Based on the results of the analyses, the soilcrete column was embedded 0.9 m into the dense till, the column was located to create a stabilizing eccentric vertical load and surplus tensile capacity in the ring beam foundation was required. A schematic arrangement of the foundations and soilcrete columns is shown in Figure 4.

Soilcrete Column Construction

Construction commenced with preliminary insitu mix trials to obtain the appropriate combinations of water-cement ratio, grouting rate and mixing rate. The water-cement-soil ratio initially was based on laboratory trials using site soils and was adjusted on site as the early test results became available. The basic mix design was 300 lbs per cubic yard of soilcrete of cement with a water-cement ratio of 1.8 to 1. The procedure was also varied by the superintendent to reflect variations in material and groundwater conditions. Some initial

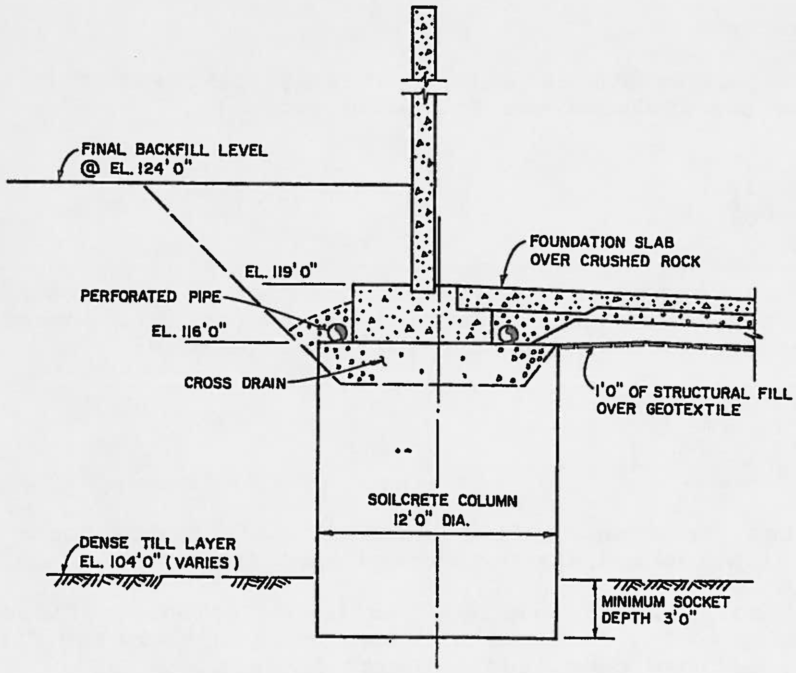

Figure 4. Foundation Detail

problems with obstructions such as boulders and logs were experienced. Where the auger paddle could not displace the obstruction, it was displaced or removed by a backhoe, obtained from nearby construction activities.

Samples were obtained from the soilcrete at various depths below the ground surface by a discrete sampler. Three cylinders were made from each sample and stored for testing. The results of the cylinder tests are summarized on Figure 5, with all samples meeting or exceeding the specified strength.

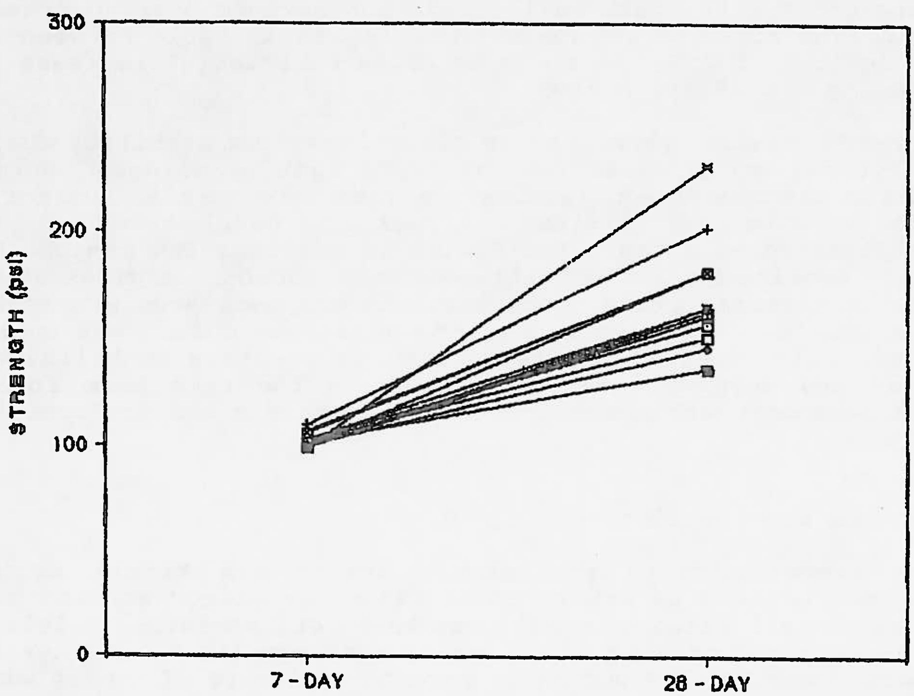

Figure 5. Soilcrete Samples U.C.S.

Figure 6 shows the SSM rig in operation and Figure 7 the completed columns.

CASE HISTORY TWO
Pittsburgh, Pennsylvania

Project Background

At 2,400 beds, the $112.5 million Allegheny County Jail in Pittsburgh, PA, will be one of the largest county jails in the U.S.A.

The disused downtown site chosen by the Urban Redevelopment Authority of Pittsburgh sits alongside the Monongahela River, adjacent to Liberty Bridge, on what was previously the old CSX Railroad facility. The southern edge of the site is bound by the retaining wall of the Parkway East (Interstate I-376); one of the main transport arteries of the Greater Pittsburgh Area.

Previous investigatory activities at the site had indicated the presence of hydrocarbon contamination from a former underground storage tank. Limited soil remediation was therefore necessary in advance of the construction of the ten-story building.

This second case study presents the use of Shallow Soil Mixing as a value engineered alternate to microfine cement permeation grouting, which was originally specified for the insitu fixation and consolidation of the contaminated soils adjacent to the Parkway structures.

Ground Conditions

Petroleum hydrocarbons had affected site soils at three separate locations over the estimated area of 32,000 sq ft. The depth of varied from 3 ft in two areas to over 20 ft contamination in the area next to the retaining wall. Total Petroleum Hydrocarbon (TPH) concentrations ranged from less than 50 ppm up to 11,000 ppm with levels varying from the surface to the groundwater table which was located at 25 ft. below existing grade.

A typical soil description for the excavation area was sandy silt to silty clay with cinders, rocks and brick pieces. Standard penetration tests ranged from 2 to 13 blows per foot.

The higher loaded sections of the retaining wall were founded on piles which apparently transferred load to the underlying bedrock. The present condition of these timber piles was unknown. In other sections, loads decreased as the Interstate ramped down, with the wall on spread footings only.

Remediation Method Considerations

The remedial approach preferred, from a cost and construction standpoint, involved the decommissioning and disposal of the existing pumping station and product lines and the removal of the affected soils down to groundwater for offsite disposal. The excavated areas would then be backfilled with clean fill and soil.

This immediately raised the question of the stability of the retaining wall and hence the Parkway if the proposed excavations were made.

The ultimate adoption of Shallow Soil Mixing and Jet Grouting to solve this combined environmental and structural problem made this site unique and of specific interest, as to the author's knowledge this was their first application in tandem.

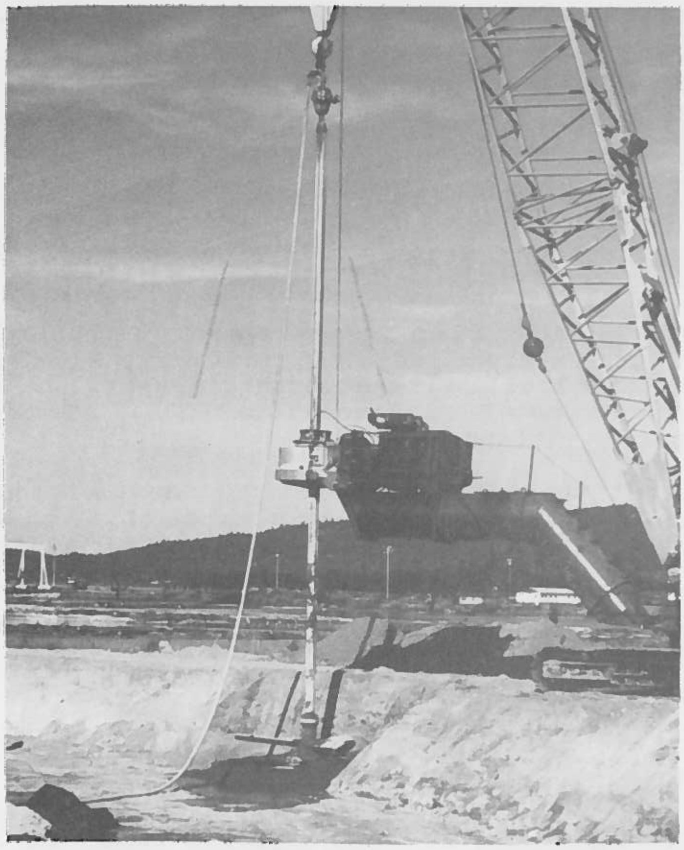

Figure 6. SSM Rig in Operation

Figure 7. Completed SSM Columns

Initial stability assessment focused on two arrangements, namely a cut slope from the footing level of the walls or the construction of a retaining wall along the Parkway. The latter method was discounted after discussions with PennDOT, the wall owners, since the required bracing or tiebacks may have interfered with the piles holding the structure in place. In addition, the lengthy design review process required by the owner and possibly the U.S. Department of Transportation, since an Interstate highway was involved, could have delayed the project by up to 12 months.

Discussions with the PA Department of Environmental Resources (PADER) indicated a willingness to consider a cut slope arrangement. However, they stipulated that any contaminants left in place must be fixated to reduce their potential mobility.

Accordingly, the Geotechnical Consultant investigated the potential for limited fixation of the hydrocarbon contamination insitu to the extent necessary to support the Parkway East structure.

It was recommended that permeation grouting with a microfine cement grout would adequately fixate the hydrocarbons and that the zone should extend 25 ft at the base of the retaining wall from the foundation level to the groundwater table. Figure 8 illustrates the area requiring stabilization. This would allow for a near-vertical excavation to be performed against the grout zone to remove the remaining contaminants, with a minimum factor of safety against failure of 1.5.

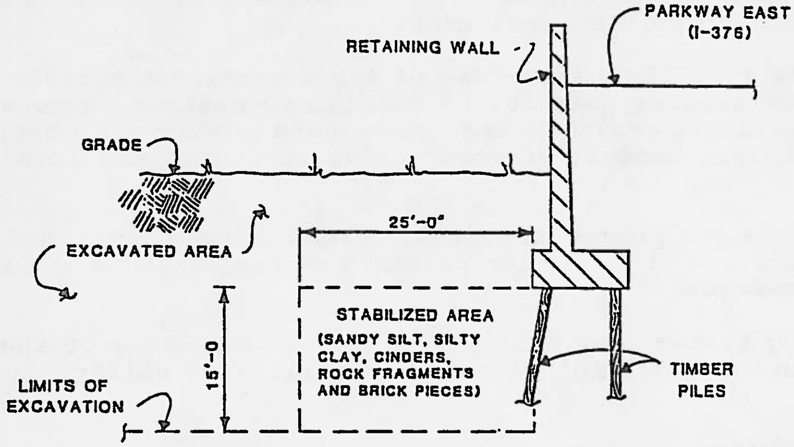

Figure 8. Cross-section of Stabilized Area

Emphasis was placed on the use of a microfine cement grout for improved permeation, injected using the end of casing method, in order to optimize filling of soil pores to fixate the hydrocarbons and densify the materials. Although Ordinary Portland Cement would have been adequate in the coarser general fill and debris, a microfine cement was selected for use since portions of the subsurface contained fine granular soils which required fixation to prevent water infiltration. The strength requirement was secondary and was specified as an unconfined compressive strength of 500 psi in order that the zone would be capable of supporting its own structure on near-vertical slopes.

PADER and PennDOT both concurred with this approach and the project was placed out for bid in the Spring of 1991.

Remediation Method Selection

Geo-Con, Inc., was the low bidder on the Remediation Contract and offered a value engineering alternate to the microfine cement grouting specified, namely the use of Shallow Soil Mixing (SSM) in conjunction with single-phase Jet Grouting. This alternate was attractive both technically and commercially and after review and acceptance the contract was awarded on this basis. The remainder of the cleanup work was implemented as planned.

The advantages of soil mixing on the site were numerous and were key in the client's decision to sanction its use.

- The technique is independent of soil type. This is a very significant advantage over permeation grouting which will prove ineffective in silts and clays. With soil mixing everything in the area is mixed and treated.

- The system has a vertical blending action which will tend to "average out" the soil stratigraphy and produce a well-mixed, homogeneous soilcrete block.

- Treatment is carried out in one pass with no additional work in problem areas as required by the split-space grouting approach.

- Cement grout is metered at a fixed rate into the ground, and the precise volume of ground treated in an identical manner is precisely known. The fixed mixing vanes assure the full column diameter and column contact. This is very important technically. With permeation grouting, the quantity of grout accepted per unit volume of ground is totally dependent on soil type, is therefore variable and is difficult to quantify due to the random nature of grout travel and actual injection elevation.

- Contaminants within the ground are locked in place within the soilcrete after thorough dispersal. Permeation grouting does not provide this dispersal effect with contaminant concentrations remaining at their original levels within a cement impregnated soil matrix.

- The result is a stabilized mass free of any significant pockets of untreated materials. The greater quantity of stabilized material generated by these processes effectively creates a more stable end product with typically higher and more consistent unconfined compressive strengths and lower constituent leaching.

- The SSM technique can produce a greater volume of treated ground per day than traditional grouting, thus easing pressure on the schedule and relieving the risk of time overruns.

- The soil mixing system does not involve the pressurizing of the ground that is required during grouting, with no possibility of uplift.

Soilcrete Block Design

In order to create the block of stabilized soil, a total of 2,200 cy of contaminated soil required treatment, extending 175 ft. along the Parkway and under Liberty Bridge.

As shown in Figure 9, three rows of 8 ft diameter columns on a 6 ft x 6.7 ft grid were installed.

They were formed on a primary and secondary sequence within each row, with the installation of the secondary columns timed to occur before the adjacent primary columns reached full strength. In this manner, block of ground over the full width were completed as the soil mixing progressed along the wall.

However, in order to stabilize/fixate areas that could not be accessed safety with the 150-ton crane jet grouting was necessary. These zones were limited to those adjacent to the timber piles and under Liberty Bridge. In these areas 3 ft diameter jet grout columns were formed, either contiguous or on a 2.5 ft triangular grid.

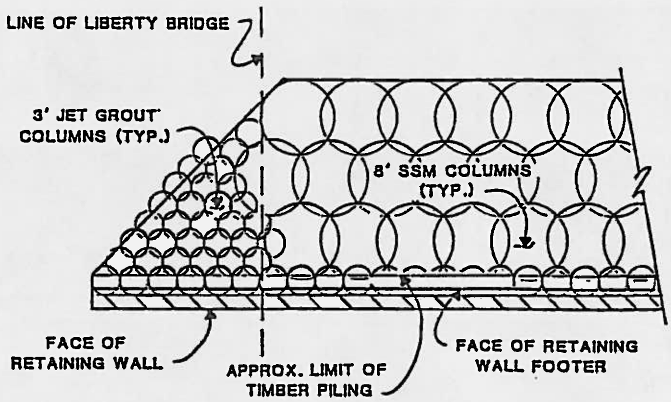

Figure 9. Layout of Soilcrete Columns

For both techniques, the stabilizing reagent was a Portland Cement slurry.

In the case of SSM, based on previous experience, a cement replacement by dry weight of soil of between 15% and 20% was adopted with sufficient water added to the grout mix to provide enough lubrication for a satisfactory auger penetration rate.

For the jet grouting, the parameters were set at:

- Grout Pressure - 5,000 psi
- Lift Rate - 1 ft/min
- Grout Flow - 40 to 45 gals/min
- Rotation - 1 rpm

A neat cement grout of water-cement ratio by weight of one was felt adequate to produce in excess of the specified 500 psi compressive strength at 28 days.

It was the intention to produce similar strengths for the SSM columns at an earlier date in order for excavation to proceed quickly after column construction, thus ensuring compliance with the very tight overall project schedule of 60 days.

Grout control was performed by frequent checks on the grout mix unit weight by use of a mud balance.

Construction

The SSM rig consisted of a high torque turntable mounted on a 150-ton crane which powered the 8 ft diameter auger. Figure 10 illustrates the rig in operation. Grout was supplied by a high-speed, continuous-mix, colloidal grout plant. This consisted of a storage silo, 1,000-gallon colloidal mixer and a progressive cavity pump. This same setup was used for the jet grouting with the exception of the use of a 350 HP high pressure, triplex piston jet pump. This pump was rated at pressures up to 20,000 psi and flow rates up to 170 gpm. While plant was being assembled, initial shallow excavation of contaminated material away from the retaining wall took place, along with concrete removal operations and soil sampling to establish waste characterization profiles. Test pits were also dug along the line of the wall to confirm the location of the piles.

The jet grout drill stem was mounted on a diesel hydraulic DK70 drill rig fitted with a Wirth Rotary Head as shown in Figure 11. The 2-1/2 in. grout pipe was advanced to the groundwater table and a check ball seated at the end of the grout pipe to initiate lateral flow through jet nozzles located on the sides of the

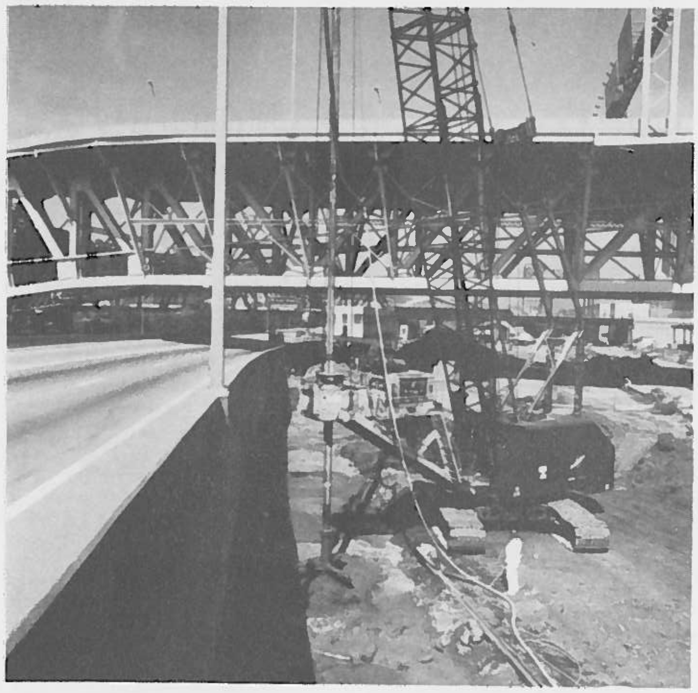

Figure 10. SSM Rig in Operation

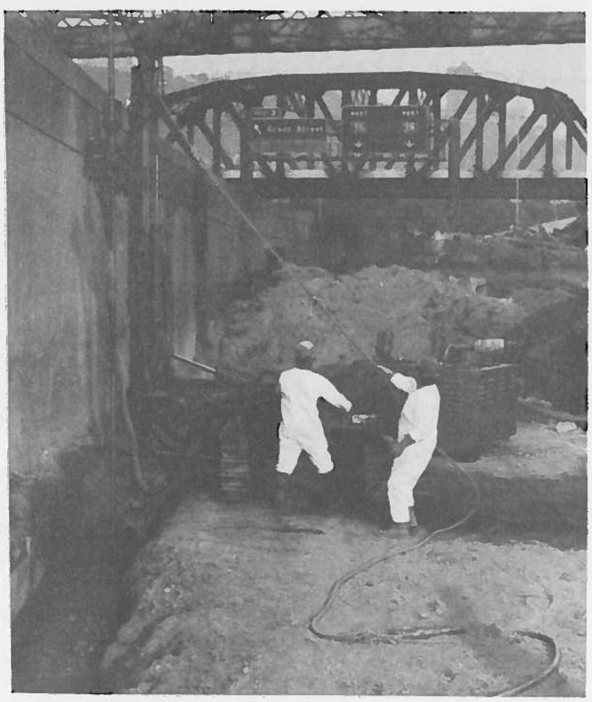

Figure 11. Jet Grouting in Progress

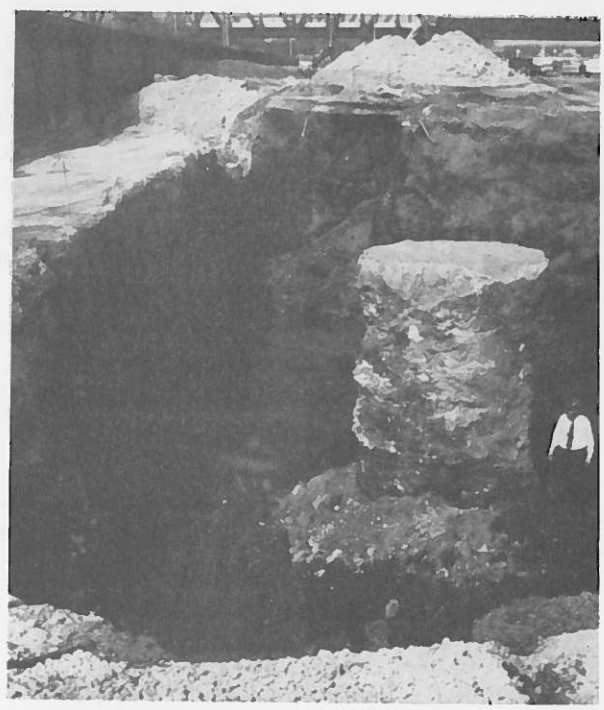

Figure 12. Trial SSM Column with Completed Block to Left

grout pipe. As the grout was pumped, the pipe was rotated and extracted at the set levels thus creating the jet grout columns. Exhaust material in small quantities, of very similar properties to the insitu soilcrete, was channeled into the open excavation to be incorporated as suitably fixated backfill material.

All grouting work was completed within twenty days with initial excavation of a vertical face against the stabilized block taking place only four days after column construction, thanks to the excellent early soilcrete strengths obtained. (See Figures 13 & 14.) Figure 12 gives a good indication of the columns produced.

Excavation and backfilling operations involving 10,000 cy of soil went smoothly with no unforeseen difficulties.

Throughout all operations the requirements of 29 CFR 1910.120, the Occupational Safety and Health Administration's (OSHA) Hazardous Waste Operations and Emergency Response Standard were strictly complied with.

Strength Results

Wet samples were retrieved from columns for testing from each day's work. These samples were taken by a special sampling tool below the surface of the column immediately following installation. The tool, mounted on a beam and deployed by the crane, consisted of a cylinder with a bottom flap that could be activated from the surface.

Compressive strength tests were performed in accordance with ASTM C29, the results of which are shown in Figures 13 and 14.

These results indicate some interesting general trends:

1. Higher soilcrete strengths are produced by SSM than by jet grouting for both short and longer term curing periods.

2. A much quicker early strength gain for the SSM compared with jet grouting and better strength gain with age.

Even though more cement is used per unit volume of treated soil in jetting, these results demonstrate, for the particular soils present, that SSM is a more effective tool, producing higher strengths with lower material and construction costs. This is partly the result of the cement wastage inherent in jet grouting.

CONCLUSIONS

These two case histories demonstrate the technical and commercial advantages that can often be achieved by the use of Soil Mixing techniques, to treat insitu both hazardous and nonhazardous materials.

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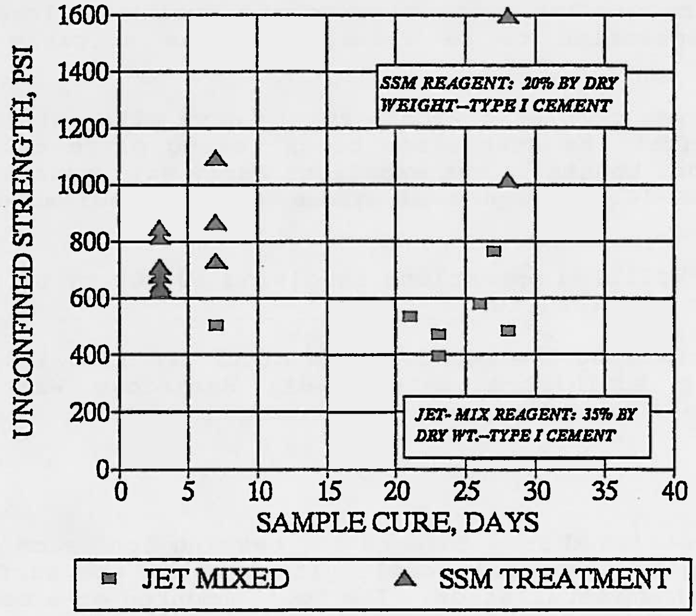

Figure 13. Soilcrete Strengths, Short Term

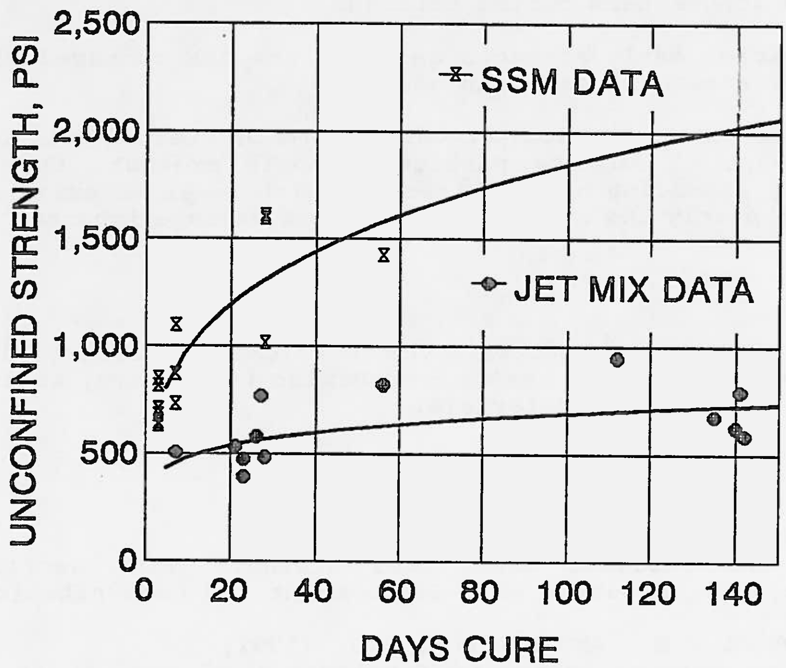

Figure 14. Soilcrete Strengths, Long Term

NEW HORIZONS IN GROUND ANCHORAGES, PINPILES AND CEMENT GROUTING

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INTRODUCTION

Developments in the specialty geotechnical engineering processes used in ground treatment, improvement and support continue at a fast pace in the United States. Much of the impetus comes from specialty contractors, usually linked with foreign partners, who are promoting new concepts and technologies for commercial edge. This is being facilitated by an increasingly wider acceptance of design-build concepts and the related contractual developments such as Partnering (Nicholson and Bruce, 1992) which are helping to create a less litigious and more equable contracting environment. A third major reason is simply the market demand: urban redevelopment and infrastructure development and rehabilitation are posing consistently difficult challenges to the foundation engineering community. A frequent consequence is the need for original, innovative solutions involving both new technologies, and modified or improved older methods.

This paper reviews developments in three major techniques, namely prestressed anchorages, pinpiles (used as in situ earth reinforcement) and cement grouting. Attention is focused only on a limited number of topics believed to have the best chance to impact practice for many years to come. The paper restricts its review to these three techniques as other methods used in ground treatment and improvement are described elsewhere in this seminar (e.g., Welsh, 1992).

GROUND ANCHORAGES

Prestressed ground anchorages have been used in the United States for almost thirty years. There still remains, however, a wide range in regional constructional methods, and a considerable unevenness in the knowledge and understanding of the fundamental engineering mechanisms. Several technical papers have helped to ease this situation (e.g., Bruce, 1989a, 1991), while a recent textbook by Xanthakos (1991) was similarly conceived. The topic has also been explored at depth in conferences (e.g., ASCE Cornell, 1990; ASCE New Orleans, 1992) while renewed efforts are being made by committees within bodies such as the Post Tensioning Institute and the Association of Drilled Shaft Contractors. The Federal Highway Administration remains a focal point for national research efforts (e.g., "Demonstration Projects Program", 1990). While overseas, a wealth of information is contained in the best of the new national standards such as the British Code of Practice (1989) and the revised international FIP recommendations (1992).

For convenience, developments can be considered in each of the two basic groups: anchorages in competent rock, and anchorages in weaker rocks and soils.

Anchorages in Competent Rock

A recent, major rock anchorage project, executed to stabilize Stewart Mountain Dam, Arizona, illustrates many significant and novel features (Bruce, et al., 1991a, b; Scott and Bruce, 1992; Bianchi and Bruce, 1992). Although many concrete gravity dams have been post tensioned to improve their stability, this was the first application for a multicurvature thin arch structure.

Background

Stewart Mountain Dam was constructed from 1928 to 1930 on the Salt River. It is 583 ft. long, a maximum of 212 ft. high, 8 ft. thick at the crest and 34 ft. thick at the base. There are concrete gravity thrust blocks at each abutment, from which wing dams extend into the abutment (Figure 1). The 10 MW powerplant is fed by a 13.5 ft. diameter penstock through the dam. There is also a 7 ft. diameter opening through the dam for bypass outlet works.

Unbonded horizontal planes within the arch concrete were the main cause of the dam's instability. At the time it was built, the importance of good cleanup on the horizontal construction joints was not recognized, so the joints were left untreated. This resulted in a layer of laitance on these joints, which compromised the bond across them.

A three-dimensional finite element analysis of the dam's performance during seismic and other loading conditions indicated that the dam would lose arch action during the maximum credible earthquake of 6.75 on the Richter scale occurring within 9 miles of the dam. This would leave vertical cantilever sections to support themselves. Because the horizontal lift lines were unbonded, the blocks in the upper portion of the dam would then be free to displace.

To stabilize the arch, 62 tendons were installed at about 9 ft. centers, with free lengths ranging up to 216 ft. and bond lengths from 30 to 45 ft. Their inclination varied from vertical to 8 deg. 40 min. from vertical. All but seven tendons, located immediately above the outlet works, were anchored in the dam foundation: these other seven anchors were bonded in the dam itself. Each arch tendon was composed of 22 epoxy-coated strands, each 0.6 in. in diameter. Design working loads averaged about 665 kips (a range of 545-740 kips) per tendon, equivalent to about 50% guaranteed ultimate tensile strength (GUTS).

In addition to the arch tendons, the design called for 22 tendons to be installed in the left thrust block of the dam to stabilize it against failure at or just below the structure/foundation contact. The free length of the thrust-block tendons varied from 40 to 125 ft., plus a 40 ft. bond length, and each was composed of 28 strands. Design load for each tendon was 985 kips (60% GUTS).

Most of the arch dam foundation consisted of hard, pre-Cambrian, medium-grained quartz diorite. The diorite was cut by irregular dikes of hard, medium-grained granite that varied in orientation and thickness. A fault divided the arch dam foundation into three distinct zones with unique mechanical properties, joint systems and permeabilities: (1) to the right of the fault, (2) to the left of the fault and (3) in the fault zone itself.

The rock underlying the right portion of the dam was hard, slightly weathered to fresh and generally of excellent quality. To the left of the fault, including the left thrust-block foundation, the rock was slightly inferior, being more fractured, sheared and weathered. The fault and the surrounding zone contained very intensely fractured and moderately to slightly weathered.

During the design phase, it was assumed that 32 of the arch tendons would be anchored in the right foundation zone (excluding the seven tendons anchored in the dam concrete), 15 in the left foundation zone and eight in the fault zone. All 22 thrust-block tendons were founded in the left foundation zone.

Test Anchor Program

Pairs of vertical "research" anchors were installed 12 ft. apart in each of three test sites representative of the three major rock zones expected to underlie the dam (Table 1). The nominal bond lengths at each site were 10 ft. and 20 ft. Each anchor was cyclically tested in 25% design working load (DL)

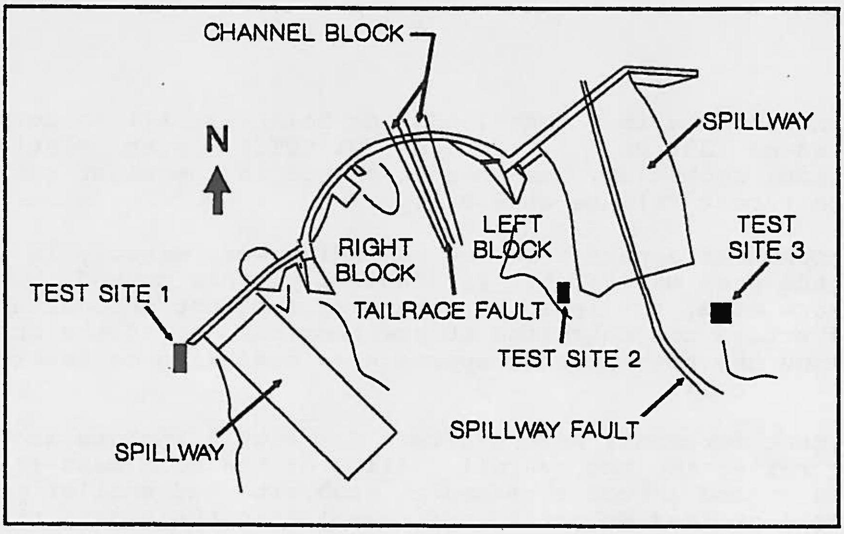


Figure 1. Simplified plan of Stewart Mountain Dam, AZ.

Hole	Strength (MPa)[8]	Dilatometer tests		Empirically derived		Settlement modulus (GPa)[3,4]
		Number of tests	Modulus (GPa)[4]	RMR[1]	Modulus (GPa)[2]	
1A	137	1	2.76	59	16.77	n/a[7]
1B	222	4	7.58	55	13.34	n/a[7]
2A	114	0[5]	-	49	9.44	n/a[7]
2B	143	4	2.07	46	7.94	n/a[7]
3A	n/a[9]	1	0.07	-	n/a[6]	0.10
3B	n/a[9]	0[5]	-	-	n/a[6]	0.17

[1]Rock mass classification according to Bieniawski (1984)
[2]Mass modulus using equation of Serafim and Pereira (1983), $E = 10^{(RMR-10)/40}$
[3]Inferred using influence factors from Poulos and Davis (1974)
[4]Poisson's Ratio assumed to be 0.2
[5]Obstruction in hole prevented testing in bond zone
[6]Out of data range used to develop empirical relationship
[7]Surficial fill at sites 1 & 2 precluded evaluation
[8]Compressive strength based on average of 5 point load tests (on core)/hole
[9]Rock core at site 3 was too fractured for point load tests

Grout age (days)	Compressive strength (MPa)	ASTM modulus (Gpa)
7	35.2	13.1
14	40.7	14.5
28	45.5	16.5

Table 1. a) Rock mass modulus and core strength estimates; b) Grout test results (Scott and Bruce, 1992)

increments to the safe maximum test load - or failure. All achieved the maximum test load of 133% DL - 1,310 kips (80% GUTS) - with relative ease, with one exception: Anchor 3A, the shorter anchor in the worst rock, demonstrated grout/rock failure at 968 kips.

The relative amounts of apparent tendon debonding were exactly in line with the quality of the rock mass (Table 2): basically, this proved that the more competent the rock mass, the less the extent of apparent debonding and the higher the bond stress concentration at the proximal end of the anchor - and the more erroneous the conventional approach of designing on "average" bond values.

Permanent bond zone movements were smallest for site 1 anchors and greatest in site 3 anchors, reflecting the overall quality of the rock mass (Figure 2). In addition, the second anchor stressed at each site had smaller permanent movements (as well as less debonding and creep) than the first, strongly indicating some type of rock mass improvement during the loading of the first anchor. Clearly demonstrated, this phenomenon is easy to accept and understand, but had not been previously documented. This is significant when assessing relative production anchor performance, since later anchors may exhibit better stressing performance than those installed earlier.

Creep was not significant at sites 1 and 2. Interestingly, however, while creep generally increased with load, the highest amounts were at 75-100% DL, decreasing at higher loads (Figure 3). Also, while test anchor 3A showed the classic progressive failure pattern, 3B showed creep values at 133% DL lower than at 100% - 0.057 in. in 10 min as opposed to 0.064 in the same period. When restressed to 133% DL a second time, the creep was lower still (0.045 in. in 10 min.).

These data point to an irregular "ratchet"-type rock mass response at odds with the smoother, more predictable performance assumed in theory and usually found in soils. This rock mass improvement was probably due to a tightening up of the fissures and joints in the mass, in the region around and above the bond zone: crushing of the rock itself was not feasible, given its material strength.

Overall, the test verified that the originally designed bond lengths had satisfactorily high safety factors in the rock at sites 1 and 2, but merited a slight increase when installed in the poorest-quality site 3 material. Work on the production anchors proceeded accordingly.

Production Anchors

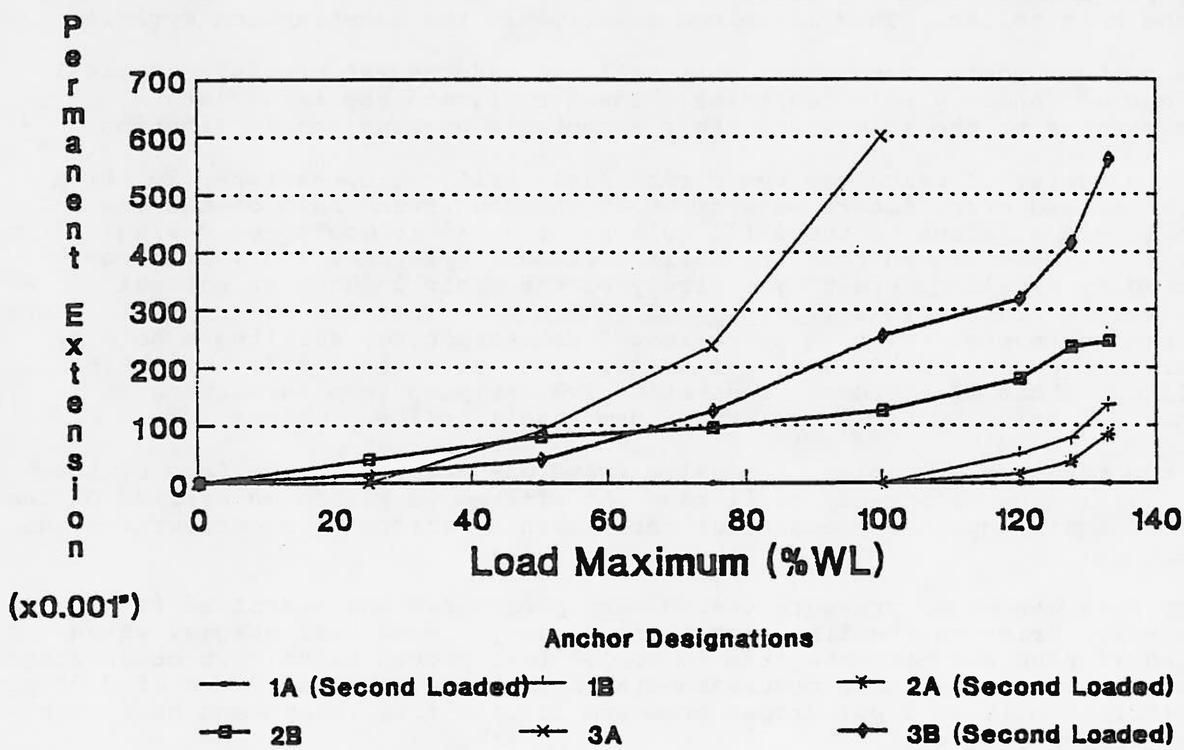
Under a previous contract, 4 ft. 9 in. square recesses, approximately 2 ft. deep, had been formed in the dam crest. At the precise location, bearing and inclination, a 12 in. diameter hole was cored about 5 ft. deep at each anchor position. A 10 in. diameter steel guide tube was then surveyed and cemented into this hole to ensure the anchor-hole drilling would have the exact prescribed starting orientation. Angles were measured by independent state-of-the-art methods to within minutes of accuracy.

A down-the-hole hammer mounted on a Nicholson Casagrande C12 diesel hydraulic track rig then drilled the 10 in. diameter anchor holes. Special hammer and rod attachments promoted hole straightness. In accordance with the specifications, the position of each hole was measured at 10 ft. intervals in the upper 50 ft. and every 20 ft. thereafter to final depth - a maximum of 270 ft.

This frequent measurement and the precision required - to within 3 in. in 100 ft. - demanded very special attention. Eastman Christensen, Bakersfield, Calif., adapted their Seeker 1 rate gyro inclinometer, normally used in oil-field applications, to this project. The Seeker could accurately measure the drill bit's position through the drill rods. Modification of its computer

Anchor	Apparent Debonding at 133%	
	Actual	Site Average
1A	21"	23"
1B	25"	
2A	40"	42"
2B	44"	
3A	Failed 142" bond	Possibly
3B	73"	108"

<u>Table 2.</u> Calculated apparent tendon debonding lengths, Test Anchors, Stewart Mountain Dam, AZ



<u>Figure 2.</u> Net permanent displacements, Test Anchors, Stewart Mountain Dam, AZ

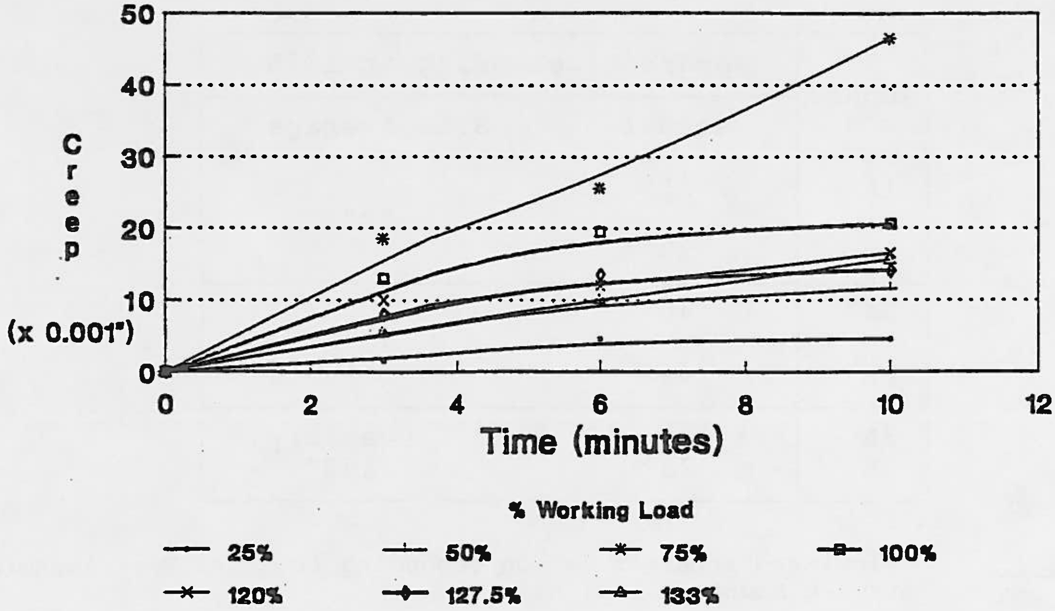


Figure 3. Creep data at each cycle maximum, Test Anchor 2A, Stewart Mountain Dam, AZ

software demonstrated the acceptability of the hole's progress within minutes at the hole collar. This minimized downtime in the construction cycle.

As a further check, government personnel ran independent precision optical surveys on randomly selected holes. These confirmed the immaculate straightness of the holes, and their acceptable bearing and inclination.

Another series of tests was run during early drilling operations, in which geophones and crack meters were fixed at the downstream face of the dam immediately adjacent to the drill hole and constantly monitored during drilling. They proved that the maximum fissure apertures and vibrations induced by drilling were tiny - barely of the order induced by natural temperature fluctuations (Table 3 and Figure 4). This has major significance for dam engineers. Even on a "delicate" dam structure, drilling a hole by rotary percussion within 5 ft. of a free face had minimal effects. This drilling method is extremely cost-effective, helping keep anchors an economical solution for a variety of dam stabilization problems.

For the thrust-block holes, a massive frame was erected up the face of the structure. The Casagrande drill mast was affixed to platforms carried on the frame. Again, special precautions were taken to ensure hole correctness and direction.

Every hole was water pressure-tested, and pregrouted and redrilled if necessary, prior to the final acceptance survey. Most test stages, which ranged in rock and concrete from 50 to 130 ft., proved tight, but other stages required as many as three pretreatments to meet the specifications of 0.02 gpm per foot of hole at 5 psi excess pressure for the free length and half that for the bond length.

Epoxy-coated strand tendons were placed in reels on special uncoilers and transported to the holes. Extreme care was taken to prevent abrasion of the epoxy coating, as each tendon was placed to full depth and tremied with a specially researched, high-strength, plasticized grout into each hole to provide the exact bond length. Fluid and set grout properties were rigorously recorded as routine quality control.

Elevation of Crackmeter (feet)	Maximum Recorded Movement (in)	Typical Daily Movement due to Temperature Effect only (in)	Approx. Distance from Meter to Hole (ft)
1520.39	0.00239	0.00284	5.0
1510.45	0.00218	0.00284	5.5
1500.34	0.00409	0.00432	5.8
1490.43	0.00510	0.00348	5.8
1480.56	1	0.00353	5.5
1470.17	1	0.00376	5.5
1460.21	1	0.00459	5.5
1450.23	1	0.00440	5.5

1 No discernable movement was detected during the drilling operation.

Table 3. Summary of crackmeter data recorded during drilling, Stewart Mountain Dam, AZ

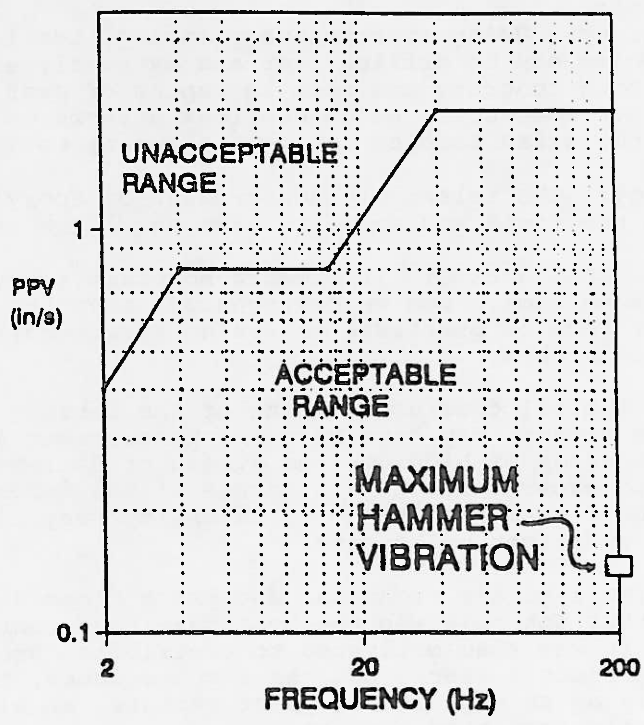


Figure 4. Hammer induced vibration monitoring, Hole 37, Stewart Mountain Dam, AZ

Stressing commenced a minimum of 14 days after grouting. To verify in detail the correct operation of the tendons, 12 cyclic Performance Tests were conducted. The remaining anchors were tested simply, under PTI Proof Test provisions. Given the high loads and long free lengths, net elastic extensions as long as 16.2 in. at the test load on the longest tendons were recored. Creep and lift-off checks rounded out the initial verification of the anchors. In all aspects, every anchor proved to have outstanding qualities, with details closely mirroring the results of the test program.

Each anchor was proved to 133% of design working load, prior to interim lock-off at 117% DL. Monitoring of the dam during stressing confirmed no significant structural deflections caused by this extra load. This was probably helped by building up the load gradually in each block of the dam to minimize any loading impact. Anchor 60 was followed by anchor 58, then anchor 6 by anchor 4, then 13 by 11, and so on. Final lock-off at 108.5% DL, and full secondary grouting of the free length, followed the 100-day observation period.

Lessons

Several features of the Stewart Mountain Dam project are unique and promise to make it one of the key dam rehabilitation projects of the decade:

- Application: The project represents the first use of high- capacity anchors to strengthen a double-curvature thin-arch dam to resist seismic effects.
- Research and development: The intensive test program confirmed many of the intricate theories of load transfer in hard rock anchors and - surprisingly - provided a clear reminder than even hard rock masses can be altered by prestressing.
- Drilling technology: Using appropriate planning, tooling, equipment and expertise, 10 in. holes can be drilled fast and extremely straight and accurately through both concrete and rock to depths of over 270 ft. Such methods appear to have absolutely no deleterious effects on the structure. And more - systems now exist to pinpoint this accuracy to within inches at this depth.
- Tendon technology: The relatively new product of epoxy coated strands appears workable in the field and seems to give excellent bonding characteristics.
- Anchor/structure interaction: If Stewart Mountain is typical of the current quality of such dams, then we can conclude that the application of tens of thousands of tons of prestress causes no structural distress to double-curvature thin arches.

Despite these technological conclusions, one of the lasting lessons of the Stewart Mountain Dam project may have been the procurement and contracting procedure. Far in advance of bidding, the Bureau of Reclamation researched current practices and experiences in all facets of the industry. As a consequence, the specifications, though by necessity very rigorous, were both eminently practical and right up to date.

The decision to invite separate technical and price proposals - independently assessed - assured that not only was the best qualified contractor for this job chosen but that it was also motivated to contribute "heart and soul" to every stage of the project's execution. As a consequence, the work was carried out virtually as an engineering joint venture, at site and head office levels, between equally committed parties.

Anchorages in Soft Rock and Soils

Littlejohn (1990a) identified four generic categories of ground anchorages, based largely on construction technique and in particular on the grouting methodology (Figure 5). Clearly rock anchorages are Type A, whereas most other anchorages are of Types B and C. The technique of underreaming

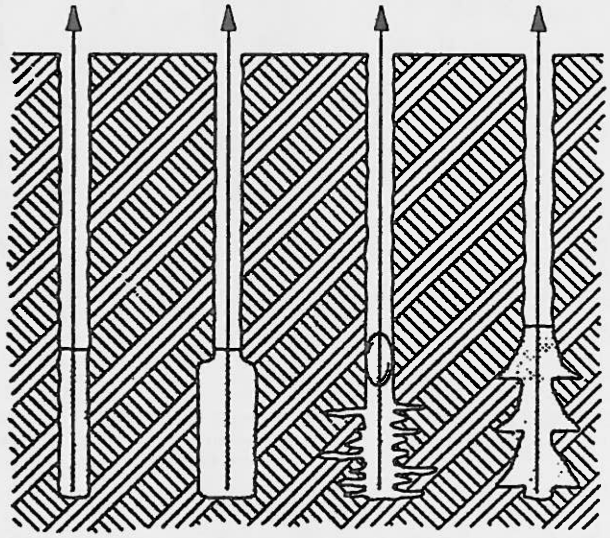


Figure 5. Main types of cement grouted anchors (Littlejohn, 1990)
Type A: straight shaft, gravity grouted
Type B: pressure grouted during installation
Type C: pressure grouted via a sleeved pipe after initial installation grout has set
Type D: underreamed, gravity grouted

boreholes to provide a large deformation in the bond zone (i.e., Type D anchor) is not common today, largely as a result of developments in other drilling and grouting methods designed to enhance pullout capacity. The following discussion concentrates on two major areas of current development and considerable future potential: grouting techniques, and corrosion protection.

Grouting

There are fundamentally four types of pressure grouting for soil (Figure 6), if the simple target of void filling is left aside. Void filling occurs when grout under its own head is simply tremied into the hole without the intention of permeating into the soil, densifying the soil or otherwise improving the soil at or away from the borehole interface. Such grouting is used in rock anchors or Type A soil anchors. Jet grouting, with the one exception of the field test run in England (Anon, 1988) is not typically a viable grouting method or concept, applicable for anchoring in the United States.

When grouting anchors in the soil, the aims are typically to permeate for some finite distance around the drill hole, to enhance the "effective bulb" diameter, and to cause some compaction of ground disturbed during the drilling process. Permeation will occur in coarse sands and gravels, but the phenomenon of "pressure filtration" will normally limit radial travel to a few inches in most cases using typical anchor grouts. This same phenomenon will squeeze out some of the integral mixing water leaving behind an anchor grout of water content considerably lower than that injected, and therefore considerably stronger than the corresponding cube results. For this reason, water/cement ratios used in cohesionless soils can be a little higher than those used for clays and tills and so on, without the drawbacks normally inherent with such mixes (Figure 7: reduced strength, significant bleed potential). Ratios for the former can be as high as 0.55 (assuming significant injection pressures are used), while it is prudent to limit water/cement ratios to 0.45 in cohesives.

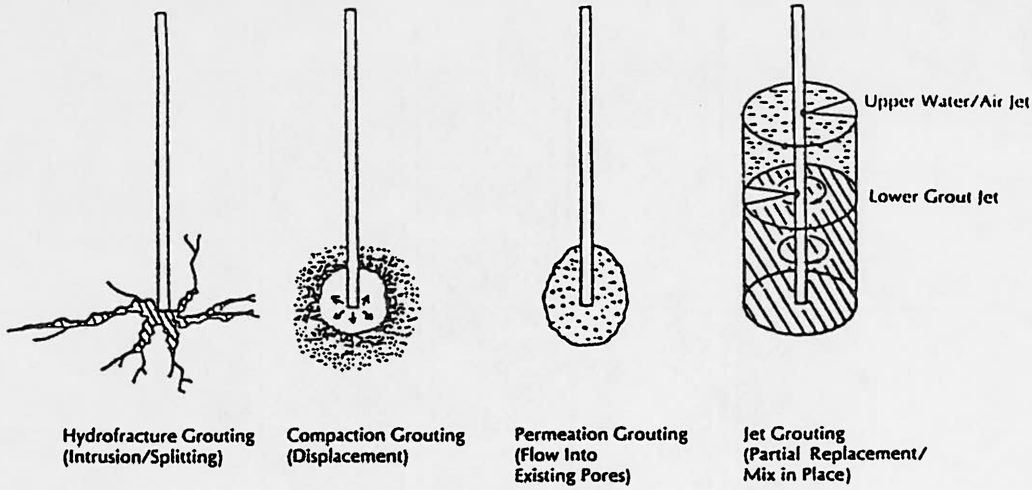


Figure 6. Basic categories of soil grouting methods

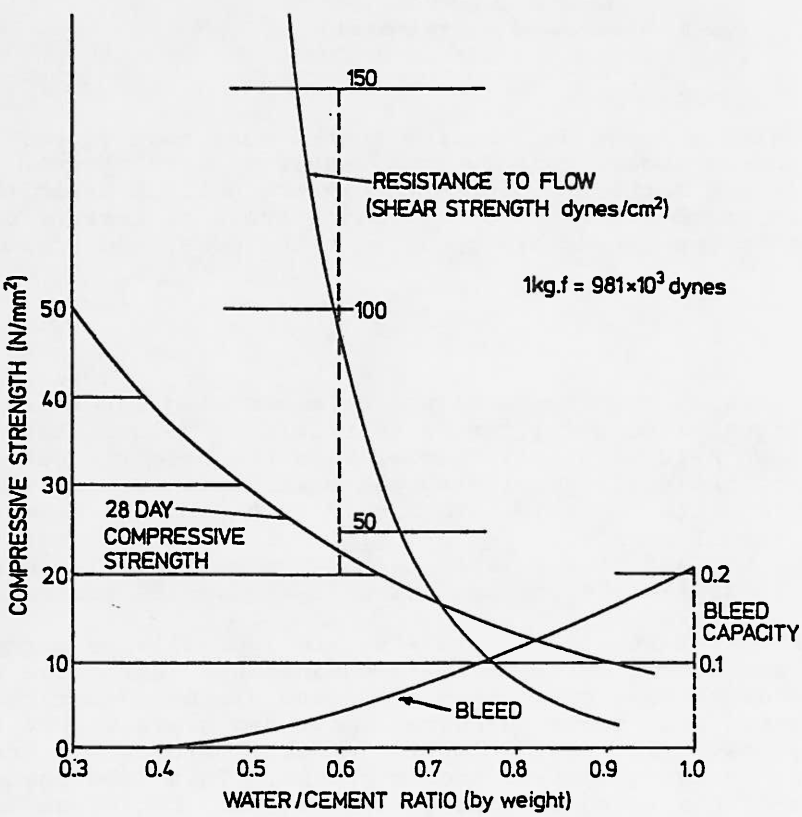


Figure 7. Effect of water content on cement grout properties
(Note $1N/mm^2 \approx 145$ psi)

Pressure grouting also causes a recompaction or redensification of the soil around the borehold thus improving its frictional properties to the benefit of subsequent anchor load/displacement performance.

During initial grouting, pressures are limited, by design - to prevent upheaval or hydrofracture, and by operational phenomena - such as escape of grout up along the outside of the casing. "Though the head" pressures are typically seldom greater than 100 psi for casing systems, and much less for augers. The role of grouting pressure is paramount in many of the design methods used to estimate safe grout/soil bond values.

Hydrofracture grouting is therefore clearly not a factor to anticipate or be considered in primary grouting, although the benefits are being increasingly exploited in the technique of post-grouting (Type C anchor). A sleeved pipe (tube à manchette: Bruce, 1989b) can be incorporated into the bond length of the anchor. A few days after initial grouting, it is possible to reinject the anchorage zone through this pipe. In some way - possibly different in different soils - post grouting improves anchor capacity and so can be used, a priori, to safely minimize anchor dimensions, or it can be used, after initial stressing, to "repair" unsatisfactory anchors.

It is likely that the grout moves along the various interfaces of the anchorage, but especially the grout/soil interface where it both permeates and compacts. It is also possible that it causes simple enlargement of the bond zone by fracturing the initial grout and thrusting the fragments further against or into the soil mass. Equally some hydrofracture into the surrounding soil or permeation up into overlying strata may also occur, in both cases giving an improved soil condition benefiting subsequent anchor performance.

In any event, the true benefits of post grouting can only be realized systematically by conducting it in the correct fashion. This involves various features including
- the use of proper quality sleeved pipes;
- grouting through a double packer, from the bottom sleeve upwards
- placing the target regrout volume in discrete batches, to control and localize the effect.

On the contrary, it is common in some regions to simply connect the top of the regrout tube to the grout pump and inject the whole target volume in one shot, with the fervent hope that some grout will exit from each sleeve, or worse that "the grout will go where it's needed." Pressure grout is fundamentally lazy, and will exit at, for it, the easiest location. This is usually the uppermost sleeve, and so grout injected in this way has no guarantee of working on the entire bond length as anticipated. Post grouting conducted in this way leads to imprecise placing of the grout, which in turn gives rise to erratic and unpredictable results during stressing.

Grouts used in post grouting are typically of slightly higher water content than those used in the initial grouting, but still require mixing - to ensure high quality grout - in a colloidal, high speed mixer. The higher pressures needed largely to overcome line and sleeve back pressures can usually only be provided by piston pumps.

Examples of tests conducted in different soils are summarized by Bruce (1991).

As a final point on grouting, the need for strict on site quality control during mixing and injection must be stressed, especially when installing tendons in marginal soil conditions. Given the variabilities and uncertainties inherent in the soil medium alone, it is clearly logical to ensure that the materials placed in the borehole are of the highest and most uniform quality. Strength of the grout is clearly a key issue, but little attention is often paid to bleed potential. Bleed water trapped in the bond zone for geological or geometrical reasons can only do harm to anchor performance. As shown in Figure 7, bleed potential, as well as strength is controlled by the water/cement ratio. By monitoring this ratio during anchor grouting we can be satisfied that the grout is being prepared to specification and that changes are not being made deliberately, accidentally or systematically. An appropriate measuring instrument is the Baroid Mud Balance, while a Flow Cone will give a qualitative indication of grout production consistency.

In special conditions, involving grouts of low water content, long pumping distances or extreme heat, an additive may be considered to plasticize and/or retard the grout. Extreme caution must be used in the design of such grout mixes and in their preparation as typically "a little additive goes a long way." It is the author's opinion that no other type of additive, including expansive agents, should be used in anchor grouting, nor are they necessary. Any mix involving an additive should be thoroughly tested, on site, for both its fluid and set properties prior to routine use.

Corrosion Protection

With the increasing awareness of the problem of corrosion protection, more use is being made of a protective, corrugated sheath over the bond length to supplement the protection in the free length. In the United States, such an application (Figure 8) would be referred to as "double protection" to the steel, i.e., protection by both the interior grout and the surrounding corrugated sheath. By certain other standards (e.g., FIP 1986) this would only be regarded as single protection as the protective role of brittle cement grout is queried. This is a question our industry must address, especially as permanent soil anchors are being increasingly installed in urban or marine environments where the threat of corrosion due to natural and manufactured chemicals is very real.

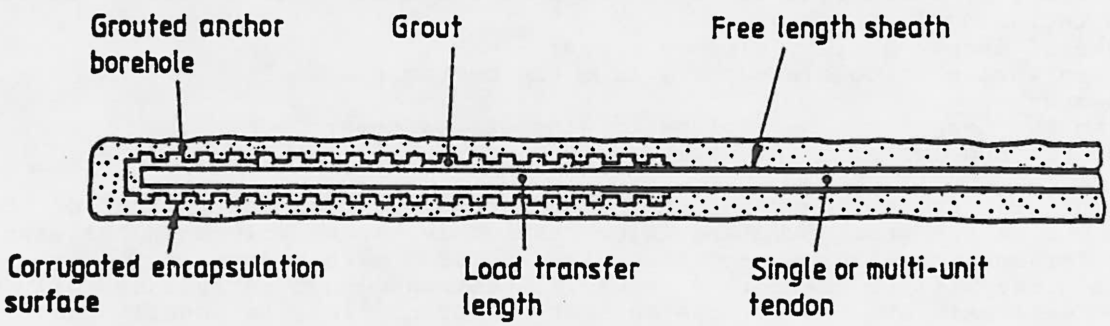


Figure 8. Encapsulation of bond length with corrugated protection

Interior grout, applied and cured prior to tendon installation is susceptible to cracking during handling unless a long rigid installation frame is used to support the pregrouted bond length. Such an option is often not practical bearing in mind the hole entry restrictions common to most tieback applications, and general logistical considerations.

The alternative is to grout simultaneously inside and outside the sheath after tendon installation. Such operations place a premium on the grouting skills on the contractor, and on the details of tendon assembly, especially the layout of the tremie tube or tube.

Although the use of spacers with intermediate clamps along the bond length has traditionally been proposed, and adopted, consideration should be given to the actual details of assembly. Without due care the strands may in fact not be correctly or efficiently separated within the corrugated tube, and may inhibit grout-steel bond development by lying in contact with the corrugated for most of their lengths, especially in shallow inclined anchors. Such problems are intensified if the corrugated diameter is imprudently minimized to reduce fabrication or overall installation costs. Close liaison between the designer, the tendon fabricator and the contractor is essential to avoid such problems, the results of which are not typically evident until stressing begins. If these cannot be resolved, there is always the option to use the epoxy coated strand as used at Stewart Mountain. Cost, however, remains a major drawback to this option.

Attention should never be relaxed on the subject of the nature of the strand surface. "Foreign substances" can be present on the tendon prior to installation. While it is conventional to think that this is a problem caused by, and therefore to be rectified by, the contractor on site during his handling and installation activities. There remains the potential for substances to be present as a result of processes used in strand manufacture, tendon assembly and/or tendon shipment activities. This is clearly the responsibility of the tendon fabricator to guard against. In general, however, the development of a uniform non-flakey rusting on the steel, prior to the installation remains a good field indicator of acceptable strand conditions. Otherwise, foreign substances have the potential, by both physical or chemical means, to reduce grout/steel bond potential. This may lead, per se, to premature tendon pull out. Alternatively and more insidiously, this reduction may push the tendon into a marginal condition where very slight and otherwise unimportant variations in anchor construction techniques may be sufficient to cause failure. Such marginal conditions, while easy to conceptualize, are very difficult to locate or prove, as the evidence may often be confusing, non-conclusive or apparently self contradictory, even though on a time related, or statistical basis the truth is patently obvious.

Finally, the most authoritative discussions of corrosion and corrosion protection have been provided by Littlejohn (1990a, b, 1992) drawn partly from his work for FIP in 1986. He also recommends that one of the "duties" of the designer is the "definition of anchorage life (permanent/temporary) and requirement for corrosion protection".

PINPILES AS IN SITU EARTH REINFORCEMENT

Background

In the last decade or so in the United States, there has been increasing use made of small diameter cast-in-place bored inclusions. Most have been designed to act as conventional load bearing piles, commonly known as pinpiles (Bruce, 1988a; 1989c, 1992a). However, these elements (4-10 inches in diameter) are finding growing popularity in the field of slope stabilization, where, installed in densely spaced patterns (Figure 9), they act as in-situ reinforcement (Bruce and Jewell, 1986; 1987). The concept of their performance is that they form a composite structure with the included soil: this structure then constitutes an in-situ barrier to arrest actual or potential slope movements.

General Features

Early applications of conventional, axially loaded pinpiles indicated, surprisingly, a positive "group effect", thought to be due to beneficial soil-structure interaction (Bruce, 1988b). This advantage was then exploited in slope stability applications in Western Europe and later - but infrequently -

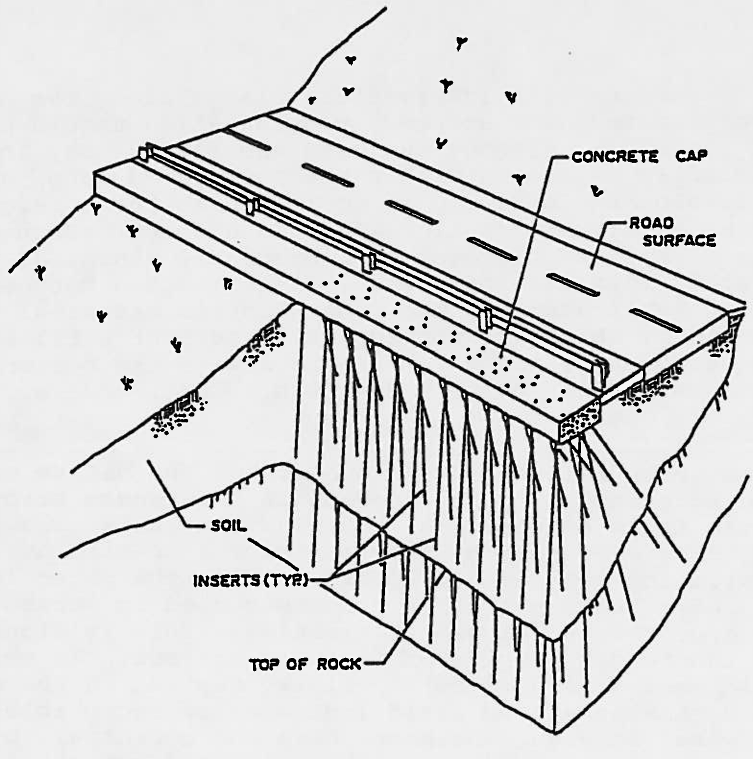


Figure 9. Typical arrangement of pinpiles as INSERTS

in the United States. In urban environments similar groups of pinpiles (or "INSERTS" in this context) can be used in cut and cover, as well as bored tunnel, construction. There the concept is to create protective structures in the ground to separate the foundation soil of the building from the zones that are potentially subject to disturbance (Figure 10). All these INSERT structures rely for their effectiveness on soil/pile interaction. This composite structure - referred to as a "Type A Wall" because of its distinctive cross sectional appearance - is intended to stop loss of soil from behind it, and to prevent sliding along potential failure planes passing through it.

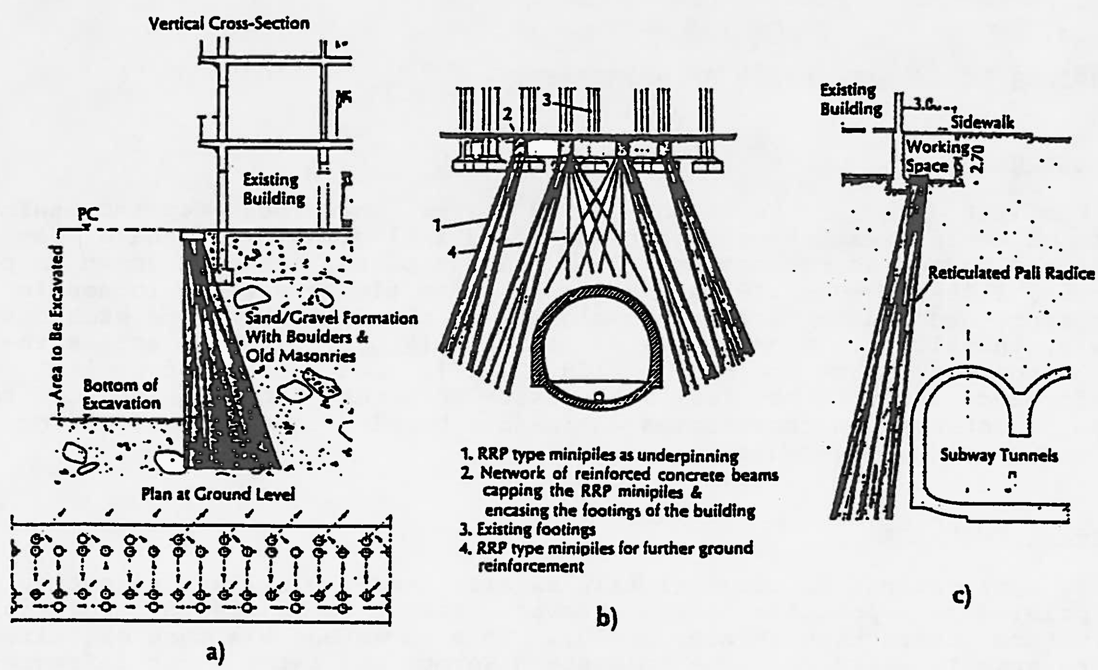


Figure 10. Applications of INSERTS: a) for cut and cover and b) and c) around bored tunnels

Design approaches continue to lag behind other aspects of the technology, but several instrumented field programs have confirmed that reinforcement stresses and overall wall movements in service are minimal, and that most probably designs have been highly conservative. Even their original proponent - Fernando Lizzi - confirmed in 1982 that "it is not yet possible to have at our disposal an exhaustive means of calculation ready to be applied with safety and completeness." In addition, an ASCE Committee (1987) also alluded to the great reliance placed on soil/pile interaction, the safe exploitation of "which is still subject to experience and intuition".

The typical approach to design is, of course, relatively simple, and involves standard basic steps:

- estimating loads (active and passive) on the wall;
- conducting a stability analysis to determine the shear force needed to maintain a required factor of safety;
- determining the number of INSERTS needed to provide the required shear resistance;
- calculations (similar to those for a conventional gravity wall) to check stability against overturning, sliding and bearing failure at the base of the wall.

Usually the INSERTS are extended into bedrock where economically possible, but, in any event, always below the potential failure plane. INSERT Walls can be constructed in close proximity to existing buildings in relatively tight access locations without the need to excavate or underpin, and without causing any decompression of the foundation soil. Given their mode of construction, as detailed below, they can be installed in any type of ground, including through boulders, old foundations or other obstructions with no constructional limitation on hole inclination or orientation.

Construction Aspects

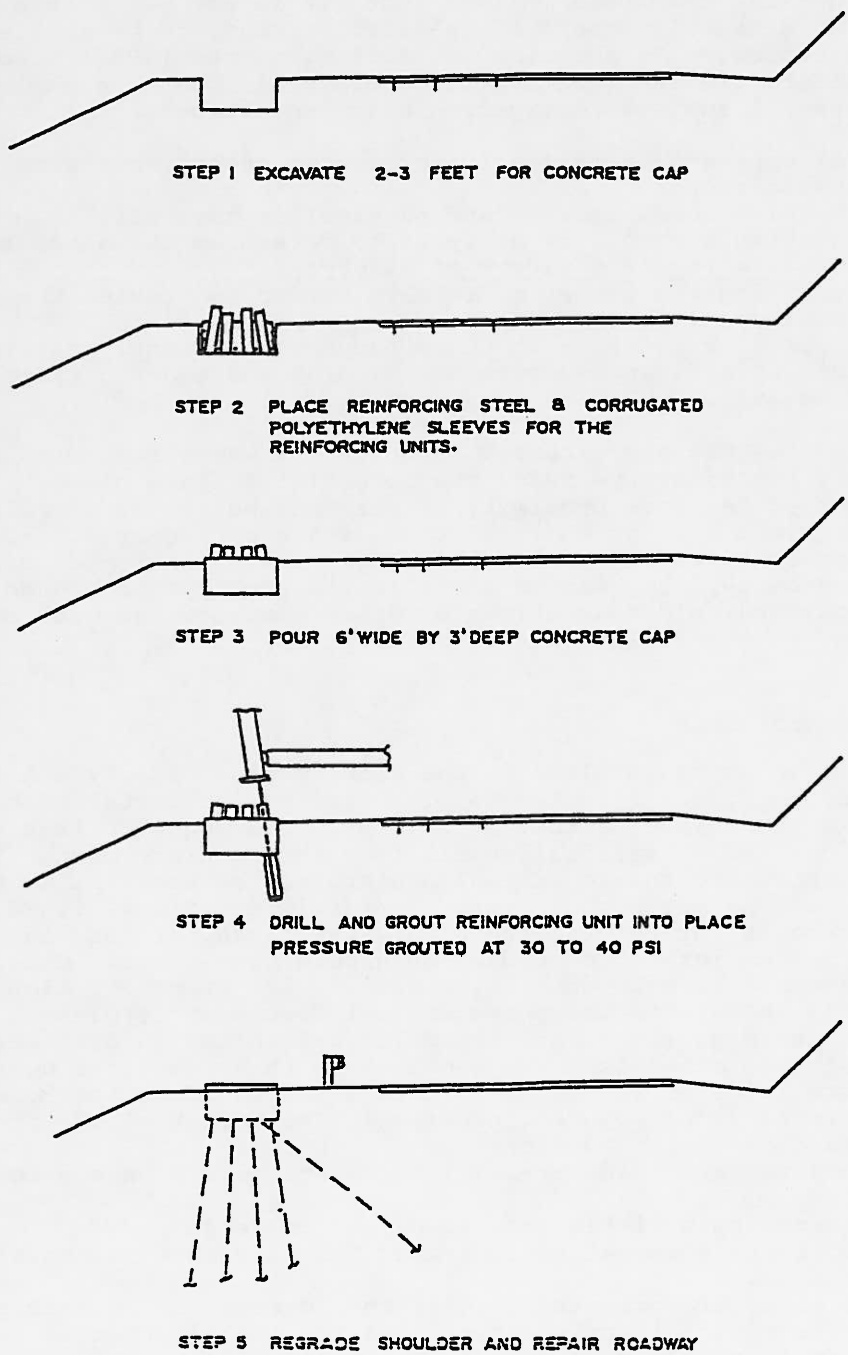
The successive steps involved in the construction of a Type A Wall are illustrated in Figure 11. The capping beam may be installed before or after the INSERTS are formed, although field evidence suggests that the latter option allows for an earlier benefit from the reinforcement. The drilling method is chosen to ensure minimal disturbance or upheaval to the soil. Of the seven generic methods of overburden drilling (Bruce, 1989d), the most common method is rotary drilling with water flush, either via a single casing or by the duplex method, depending on ground conditions. Once the casing has been advanced to target depth it is filled with a stable, high strength cementitious grout, and the permanent reinforcement is placed. This may be a solid high strength steel bar, typically 1-2 inches in diameter, or a steel pipe of suitable dimensions, as dictated by the structural design requirements. The drill casing is then withdrawn from the hole as grout continues to be injected under pressure. The effect of the pressure grouting is three-fold in most conditions:

- it ensures all voids or drilling related disturbances to the soil are filled;
- it permeates a little into sands and gravels;
- it compacts somewhat soils around the pile that is too fine to be permeated.

Individual piles are oriented in different directions in each plane to promote the most effective soil/pile network. After installation of the INSERTS, the capping beam is simply graded over, or it can form the base of a guard rail or similar: the whole wall is thus wholly out of sight and maintenance free.

Case History: Road in Armstrong County, PA

Portions of State Route 4023 north of Kittanning, Pennsylvania, were constructed on a slope adjacent to the Allegheny River. A 240 ft. long section of the two-lane road, and the railroad tracks located upslope, experienced damage caused by slope movements toward the river. In June, 1988, and January and February, 1989, the owner conducted a subsurface exploration program and installed inclinometers to monitor the slope movements. The inclinometers indicated that a slip-plane was located approximately 26-36 ft.



<u>Figure 11.</u> Typical steps in INSERT Wall construction

below the roadway and that the slope was moving at a rate of up to 0.75 inches per month downwards toward the river.

Site investigation drilling showed that a significant amount (20-30 ft.) of fill had been placed at the site apparently during the construction of the roadway and/or railroad tracks. The fill consisted of intermixed loose to medium dense rock fragments and medium stiff silty clay. Underlying the fill was a 5 to 10 ft. thick layer of stiff colluvial clay with rock fragments, in turn overlying a 3-20 ft. layer of weathered claystone. Competent rock was encountered at about 50 ft. below the roadway, and generally consisted of medium hard siltstones and sandstones.

The owner designed a repair of the failed section using an anchored caisson wall extending into competent rock. The earth pressures used for the design were based on the results of stability analyses, for which the soil along the slip-plane was assigned a residual friction angle of 17°. This design provided a minimum factor of safety with regard to the overall slope stability equal to 1.5 and 1.2 for the normal and rapid drawdown conditions, respectively. A row of 3 ft. diameter caissons were foreseen at a center-to-center spacing of 4.5 ft. and located immediately downhill of the roadway. The caissons were to be connected at the top by a cast-in-place reinforced concrete cap which was to have 90 ft. long prestressed rock anchors extending underneath the roadway at 7 ft. lateral intervals.

In 1989, the owner accepted the alternative contractor design employing an INSERT Type "A" Wall. The wall consisted of four rows of pinpiles extending across the slip-plane and into competent bedrock. It comprised two equal length sections designated as Wall A and Wall B. Wall A contained a higher density of piles than Wall B, because the top of weathered rock dipped to a lower elevation in the area of Wall A which resulted in a larger volume of soil to be stabilized in this area. In general, Wall A contained 4 piles per lineal meter, and Wall B contained 3 piles per lineal meter. Besides providing a significant cost savings over the original design, the selection of the INSERT Wall allowed for one lane of roadway to remain open during construction (February to May, 1989). The wall was constructed as described above, with the cap poured after pinpile installation for practical reasons.

To monitor the INSERT Wall performance, two sections of the wall were instrumented with strain gauges, inclinometers, telltales, and survey pins. The inclinometers yielded the most useful information regarding the performance of the wall. The data for inclinometers located relatively close to and within the wall indicated that up to 1.5 inches of horizontal movement occurred during the 75-day construction period, but that a maximum of 0.3 inches of movement occurred in the 7-month period following the completion of the wall.

Overall, the inclinometer data indicated that the wall performed as expected, and had effectively stopped the slope movements at the site. These data also confirmed that some deflection of the relatively flexible INSERTS was required to mobilize their lateral resistance.

CEMENT GROUTING

Rock grouting has been conducted in the United States for almost a century and soil grouting is well into its fifth decade (Bruce, 1992b). Various aspects of grouting are discussed elsewhere in this seminar by Welsh, and others, while this country has recently hosted the major international grouting conference of the period (ASCE, New Orleans, 1992). In this section, attention is focused on certain novel developments in the practice of cement based grouting for fissure injection and soil permeation.

Rock Grouting Methodology

In general, U.S. grouting practice as described comprehensively by Houlsby (1990) and Weaver (1991) may be regarded as "conservative" in comparison with that of other countries. This traditionalism is reflected in specifications relating to drilling type (rotary), permeability testing (simple), grout mix design ("thin"), and pump type (Moyno). However, there are two areas where major change is occurring: automatic parameter recording, and staging philosophies

- Parameter recording by electronic means has become standard practice on all federal jobs and on most others also. This may range from a simple "in the field" chart recorder, to the telemetric system, devised by the Bureau of Reclamation at their massive New Waddell Dam project in Arizona (Aberle, et al., 1990). There, electronic pressure transducers, magnetic flow meters and density meters in the field constantly relay data via a Remote Telemetry Unit to a Central Telemetry Unit, where all the grouting parameters are displayed in real time. Graphical data consist of flow rate, pressure, bag rate, and water-cement ratio. Numerical data include hole and stage number, target pressure, volume, density, w/c ratio, take rate, depth, cumulative take, date and time. Numerical data from six stages can be monitored instantaneously. The field inspector is in constant communication via radio with the CTU office to exchange information and instructions. Data are stored for future technical analyses and reports, and also for payment purposes. Aberle et al. concluded that these systems are extremely valuable and greatly help to direct and optimize the grouting.
- Regarding staging practices, the competent rock available and selected for past sites was ideally suited to ascending stage operations, and this method has become the traditional standard. Descending stage grouting is becoming more com- mon, reflecting the challenges posed by more difficult site conditions in the remedial and hazardous waste markets. The work described by Weaver et al. (1992) related to the sealing of dolomites under an old industrial site at Niagara Falls, NY, represents a statement of the best of American practice.

In some cases of extremely weathered and/or collapsing bedrock, even descending stage methods can prove impractical, and two recent projects illustrate innovative trends. Firstly, at Lake Jocassee Dam, SC, a remedial grouting project was conducted (Bruce, et al., 1992) to reduce major seepages through the Left Abutment of the dam. *Given the scope of operating within innovative contracting procedures*, the contractor was able to vary his methods in response to the extremely variable ground conditions actually encountered. Some holes permitted ascending stages, others needed descending stages, while the least stable holes had to be grouted through the rods during their slow withdrawal.

A second example is the grouting of poorly cemented hard rock backfill 2700 ft. below ground level in a copper mine in Northern Ontario, Canada (Bruce and Kord, 1991). This medium proved so difficult to drill that none of the conventional grouting methods could be made to work. Instead, the first North American application of the MPSP system, devised by Rodio, in Italy, was called for. The Multiple Packer Sleeved Pipe System is similar to the sleeved tube (tube à manchette) principle in common use for grouting soils and the softest rocks (Bruce, 1982). The sleeve grout in the conventional system is replaced by concentric polypropylene fabric collars, slipped around sleeve ports at specific points along the tube. After placing the tube in the hole, the collars are inflated with cement grout, via a double packer, and so the grout pipe is centered in the hole, and divides the hole into stages (Figure 12). Each stage can then be grouted with whatever material is judged appropriate, through the intermediate sleeved ports. Considerable potential is foreseen in loose, incompetent, or voided rock masses, especially karstic limestones (Bruce and Gallavresi, 1988).

As a final note, there remains considerable activity in bulk infill, principally associated with older, shallower mining operations in the

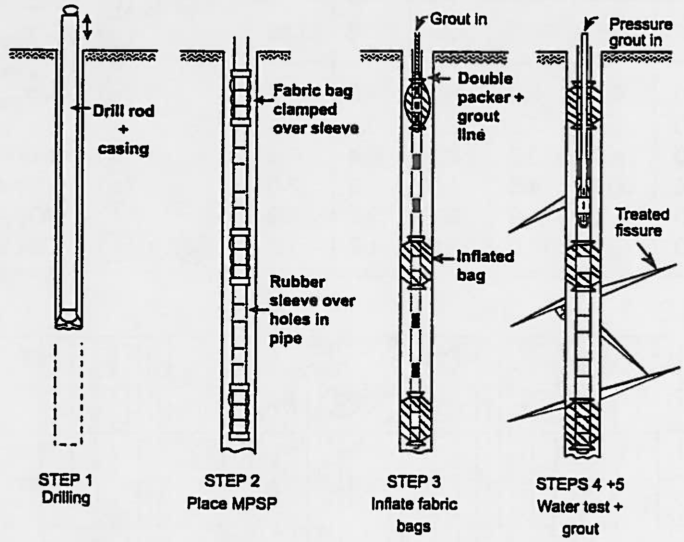


Figure 12. Typical steps in MPSP installation and grouting

Appalachians, and in Wyoming. Rotary and rotary percussive drills, often of water well drilling type, are common, with the void filling (either partial or total) being executed with cementitious grouts or concrete prepared in large scale site batching plants. Innovations are restricted to improved automated parametric recording and the development of special foamed grouts intended to extinguish mine fires.

Materials

Microfine cement grouts were introduced into the United States in 1984. Manufactured in Japan, the earliest example (MC500) is a mixture of finely ground Portland cement and slag in the ratio of about 4:1 (Karol, 1990). It can be used like a conventional cement grout with 4-5 hour setting time, or with sodium silicate to accelerate set to 1-3 minutes. It has been used on many relatively small projects in North America.

Clarke, et al., (1992) describe the use of two new products, MC300 (an ultrafine Portland of Greek origin) and MC100 (ultrafine slag) which can be mixed in varying amounts with dispersant to give a range of hardening times. Both are finer ground than MC500, and so have enhanced penetration potential. Other foreign manufactured materials are also available, including the aptly named "Stealth" grout. All these prebagged materials, however, despite their technical attractions, do share certain problems associated with availability, handling, preparation and cost, and much favorable attention has recently been focused on an alternative principle.

The Cemill^R technology (DePaoli, et al., 1992a) permits microfine grouts to be produced, on site, from normal cement grouts, in a wet regrinding process. Excellent grain size characteristics are produced (Figure 13), resulting in enhanced penetrability characteristics (Figure 14). Yet to be exploited in the U.S., this method is proving highly successful - technically and economically, in Italy.

Equally attractive to the U.S. market is the concept of improving the penetrability of cementitious grouts by fundamentally examining their rheological and internal stability characteristics. The Mistra^R series of grouts (DePaoli, et al., 1992b) has already been successfully exploited in Europe and provides extremely stable mixes with greatly reduced cohesion (Figure 15). Both these features generate major technological and economical benefits, and the concept is attracting favorable interest in the U.S.

	grain size (μmm)					
	D 95	D 85	D 60	D 50	D 15	D 10
CEMILL® 6	15.0	9.0	6.0	5.0	1.3	0.9
CEMILL® 9	9.0	5.5	3.5	2.5	0.6	0.4
CEMILL® 12	6.0	4.0	3.0	2.2	0.4	0.3
ONODA MC-500	8.0	60.0	4.5	4.0	2.5	2.0
Portland 525	40.0	22.0	11.0	8.0	2.5	2.0
bentonite	60.0	40.0	15.0	10.0	1.7	1.2

a b c sands for injection tests
a $\gamma = \gamma_{max} = 1.713$ g/cm^3
b $\gamma = \gamma_{max} = 1.701$ g/cm^3
c $\gamma = \gamma_{max} = 1.690$ g/cm^3

d bentonite
e Portland 525 cement
f ONODA MC-500 cement
g h i CEMILL® mixes

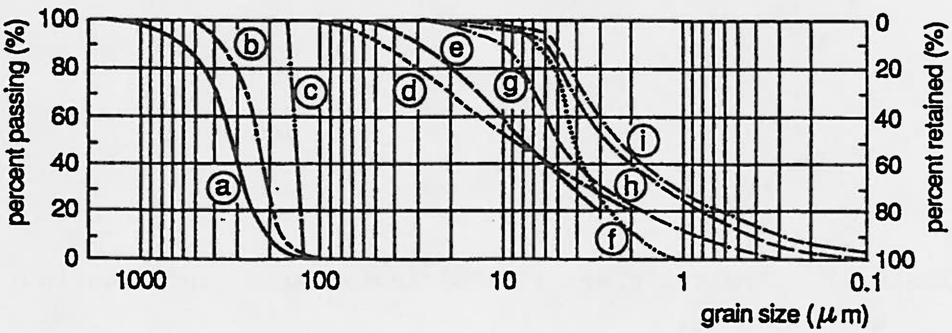


Figure 13. Grain size distribution curves for sands, dry materials and grouts (DePaoli et al., 1992a)

filter no.	permeability (m/s) theoretical Hazen (C = 1.45)	permeability (m/s) experim. (permeam.)	grain size (μm) D 95	D 60	D 15	D 10	U	porosimetry (μm) theoretical (Kozeny) D 80	D 50	D 30	experim.(Hg porosimetry) D 95	D 85	D 15	D10	specific surface cm^2/g	retaining capacity (μm)
07	$5.9 \cdot 10^{-3}$	$3.8 \cdot 10^{-3}$	1500	900	700	640	1.41	300	240	150	380	300	170	160	28	70
06	$2.3 \cdot 10^{-3}$	$8.3 \cdot 10^{-4}$	750	620	450	400	1.55	160	133	90	360	260	130	124	37	60
04	$7.7 \cdot 10^{-4}$	$4.5 \cdot 10^{-4}$	700	480	250	230	2.09	110	90	60	300	140	70	64	56	40
01	$2.8 \cdot 10^{-4}$	$1.6 \cdot 10^{-4}$	400	230	160	140	1.64	58	49	32	120	64	46	44	111	10
005	$1.4 \cdot 10^{-4}$	$9.5 \cdot 10^{-5}$	180	120	110	100	1.20	35	25	18	90	46	32	30	125	5

a

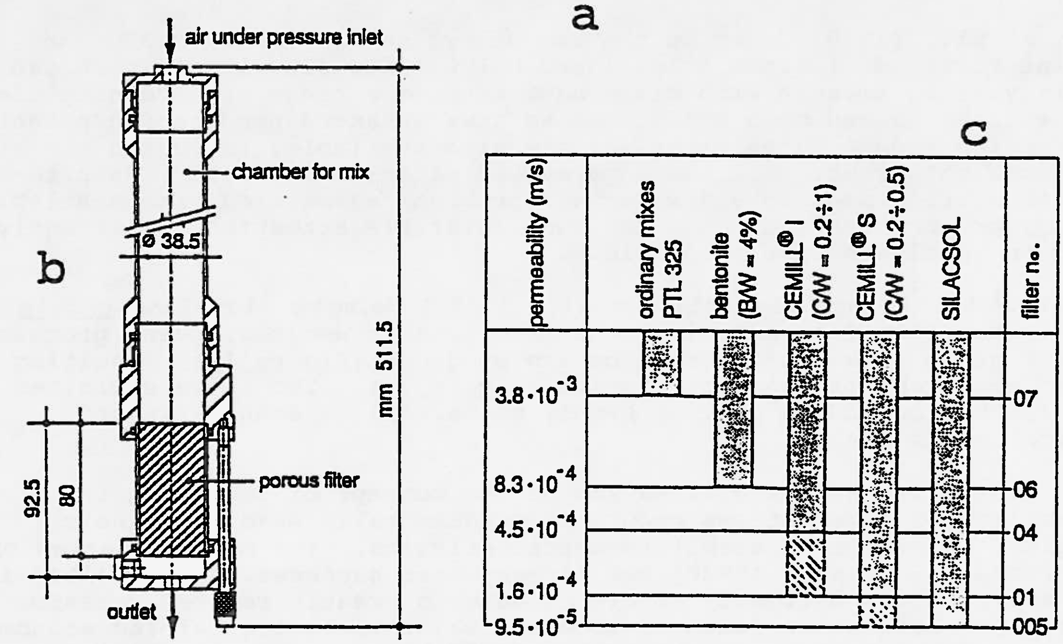


Figure 14. Injection test details: a) porous stone filter characteristics, b) apparatus, and c) penetrability limit of different mixes into filters (DePaoli et al., 1992a)

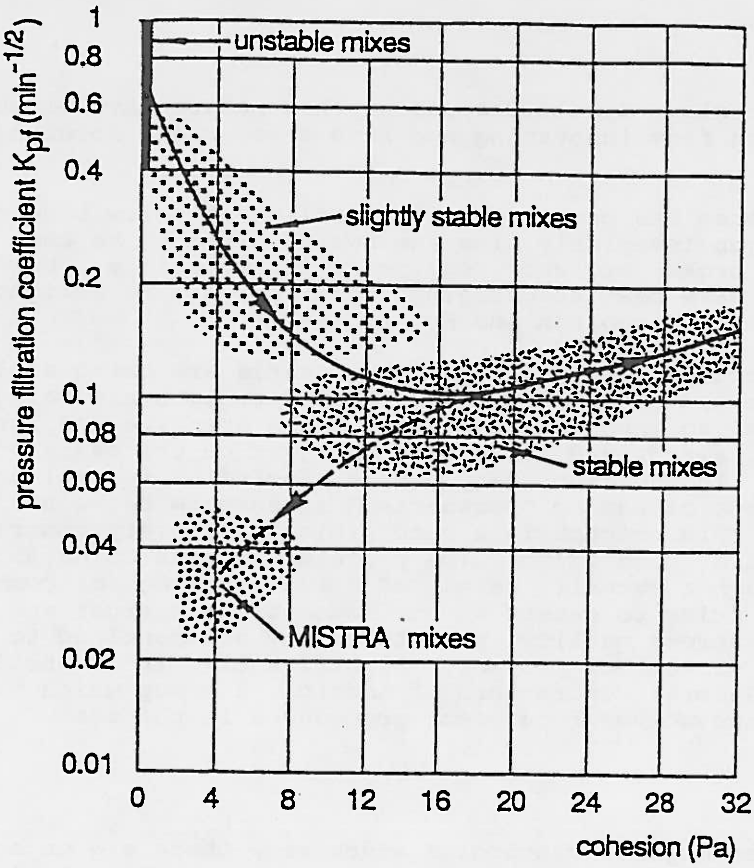


Figure 15. Relationship between stability under pressure and cohesion for the different types of mixes (DePaoli, et al., 1992b)

Contracting Practices

Weaver (1991) lists the elements necessary for a successful grouting project as follows:

- o a design accommodating the site geological conditions;
- o specifications that allow or facilitate modifications to the grouting program as the site conditions are revealed;
- o an "experienced, competent, cooperative and honest" contractor;
- o appropriate materials, equipment and techniques;
- o knowledgeable inspection staff, and
- o an effective quality assurance program.

While reviewing the history of grouting in the U.S., however, it is clear that rarely have these elements been simultaneously in place. The author believes that there are two fundamental reasons: inflexible specifications, and "low bid" procurement systems.

Regarding specifications, these must "be tailored to the project in hand and to the objectives to be accomplished" (Weaver, 1991). Instead, successive generations of specifications have been cobbled together from sections lifted from previous documents, and often contain "boiler plate" sections which may be contradictory and always perpetuate the use of outmoded procedures and/or inappropriate materials. Specifications of this nature have dissuaded domestic contractors from innovating and have discouraged foreign specialists from competing.

The procurement system has proved equally stifling: the low bidder on a tightly specified job invariably wins the award, although he then operates as little more than a broker of labor, equipment and materials. However, in recent years there have been encouraging signs that a more enlightened approach is surfacing (Nicholson and Bruce, 1992).

As a first step, stronger prequalification criteria are being applied to prospective bidders and their personnel. Specifications are being changed to "performance" types, so encouraging bidders to be creative and innovative, and most significantly, awards are being made not just on the basis of a low bid (Nicholson, 1990). In addition, many owners, including federal agencies, are promoting the concept of having "partnering" agreements between all the involved parties. This concept is a recognition that every contract includes an implied covenant of good faith. The process attempts to establish working relationships through a mutually developed formal strategy of commitment and communication. It tries to create an environment where trust and teamwork prevent disputes, improve quality, promote safety and continue to facilitate the execution of a successful project. Significantly, it is wholly endorsed by the Associated General Contractors of America, a group which has not always favored the more innovative procurement procedures in the past.

FINAL REMARKS

In each of the three major technologies addressed, there are major advances being made. In the case of ground anchorages these are largely in construction technique, and in the improved understanding of anchor-ground-structure interaction. For pinpiles used as Type A Walls, the developments are in design approaches and the benefits will accrue in the form of more cost effective schemes. Changes in grouting cover a wider range of facets including methodologies and materials. Perhaps the most significant changes, however, may arise from the alternative bidding practices, and other contractual changes which are being promoted. If allowed to flourish, they will encourage and reward innovation and imagination, while at the same time they will foster the spirit and reputation of geotechnical specialists in this country: their horizons have been too often lowered in the past by the pressures exerted by the legal profession and its heavy handed acolytes.

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FOUNDATION STABILIZATION SYSTEM USING JET GROUTING TECHNIQUES

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ABSTRACT

The Florida Suncoast Dome Stadium in Saint Petersburg, Florida is supported by a series of ring beam columns founded on large spread footings. The stadium was suffering damage due to settlements of the footings of up to six inches along its southeast perimeter. The results of geotechnical studies indicated that a thick interval of soft organic soil was present adjacent to this area of the stadium. Because as much as 3.8 inches of the total settlement was differential across the footing and toward the organics, it was agreed that the ongoing settlement of the footings was due to load being transferred into the compressible organic soils.

In order to reduce the potential for further settlements of the footings, the installation of a series of soilcrete columns using jet-grouting techniques was selected as the most attractive alternative. The soilcrete stabilization system was designed to transfer the majority of the vertical and lateral loads being imparted to the organic soils by the stadium footings to deeper, more competent strata.

This paper will present the layout and design of the soilcrete system and also a compaction grouting system. In addition, the results of engineering analyses, including settlement analysis, are included. Finally, performance monitoring data during construction and long-term settlement monitoring data after construction are provided.

DESCRIPTION OF PROBLEM/SUBSURFACE CONDITIONS

The Florida Suncoast Dome Stadium was completed in 1990, in anticipation of attracting a major league sports team to the Tampa Bay area, as well as providing a multipurpose facility for a variety of entertainment and sporting events, including concerts, shows, circuses, and tennis tournaments.

The stadium is a circular structure supported by a series of ring beam columns and founded on 20 feet by 24 feet spread footings. Geotechnical and exploration data showed a relatively thick deposit of very soft, compressible organic material interbedded with silty, fine sand. The construction of the large spread footings and interior floor necessitated dewatering and conventional excavation and replacement of these organic soils. Cone Penetration Tests (CPT) were performed in 1987 beneath each footing location prior to footing construction. An example CPT log is included as Figure 1. In addition, a relatively high water level was present, and up to 15 feet of fill were to be placed outside the footing locations.

Predicted settlements for the footings ranged from one to four inches. However, by 1990, nearly six inches of total settlement had occurred at one of the footings, with nearly four inches of differential settlement across the footing. Several holes were drilled to investigate the cause of the settlements. As shown on the cross-section included as Figure 2, it was discovered that the removal of the organic materials extended only to the edge of the footings and not past the zone of influence of applied stress from the footings.

It was theorized that although the footings were bearing on engineered fill, the loads from the columns were being partially transferred to the compressible organic soils, resulting in the ongoing settlements of the footings, especially in the direction of the organics. Further, vertical and lateral compression of the organics has occurred due to the loading from the placement of up to 15 feet of compacted fill outside the limits of the footings. The affected footings not only included the main ring beam column footings, but also the exterior wall foundations (see Figure 2).

The most serious consequences of the movements were the stress changes in the steel cross-bracing and the cable-stayed roof of the domed stadium. The stadium roof structure was supported by a large ring beam held in place by cross-bracing and columns resting on the spread footings. It was determined that if the settlement went unchecked, costly damage would result.

The challenge for the engineers investigating the problem was forensic in that the as-built conditions and construction activity had to be reconstructed. This was a difficult task as the excavation face was not the designed 1.5 horizontal to 1.0 vertical (1.5H:1.0V) slope, but was more like 0.4H:1.0V. Locating the near vertical plane of the excavation limits was critical to the selection of a remediation system and required an angle drilling approach during the investigation.

SELECTION OF THE SOIL STABILIZATION SYSTEM

Requirements for the selected stabilization system included:

1. Limit wall and column footing settlements without additional treatment in the future;

2. Minimize disturbance to existing foundation soils;

3. System must be economical;

4. Installation time must be relatively short; and

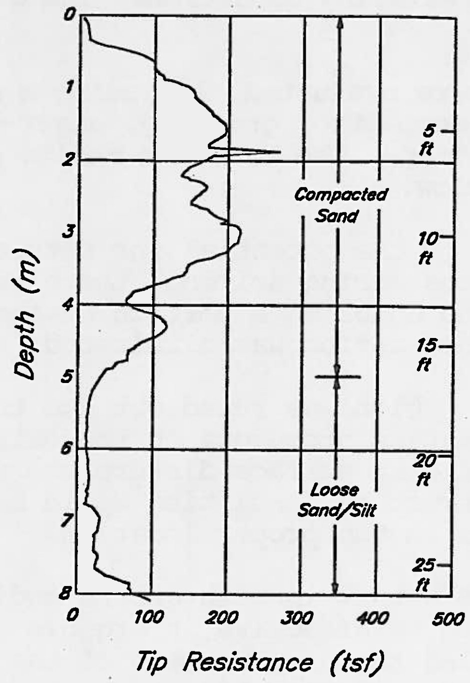

Figure 1. Typical CPT log (beneath footing R-8).

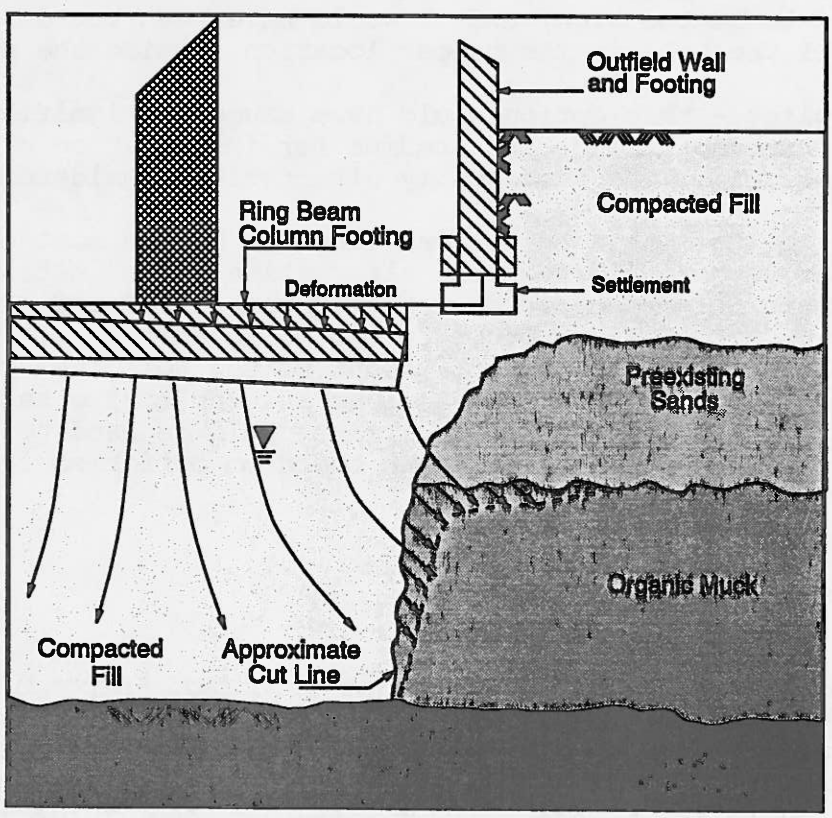

Figure 2. Cross-section showing subsurface conditions.

5. Minimize impact on existing conditions, inside and outside the stadium.

Several repair methods were evaluated, including a sheet pile wall, structural slurry wall, compaction grouting, auger-cast pile wall, mini-piles, and jet grouting. The relative merits of the various options are discussed below.

<u>Sheet Piling</u> - due to the potential for settlements of the loose sands from vibrations during driving, the high material costs, and the necessity to demolish a portion of the structure for proper location, this option was eliminated.

<u>Slurry Wall</u> - this option was ruled out due to the potential impact of the excavation procedure on the existing foundation. In addition, significant surface disturbance was certain for this option, and some structure demolition would have been necessary to install the wall in the proper location.

<u>Compaction Grouting</u> - this in-situ ground modification technique is not considered to be effective in organic material. In addition, controlling the displacement of the stiff grout under high pressures would be difficult.

<u>Auger-cast Pile Wall</u> - although this option was considered to be a feasible alternative, the relative installation costs were deemed to be too high, and it would have been too disruptive to install the wall in the proper location (inside the structure).

<u>Mini-piles</u> - this option would have caused a significant amount of disturbance inside the stadium for installation of the piles, and was deemed the most costly alternative considered.

<u>Jet Grouting</u> - this option was found to be the most economical as well as technically feasible alternative. The installation of soilcrete columns could be strictly controlled and produce only a minor amount of disturbance to the surrounding soils. Because the procedure involves replacement of the soil, the organic materials could largely be removed and replaced with grout. Finally, relatively small equipment could be used to install the soilcrete columns, and all work could be performed from outside the stadium structure.

SOILCRETE STABILIZATION SYSTEM

General

Soilcrete is the product created by jet grouting. Depending on the system of jet grouting employed, it can be a product of mixing cement slurry with soil or it can be a nearly complete replacement of soil with enriched cement slurry. As shown on Figure 3, there are three systems of jet grouting; the Single-, Double-, and Triple-Rod systems. The Single- and Double-Rod systems inject cement-slurry at high velocity (approximately 5,800 psi back-pressure) to mix with the soil

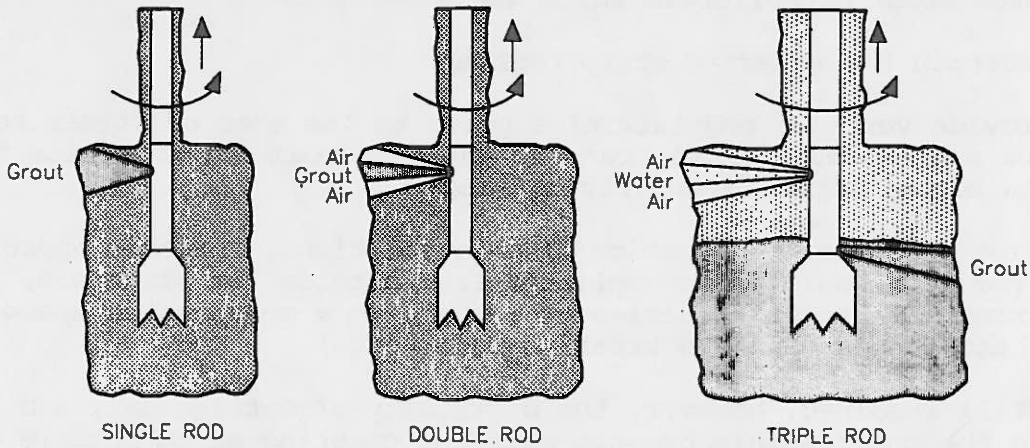

Figure 3. Jet grouting systems.

and create a hardened soilcrete. The Double-Rod system differs from the Single-Rod system by the addition of an air sheath around the grout jet nozzle to enhance cutting distance. In each case, slow, uniform rotation and lifting of the rods produce a columnar geometry of soilcrete. The Triple-Rod system differs markedly by using high velocity water injection, sheathed in a cone of air, to cut away the soil and simultaneous tremie injection of an enriched cement slurry to displace and replace the soil.

Typically, the Single- and Double-Rod systems are best suited to cohesionless sandy soils (wherein sand makes for good aggregate) and are not as effective in cohesive and organic soils (where the soil will negatively affect the quality of the soilcrete). The Triple-Rod system can work well in sandy soils, but has the unique ability to perform in-situ an almost surgical removal of poor quality soils and replace them with a soilcrete with strength properties 10 to 100 times greater.

Options

On review of the project and the structural support requirements, it was first considered to completely underpin all foundations to a suitable depth beneath the peat strata. This would require working through the peat for the exterior wall strip footing, but for the large ring-beam support columns, the soil would typically be sand. Due to the ring-beam column loading (dead loads ranging from 1,000 to 1,450 kips), a significant quantity of soilcrete would have been necessary due to the low bearing capacity of the deeper sands. This soilcrete work would have to be accomplished using low headroom drilling rigs working inside the structure and through the existing footings. The impact of working inside the structure, as well as the significant cost to be incurred for this solution, urged another approach to the problem.

After much discussion, it was decided to review the option of creating an in-situ block of soilcrete which would serve to:

1. Underpin the exterior strip footing.

2. Provide vertical and lateral support to the zone of stress beyond the engineered fill to transfer the ring-beam column stress to the higher capacity underlying sandy soils.

See Figure 4 for an illustration of this solution. For this approach, all of the work could be accomplished from outside the structure, eliminating all the difficulties of working in a constricted space (waste handling, adding and breaking rods, etc.).

There still remained, however, the difficulty of cutting away and removing the compressible organic soil and creating an acceptable strength soilcrete. To achieve this, the contractor employed a procedure which:

1. Allowed the jet-grout holes to be staggered, thus permitting the soil to arch and redistribute stress while the soilcrete hardened.

2. Double cut the organic strata to provide a high percentage of organics removal.

3. Injected an enriched cement-slurry to provide the highest displacement and strength properties.

Another problem that was foreseen was that the in-situ sands below the engineered fill interval under two of the ring-beam foundations seemed to be "looser" than what might have been desirable for a structure of this type. Since the CPT profiles indicated these soils were clean sands, it was decided to compaction grout these zones and subsequently enhance their bearing capacity. Compaction grouting consists of injecting a stiff, mortar-like grout to densify soils in place. As will be discussed later, this compaction grouting provided evidence that these soils were not clean sands, but rather, sands thinly bedded with organic silt.

QUALITY CONTROL

When performing any type of in-situ ground modification, the quality control program must be capable of assuring that the design guidelines are achieved. The first part of any jet grouting program is the test section. For this project, a series of preproduction columns were installed while varying jet grouting parameters, which included water injection velocity, air injection velocity, grout injection flow rate, and rotation and lift speeds.

The completion of the test program provided enough qualitative data to select the actual production parameters for the work.

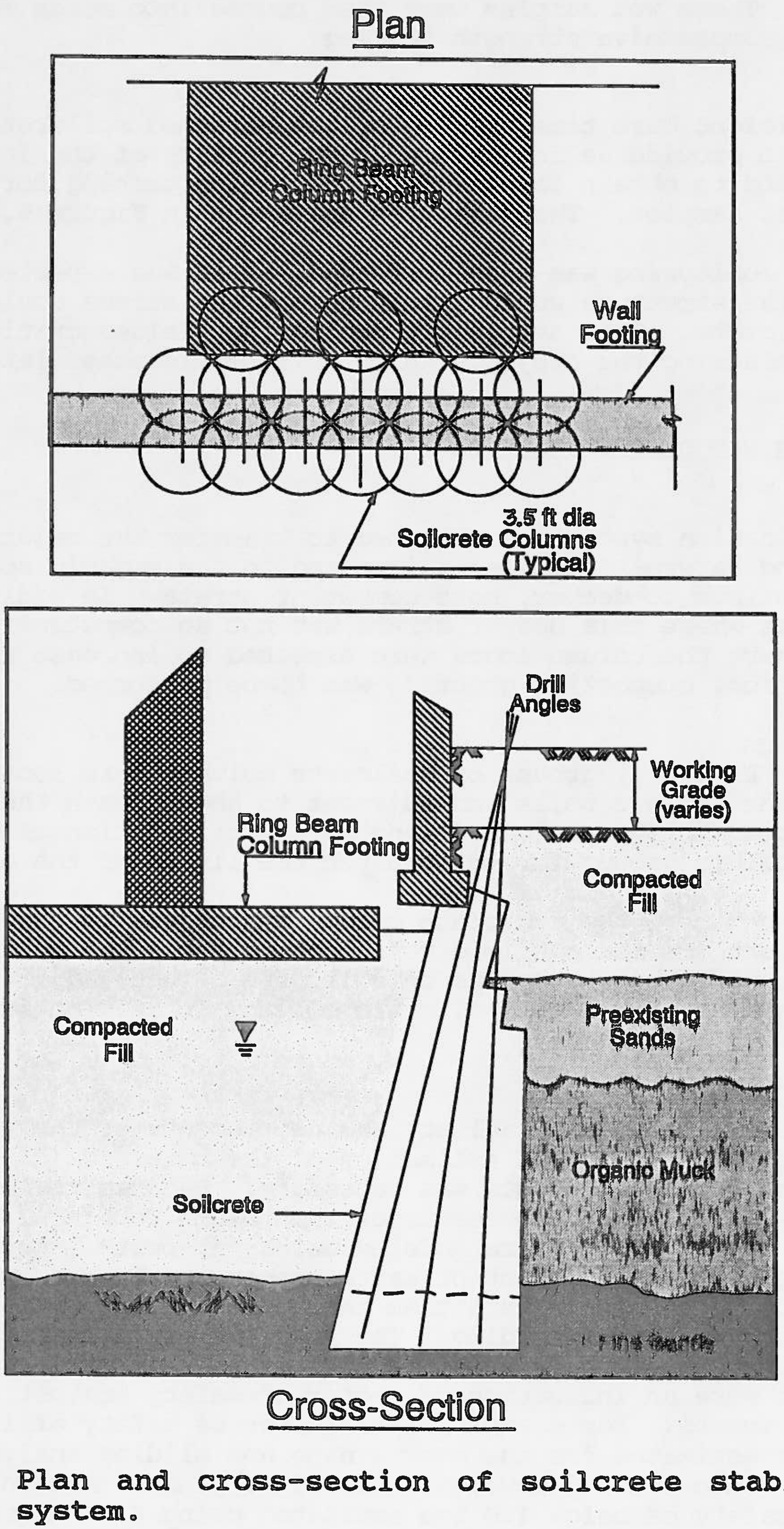

Figure 4. Plan and cross-section of soilcrete stabilization system.

During production work, uncured soilcrete samples were retrieved using a specially developed in-situ sampler which could capture a sample from any depth. These wet samples were then poured into molds for future unconfined compressive strength testing.

After sufficient cure time, coring of the hardened soilcrete mass was performed to provide an indication of the quality of the in-situ soilcrete and to obtain samples for compression testing for comparison with the wet samples. These results are shown in Figure 5.

Settlement monitoring was also performed, as it was expected that some strain of the structure would result before the stress could be applied to the soilcrete. This settlement monitoring yielded critical information during the project, which will be discussed later.

ENGINEERING AND DESIGN ANALYSES

General

The stabilization system was designed to transfer the majority of the vertical and lateral loads being imparted to the organic soils by the stadium footings to deeper, more competent strata. In addition, in select areas where this deeper strata was not so competent (i.e., loose sand) or where the column loads were expected to increase after load redistribution, compaction grouting was to be performed.

System Design

As shown on Figure 4, groups of soilcrete columns were located at the limits of the organic soils and adjacent to and beneath the larger column footings. The projected lengths and orientation of the soilcrete column groups varied to match the limits of the organics.

Another function of the soilcrete column system was to provide some direct support for the outfield wall footing above the column footings. This was provided by the groups of soilcrete columns adjacent to the footings, and by individual soilcrete columns located at intermediate points between the groups.

Engineering Analyses

In order to evaluate the stability and settlements of the various soilcrete column groups, an estimation of the magnitude of lateral earth pressures and movements was necessary. Because the purpose of the system was to reduce movements of the column footings, the various components of the system were modeled using "at-rest" lateral earth pressures (i.e., coefficient of earth pressure = K_o, no lateral movements). The footings were then analyzed for factor of safety against overturning and sliding. The results were not interpreted as an indication of factor of safety against failure of the soilcrete system, but were an indication of factor of safety against undesirable lateral movements. For example, if a factor of safety of 1.0 or greater was estimated for the overturning and sliding analyses using K_o conditions, then lateral movements should be minor. Alternately, if a factor of safety of below 1.0 was estimated using K_o conditions, then

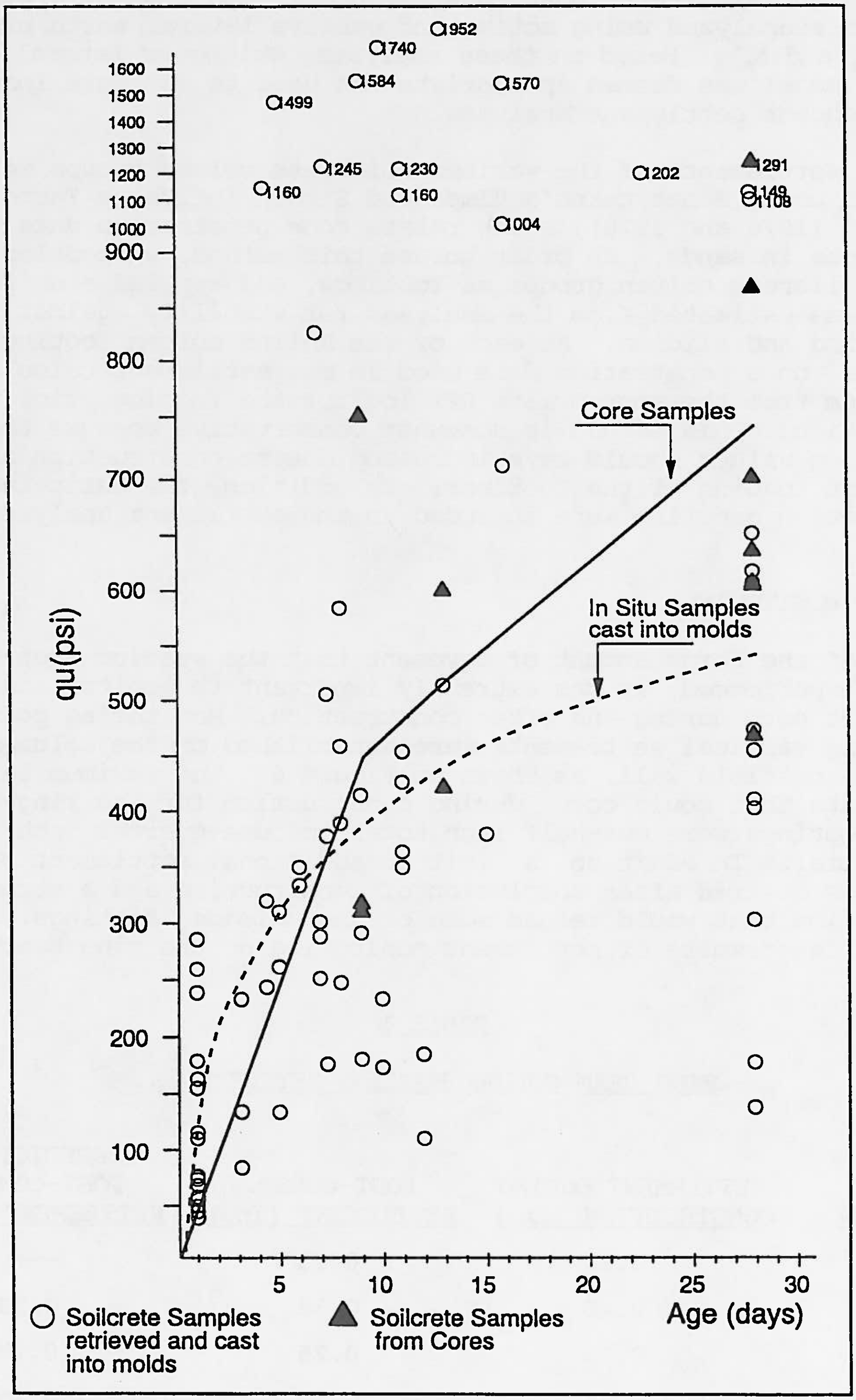

Figure 5. Graphical results of soilcrete unconfined compression strengths (all samples from organic silt/peat zone).

more substantial lateral movements may occur, and the soilcrete column group was reanalyzed using active and passive lateral earth pressures (i.e., K_A and K_p). Based on these analyses, whichever lateral earth pressure model was deemed appropriate was used to estimate loads for the subsequent settlement analyses.

Vertical settlements of the various soilcrete column groups were estimated using Schmertmann's "Improved Strain Influence Factor Diagrams" (1978 and 1970), which relate cone penetration data to settlements in sands. In order to use this method, we modeled the base of the soilcrete column groups as footings, and applied a uniform pressure as estimated from the analyses for stability against overturning and sliding. At each of the R-line column footing locations, cone penetration data used in our settlement calculations were taken from the appropriate CPT log for the footing prior to its construction. This method is somewhat conservative because the cone penetration values should have increased due to construction and subsequent loading of the footings. In addition, the estimated effects of compaction grouting were included in the settlement analyses.

MOVEMENT MONITORING

Because of the large amount of movement that the stadium footings had already experienced, it was extremely important to monitor additional settlement both during and after construction. Monitoring points for estimating vertical settlements were established on the columns and along the outfield wall, as shown on Figure 6. The maximum target settlements that could occur during construction for the ring-beam column footings were one-half inch total and one-quarter inch differential. In addition, a limit on additional settlement of 0.4 inches was desired after completion of construction and a structural reconnection that would reload some of the unloaded footings. Table 1 includes the results of settlement monitoring on the ring beam column footings.

TABLE 1

RING BEAM COLUMN FOOTING SETTLEMENTS

COLUMN	SETTLEMENT DURING CONSTRUCTION (in.)	POST-CONST. SETTLEMENT (in.)	PREDICTED POST-CONST. SETTLEMENT (in.)
R-7	0.12	0.25	---
R-8	0.45	0.38	0.53
R-9	0.25	0.25	0.25
R-10	0.26	0.25	0.35

Figure 7 presents a graphical representation of settlement monitoring along the outfield wall, as well as predicted settlement of each

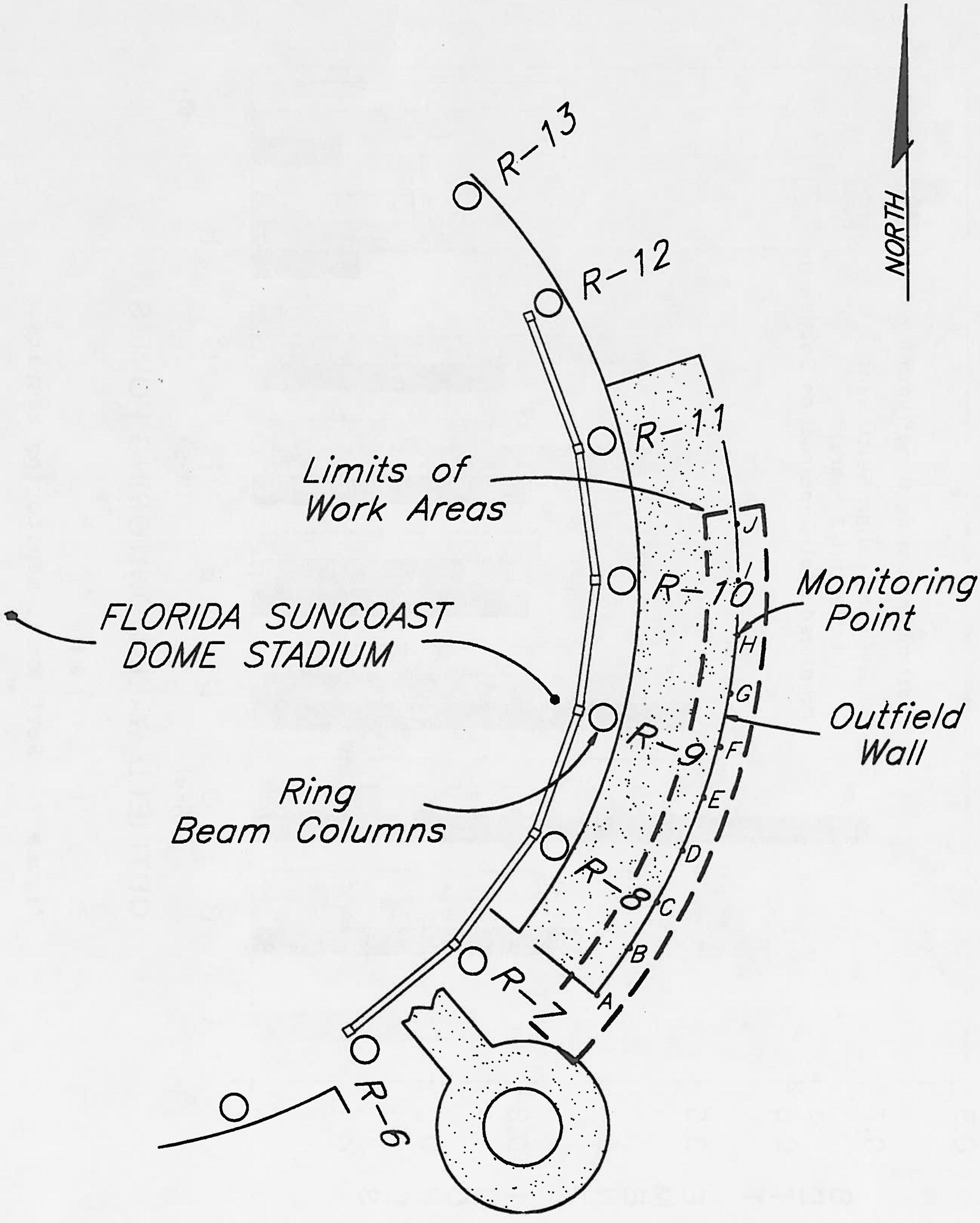

Figure 6. Plan view showing monitoring point locations.

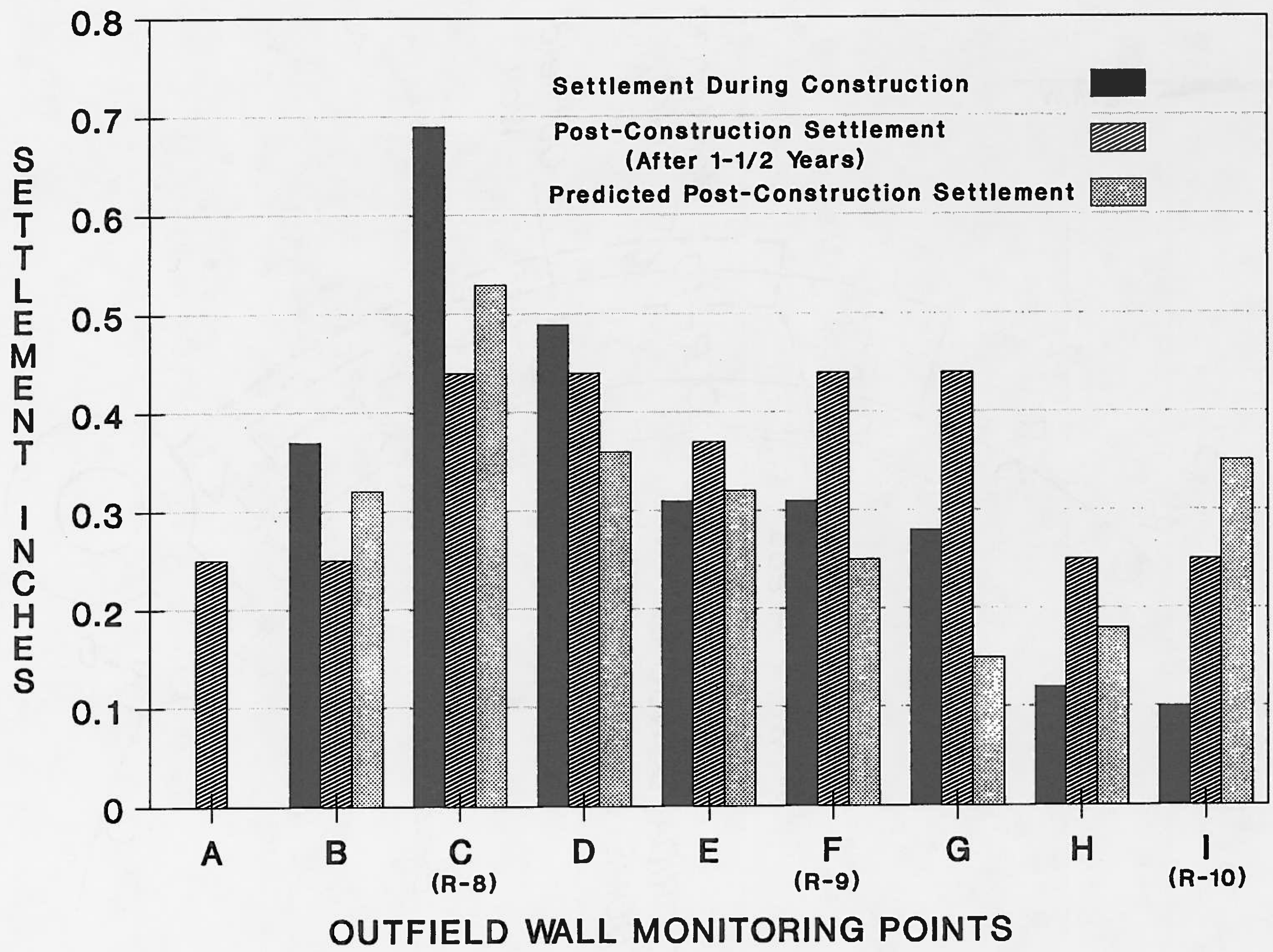

Figure 7. Settlement monitoring results.

soilcrete mass. As shown thereon, the settlements along the outfield wall (which were interpreted as a direct settlement of the soilcrete mass) were typically greater than the subsequent settlement of the ring beam column footings.

It should be noted that column R-8 experienced nearly 80 percent of its construction settlement during the compaction grouting phase of work which was accomplished post-jet grouting. Compaction grouting was designed to be directed into the stress zone beneath the footing and beneath the compacted fill. The quantity of grout injected and pressures monitored were as expected, and a slight heave of the foundation was noted at the end of each shift. However, the next morning after each of these shifts there was a net settlement of the footing.

This effect had been experienced on previous projects, and was potentially indicative of the remolding effect on lenses of organic silt which was observed during the grout pipe installation. Compaction grouting may cause a temporary increase in pore water pressure in these soils, as well as significant disturbance from displacement, which results in loss of strength and increased settlement potential. The post-construction settlements confirm this effect, as these settlements were more time-dependent than would be expected from sands.

CONCLUSIONS

Based on the results of movement monitoring and testing of the soilcrete columns, we conclude that:

1. Jet grouting with the Triple-Rod system was capable of creating very high quality soilcrete even in materials as poor as peat.

2. Settlements during construction could be limited to approximately 0.5 inches working in the most difficult soil conditions.

3. Post-construction settlement monitoring indicates that using Schmertmann's strain diagrams for estimating settlements in sand were fairly accurate. Differences between the predicted and actual settlements were probably due to variable soil conditions (i.e., the inclusion of silt or organic layers that are susceptible to long-term settlements) and limited soil data.

4. Compaction grouting beneath select footings provided an improved support system, yet could have been performed with much less strain had the subsurface conditions been more accurately known. The pressures achieved, grout takes, and subsequent induced settlements exhibited behavior not typical of clean sands.

5. Ground improvements using jet grouting and compaction grouting techniques can provide a cost effective solution to problems of this type. Moreover, the work can be performed in a timely manner, even where space constraints are present, as was

demonstrated by the time to complete the field work (less than four weeks in this case).

Photographs of the actual work are included as Figures 8 through 11.

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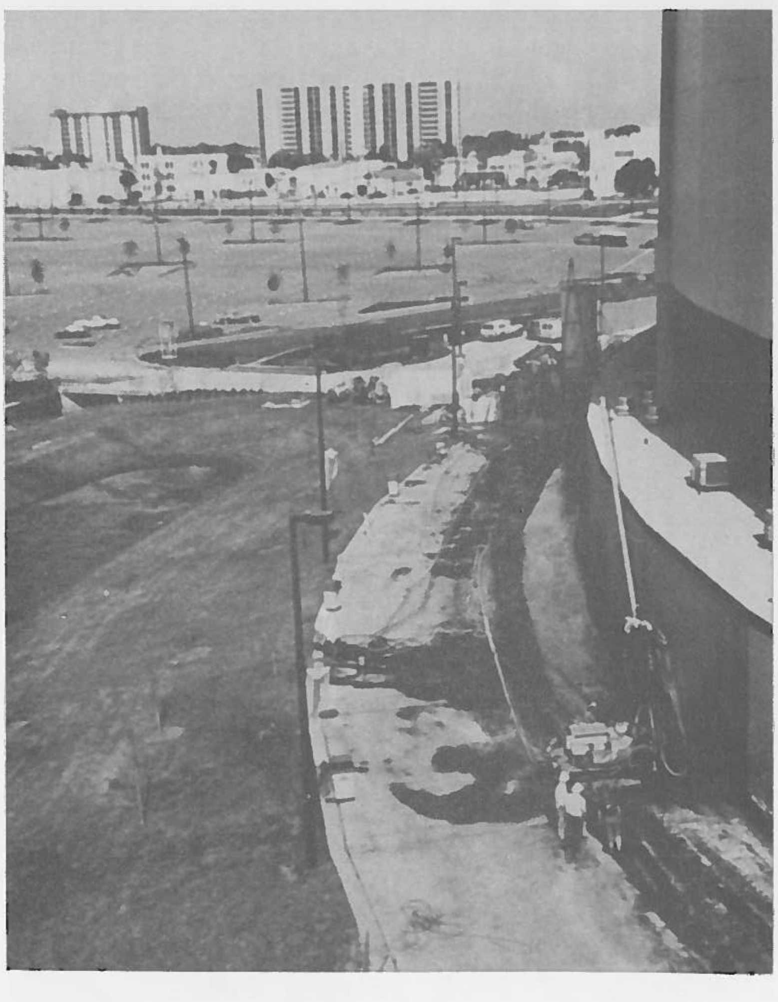
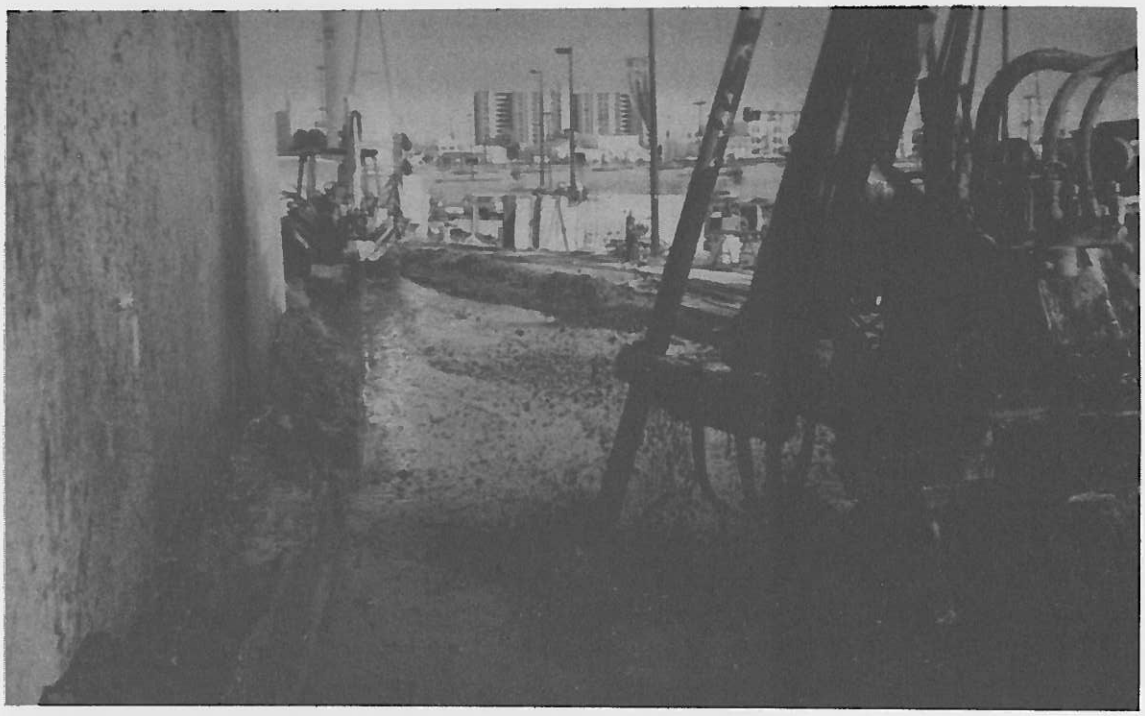


Figure 8. Florida Suncoast Dome Stadium.

Figure 9. Angle drilling beneath stadium prior to jet-grouting.



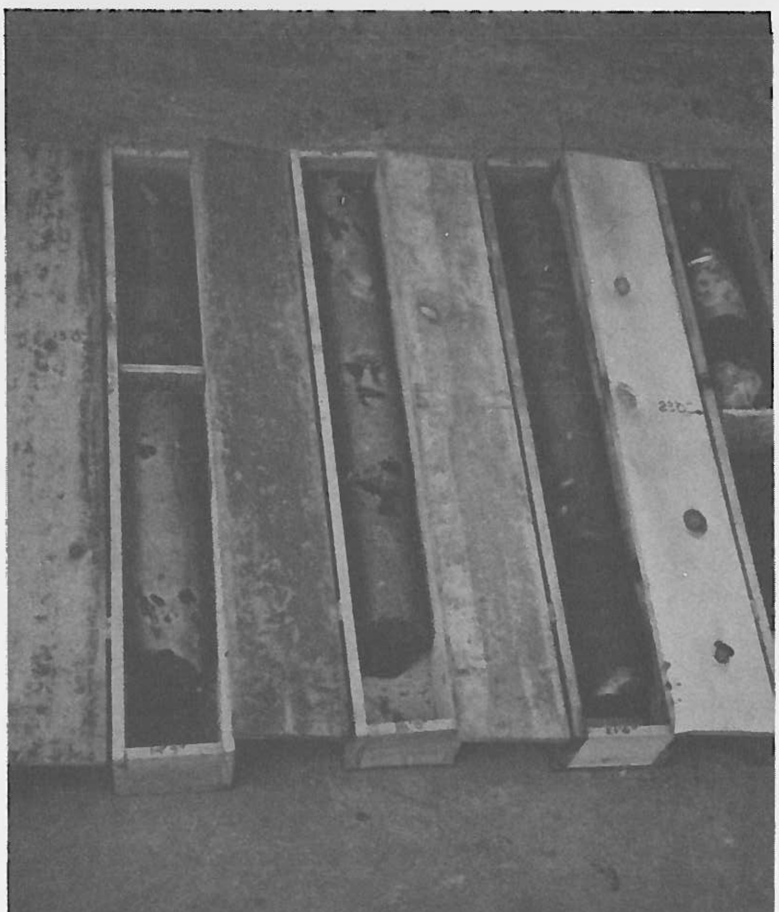


Figure 10. Jet-grouting operations displacing organic materials.

Figure 11. Soilcrete cores taken from completed columns.

APPLICATION OF LIME-FLY ASH INJECTION IN RUNWAY REHABILITATION

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ABSTRACT

Soils are an integral part of a pavement system. It is safe to say that the quality of the soils underlying pavements has an effect on the long-term performance of a pavement. Most soils found to be inefficient for supporting pavement can be treated or replaced prior to paving. However, once a pavement has been constructed, the options for improving the soils beneath it are limited. Many of the governmental agencies (D.O.T.'s, F.A.A., county and city) responsible for maintaining pavements are faced with problems in existing pavement arising from poor subsoil conditions. Few options exist in these situations other than continuing long-term maintenance, such as repeated overlays and patching.

The method presented is the injection of lime/fly ash slurry into the problem soil mass. The case history is explained in detail along with the mechanism of lime/fly ash injection (L/FAI) and the before and after results of field tests. The paper presents a description of an in situ stabilization method which was implemented to study a soil beneath an existing runway pavement. The procedure entails the injection of lime-fly ash slurry into the problem soil mass.

INTRODUCTION

The Delaware County Airport Authority contracted Wetzel Engineers, Inc. to prepare plans and specifications for an asphalt overlay to strengthen Runway 2-20 at the Delaware County Airport. The runway serves aircraft with a maximum dual wheel strength of 66,000 pounds. This is a synopsis of the project from design through construction that used an innovative and cost-effective method for stabilizing and strengthening the subgrade without removing the pavement resulting in reduced overlay thickness - "Soil Stabilization with Lime-Fly Ash by Injection Method."

DESIGN

Location

The Delaware County Airport, shown in Figure 1, is located 3 miles north from the center of the City of Muncie, Indiana, approximately a 1½

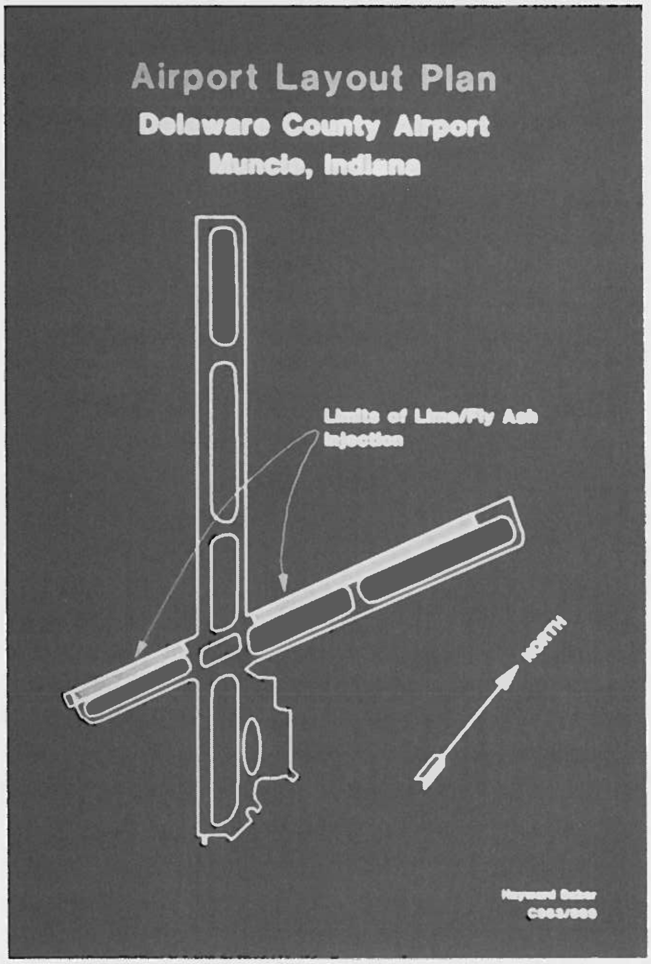


FIGURE 1 Project limits.

hour drive from Indianapolis. Runway 2-20 is the Airport's busy secondary runway because of the orientation into the prevailing winds.

Runway 2-20 was first constructed in 1955. Since 1955, the runway has been extended twice and overlayed three times. The last overlay was in 1979 and the last extension was in 1988. The pavement sections were either full-depth asphalt or asphalt over aggregate base. Full-depth pavement thicknesses ranged from 9 inches to 13½ inches. The asphalt over aggregate pavement thicknesses ranged from 5 inches to 10 inches of asphalt over 7 inches to 14 inches of aggregate. The average section in the injection area was 8 inches of asphalt over 12½ inches of bank-run gravel.

FIGURE 2 Obtaining pavement and soil samples.

Design Parameters

The design followed Federal Aviation Administration (FAA) design and evaluation criteria. The critical aircraft was the Grumman Gulfstream II with a maximum gross weight of 66,000 pounds. The overlay was asphalt with a design life of 20 years. Annual departures were assumed to be less than 1,200.

Pavement Condition Survey

Although a formal pavement condition survey and a subsequent pavement index were not required, field data was obtained to determine the structural integrity and operational surface condition. Field data obtained included pavement distresses and severity, pavement thickness, surface and subsurface drainage conditions, and climatology. Through evaluation of the data the existing pavement was adjusted for comparison to a minimum pavement section.

Subsurface Soil Exploration

Materials Inspection and Testing, Inc. (MIT) of Fort Wayne, Indiana was contracted to perform a geotechnical investigation of various pavement and soil horizons with the rig shown in Figure 2, report on the engineering characteristics and provide criteria for the design. Soil borings at 200-foot intervals were taken and the soil horizon and ground water level visually recorded. Soil samples were generally obtained at

FIGURE 3 Conducting field CBSs on subgrade.

2½-foot intervals with split spoon and standard penetration values recorded as the split spoon was driven into the soil sample. Laboratory tests included unconfined compressive strength, hand penetrometer, moisture content, soil density, Atterberg limits, grain size analysis and California Bearing Ratios (CBRs). Laboratory CBR values for 0.1 and 0.2 penetrations of soaked samples were recorded at varying densities. The CBR values for the 0.1 penetration ranged from 6.9 to 10 percent. Because the majority of the soils sampled had high moisture content, two CBR samples were prepared to simulate actual soil samples. The simulated field condition CBR values were 2.9 and 1.3 percent. These simulated CBR values indicated field CBRs were necessary to fully understand the subgrade support conditions for the overlay. Subsequently, Engineering and Testing Services, Inc. of Indianapolis, Indiana was contracted to perform three field CBRs as shown in Figure 3. The plasticity index of the soils ranged from 16.9 to 20.24 and the percent of swell was generally above two, indicating a moderately expansive soil. The unified soil classification for the majority of the soils was CL.

Problems

It became apparent through the data collection process that an overlay would be ill-advised without first addressing other problems such as cracking, weathering, high-moisture content, varying soil densities and unstable soil.

Longitudinal Cracks
(Medium to high severity)

Block Cracks
(Low severity)

FIGURE 4 Pavement distresses encountered.

Cracking was present in several types of densities and severities. Types of cracking encountered were alligator, longitudinal, transverse and block. Alligator cracking, caused by loss of support and structural failure, generally occurred left and right of the runway centerline and was of low severity. Longitudinal and transverse cracks were caused by shrinkage due to low temperatures, asphaltic cement hardening or reflecting paving joints and were of medium to high severity. Block cracking was transformed from longitudinal and transverse cracks, asphaltic cement shrinkage and temperature cycling. Block cracks were low severity. Examples of distress encountered are shown in Figure 4.

The major concern was not the cracking but the unstable subgrade conditions. Organic clays, silty clays and poorly compacted fills were encountered. Almost all of the soil had high moisture content. The dry densities varied from 90.4 to 104.8 lb/cft. A subsurface drainage system was installed at the edge of the pavement in 1983, but high moisture contents indicated limited effectiveness of the subsurface drains. The weak soil strengths (CBR) encountered were inversely proportionate to the moisture content and proportionate to soil densities.

Alternatives

Alternatives for the flexible pavement design were:

a. Overlay
b. Remove and Replace

c. Remove, Modify and Replace
d. Lime-Fly Ash Injection (L/FAI) and Overlay

Overlaying would not correct the high moisture content and weak subgrade problems.

Remove and Replace involved removing to four feet below finish subgrade, replacing the same soils under controlled conditions to achieve optimum densities, then placing the required pavement section. Although in time, the subgrade would accept moisture, reducing the density below optimum.

Remove, Modify and Replace involved removing to four feet below finish subgrade, modifying the soils with lime and fly ash, replacing the modified soils, then placing the required pavement section. This alternative was the most effective method from the standpoint of subgrade modification but was not cost effective and increased construction time.

The soils were a good candidate for lime modification because of low sulfate content (25.1 ppm) and favorable pH values (8.13 units). Fly ash was added to enhance the strength characteristics of the soils because of the predictable pozzolanic reaction with lime.

Lime-Fly Ash Injection and Overlay involved stabilizing the subgrade to five feet below the surface. The process of soil modification by injection, although not new, has never been attempted through existing pavement and has not been used in this region. However, historical data was available on comparative soils at Dallas/Fort Worth (D/FW) International.

After communications with MIT, the National Lime Association Headquarters, and Hayward Baker Inc.'s personnel, Otto Mueller (a retired local expert in soil stabilization with lime and fly ash), review of available data and interviews of references, the decision was made to select the L/FAI and Overlay Alternative.

<u>**Benefits and Mechanisms of Lime-Fly Ash Injection**</u>

While the stabilization of an existing runway pavement was a new application for L/FAI, the process has been used successfully to treat clays and silts since the 1960's. Although it is sometimes difficult to quantify the exact degree of improvement derived from injection in a soil mass, it has been determined through successful application that certain benefits do occur. One of the primary benefits of L/FAI is the creation of a moisture barrier in the soil mass as illustrated in Figure 5. During the injection process the lime-fly ash slurry flows through the crack pattern of the soil mass. Free water is squeezed out and the slurry then sets up via pozzolanic reaction to prevent future water intrusion.

An additional benefit that comes about as a result of reducing the moisture content of the soil is an increase in the shear strength of the soil mass. L/FAI is commonly used to repair slope failures because of the ability of the process to cause an increase in the shear strength in the weak planes of the soil mass. Bearing strength of the soil also increases after L/FAI. The actual increase in bearing strength can be measured utilizing CBR tests.

Evaluation of Candidate Soils

The injection of lime/fly ash was primarily utilized for improving fine-grained cohesive soils (clays and silts). Candidate soils usually have 75 percent or more material passing a #200 sieve, low sulfate content and pH values of not less than 6.5. The primary characteristic of the soils is a loss of strength in the presence of water. Good candidate soils show an increase in strength when treated with lime-fly ash. A reactivity test could be run in a laboratory using a remolded cylindrical sample of soil treated with a lime-fly ash glaze. The sample is then measured for soil strength by an unconfined compression test. This test, however, was not conducted because it would not give conclusive results of what would occur in the field. Before and after field CBR tests were selected to measure the increase in soil strength.

Lime-fly ash has been successfully utilized to treat soils prior to construction of runway subgrades at D/FW International Airport. Prior to constructing three apron areas in 1990, D/FW Airport engineers investigated L/FAI as a means of increasing the strength of the subgrades. Field CBR values in these areas were 2.0 to 4.0. A test section performed in the proposed areas increased the field CBR values to 25 on an average. The excellent results were due to the desiccated

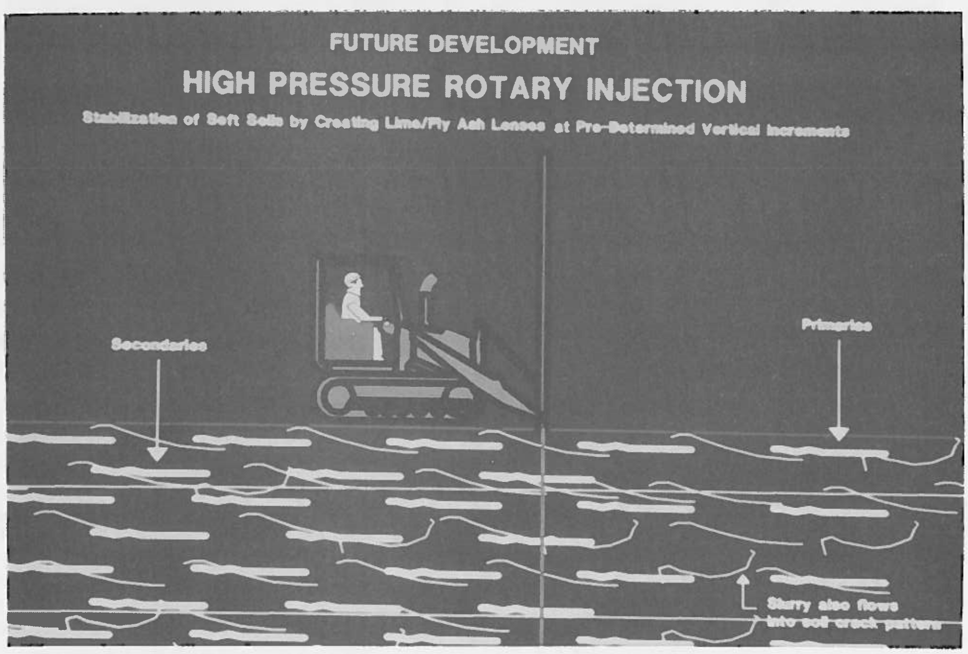


FIGURE 5 Systematic soil stabilization process.

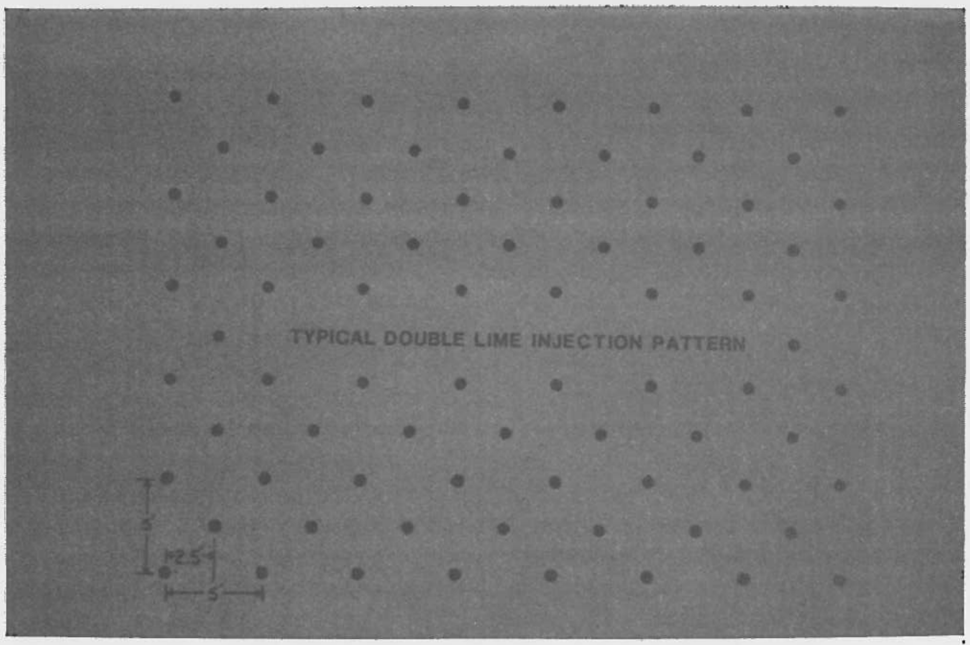


FIGURE 6 Typical injection hole pattern.

nature of the clay soils which allowed more material to be injected into the soil mass.

The slurry was usually specified to be used at one part lime to three parts fly ash. Fly ash should be a type "C." If type "F" was used then an additive such as "sure-set" was added to ensure the pozzolanic reaction occurs.

CONSTRUCTION

In order to prepare the site for the injection process, it was necessary to drill holes through the runway surface and into the subgrade. An illustration of a typical pattern of injection holes is shown in Figure 6. The holes were needed to allow the injection rods to penetrate into the soil. The holes were 2½ inches in diameter and drilled on 2½ foot centers. In all, over 30,000 holes were drilled through the runway. Included in Figure 7 is a picture of the rotary percussion drill unit used to drill the holes at an average rate of 1,000 per day.

The injection was accomplished in two phases. Each injection phase (primary and secondary) was performed on five-foot centers with the second injections spaced in between the first, resulting in the 2½ foot grid pattern. The injection rods were mounted on a mast which hydraulically forced the rods into the ground three at a time. As the rods were inserted into the soil, the slurry was pumped through under pressure at a rate of 50 psi. The rods were injected downstage in 12 to 18-inch intervals. As the slurry came to the surface through previous holes, the rods were moved down to the next interval until the total

FIGURE 7 Rotary percussion drill unit with 3100 cfm compression trailer.

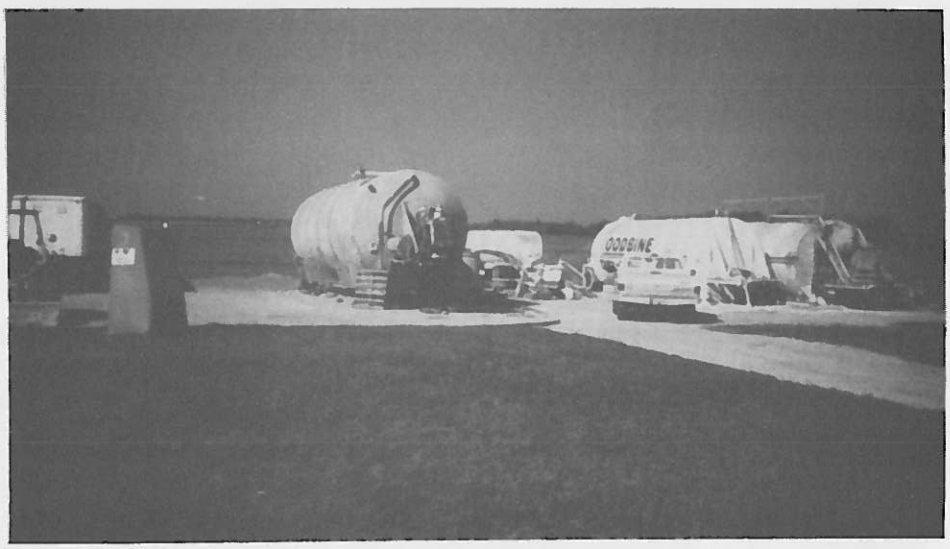


FIGURE 8 Batch plant (fly ash tank left, lime and slaking tanks right).

desired depth had been treated. The target quantity of slurry was two pounds of lime-fly ash per cubic foot of soil, but the final rate was 1.4 pounds per cubic foot.

The slurry was processed at a centrally located batch plant, Figure 8. Lime was stored as a slurry and fly ash was stored in dry bulk. The two components were mixed through a jet-valve and then pumped to a deaeration tank, Figure 9, where the specific gravity of the material was checked. From the deaeration tank the material was sent to a secondary deaeration unit at the injection site which fed each individual injection unit, Figure 10. After the injection was completed, each injection hole was filled with grout to seal the pavement.

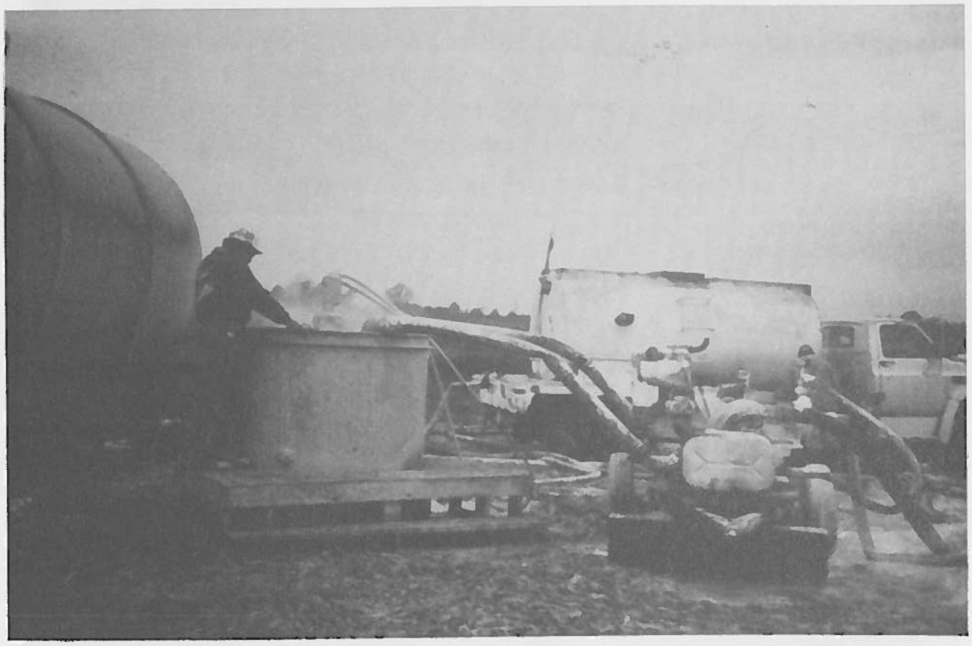
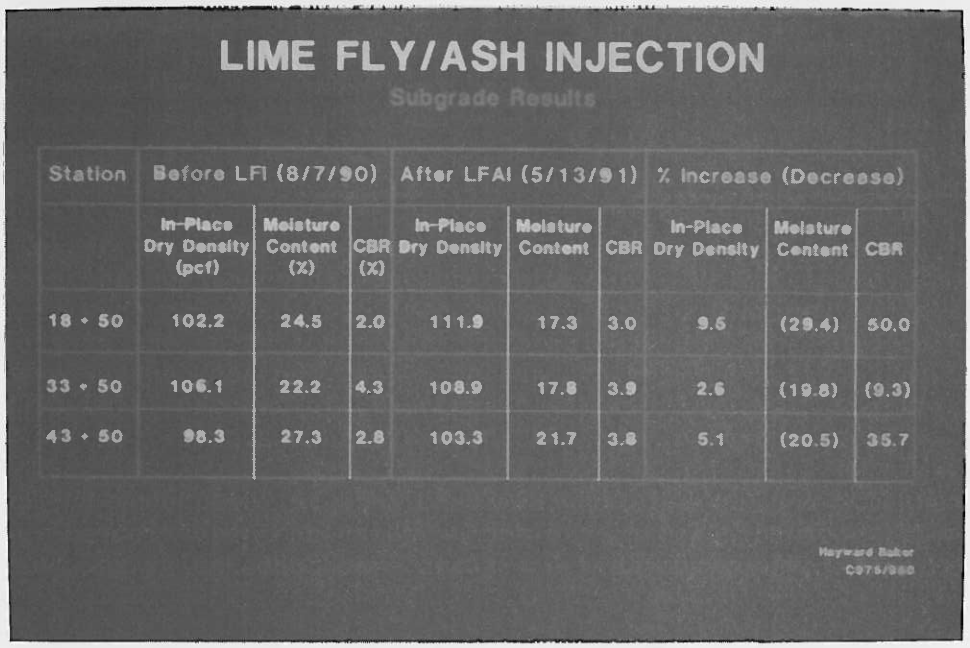


FIGURE 9 Deaeration tank (jet valve right).

FIGURE 10 Injection units.



LIME FLY/ASH INJECTION

Subgrade Results

Station	Before LFI (8/7/90)			After LFAI (5/13/91)			% Increase (Decrease)		
	In-Place Dry Density (pcf)	Moisture Content (%)	CBR (%)	In-Place Dry Density	Moisture Content	CBR	In-Place Dry Density	Moisture Content	CBR
18 + 50	102.2	24.5	2.0	111.9	17.3	3.0	9.5	(29.4)	50.0
33 + 50	106.1	22.2	4.3	108.9	17.8	3.9	2.6	(19.8)	(9.3)
43 + 50	98.3	27.3	2.8	103.3	21.7	3.8	5.1	(20.5)	35.7

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C976/980

FIGURE 11 Summary of injection results.

The cleanup operation for the runway became extensive. Lime-fly ash (L/FA) could not be left on the surface to dry. A separate crew was needed with a water truck to wash off the runway behind the injection vehicles. In some areas where L/FA set up on the runway, a water blasting unit was used to clean the pavement. Because the application was new to a large paved surface, the extent of the cleanup was not anticipated; however, future applications allowed for effective methods of removing excess slurry.

CONCLUSIONS

The process reduced the moisture content, increased soil strength and stabilized the subgrade. But because of the lack of empirical or test data, we were prepared to adjust the pavement section by as much as one inch. The target CBR value was four percent.

The three-field CBR test sites with before-L/FAI and after-L/FAI values for dry density, moisture content and CBR, quantified the relative changes in the engineering characteristics of the subgrade. The field test sites were selected from test boring logs indicating the worst soil conditions in the injection area. Favorable results were accomplished in reducing moisture content and increasing soil density. Also, CBRs reached 95 to 97 percent of the target value. Figure 11 summarizes the results of the before and after field CBR test locations.

The CBR reduction at station 33+50 could have been the result of reducing the moisture content past optimum. Organics were reported in the boring log which could have created a high optimum moisture content.

The bid amount for L/FAI was $301,000 for approximately 335,000 square feet of runway. Figure 12 illustrates the economic benefit of the L/FAI process. Cost for removing the runway pavement, undercutting four feet, replacing the soil and constructing the original strength pavement section was estimated to be approximately $770,000. Resultant savings were approximately $1.40 per square foot.

The treated soil continues to benefit from the L/FAI as the silica cementing compounds created by the L/FAI process continue to gain strength. The paving contractor also benefited because the L/FAI area required less compaction effort to achieve the target density of the asphalt overlay than in the area not injected.

Laboratory tests were conducted on similar silty clay soils to determine the optimum changes in soil characteristics. The results were exceptional as shown graphically and tabular form in Figure 13.

Although L/FAI is not a panacea for all soil stabilization projects, L/FAI does offer an economical alternative for improving the engineering characteristics of subgrade soils without removing a functionally sound pavement section.

Further research should be conducted in the areas of in situ testing methods and field methods to recover excess slurry to more accurately determine the degree of success in the field.

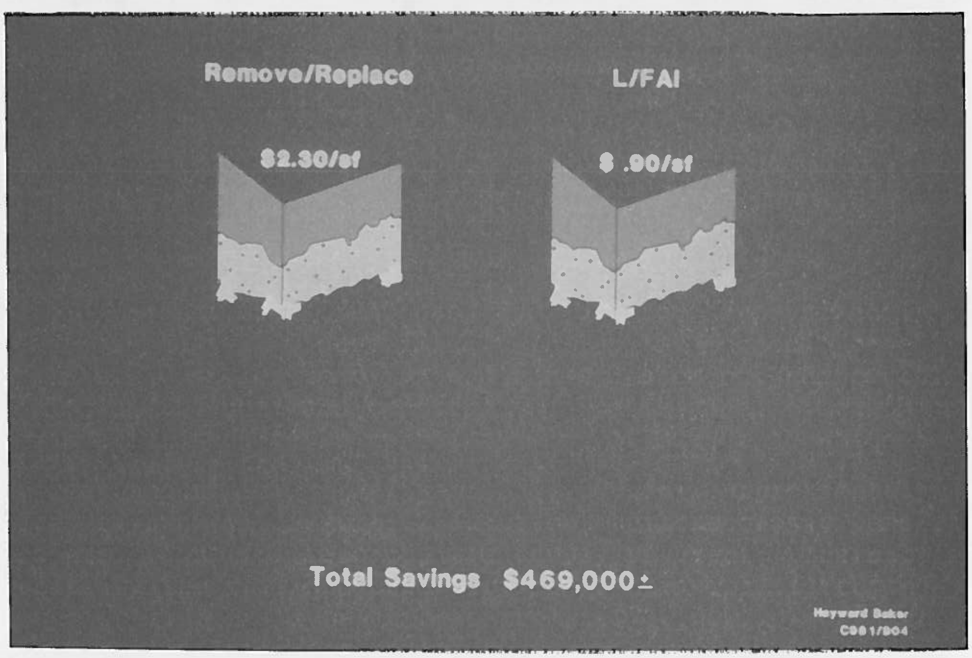

FIGURE 12 Cost benefit.

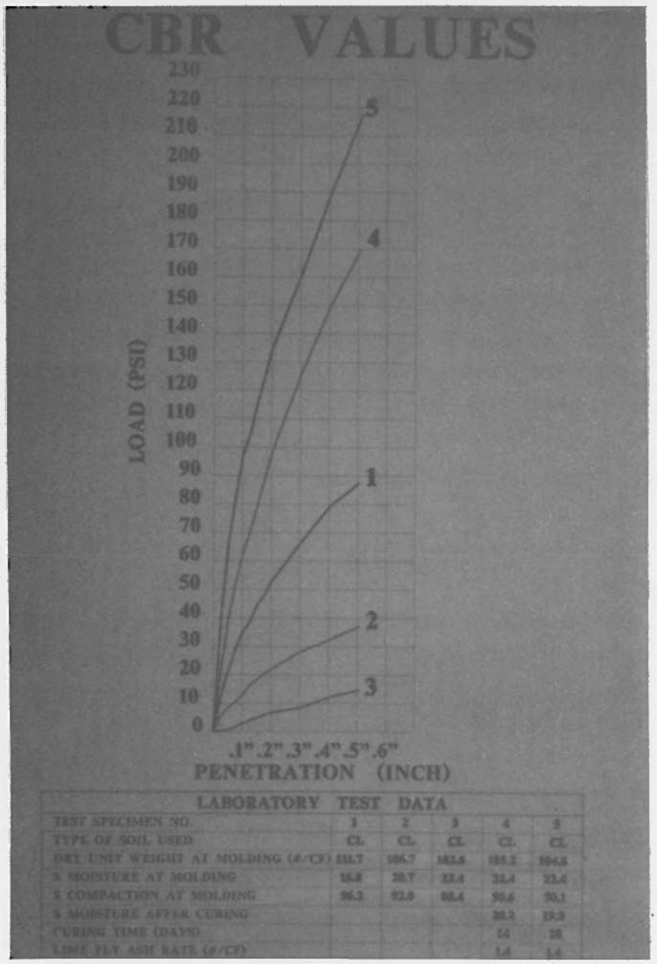

FIGURE 13 Laboratory results of L/FAI on similar silty clay soils.

ACKNOWLEDGEMENTS

The authors are pleased to acknowledge the support of the Delaware County Airport Authority, Federal Aviation Administration and Indiana Department of Transportation, the sponsors of the project; and the assistance of Otto Mueller, Materials Inspection and Testing, Inc.; Engineering and Testing Services, Inc; Sherry Laboratories and National Lime Association.

THE USE OF COMPACTION GROUTING TO REMEDIATE A RAILROAD EMBANKMENT IN A KARST ENVIRONMENT

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ABSTRACT

A 1500 feet length of mainline railroad in north Georgia is located in a sinkhole prone (i.e., karst) geologic setting. In the mid to late 1980's the frequency of sinkhole activity near and beneath the railroad increased dramatically and has affected railroad operations ever since. The railroad posted full-time flag protection in conjunction with greatly reduced rail speeds throughout this area. The cause(s) of the increased sinkhole activity included drought conditions coupled with substantial groundwater withdrawals from an adjacent quarry. In 1989, the quarry ceased operation, and rainfall and groundwater levels also returned to normal. Although the frequency of sinkholes appearing beneath the rail has decreased, small "remnant" sinkholes which were initially formed when groundwater levels were depressed continue to appear. In order to reduce the risk of drop-outs occurring which could affect railroad traffic, the owner selected a remediation system using compaction grouting techniques.

Compaction grouting has been used extensively in sinkhole repair in Florida, where the soil above the rock is typically sand. However, in this area of Georgia, the highly irregular limestone bedrock surface is typically overlain by thick deposits of soft clay. A compaction grouting system was designed to treat the upper weathered bedrock interval, the soil-bedrock interface, and the soft soil intervals above the rock.

This paper presents a case history including geologic data, a discussion of previous attempts to remediate the railroad embankment and various options available, the design and development of the selected compaction grouting system, and field results obtained during grouting.

INTRODUCTION

Throughout the history of the railroad, engineers have been confronted with the difficulties involved in maintaining rail traffic through difficult and complex geologic settings. Whether it be the repair of tunnels in soil or rock, landslides or washouts of hillside rail, or bridge supports on compressible or liquefaction-prone soils, in-situ ground modification systems have, and will continue to provide innovative and economical solutions. This paper presents a case history of the problems encountered along a mainline rail through a sinkhole-prone karst environment in northern Georgia, and the compaction grouting system used for remediation.

Prior to 1986, the subject section of rail had not experienced any surface subsidence or sinkhole problems. However, during the period from October 1986 to December 1987, the rail line experienced over 40 sinkholes beneath the track which had to be repaired. The repairs were generally temporary in nature and consisted of dumping tons of ballast into the depression to maintain rail grade. In addition, a railroad bridge in the section suffered severe damage as a result of sinkhole activity and required major repair. The frequency and severity of the sinkhole activity prompted the railroad owner to issue an order slowing all rail traffic from 60 mph to 10 mph, and a watchman was posted to patrol one section of track more than 2000 feet long, 24 hours a day.

CAUSE(S) OF INCREASED SINKHOLE ACTIVITY

The causes of the increased sinkhole activity were generally considered to be a combination of drought conditions, groundwater drawdown from a nearby quarry, and the natural sinkhole process usually associated with karst topography.

Subsurface/Geologic Conditions

The subsoils within the site area consisted primarily of alluvial materials or residuum. The alluvium typically consisted of silt, sand, gravel, or a mixture thereof. The residuum consisted of silt and clay mixtures. The subsoils ranged in thickness from as little as 15 feet to as much as 80 feet. A plan and schematic profile of the soils along the subject section of rail are included as Figures 1, 2a, and 2b.

The bedrock surface, as indicated by borings, is extremely irregular and is characterized by some deeply weathered slots (both open and soil-filled) and by pinnacles which extend above the general bedrock surface. Moreover, there are cavities in the very soluble limestone bedrock mass ranging in size from very small to very large (thicknesses ranging from less than two-tenths of an inch to greater than ten feet).

A large creek parallels and crosses beneath a portion of the problem rail area. The internal migration of the soil within the bedrock cavities and the subsequent internal erosion of the overlying alluvium has resulted in the formation of numerous sinkholes in the flood plain of the creek. Generally, these sinkholes occur when internal erosion causes a loss of soil support "from the bottom up." Cavities often form in the lower portion of the overburden due to migration of soil into the bedrock cavities below (see Figure 3). Loose deposits of alluvial soils or extremely soft zones of residual soil are the most susceptible to raveling. As this internal erosion progresses, the cavity in the soil system will become larger and larger, growing outward and upward towards the ground surface. As this phenomenon progresses, the overlying soil will form an arch that will temporarily support itself. However, as the erosion progresses to the point that

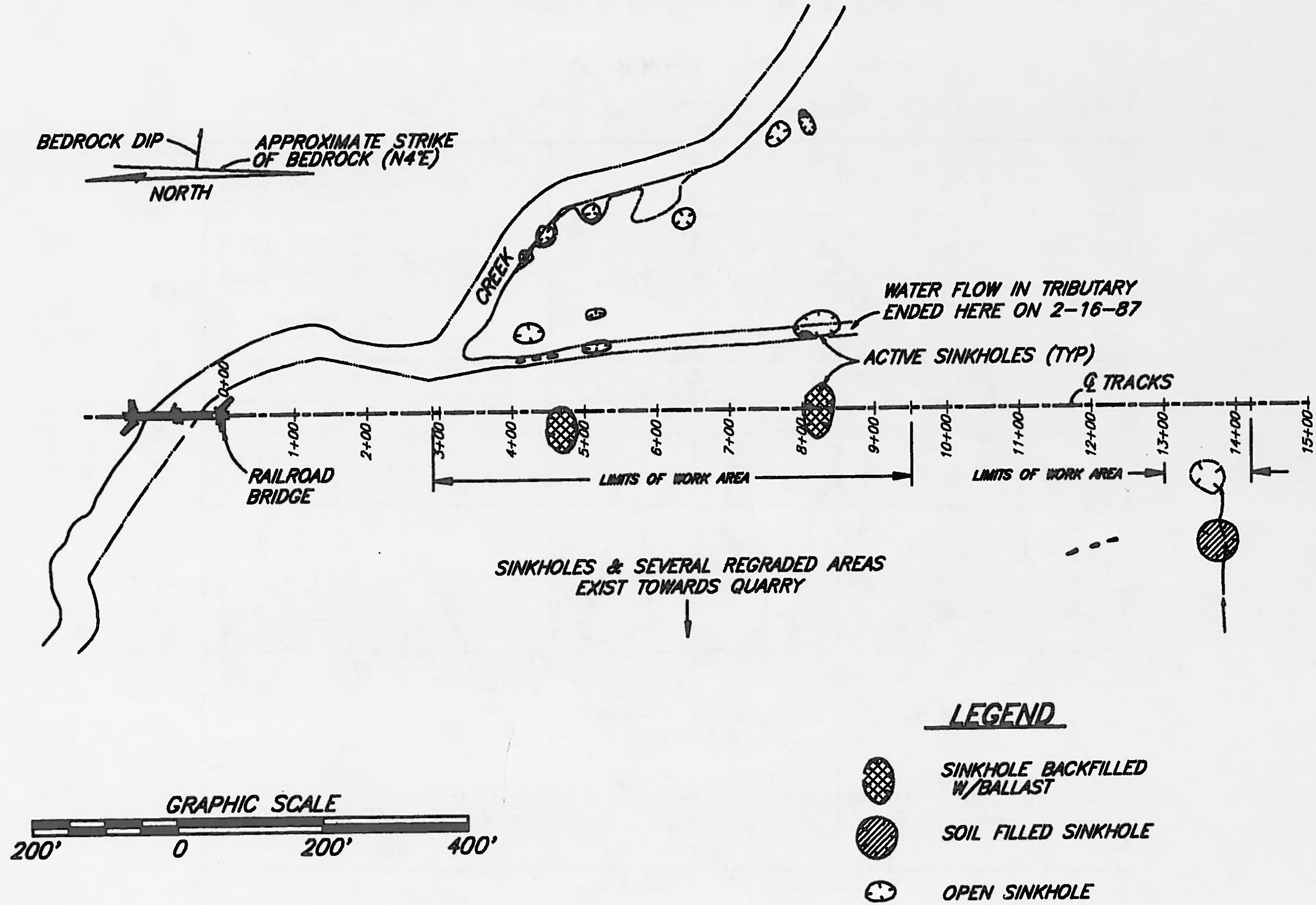


Figure 1. Plan View of Work Area

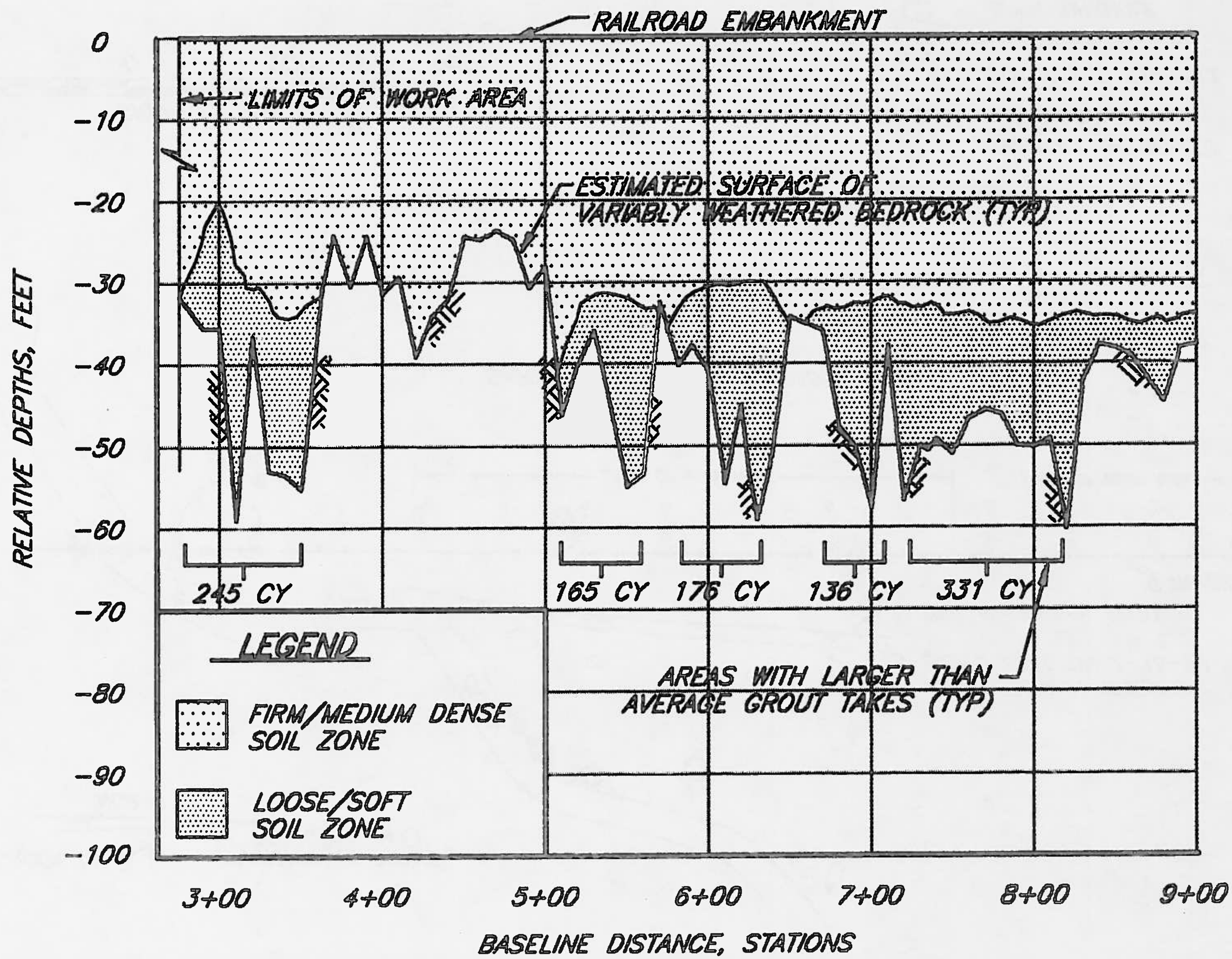


Figure 2a. Geologic Profile Along Work Area.

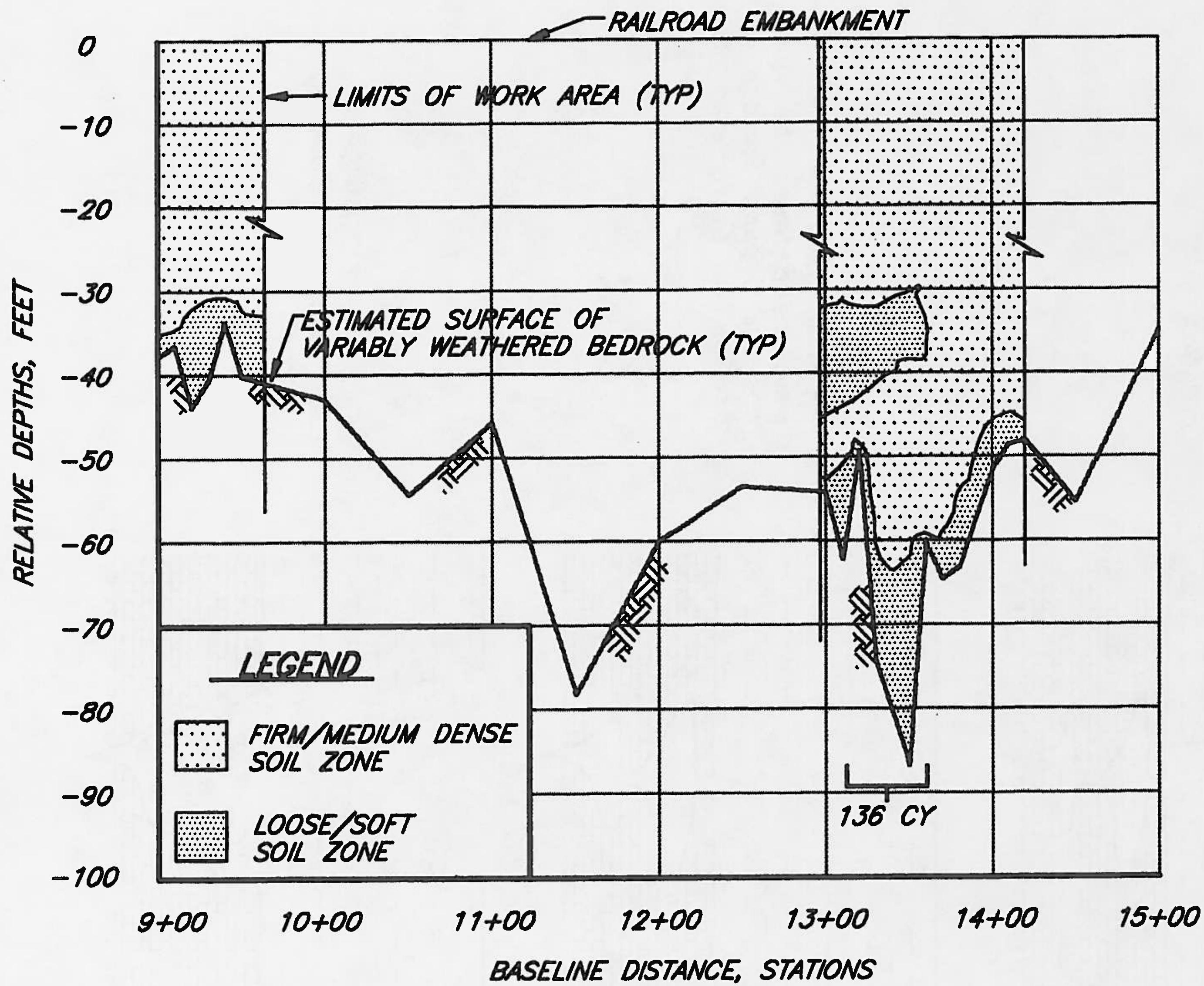
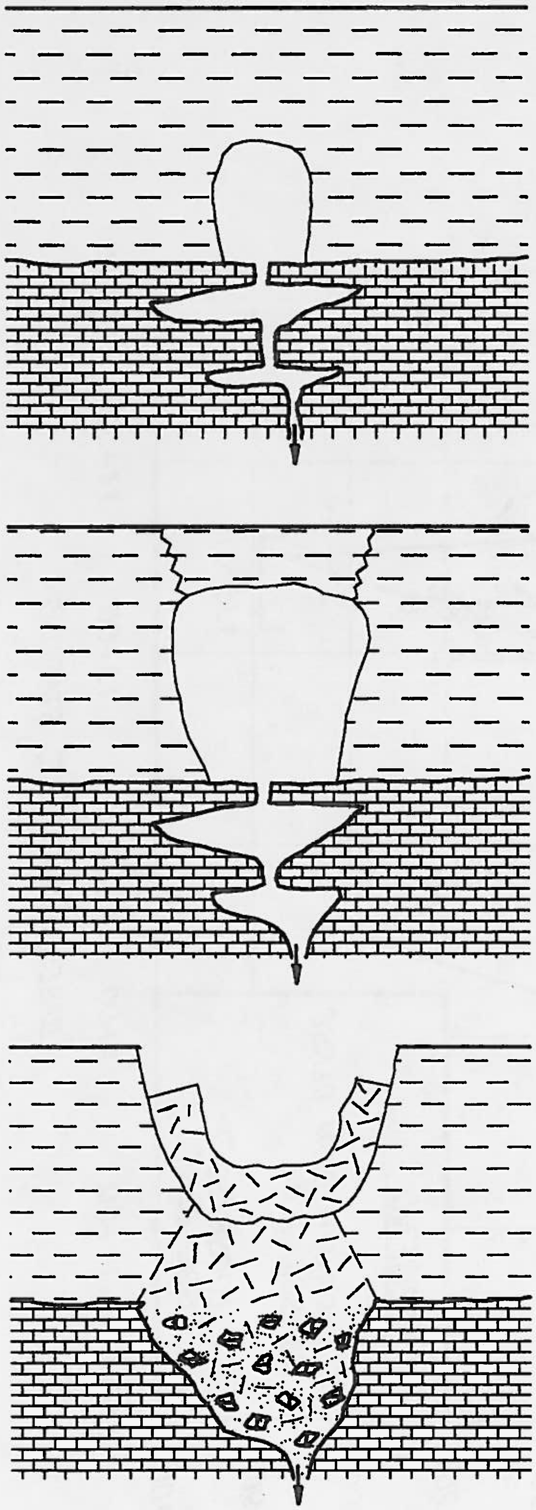


Figure 2b. Geologic Profile Along Work Area (continued).



INITIAL STAGE – Void Formation Near Dominant Fracture Inlet.

CRITICAL STAGE – Shear zone Develops Between the Void and the Ground Surface, as Solution Cavity enlarges.

SINKHOLE COLLAPSE – Soil Plug Falls, Covering Bedrock Inlet.

Figure 3 Schematic Representation of Sinkhole Formation.

the arch is no longer strong enough to support itself, a collapse will occur, thus forming the surface expression of the sinkhole. Under varying localized subsurface conditions and climatic conditions, the collapse can occur suddenly,with no surficial warning indications. Alternatively, the beginning of the sink is sometimes observed as a general but limited subsidence with corresponding tension cracks on the ground surface.

Sinkholes are generally a very slow-developing phenomenon (when measured in man's time as compared to geologic time). However, they can be greatly accelerated by severe climatic conditions and/or by man-made changes or disturbances to the subsurface. Specifically, any large introductions of water into the subsurface can greatly accelerate the internal erosion phenomenon. Also, significant groundwater withdrawal, resulting in a depressed and/or widely fluctuating piezometric surface, can also greatly accelerate the internal erosion and associated sinkhole development.

Groundwater Conditions

The sinkhole activity experienced at the site coincided with drought conditions in northern Georgia during 1986-87 and the pumping of large quantities of water from a nearby quarry.

Climatological data indicate that northwestern Georgia received as little as 40 to 50 percent of the normal precipitation during 1985 and 1986. This period was followed by several months of above average precipitation. This significant fluctuation in rainfall was considered to be one of the primary causes of the increased sinkhole activity.

Between 1986 and 1987, the nearby quarry pumped between 6 and 16 million gallons of water a day. It is believed that the further lowering of the groundwater levels by pumping during the drought triggered most of the problem sinkholes. Lowering of the groundwater table can result in a loss of support to the roofs of cavities in the bedrock that were previously filled with water.

An aquifer recharge program was conducted between the fall of 1987 and the spring of 1988 along the problem stretch of rail. This was followed by the eventual shutdown of the quarry in late 1989.

Since the shutdown of quarry operations, the frequency of sinkhole occurrence in the rail area had decreased significantly. However, problems due to remnants of sinkholes, or small sinkholes that were formed when groundwater levels were depressed, continued to occur. To eliminate the train slow order and the full-time watchman, the railroad owner desired a repair system that would significantly reduce the risk of sinkholes occurring which could affect the railroad.

ATTEMPTED SOLUTIONS/REMEDIAL REPAIR OPTIONS

During the summer of 1987, a specialty contractor injected about 1800 tons of lime/fly ash grout to treat about one mile of railroad embankment. However, sinkhole activity continued after treatment. The lime/flyash grout used had a low strength and specific gravity and was likely diluted and ran off into cracks in the bedrock and never filled the voids beneath the track.

In 1987, a survey using Ground Penetrating Radar was conducted in an attempt to delineate sinkhole-prone areas. In addition, during the summer of 1989, a microgravity survey was conducted on 6000 lineal feet of track to detect gravity lows (possible sinkhole areas). However, the results of both surveys were less than conclusive.

At this point the owner began looking for a solution that would provide a more positive reduction in risk to railroad operations. The alternatives considered included:

1. Installing an automated monitoring system to detect sinkhole occurrences;

2. Reconstruct the railroad embankment either partially or completely by excavating to within the natural ground and rebuilding a "structural" fill;

3. Remediating the railroad embankment by deep dynamic compaction; or

4. Remediating the railroad embankment using compaction grouting techniques.

From a feasibility standpoint, and with due consideration to economics and perceived effectiveness, compaction grouting was selected as the most attractive alternative because it is a positive stabilization method that can be performed while maintaining rail traffic.

COMPACTION GROUTING

Background

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

In the early 1980's, compaction grouting was first used to treat sinkholes in central Florida. Previously, sinkholes had typically been treated using a slurry grout, which consisted of a high-slump cement/water mixture. Although the technique was successful in stabilizing many sinkholes, the high-slump grout would travel along every small crack in the rock, resulting in large grout quantities.

Since grouting is generally performed on a unit rate basis, the final cost could be quite high. Occasionally, slurry grouting had to be discontinued at a pre-selected dollar amount even though the problem had not been fixed. In addition, little to no densification of overburden soils is achieved with slurry grout. Slurry grout has also been known to aggravate raveling and cause additional settlement.

Compaction grout does not travel along small cracks in the rock because it is a low slump mortar grout. Therefore, the grout remains more localized to the injection location, allowing treatment of a specific area rather than an unlimited area. In addition, the grout can be pumped into water-filled voids without becoming overly diluted as a slurry grout can. The use of compaction grouting resulted in a technically more thorough fix, more predictable grout quantities, and a lower final cost.

Initially, compaction grouting was used to treat sinkholes in granular soils after they occurred. The typical procedure consists of installing a casing down the throat of the sinkhole to the base of the cavity. Grout is then pumped under high pressure as the casing is extracted. The grouting is continued at a specific depth until a specific volume, injection pressure, or surface heave is achieved. Within the cavity, any voids are filled, and any soil which may have collapsed into the void is compacted. After treatment of the cavity, grouting is continued through the overburden soils to compact soils loosened as a result of the dropout. Additional pipes are then installed around the perimeter of the sinkhole to complete the densification of the overburden soils. In addition, if overlying structures settled as a result of the sinkhole, they can be carefully leveled by controlled heave during grouting.

In addition to treating sinkholes after they occur, numerous sites in sinkhole-prone areas have been treated as a preventative measure to reduce the risk of sinkhole occurrence. In this application, the casing is generally installed on a grid pattern. The pipes are installed to the top of rock or to penetrate the upper bedrock surface. If a void is encountered in the upper rock surface, the void is generally filled. The casing is then extracted to the soil/bedrock interface where grouting is performed until either a predetermined grout volume, pressure, or surface heave is achieved. As the grout is pumped at this interface, the grout travels along the interface, forming a grout barrier which will enhance the ability of the soils to arch over a potential collapse in the lower bedrock. Alternately, the grout under pressure may trigger an impending collapse that could have eventually resulted in surface subsidence; the void can subsequently be filled with grout. In either case, the compaction grouting is then continued as the casing is extracted through the overburden soils to compact loosened zones in the soil.

Grouting Program for the North Georgia Site

For this project, compaction grouting was performed as a preventative measure for an 850 linear feet treatment area of railroad embankment. A drill rig began installing the casing at one end of the treatment area. Since the rock surface dipped to the east, the holes were angle-drilled from the east edge of the embankment to best intercept the surface of the limestone directly beneath the track centerline (see Figure 4). Casing was installed and the holes were advanced to penetrate approximately five feet into the upper bedrock surface. The drilling process was monitored and logged to locate soft/loose soil zones and/or void areas.

The grouting program was split into primary holes (on 20-foot centers), secondary holes (splitting the primary holes to 10-foot centers), and tertiary holes (on 5-foot centers) at select locations. The grouting procedures were based on the results of the drilling information, anticipated return injection pressures, and volume of grout injected. In general, if high return pressures were not observed, the primary hole grouting consisted of injecting 3 cy/1 foot lift of casing for the first three feet above rock, followed by 2 cy/ft through the loose/soft soil zones encountered during drilling. Assuming a uniform geometry, this would result in a maximum 10.2-foot- to 8.3-foot-diameter mass at each primary hole injection location. The secondary hole grouting consisted of injecting a lesser volume of grout (3 cy for the first foot above rock, 1 cy for the next two feet, and 0.5 cy/ft for loose/soft soil zones) and was also intended to help evaluate the effectiveness of the primary hole grouting by monitoring for surface ground rise and grout backpressure (indications of "tight" soils) while performing the grouting. Finally, tertiary holes were added where secondary holes did not encounter tight conditions.

As the casing was extracted, a higher slump sand/cement grout was initially pumped under pressure to seek rock fractures and soft soil zones. As rock voids or soft soil zones were encountered, the grout was modified to a lower slump compaction grout.

Results

During performance of the compaction grouting, nearly all of the injections encountered loose soils immediately overlying the bedrock surface. As a result, the 3 cy per linear foot was typically the refusal criteria first met in the three-foot zone overlying the bedrock. There were several indications that the grouting program effectively densified the soil and filled voids beneath the railroad embankment. While grouting the majority of the secondary grout holes, ground surface heave was observed with the drill casing still 20 feet or more in the ground. In addition, while removing pieces of casing, backpressure that forced grout back out of the remaining casing was observed. Furthermore, traces of grout, and in some cases, solid grout was encountered while drilling some of the secondary holes. In summary, 88 holes were drilled and 1734 cubic yards of grout were injected beneath the railroad embankment.

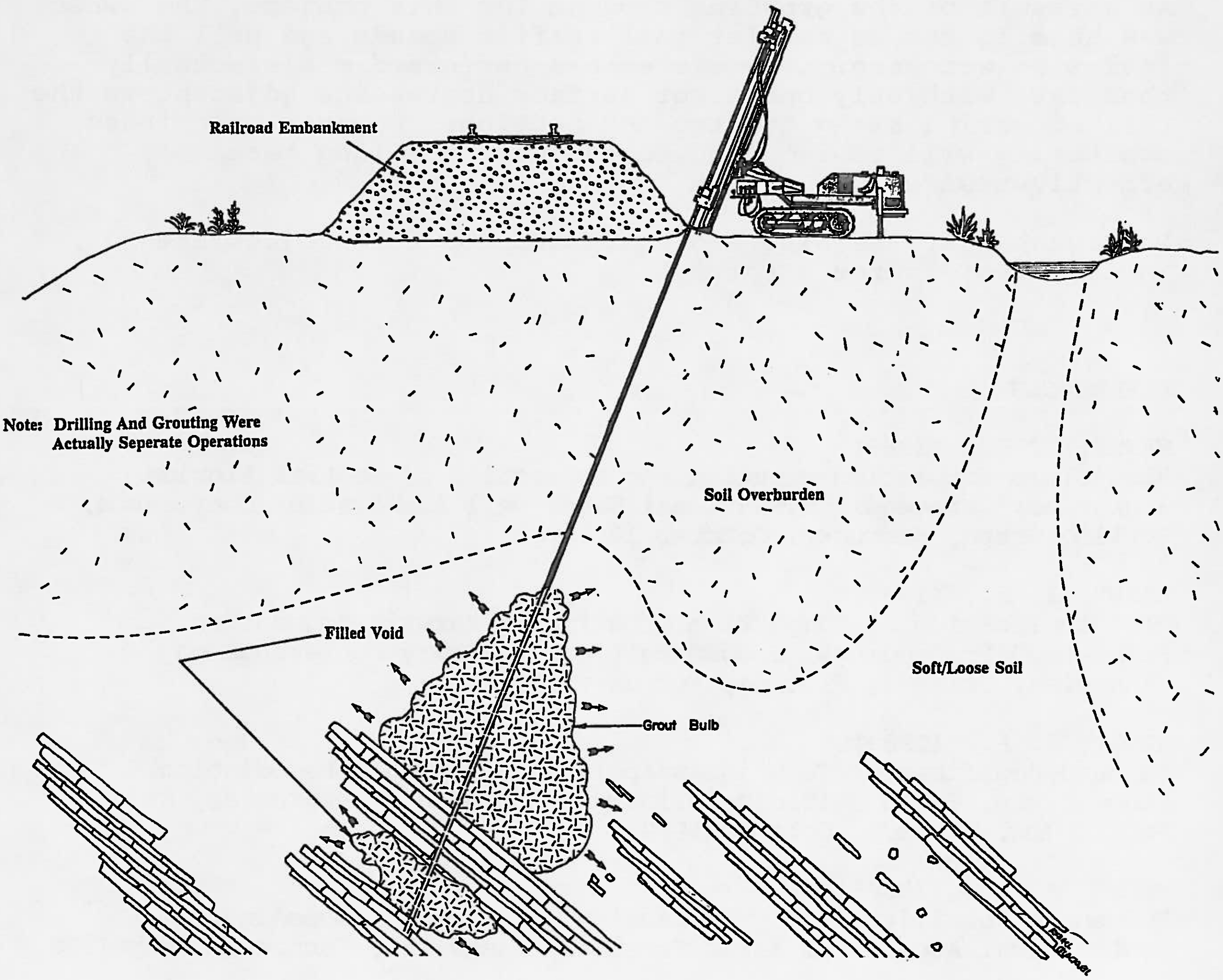


Figure 4. Schematic Cross-Section of Compaction Grouting Remediation System.

CONCLUSIONS

Compaction grouting can be an effective and economical method for sinkhole repair and/or prevention of surface subsidence due to sinkholes. The grouting program procedures can be established to evaluate the effectiveness of the program in progress and adjustments can be made as necessary to match variable subsurface conditions.

As a result of the grouting program for this project, the owner was able to resume regular rail traffic speeds and pull the full-time watchman. The system has performed satisfactorily thus far, with only one minor surface depression adjacent to the rail occurring along the treated section. However, continued monitoring will be required to evaluate the long term effectiveness of the system.

Photographs of the site and equipment for the project are included as Figures 5 through 8.

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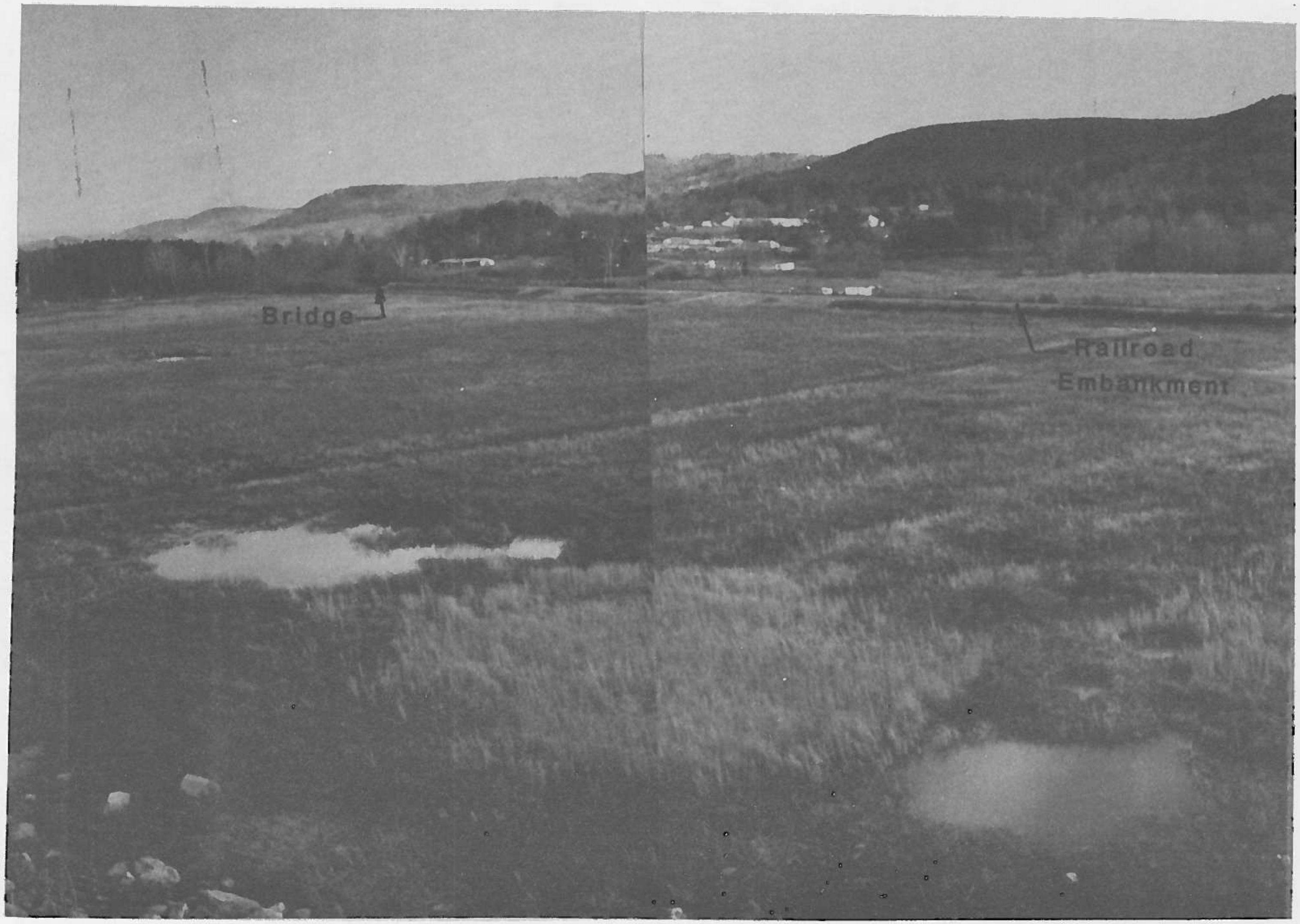

Figure 5. Photograph of Work Area (note numerous sinkholes in foreground).

Figure 6. Photograph Facing Railroad South Showing Angle-Drilling Operations.

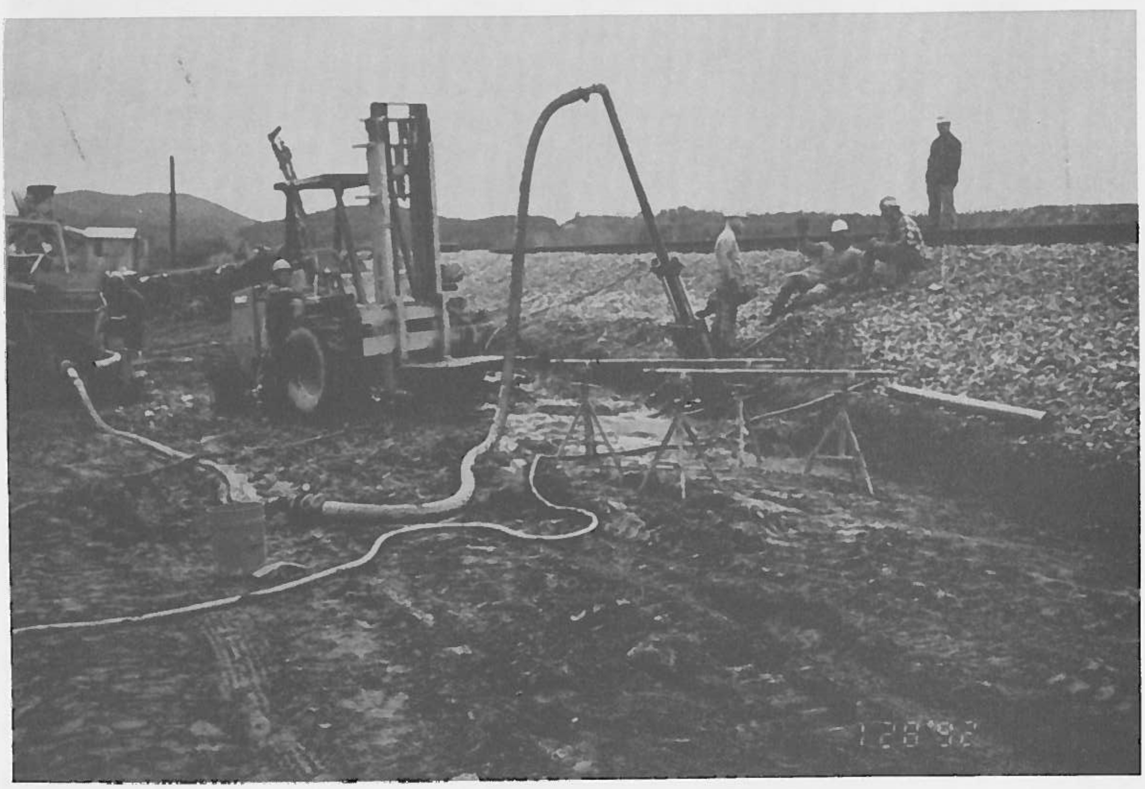

Figure 7. Photograph Showing Grouting Operations.

APPENDIX

Past Proceedings of Ohio River Valley Soils Seminars

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, CONSTRUCTION, AND PERFORMANCE, October 14, 1978, 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 PRACTICE, October 8, 1982, Lexington, Kentucky

ORVSS XIV: FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION, 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, Lexington, Kentucky

ORVSS XX: CONSTRUCTION IN AND ON ROCK, October 27, 1989, Louisville, Kentucky

ORVSS XXI: ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING, October 26, 1990, Fort Mitchell, Kentucky

ORVSS XXII: DESIGN AND CONSTRUCTION WITH GEOSYNTHETICS, October 18, 1991, Lexington, Kentucky