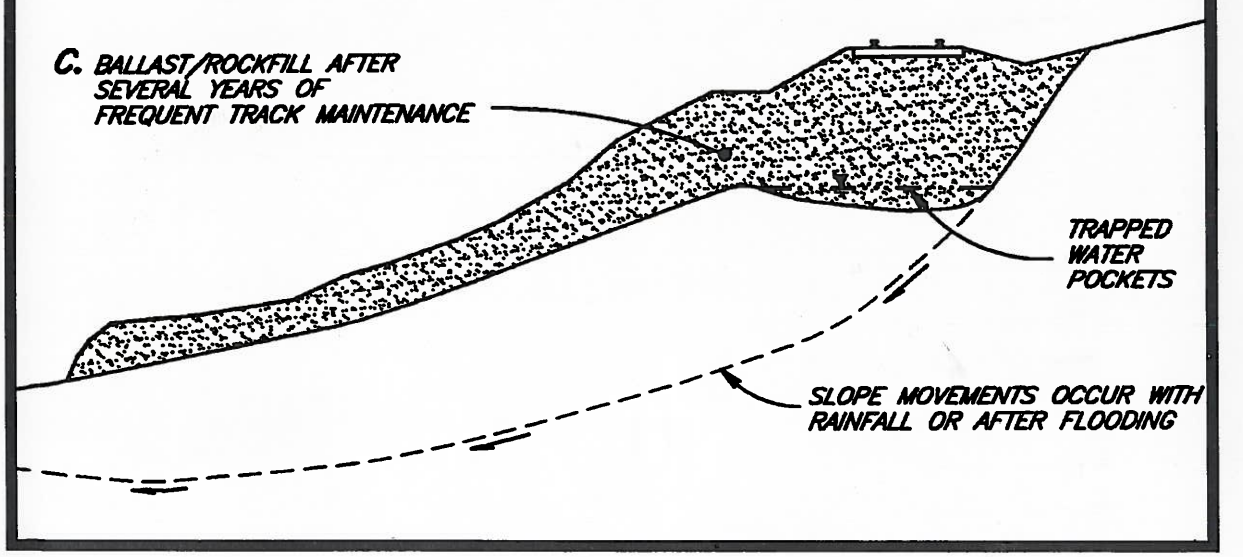
OHIO RIVER VALLEY SOILS SEMINAR XXIX

PROBLEMATIC GEOTECHNICAL MATERIALS



Louisville, Kentucky
October 16, 1998

ORVSS XXIX
PROBLEMATIC GEOTECHNICAL MATERIALS

8:50 am	**Welcome**
9:00 am	**Seismic Renovation of the South Carolina Statehouse**, by Larry Rayburn, Terry Tucker, and Bill Heckman, Richard Goettle, Inc.
9:45 am	**Railroad Subgrade and Slope Repair Along the Ohio River**, by Gary T. Brill, Geo/Environmental Associates, Inc.; Kenneth E. Darnell, Darnell Associates, LLC; and Charles E. Gullakson, CSX Transportation, Inc.
10:15 am	**Break**
10:45 am	**Hickman Bluff Slope Stabilization Project**, by Thomas N. Anderson, Schnabel Foundation Company
11:15 am	**Refined Field Methods for Identifying, Describing, and Testing Shale and Weak Rock**, by Paul M. Santi, Department of Geological and Petroleum Engineering, University of Missouri-Rolla
11:45 am	**Unconventional Earthwork Construction Minimizes Undercutting** by Jon H. Gould, Raytheon Engineers & Constructors
12:15 pm	**Lunch**
1:30 pm	**Papa John's Cardinal Stadium: Foundation Construction Over 100 Years of Railroad Repair Shop Debris**, by Timothy M. O'Leary and Nicholas G. Schmitt, Law Engineering and Environmental Services, Inc.
2:00 pm	**Railroad Tunnel Construction in the Kope Formation**, by Mark J. Doller and Bernard H. Voor III, Ogden Environmental & Energy Services Company, Inc.
2:30 pm	**Landsilide Repair at Mission Viejo Mall,** by John R. Wolosick, Hayward Baker, Inc.; Paul B. Gronek, DBM Contractors, Inc.; and Seth L. Pearlman, Nicholson Construction Company
3:00 pm	**Break**
3:30 pm	**Design and Performance of Foundations in Expansive Shale**, by James J. Belgeri and Timothy C. Siegel, S&ME, Inc.
4:00 pm	**Problematic Soils Improved by Compaction Grouting** by John W. Reed and Dennis L. Boley, Denver Grouting Services
4:30 pm	**Soil Displacement Piles in Coastal Deposits: A Case Study**, by Michael D. Smith and Daniel C. Brahana, Law Engineering and Environmental Services, Inc.
5:00 pm	**Social Hour**

Proceedings of the Twenty-Ninth
Ohio River Valley Soils Seminar

PROBLEMATIC GEOTECHNICAL MATERIALS

October 16, 1998
Club Hotel by Doubletree
Louisville, Kentucky

Sponsored by

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SEISMIC RENOVATION

SOUTH CAROLINA STATE HOUSE

COLUMBIA, SOUTH CAROLINA

RICHARD GOETTLE, INC.

Larry Rayburn, Terry Tucker, and Bill Heckman

Introduction

The history of the South Carolina State House is an intriguing story that parallels much of the rich history of the state. During the post-revolutionary era, the capitol was moved from Charleston to a more central location in Columbia. In 1849, a desire to protect the state's records led to the decision to construct a new building to house them. Originally designed to accommodate expansion, the legislature decided to proceed with a larger plan. However, by 1854 it became clear that there were structural faults in the uncompleted building. At this point the Commissioners for the State House called in a new consulting architect, John Rudolph Niernsee, who was born in Vienna, and studied architecture and engineering in Prague. On the basis of his recommendations, the legislature ordered the building razed.

The design of the State House was in the Roman Corinthian style (See Figure 1). Essentially, the State House is the product of two building periods, 1855-1863, 1883-1902 and one major period of alteration, 1959-1969. In the first period, the exterior walls and part of the ground floor were completed; in the second, the interiors were completed and porticos and dome added. Finally, in the mid-20th century, major alterations removed or covered much of the 19th century work.

The building was supported by 7 foot wide granite walls bearing directly on the soil. The exterior walls are 10 foot wide at their bases, while some of the interior walls are only 5 foot wide. The granite was discontinued at ground level and brick was used to complete the walls to the roof level. The exterior wall have a skin of finish granite. Most of the first floor is constructed with brick arches. The upper floors are framed with wood or iron joist. The steel framed and masonry exterior dome is supported by two 6 foot deep riveted box girders that span from the south corridor wall to the north exterior wall. The interior dome is very light, consisting of sheet metal frame, glass panels, and paper. Figure 2 gives the building dimension and the layout of the foundation walls.

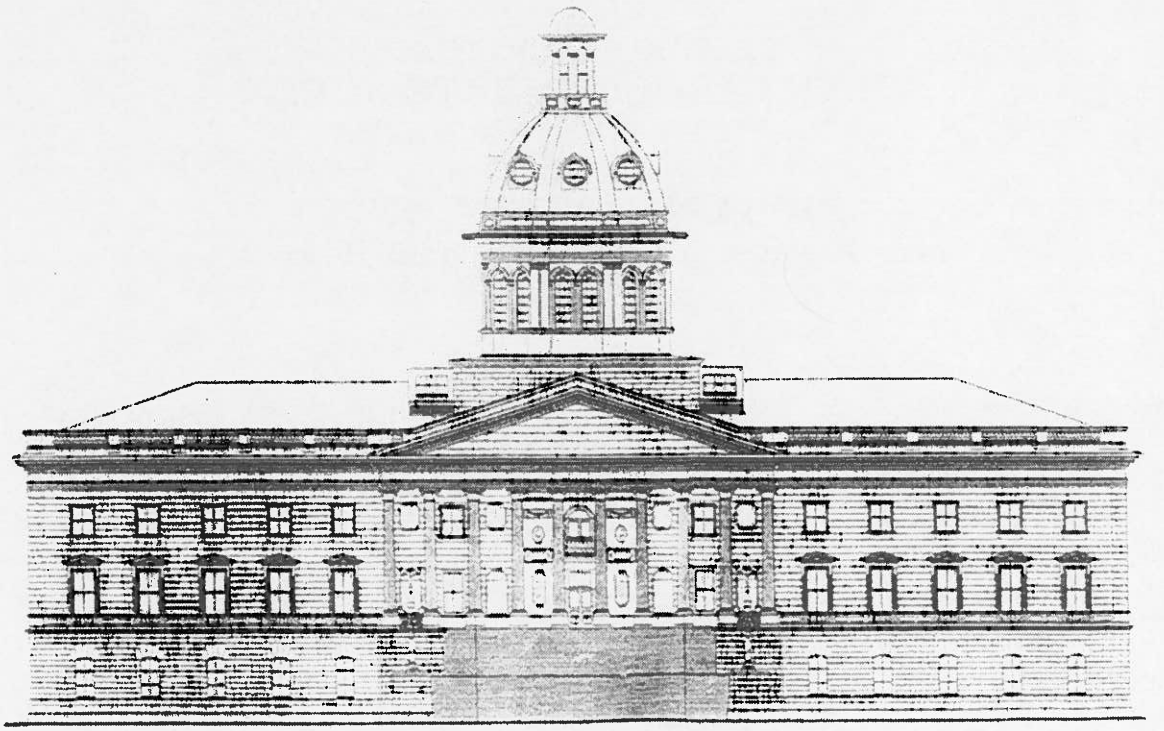


Figure 1 – North Elevation

In February of 1865, the 13th Iowa Regiment of Sherman's 17th Corps attacked Columbia. However, the bombardment and heat from the burning of the old State House did minor damage. To this day, six gold stars mark the locations where cannonballs struck the west front of the building.

Columbia, SC is located on the western edge of the Coastal Plain Province. The near surface soils at the State House site are made up of Tuscaloosa sands and clays of the late Cretaceous age. The soils above the bottom of the new footings (Elev. 300 U.S.G.S) to the existing ground surface (Elev. 320) consist of medium dense brick red silty fine sand and brick red clayey fine sand. The new footings bear on a dense yellow clayey silty fine sand. (The bottom of the existing granite walls are at elevation 310.)

Ground water is located approximately 50 feet below the ground surface. Natural moisture content varies from 8% at the foundation level to 15% at the surface. Moisture contents vary greatly under the building down to the yellow fine sand layer. Under the center of the building the top 5 feet or so is very dry and hard. In the boiler rooms on the east side of the building the soils are very moist and soft down to the yellow fine sand layer. This was caused by many years of leaking boilers and pipes.

The bearing capacity of the brick red clayey and silly fine sand is very sensitive to the moisture content. The exterior wall underpinning, founded in this material, required extensive undercutting and backfilling with lean concrete fill. The exterior walls of the boiler room started settling as the underpinning operation approached this area. Extensive compaction grouting under these walls was required just to slow the rate of settlement during the underpinning and excavation for the new footings.

In the early 1990's, the State of South Carolina retained the services of Stevens & Wilkinson, an architectural & engineering firm located in Columbia, to design a major renovation and retrofit effort. Owing to the geological history of the region, (1886 Charleston Mb=6.8 Earthquake), and with the understanding that 9 major tectonic features have been identified in the area surrounding Columbia, Stevens & Wilkinson was charged with developing a seismic protection plan to ensure that the history of the State House will be preserved for centuries to come.

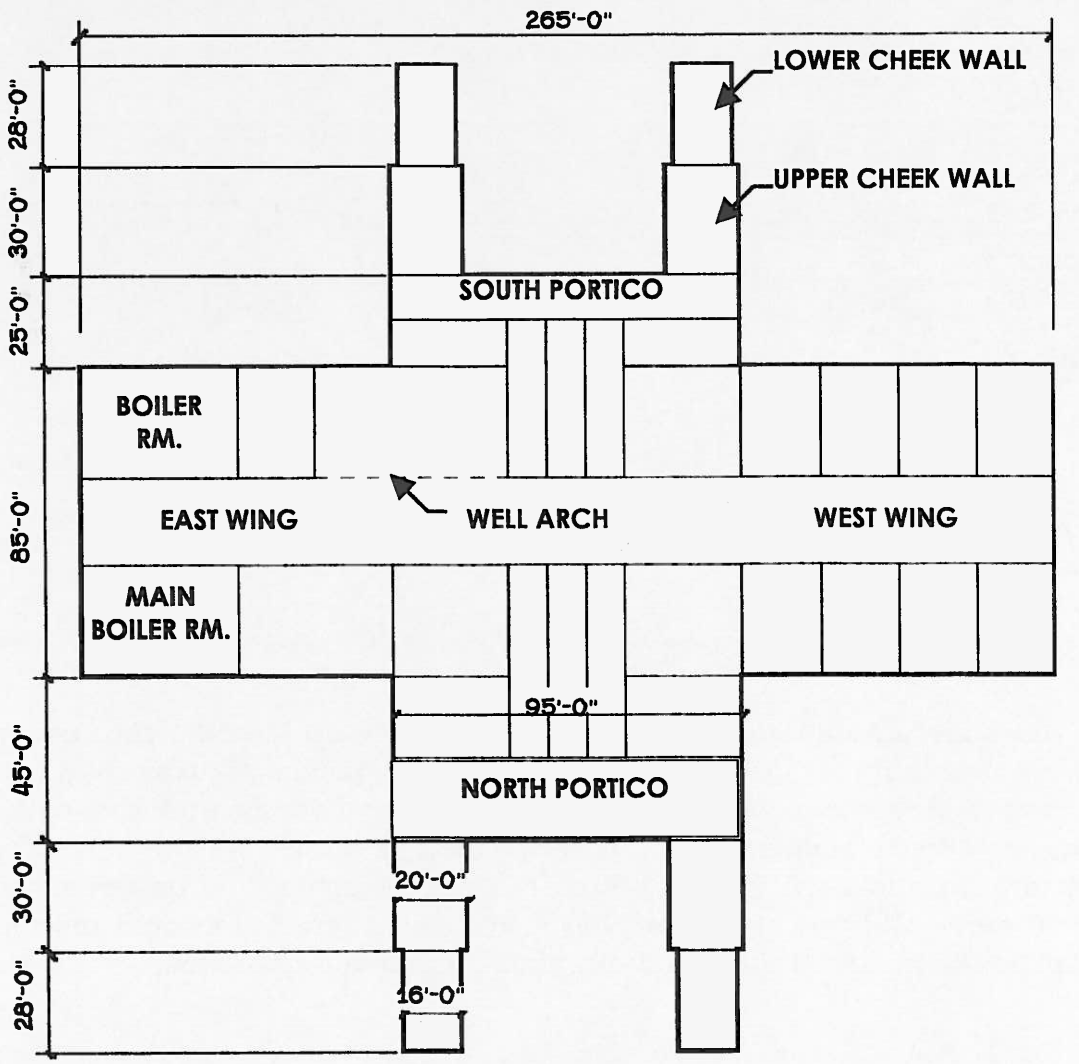


Figure 2 – Schematic Plan of the State House Foundation

To develop the seismic design, Stevens & Wilkinson engaged the services of Structural Affiliates International, Inc. (SAI), of Nashville, Tennessee. The principal features of the seismic isolation work are; engineering & monitoring, excavation, foundation underpinning, wall stiffening and doweling to underpinning, transfer beams, shoring & bracing, new foundation elements, adjustable loading & jacking system, base isolators, and the construction of an isolation or mote wall surrounding the structure (See Figure 3). Richard Goettle, Inc. (RGI) was responsible for the foregoing work with the exception of concrete placement. The seismic effort was planned and sequenced as follows:

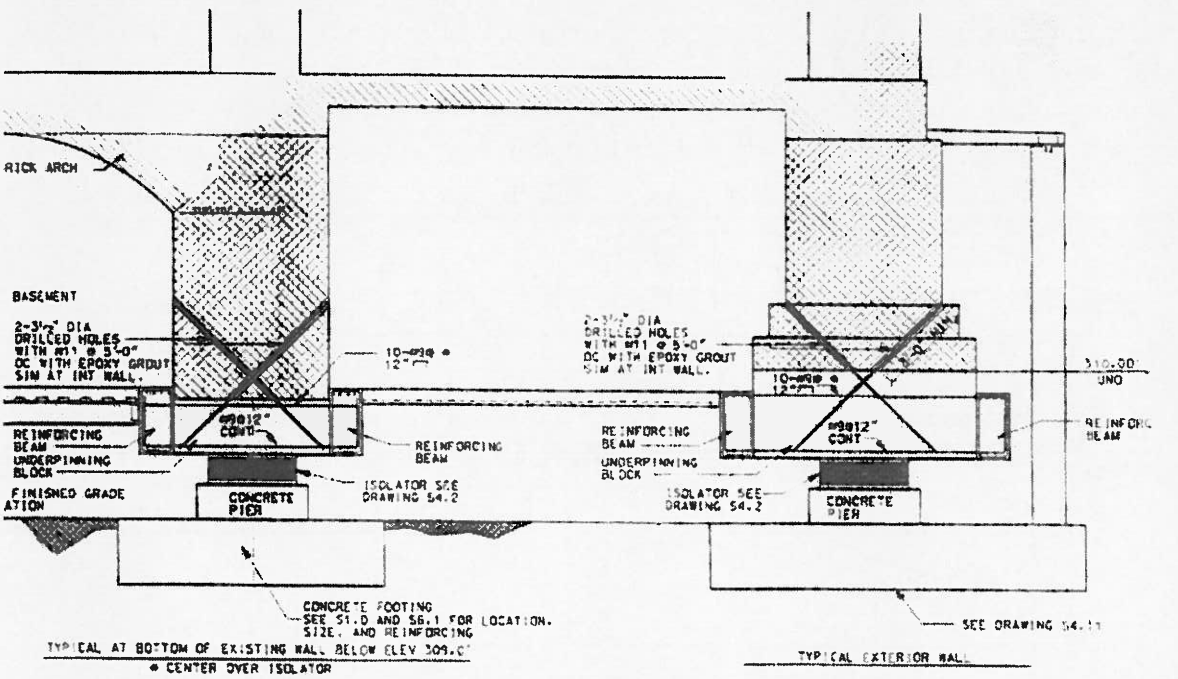


Figure 3 – Typical Section

ENGINEERING & MONITORING

As the underpinning and seismic contractor, Richard Goettle, Inc. assumed the responsibility for design, construction, and sequence of excavation for all shoring and bracing capable of supporting excavations and construction loads. Notwithstanding the conceptual design which was a part of the contract specifications, it was necessary to develop the final design scheme, which was verified on an ongoing basis, utilizing a series of model runs which depicted the results through a finite element analysis approach.

During the underpinning phase, the basic approach to underpinning was to excavate 5' wide segments on 20' centers to keep construction loads to a maximum of 8250 psf. During the initial analysis, hot spots were identified beneath new door openings which would be cut through granite in the basement walls. This condition necessitated a re-sequencing of the underpinning blocks to maintain construction loads within the contract limits.

Similarly, the same method of analysis was used to predicate the optimum sequence of new footing construction while maintaining the structural integrity of the building as a primary consideration, and schedule as a secondary concern. Prior to construction it was predicted that approximately 1.5 inches of settlement could be expected, with the majority of the movement occurring during the initial phase of the work when the bearing pressure was the highest. Through the underpinning phase, average movement of about 1 inch has been recorded. Careful consideration was given to the angle of distortion, to control the opening of existing cracks and to minimize the size and location of construction related cracks.

In addition to the extensive circuit of optical monitoring which was performed by RGI, the owner retained the services of Atlanta Testing & Engineering (AT&E), to implement an independent electronic monitoring program. The principal feature of AT&E's monitoring was a network of rod settlement points connected to transducers, which were wired to a central control panel. Readings were recorded 4 times daily, and weekly the information was down loaded and a comparison was made with the optical data. This information was plotted and presented in a weekly monitoring meeting. Interestingly, the optical and electronic readings reconciled on a consistent basis.

EXCAVATION

The initial phase of excavation consisted of lowering the grade around the periphery of the building approximately 10'. Additionally, the grade within the building itself had to be lowered approximately 2', to expose the bottom of existing footings, prior to commencing with the underpinning work. The bottom of the exterior wall footings were at elevation 310, while the interior footings varied between elevation 307.5 and 310. The existing basement area consisted of a series of rooms with catacombs or adits extending in a maze fashion. As a result of close quarters within the basement area, and owing to the fact that new 5' wide door openings were to be cut through the basement walls, it was initially envisioned that much of the 17,000 cubic yards of excavation would have to be performed by hand.

Factoring schedule constraints, it was determined that 5 each, 36" diameter horizontal bores extending up to 300' in length, would be advanced under the building from bore pits located at the east end of the building. A series of vertical shafts strategically located throughout the basement, were connected to the horizontal bores. Small mini-excavators and skid steer loaders assisted small boring machines which augured the soil out from beneath the footings. In general, the underpinning blocks were 5' wide, 4' high, and extended from 6' to 10' beneath the footings. As the silty sand was removed from the beneath the footings into the rooms, it was dumped into the vertical shafts. A continuous auger extended the length of the horizontal bores was rotated by the boring machine located in the bore pit. As the soil was pulled back into the bore pit, a large hydraulic excavator removed the material from the exterior bore pit into trucks and discarded off site. Approximately 12,000 cubic yards was removed through the horizontal bores by the time the new foundations were installed at elevation 300.

UNDERPINNING BLOCKS

Within a day after the material was excavated from an underpinning block, preparation for concrete placement began with cleaning the underside of the footing and the application of a bonding agent. The exact location of the blocks, along with the elevation of the bottom of the block, as well as the alignment, was very critical since the blocks and the transfer beams were heavily reinforced. Due to the size of the underpinning blocks and given design considerations, lap splices were not allowed which required approximately 10,000 mechanical connections. In the absence of adhering to critical digging and pouring tolerances, the steel connections would not have aligned. In underpinning blocks which would support a base isolator, additional reinforcing was required to extend through the forming system into the transfer beam. At base isolator locations, a template consisting of a 1/4" plate with 8 each box sleeves for connection to the base isolator, was cast into the underpinning block. To ensure positive contact between the bottom of the granite footing and the underpinning block, pre-positioned sleeve port grout tubes were placed and subsequently grouted with a pre-blended non-shrink grout.

DIAGONAL DOWELS

On 5' spacing from alternate sides of the granite walls, 2.5" diameter holes were drilled on an angle through the bottom two courses of granite. A #11 bar extending into the underpinning block was grouted with a non-shrink cementitious grout. The dowel provided a number of benefits. From a construction standpoint, it knits the granite blocks together, allowing for a safer environment while working in the underpinning blocks. From an engineering perspective, it provided a shear connection between the granite walls and the blocks.

TRANSFER BEAMS

The transfer beams were cast after a section of wall has been underpinned to elevation 306. The beams are 1.5' wide and 3' in depth and contain 4 each #9 bars top and bottom. The transfer beams perform a dual purpose; during the excavation process from elevation 306 to 300, they transfer load beyond the 20' long excavated area. Secondly, the beams are an integral part of the final design, allowing the load to transfer between base isolators.

SHORING & BRACING

Once the underpinning blocks were complete and the transfer beams poured beneath approximately 25% of the walls, excavation proceed in as many as 7 concurrent locations. The limit of excavation was 20', and during this process, the building was shored adjacent to base isolator locations. The maximum load at any given isolator was 1620 kips. To support this load, a series of jacks connected to a common manifold supported on both structural sections and heavy wall pipe, was used to transfer load from the underside of the underpinning blocks to the top of the new footings. Once the base isolators were installed and the isolator pedestal cured, the load was transferred to the isolators and the temporary shoring removed. As many as 20 bents of shoring were required during this phase of the work.

Monitoring during this operation was again performed optically, with a cross-check to the rod settlement points connected to transducers and wired to a common control panel.

FOUNDATION ELEMENTS

Once the underpinning was completed in a given area and an engineering evaluation on load transfer was performed, the excavation proceeded from elevation 306 to 300, the bottom of the new footing. The footings are 12' to 17' wide and 3' thick, and the contributory area at any given base isolator is a minimum of 12'. The maximum distance for excavation beneath the underpinning blocks was 20'. The reinforcing steel in the footings is coupled, since the splice distance required exceeds the distance between the base isolators.

ADJUSTABLE LOADING SYSTEM FOR NEW FOUNDATIONS

Once the lead core rubber isolators were installed by connecting the top plate with through-bolts into the box assembly, which are cast in the underpinning blocks, a 1.25" plate jack was positioned between the top of the isolator and the bottom of the underpinning block. At this point, hydraulic oil was pumped into the plate jack. The system was adjustable, and capable of transferring the building structure gravity loads from elevation 306, (bottom of underpinning blocks) to elevation 303, (top of foundation). A principal feature of the system was the ability to allow for adjustments of the loads during the initial settlement of the foundation. The plate jack was designed as a permanent non-removable component of the base isolator system. The jacking system was capable of 1" of travel which will allow for differential movement to be realigned. The plate jacks were under fluid until the building stabilized, at which time the hydraulic oil was replaced with epoxy, allowing the jack to be permanently secured as part of the structure.

LEAD RUBBER SEISMIC ISOLATION SYSTEM

The base isolators were designed and manufactured by Dynamic Isolation Systems, Inc. (DIS) of Lafayette, California. The DIS Lead Isolation System consists of a total of 127 production isolators of two distinct types. There are 117 type "A" isolators, and 10 type "B". Type "A" isolators are 27.5 inches in diameter and 8.065 inches tall; type "B" isolators are 29-.5 inches in diameter and 8.875 inches tall. Both isolator types have a lead core 5.25 inches in diameter. The isolators weigh 1100 pounds and 1300 pounds respectively.

Although there are two distinct isolator types from a physical size standpoint, both types have essentially identical horizontal stiffness and energy dissipation properties. All isolators are designed for a maximum displacement of 7.5 inches in any given direction. All bearings were delivered to the job site pre-loaded with a retaining device to 200 kips.

RETAINING (MOAT) WALL

In order to isolate the building from the adjacent ground, a 16' high retaining wall has been constructed for a distance of 940 LF. The wall sits on the exterior foundation footing and is located to allow 8" of displacement of the structure. A foundation drain was installed at the base of the retaining wall.

MOVEMENTS

Table 1.0 gives the settlements across the east - west axis and north - south axis of the building. Despite the heavy loads on the center of the building from the dome above, very little settlement was experienced. In addition to the very dry and stiff soil under the center of the building, lost bearing was easily distributed away from the excavated area by network of intersecting walls. Settlements increased under the exterior wall with the maximum settlement occurring around the boiler rooms where the soil was very moist.

TABLE 1.0

SETTLEMENT AFTER UNDERPINNING PHASE

East Wall	Center	West Wall
1.92"	0.48"	1.2"

North			South	
Cheek Wall	Portico	Center	Portico	Cheek Wall
0.36"	0.84"	0.48"	0.84"	0.6"

SETTLEMENT AFTER FOOTING INSTALLATION

East Wall	Center	West Wall
2.76"	0.84"	1.68"

North			South	
Cheek Wall	Portico	Center	Portico	Cheek Wall
0.6"	1.2"	0.84"	1.44"	1.2"

These movements caused cracking in the floors and walls along the both axis of the building. Only minor cracking occurred in the resulting quadrants of the building. Very little cracking occurred in the granite facing. Most of the movement was taken by the separation of the mortar joints. The northeast cheek wall experience extensive damage due to very poor existing construction details that where unknown until the underpinning excavation uncovered the problem areas.

An epoxy resin system was used to repair the cracks in the floors and brick interior walls. Stone masons repaired the exterior mortar joints and fill the cracks in the granite facing with a mixture of epoxy and ground up granite from granite removed for other reconstruction.

While many buildings in the United States have been retrofitted with vibration isolators, the State House in Columbia, South Carolina is, to our knowledge, the first building in the eastern United States.

ACKNOWLEDGMENTS

We would like to acknowledge the assistance of the project architect, Stevens & Wilkinson, the seismic consultant, Structural Affiliates International, Inc., the geotechnical consultant, Atlanta Testing & Engineering, the general contractor, Caddell Construction Company, Inc., and the excavation subcontractor, Rowe, Inc.. to complete this project successfully, we had the continuous support and cooperation of all these parties.

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RAILROAD SUBGRADE AND SLOPE REPAIR ALONG THE OHIO RIVER

by

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ABSTRACT

Increased traffic and heavier rail and train loads in the challenging geologic setting along the Ohio River resulted in the need to repair numerous roadbed/slope problem sites. With over 80 problem areas but having limited roadbed repair budgets, CSX Transportation employed innovative repair methods to keep the trains moving. A three-year roadbed improvement program was performed to repair more than 30 sites. In order to stretch the finite budget to the greatest number of sites, the program applied the observational approach to subgrade and slope repairs while implementing a wide variety of technologies and repair methods, including

- subgrade injection
- rockfill buttresses
- cantilever H-pile walls
- drilled friction piers
- socketed pipe pile walls
- tied-back pipe and H-pile walls
- tied-back concrete reaction blocks
- improved drainage

The repairs have been in place three to four years and have performed well under rapid drawdown after flood conditions.

History of the Ohio River Subdivision

The Ohio River Subdivision is a segment of CSX Transportation's railroad system connecting the cities of Wheeling and Huntington, West Virginia along the Ohio River (refer to Figure 1). The line was constructed by a predecessor company of CSX Transportation in the 1870s to provide railroad service to communities situated along the river. Its purpose was to carry a variety of freight commodities and passengers, and to serve as a feeder route to other routes in the region. Traffic volume on the line progressively dropped during the 1960s through the 1980s. However, coal transportation contracts executed in the early 1990s brought substantial additional traffic to the line.

Route Location and Construction Methods

The railroad was constructed in the Ohio River's often broad and flat valley between Wheeling and Huntington because the valley created a natural avenue. While carrying the risk of occasional flooding, the valley location significantly reduced the grading necessary to establish a roadbed through the otherwise rugged terrain of the region. The line was constructed before the advent of "modern" earthmoving equipment, using wagons and scrapers pulled by horses and mules. Fills were loosely constructed using end dumping techniques typical for the era and were composed primarily of silts and clays from the river's sediments, and to a limited extent, of the rock and clay excavated from cut sections made at selected locations where the valley narrowed. Since railroad locomotives and cars were much smaller in size and weight compared with those used by modern railroads, the constructors of the line could use narrow fill sections, small timber drainage structures, and lightweight rail and crossties.

Roadbed Maintenance and Repair

For the majority of its length, this floodplain subgrade is relatively stable alluvium. However, at a number of individual locations (some of which exceed 1000 feet in length) the valley narrows and the river side slopes are unstable colluvium, and/or poorer alluvium. Because the line is essentially on the flood plain of the Ohio River, high water levels from the river and its tributaries have historically affected the line causing local washouts. Moreover, localized severe settlements of the track roadbed are common primarily in the colluvial settings. Since interruptions in train service are very costly because of shipment delays to customers, crew delays, and reduced equipment utilization,

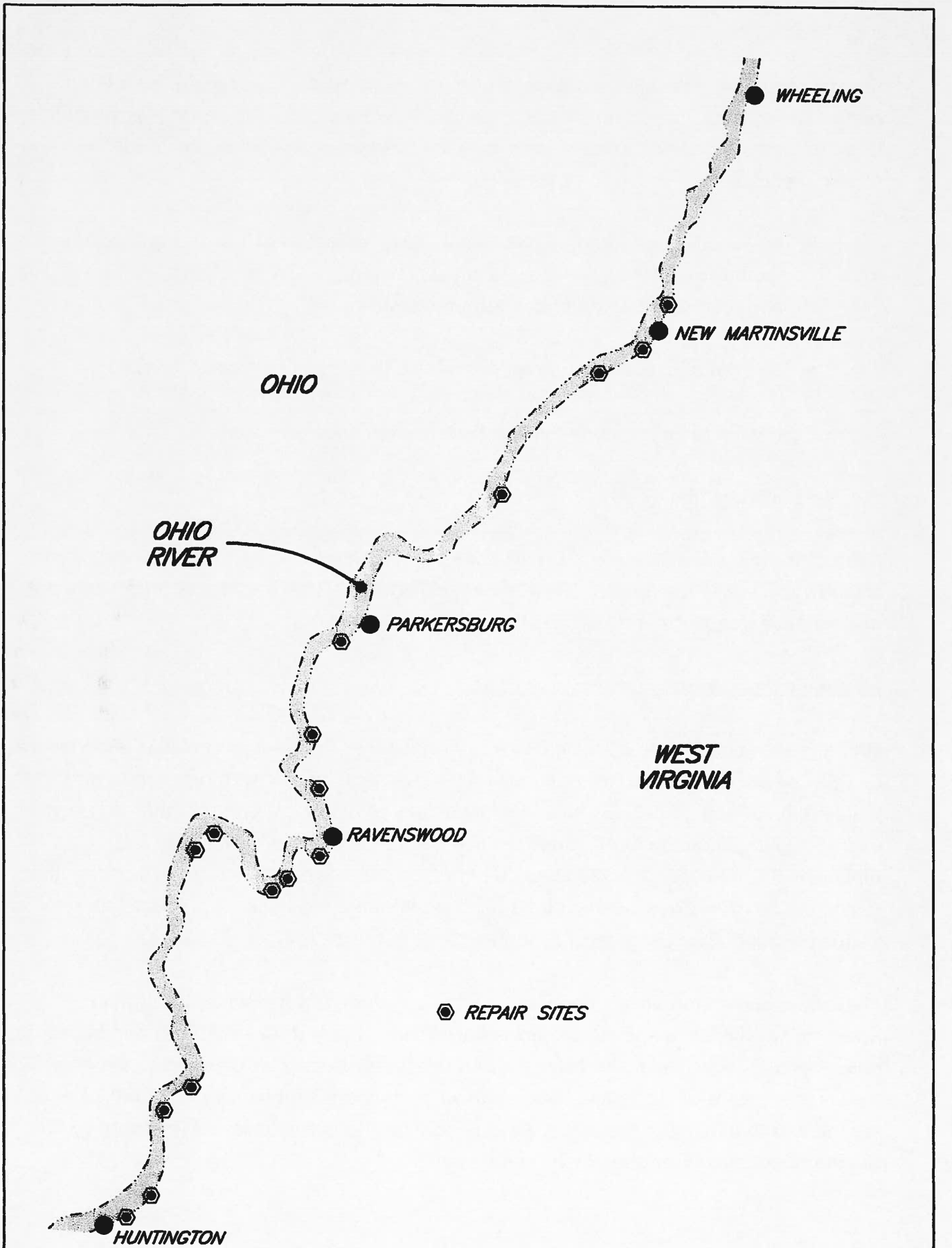


FIGURE 1 - LOCATION MAP

restorations and repairs have necessarily been made quickly. These repairs have often proved to be "Band-Aids" primarily addressing the symptoms not the underlying problems. Depending upon the location and severity of the washout or settlement, methods to restore, maintain, and repair have included

- track surfacing (dumping relatively large quantities of ballast and compacting them beneath the crossties, known as "tamping,)" to reestablish a uniform profile and relative elevation of the rails of the track;
- end dumping cinders, riprap, or shotrock to reestablish eroded roadbed;
- driving rail piles along the roadbed shoulder; and
- injecting lime-flyash.

If the cause of track settlement was embankment slope movements, track surfacing often actually aggravated the movements as shown in Figure 2. After decades of track surfacing, maintenance frequency at these sites has increased.

Ohio River Subdivision Improvement Project

With the substantial increase in coal traffic, a capital improvement project was undertaken by CSXT to prepare the line for unit coal train movements. The coal movements were planned to be made using cars having a higher total gross weight than the line had been maintained to handle up to that time. Hence, bridge and track rehabilitation and replacements, and drainage and roadbed improvements were planned to upgrade the line to handle the increased wheel loadings resulting from these cars, and to reduce the frequency of periodic maintenance resulting from the traffic.

The nature of the landslides/embankment problems along this line were not unique; however, two factors made the project extraordinary. The first was the sheer number of problems to be examined, characterized and selectively treated (as opposed to the more typical investigate, analyze, and then repair when only one problem is identified). The second was that many of the sites were at remote locations where access was only by rail, but the rail could not be shut down to train traffic.

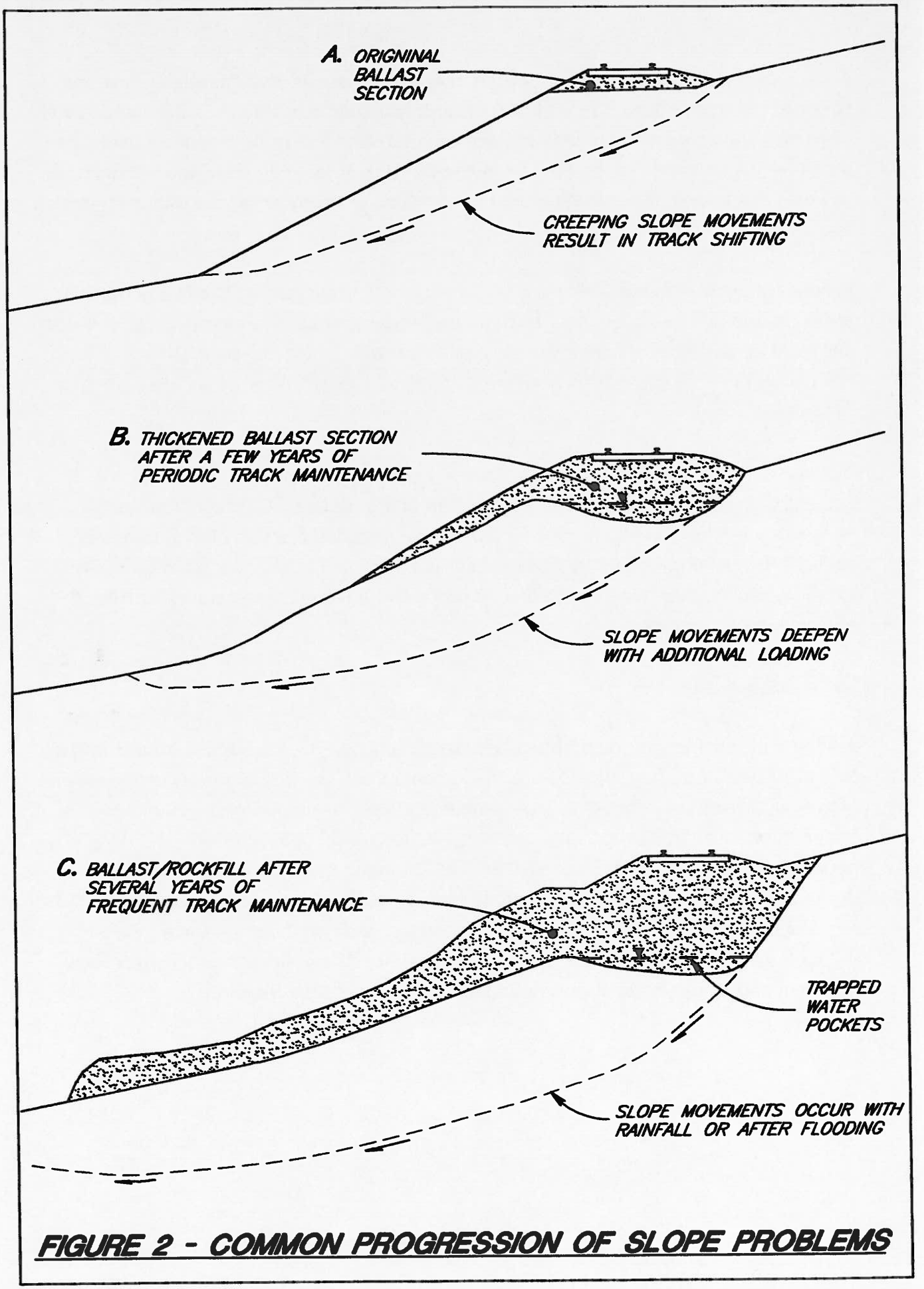


FIGURE 2 - COMMON PROGRESSION OF SLOPE PROBLEMS

Thus, the program could not be approached in a "business as usual" fashion. First, the sites had to be prioritized. Second, a decision was made to develop solutions which were often observational and included potential add-ons (such as first try cantilever walls with the provision to add tie-backs later) to spread the budget to repair more sites. In fact, we estimate that using this approach more than doubled the number of sites that were treated and improved.

Finally, for many repaired sites a varied "bag-of-tools" is necessary. While a number of these are routine landslides repair techniques, in a program this large and varied it is often desirable to develop and implement new solutions which push the state-of-the-art. That solution in this case was the anchored reaction blocks as will be described in more detail in the text.

Obviously, a comprehensive identification and geotechnical analysis of unstable roadbed and embankment locations was an integral part of the project. A variety of mitigation concepts were developed and constructed. These included reaction blocks, tied-back soldier pile retaining walls, cantilever soldier pile retaining walls, rock buttresses, cantilever drilled pier retaining walls, Geocem slurry injection, and comprehensive drainage improvements.

Site Investigations

A good understanding of the problems is essential in applying the observational approach. Adequate subsurface investigation and instrumentation is the first and most important part of this understanding. The scope of site investigations was proportionate to the severity of the problem. As a minimum, surveyed cross-sections and limited subsurface drilling were performed at the sites. At narrow sites with no shoulder, geologic sections often had to be developed from exposed surface materials and a single boring drilled through the tracks. For wider sites with more extensive slope problems, additional borings were typically drilled to develop geologic cross sections. At most locations, slope inclinometers were installed, and these almost always provided the most useful information.

Important information derived from the drilling included the following:

1. depth and slope of the top of rock;

2. material breaks between embankment fill and alluvium, and the nature of these materials;

3. depth of sliding (if inclinometers were used); and

4. groundwater levels.

One difficulty in estimating the sliding plane(s) with inclinometers was having enough time to develop adequate data. During the "sliding" season (December to May), distinct movements at the sites of one to four inches might be observed. However, the total movement during the remainder of the year might be less than one-fourth of an inch. Therefore, timing and sound engineering judgement were important in interpreting the data. Flood records and other information about the Ohio River were also important for characterizing the problems at the sites. Although the U. S. Army Corps of Engineers maintains a series of locks and dams for this section of the river, dramatic changes in the river level are still common. Further, the projected success of repair systems was affected by the river bottom profile, and river soundings were performed at several sites.

Another very important aspect of understanding the slope problems and their impact on the railroad tracks were interviews with track roadmasters and other track maintenance personnel. Information obtained from these interviews on the limits of the problems, history, frequency, and nature of the track movements was added to the site investigation data.

Nature of Problems/Materials

The track problems on the Ohio River Subdivision ranged from constantly shifting track foundations founded on deep, creeping slide planes, to steep, cliff-hanging roadbed sections where the risk of a complete track washout was high with only a moderate flood event. Examples of specific subgrade materials are shown on the figures. A typical geologic cross section includes a combination of the following

- a thickened ballast section
- fill material
- alluvium (combinations of sand, silt, and clay)
- intermittent colluvium
- bedrock (interbedded claystone, siltstone, shale, or sandstone)

Slope sliding was prevalent at most of the problem sites, commonly at the contact between the fill and the alluvium, within a soft interval of the alluvium, at the contact of the alluvium and the lower bedrock, or especially at the larger sites on residual colluvial sliding planes. Sliding was usually triggered by heavy rainfall or flooding and drawdown of the Ohio River. For the steep side hill fill sites (Figures 3A and 4A), the railroad fill embankments were constructed of cinders founded on an excavated rock shelf. Slope movements at these sites were dramatic and usually followed rapid drawdown of the Ohio River. The fill materials were easily erodible cinders with only moderate shear strength. For sites with the railroad fill embankments founded on alluvium/colluvium (Figures 5B, 8, 9, and 10), the slope movements were typically less dramatic and more "creep-like." The sliding plane at these sites ranged in depth from about 15 to 40 feet.

Engineering Analyses

One of the most difficult tasks of the site evaluations was estimating the sliding mechanism and related shear strengths. For sites with slope inclinometer data, visible escarpments, or obvious soft zones indicated by the drilling, the shear strength was derived from a "forensic," or factor of safety equals 1.0 analysis. The resulting shear strengths were used in evaluating site repairs, and were as low as $\phi = 12°$ at some of the sites. While this type of analysis is commonly used in evaluating embankment or landslide problems, the impact of train loading, multiple slide planes, elevated groundwater, and river drawdown effects also had to be considered, especially when designing repairs. Determining the cause(s) of

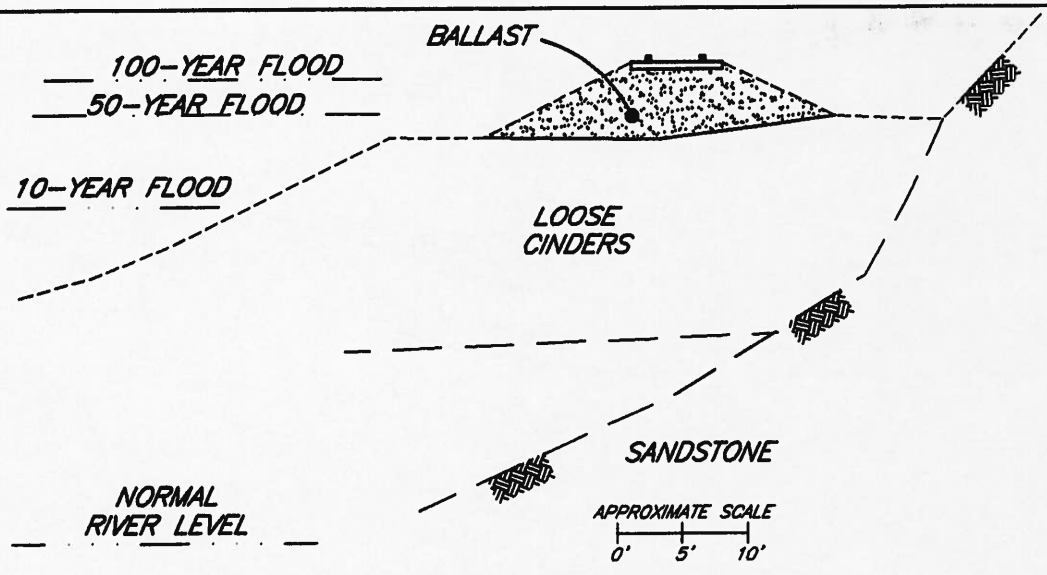


FIGURE 3A - PROBLEM CINDER FILL

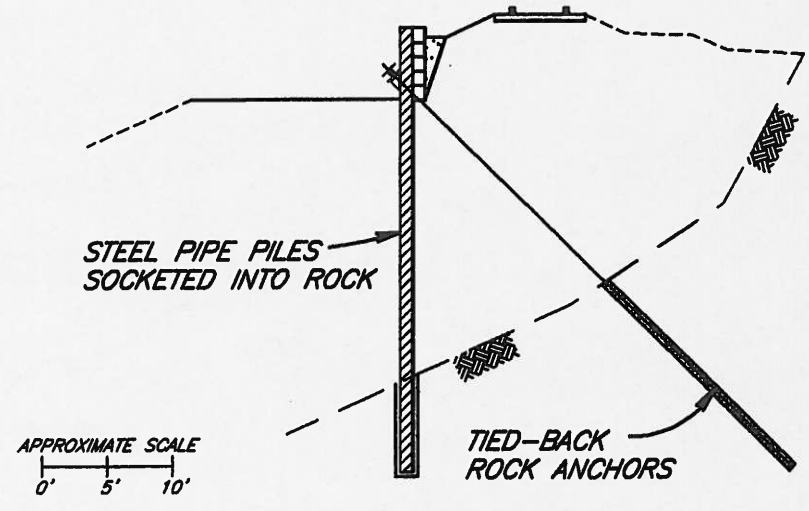


FIGURE 3B - TIED-BACK RETAINING WALL REPAIR

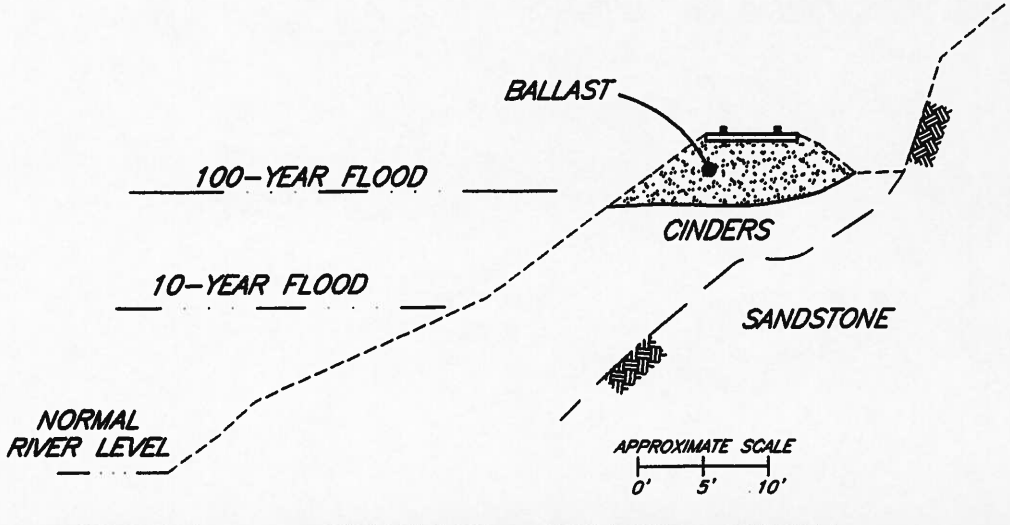


FIGURE 4A - PROBLEM SITE ON ROCK SHELF

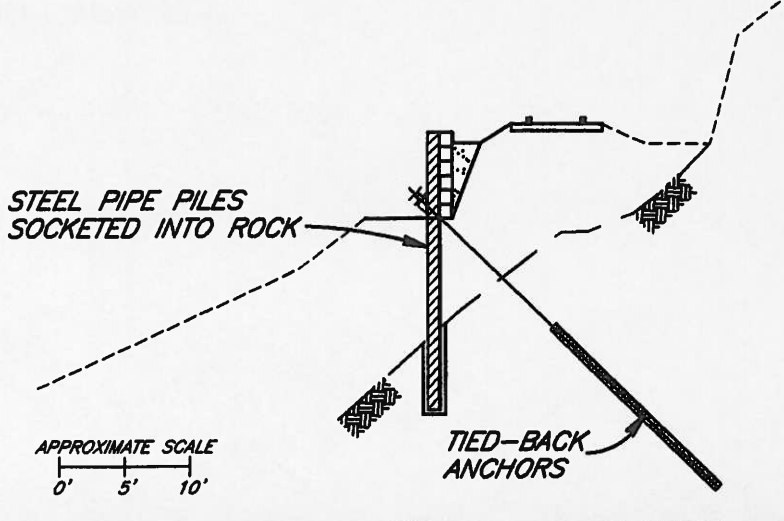


FIGURE 4B - TIED-BACK RETAINING WALL REPAIR

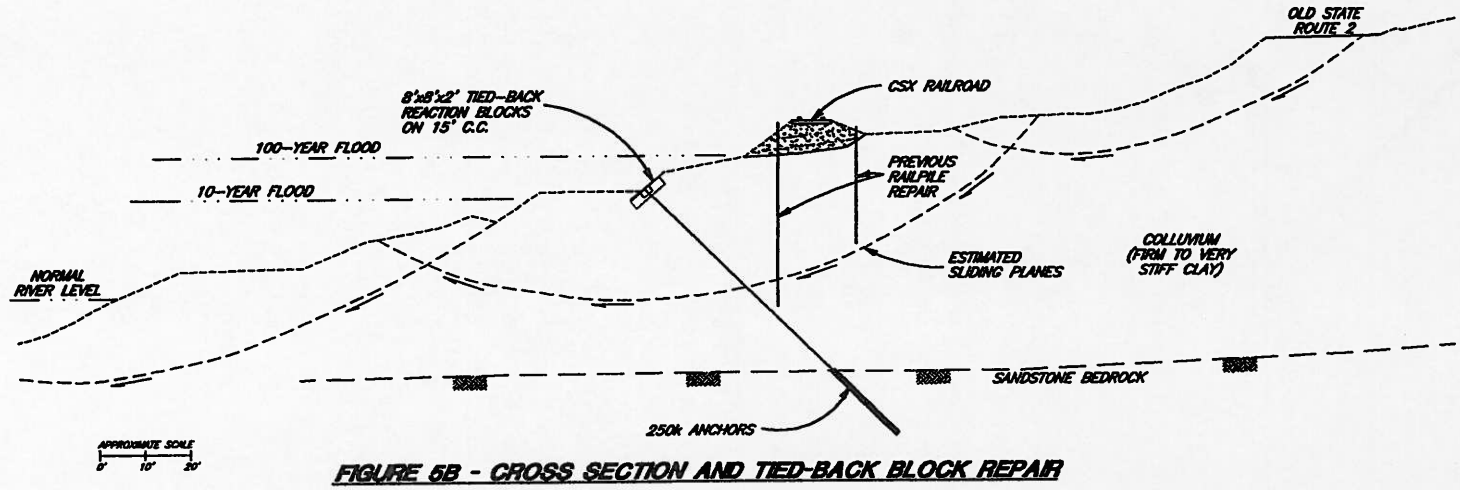


FIGURE 5B - CROSS SECTION AND TIED-BACK BLOCK REPAIR

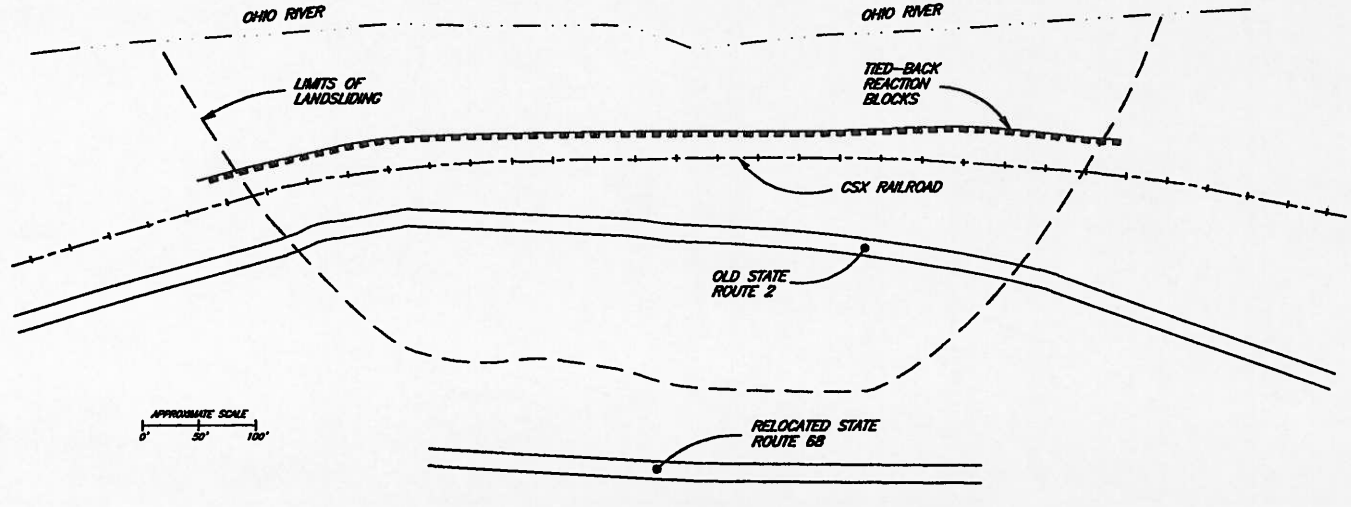


FIGURE 5A - SHERMAN SLIDE PLAN VIEW

the sliding was only important to developing a repair system that would work safely in most scenarios, or a system that could be strengthened incrementally.

Site Prioritization and Repair Criteria

After a few obvious very bad sites were identified, one of the most difficult problems was simply deciding which sites to address first. The problem sites were divided into three levels: A, B, and C, with the A-level sites having the highest priority. A-level sites were those characterized by a high probability of track problems after a minor flood event, or sites with an extensive history of problems and previously attempted slope repairs. The B sites were primarily locations that required frequent maintenance and where slope movements were evident. The C sites were periodic maintenance sites or sites where problems were anticipated to worsen along an extensive length of track with increased traffic and heavier rail and train loads.

Using the subsurface data, shear strengths derived from a factor of safety equal to 1.0 analysis, and estimated failure planes, a minimum improvement up to a factor of safety of 1.2 was targeted. In general, this safety factor was not a 1.2 against mass slope failure, but against creep movements that could result in shifting of the tracks. This method was used only at sites where an adequate understanding of the sliding mechanism had been developed. Higher safety factors were used at sites where this was not true.

Other factors which affected the selection of repair systems included:

- Access to the site
- Geometry of the work area
- Proximity to the highway
- Depth to rock
- Cost
- Disruption to train traffic
- River inundation potential
- Ability to supplement repairs, if necessary

Considering all of the above, repairs were then developed.

Constructed Repairs

For the A-level sites, repairs were performed which would provide a high level of confidence for these sites. Often, the limits of the repairs were extended beyond where the problem was evident, because returning to these sites for additional work would be difficult. Further, positive support derived from the lower bedrock was generally incorporated into the A-level sites repairs.

For the "cliff-hanger" sites, tied-back retaining walls were constructed as shown on Figures 3B, 4B and 6. Rock sockets at the base of the piles were included in the repair requirements because of unknowns regarding the sliding mechanism, and the slope of the bedrock beneath the roadbed. Utilizing special track-mounted drilling equipment, pipe piles were drilled and socketed into the lower bedrock. After lagging and backfilling behind the wall, tendon anchors were installed into the hard sandstone bedrock using a removable mast suspended at an angle in front of the walls.

The most significant of the A-level sites was a landslide at Sherman, West Virginia, dubbed the "Mother-of-all-slides" by railroad personnel. The landslide had been a problem for several decades and encompassed both the railroad and the main highway (Route 2) with multiple sliding planes for a length of over 1000 feet (see Figures 5A and 5B). In the mid-seventies, the West Virginia highway department extensively studied the slide and developed repair options, including large tied-back retaining walls. Sliding at the site was deep, and the available friction along the sliding plane was only 12° to 14°. Due to the projected cost of the repairs, they elected to relocate the highway (now Route 68) to a rock cut well above the slide. Because the railroad could not easily change grade or alignment, relocation of the tracks off of the slide was not a viable option. Numerous unsuccessful repair attempts were made to stabilize the track roadbed including several generations of rip rap, rockfill, and driven railpiles. An innovative repair approach was developed that would significantly reduce track movements by stabilizing only the portion of the slopes directly affecting the railroad. As shown on Figures 5A, 5B and 7, tied-back concrete reaction blocks were installed on a 15 foot spacing. The blocks were designed to impart the load from 250 kip anchors to the surrounding embankment to prestress the soils similar to a spread footing supporting a heavy column load. The blocks were designed such that they could be restressed if necessary after flooding events, and their cost was roughly half of the projected costs of a tied-back retaining wall. The blocks have been in place almost 4 years, and have stabilized the railroad embankment.



FIGURE 6 - TIED-BACK AND SOCKETED RETAINING WALL



FIGURE 7 - TIED-BACK REACTION BLOCKS AT SHERMAN SLIDE

Repairs for the B-level sites were developed to stretch the railroad's repair budget by providing repairs that could be incrementally applied (i.e., the observational approach originally conceptualized by Terzaghi). Cantilever retaining walls were proposed for many of the sites with the provision that they could be tied-back with rock anchors in the future if necessary (Figures 10 and 11). This approach could only be used with an adequate understanding of the sliding mechanism from slope inclinometer monitoring, and by performing periodic inspections of the sites after completion of the repairs. For B-level sites where the sliding was at the contact of the fill and a deep interval of alluvium or colluvium, the rock was too deep to economically install tie-backs. Friction pile support systems were designed for these sites, and utilized drilled concrete piers or driven closed-end pipe piles installed on a batter as shown on Figures 8 and 9.

Repairs for the C-level sites were scaled back even further by treating only limited sections of problem embankments, widening the spacing of support points, or using lower cost treatment methods for long problem sites. Based on the success of the reaction blocks at the Sherman slide, reaction block spacing was widened to 20 feet and blocks were used at select locations for the C-level sites. Further, many of the C-level sites had extensive lengths of deep intervals of ballast or cinders from decades of track leveling. Subgrade injection with a cement-based slurry product GEOCEM was performed at these sites to fill the void spaces in the ballast, and to displace trapped water in the roadbed.

Other repair sites of interest involved emergency reconstruction and protection of slope shoulders. While slope movements effecting the railroad tracks had not occurred at these sites, erosion of the railroad embankment materials had progressed to where the risk of embankment failure was high with the next river flood event. Rockfill buttresses were constructed as shown on Figures 12 and 13. Understanding the slope materials and providing an adequate keyway at the base of the slopes were pivotal in the success of these projects.

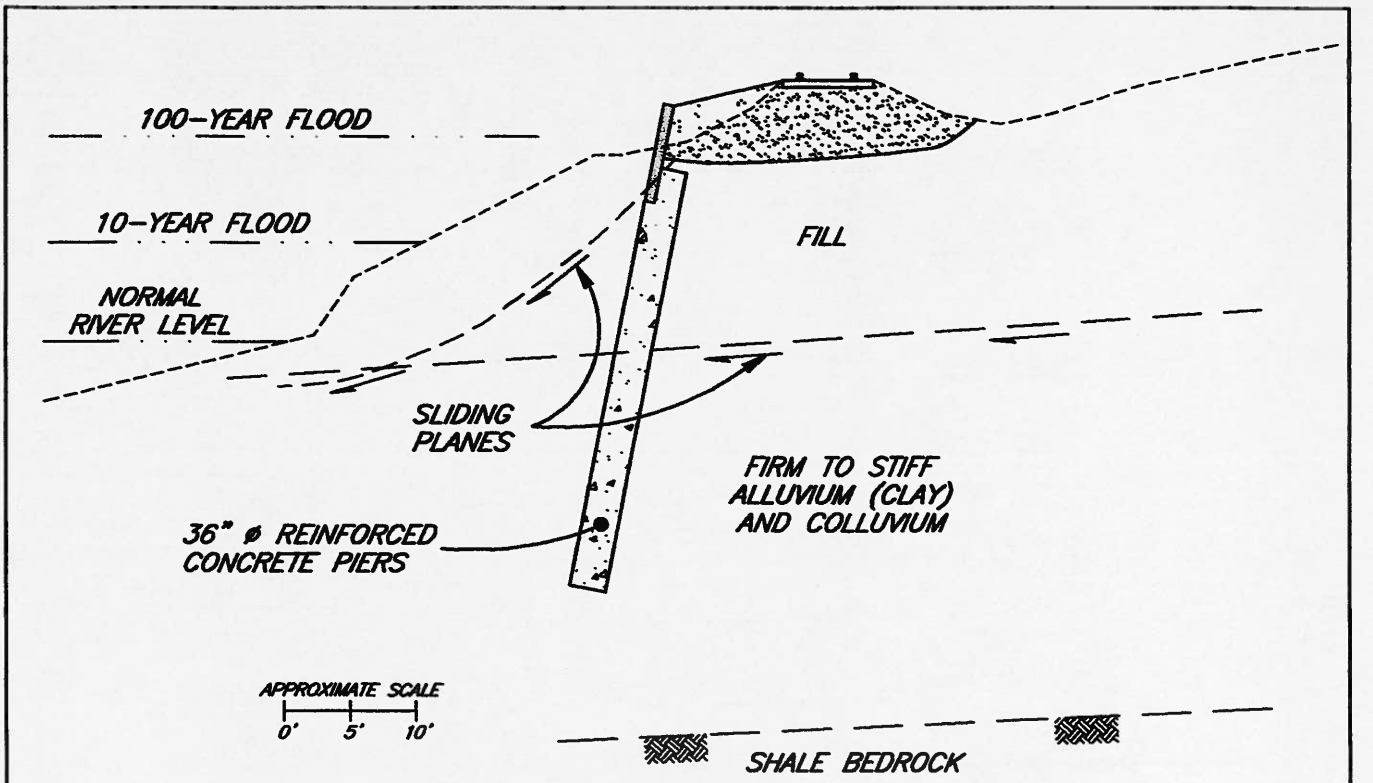


FIGURE 8 - DRILLED FRICTION PIER REPAIR

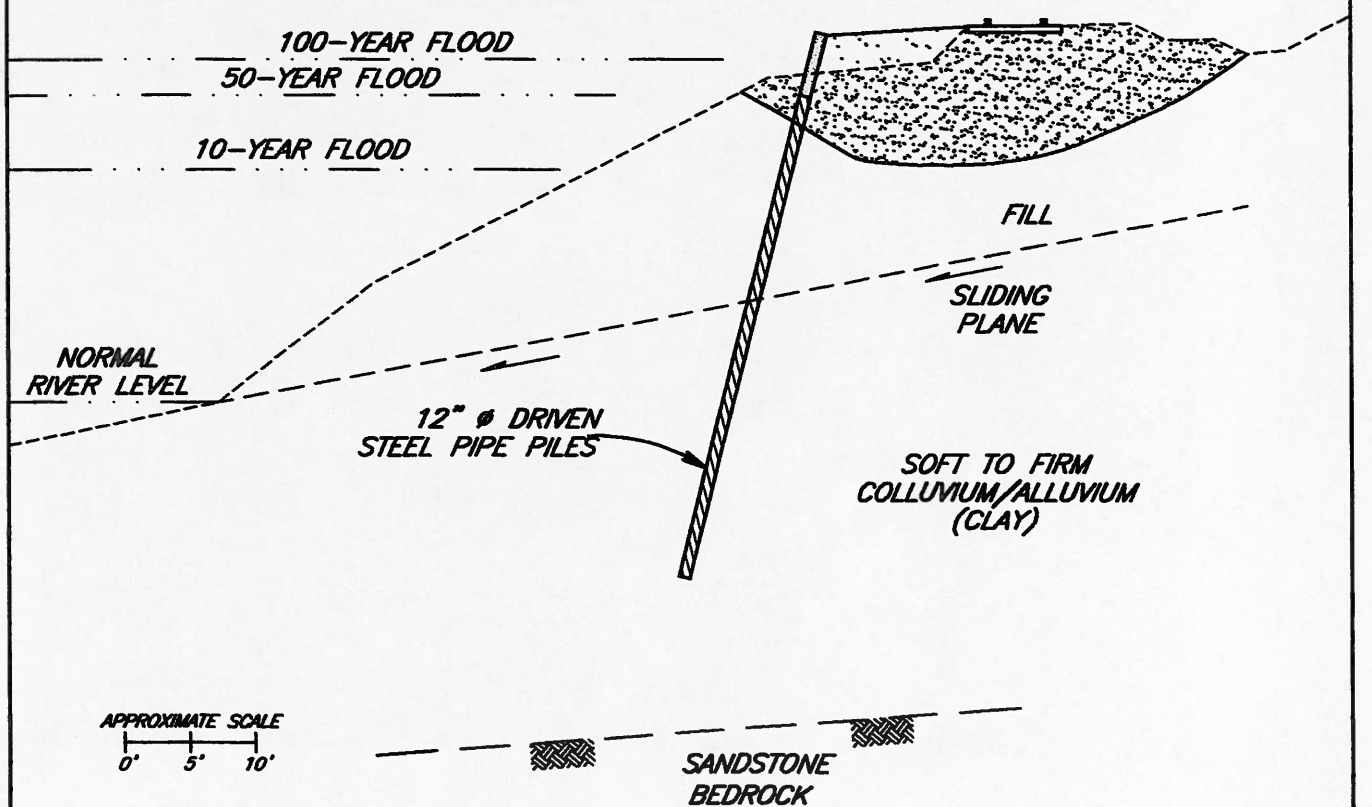


FIGURE 9 - DRIVEN FRICTION PILE REPAIR

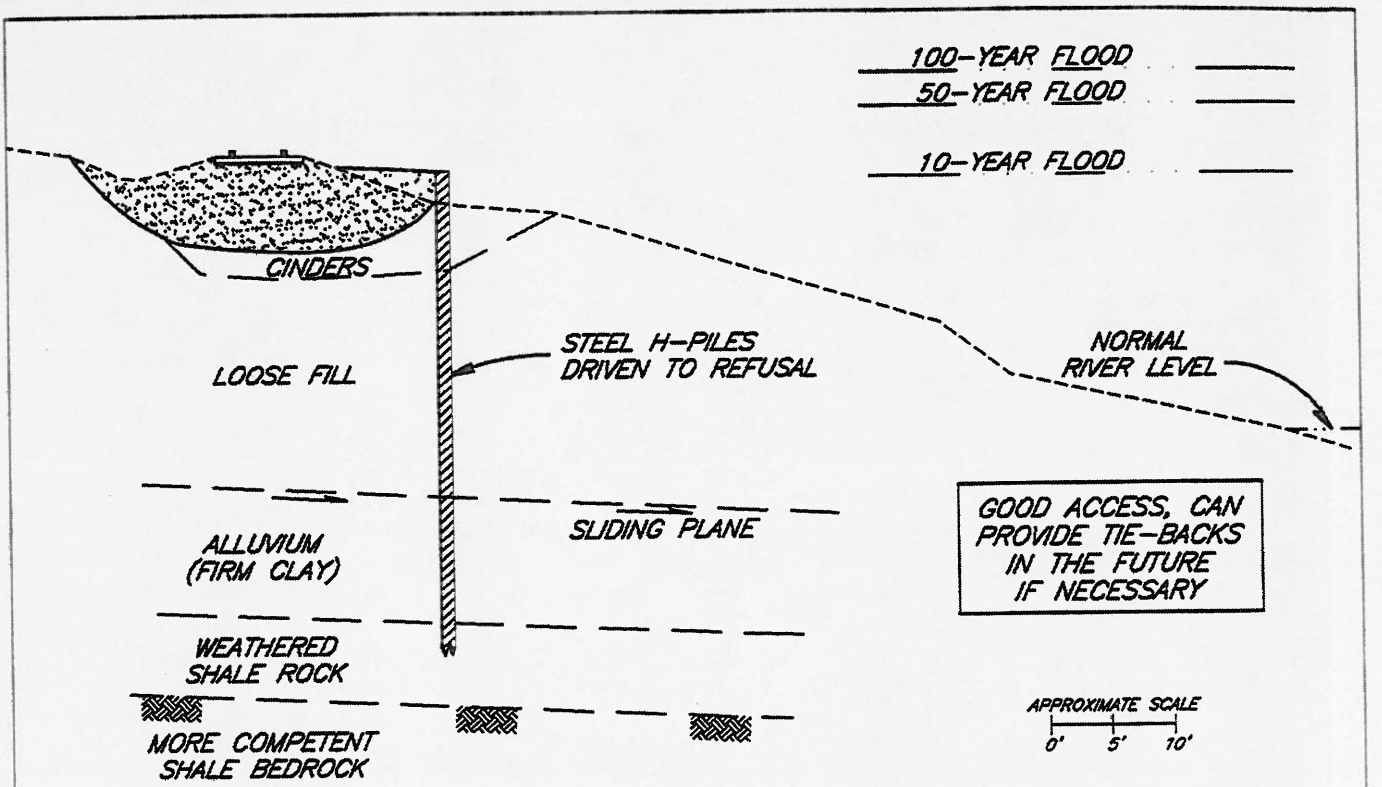


FIGURE 10 - DRIVEN CANTILEVER PILE REPAIR



FIGURE 11 - DRIVEN CANTILEVER PILE REPAIR

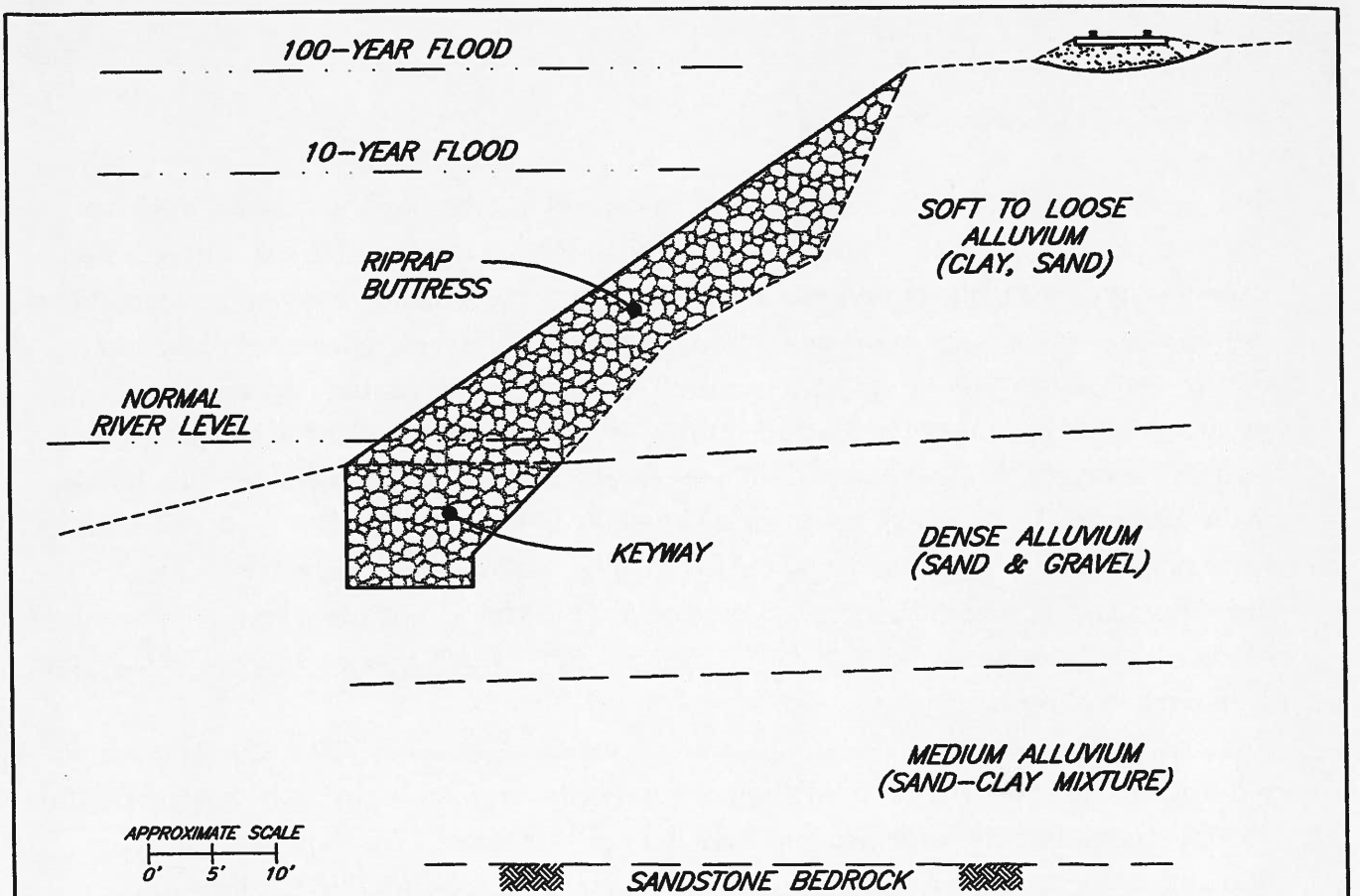


FIGURE 12 - SLOPE PROTECTION WITH RIPRAP

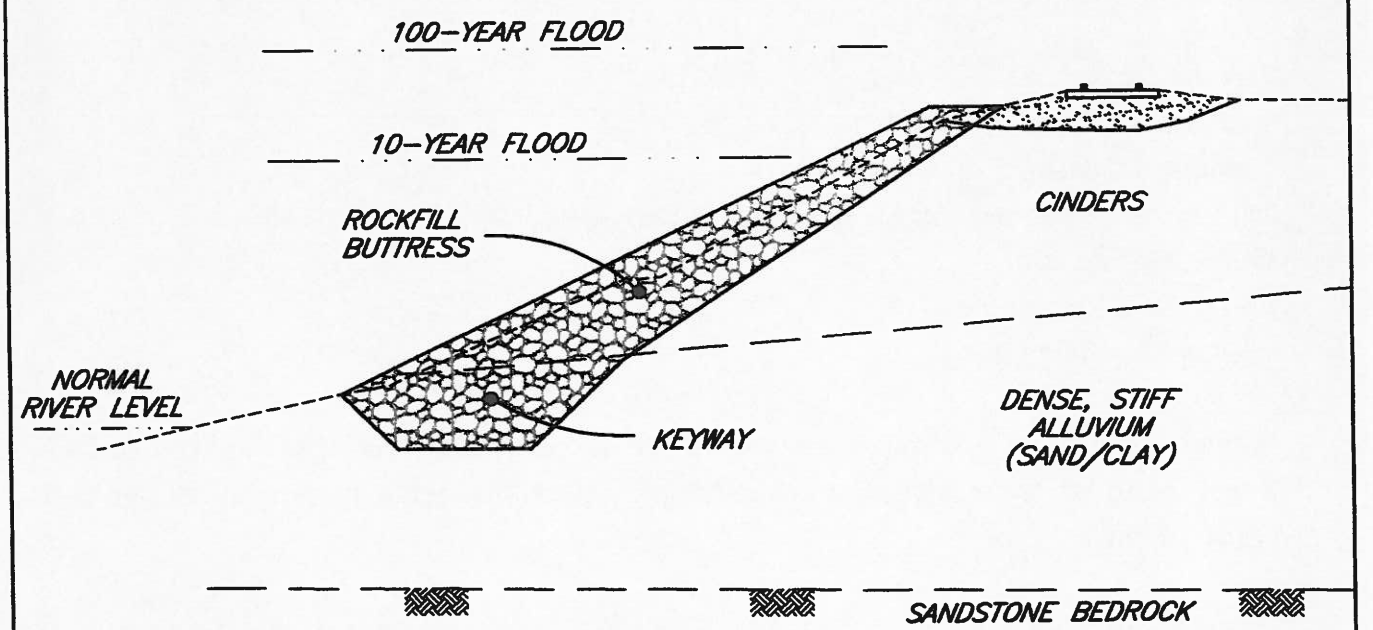


FIGURE 13

SHOULDER RECONSTRUCTION WITH ROCKFILL BUTTRESS

Performance of Repairs/Conclusions

The repairs on the Ohio River Subdivision have been in place for 3 to 4 years. CSX has instituted a monitoring program whereby the tops of all of the retaining walls have been carefully surveyed, and are periodically monitored for movements. Further, periodic lift-off testing of the anchors are being performed for the reaction block repair sites. Seasonal flood events of the Ohio River have occurred since completion of the improvement project, including an event approaching the 100-year flood level. Several of the retaining walls were inundated by the river during this event, as were the tied-back reaction blocks at the Sherman slide. As expected, some sloughing of the level bench in front of the repair walls has occurred, but no remedial repairs to the walls have been necessary. The innovative anchor blocks have performed very well in almost all cases. However, at some of the C-level sites a few blocks have "floated around" a small amount when inundated and required some restressing.

It is obvious that the observational approach to roadbed repairs is currently very successful. The repair dollars were stretched and the subdivision is stable. Time and additional flooding will be necessary to judge just how effective the program is in the long run.

ACKNOWLEDGEMENTS

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Walker Excavating, Inc.

Further, the authors would like to acknowledge the dedicated work of Mr. Joe Lovetro, Mr. Roger Cecil, Mr. Mitch Halsey, and Mr. Craig Smith on managing the program design and construction.

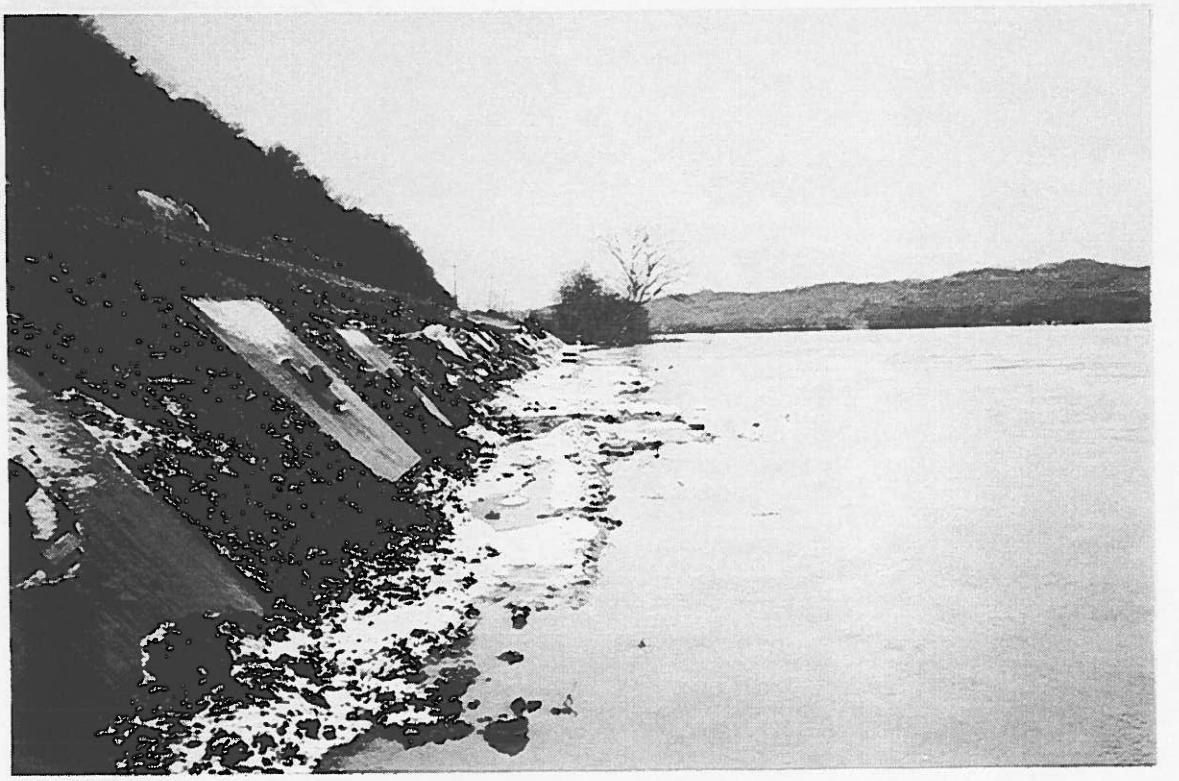


FIGURE 14 - REACTION BLOCKS DURING FLOOD



FIGURE 15 - CANTILEVER WALL DURING FLOOD

REFINED FIELD METHODS FOR IDENTIFYING, DESCRIBING, AND TESTING SHALE AND WEAK ROCK

by

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ABSTRACT

Many classification and testing methods for shale, slaking rock, weak rock, and weathered rock have been developed. However, few of these methods are suitable for field use or are applicable to a wide range of material types. The purpose of this paper is to present a summary of recent research and analysis evaluating classification and testing methods. This summary will assist the practitioner in properly identifying problem shale and weak rock units, in adequately describing their characteristics in the field, and in selecting and performing field tests to quantify their behavior.

For identification of shale and weak rock units, three increasingly site-specific tools are presented. First is a map showing locations of weak rock units in the U.S. Second is a detailed list of suspect rock types which may appear on regional or local geologic maps. Third is a summary of engineering properties which describe weak rock units, based on a review of technical literature. Some of these properties are compressive strengths between 1 and 20 MPa, slake durability less than 90%, clay content greater than 15%, poor induration, a significant amount (50 to 75%) of matrix between hard blocks, or high moisture content.

Methods of field description were chosen based on a review of existing methods to assess their ease of application, breadth of application, and the usefulness of the engineering properties indicated by each system. A modified method based on Geological Society (1995), Dearman (1976), and Palicki (1997) is presented.

Finally, field testing methods which estimate strength, permeability, durability, and reaction to water are identified. These include point load index, Norwegian Geotechnical Institute Rock Mass Classification, jar slake, and hammer rebound classification. Predictive equations to estimate unconfined compressive strength, slake durability and slake index are given.

INTRODUCTION

In spite of the large number of classification and testing schemes for shales and other weak or weathered rock types, there exists no single accepted method for characterizing these materials. For example, at a recent symposium, researchers and practicing professionals who deal with weak rock were asked to suggest the classification system they prefer to use. Of ten respondents, nine different schemes were suggested (Santi and Shakoor, 1997). In an overview of classification systems, Chapman and others (1976) note that all shale classification systems have fallen into one of three categories: systems developed by geologists emphasizing genesis, systems developed by

engineers emphasizing quantitative laboratory tests, and systems developed by agencies which prove to be limited regionally or occupationally.

In particular, field recognition and field identification of adverse properties needs to be consistently addressed. Santi and Shakoor (1997) note that many of the common blunders in dealing with weak rocks stem from poor field data, lack of recognition of sensitivity to water, and lack of recognition of the influence of rock mass properties such as remnant bedding planes. The goal of this paper is help prevent these blunders by summarizing recent research and analysis for field identification, description, and testing of weak and weathered rock materials.

IDENTIFICATION OF WEAK ROCK

For field identification of weak rock units, three increasingly site-specific tools are presented. The first is a map showing location of weak rock units in the United States. The second is a detailed list of suspect rock types which may appear on regional or local geologic maps. The third is a summary of engineering properties which describe weak rock units, based on a review of technical literature.

General Weak Rock Units

Figure 1 (from Santi and Doyle, 1997) shows generalized locations of a number of types of weak rock in the United States. Because of the small scale of the map, many small weak rock bodies could not be included. The units shown on Figure 1 are expected to have less than thirty feet of soil cover, and to be at least ten feet thick.

Rock Types for Local Recognition

While some rock minerals like gypsum are inherently weak, the genetic and post-depositional processes are typically what "create" weak rock. In light of this, any list of weak rock types should be grouped by both mineral content and post-depositional processes. Such a list is presented in Table 1, which should serve the user in identifying weak rock units on local geologic maps.

Recognition of Weak Rock by Engineering Properties

If even a small amount of laboratory or field data is available, it may be used to help identify problematic weak rocks. While the map and rock type information presented above is useful as a general overview for a region, the engineering properties may be used for site-specific confirmation. A summary of methods of identifying weak rock by engineering properties is given in Table 2.

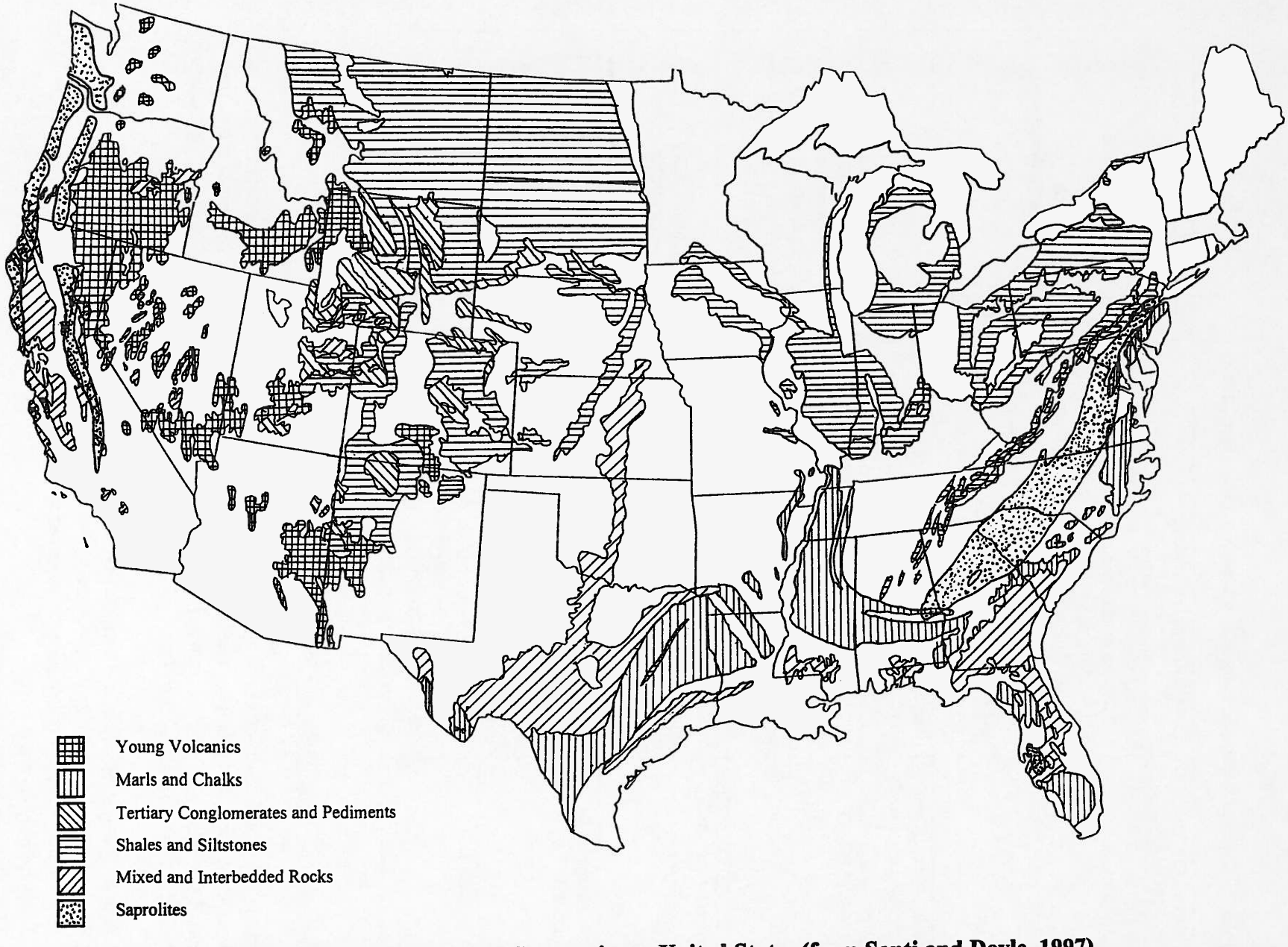


Figure 1. Weak Rock Locations in the Conterminous United States (from Santi and Doyle, 1997).

Table 1. Types of Weak Rocks (modified from Santi and Doyle, 1997).

Mineralogy or Processes Affecting	Explanation of Importance	Rock Types
Materials with high clay content	Clay minerals are very sensitive to water, have low friction angles, and are generally poorly cemented, all of which reduce material strength. Although an engineering definition of clay requires at least 50% clay-sized particles, clay-like behavior will be observed with a much smaller clay component.	overconsolidated clay cemented clayshale, shale siltstone mudstone, mudrock tightly interbedded sedimentary rocks marl flysch turbidite
Materials with other undesirable minerals	Other minerals may be weak or reactive to water.	chalk anhydrite gypsum
Young materials	Young materials generally are poorly cemented, poorly consolidated, and retain a significant component of unstable and easily-weathered minerals.	Quaternary carbonates Tertiary sedimentary rocks (conglomerate, sandstone) Tertiary volcanic rocks (basalt, tuff, volcanic breccia) pediments
Highly weathered materials	Weathering removes cement, dissolves mineral framework, loosens cohesive bonds, creates clays, lower friction angles, yet often leaves relict planes of weakness such as joints and bedding planes.	saprolite weathered igneous rocks weathered metamorphic rocks
Metamorphosed materials	Metamorphism often produces unstable and easily-weathered minerals, schistosity or other layered weakness planes, or block-in-matrix structure which is problematic for sampling and testing.	melange metashale
Hardened soils	These materials are often underlain by weaker soils, and they may also break down upon extended saturation.	hardpan caliche laterite tropical duracrust

FIELD DESCRIPTION OF WEAK ROCK

Santi (1997) evaluates and compares fourteen different popular classification systems for weak or weathered rock, and proposes modifications to enhance the applicability of some of the more useful systems. The evaluation was based on a series of ten questions, which should be answered "yes" for the most useful systems:

1. Can it (the system) be applied in the field without special equipment?
2. Can it be used without any laboratory work?
3. Is the classification confirmed and enhanced with laboratory data?
4. Does it apply to weak rock?
5. Does it apply to weathered rock?

Table 2. Summary of Engineering Properties of Weak Rock (modified from Santi and Doyle, 1997).

Test or Property	Value or Range for Weak Rock	Reference
Compressive strength	1 – 20 MPa	Afrouz, 1992
Standard Penetration Test (SPT)	50 – 300 blows per foot	White and Richardson, 1987; Sowers, 1973
Rock Quality Designation (RQD)	< 25 –75%	Santi and Doyle, 1997
Seismic wave velocity	< 7000 feet per second	Caterpillar, 1996; White and Richardson, 1987
Ratio of weathered matrix to unweathered blocks	> 75% matrix	Geological Society, 1995
Jar slake	≤ 4	Santi, 1995
Slake durability, $I_d(2)$, ASTM D4644-87	< 90%	Santi and Doyle, 1997 Lee and de Freitas, 1989
Free swell	> 3 – 4%	Welsh and others, 1991 Underwood, 1967
Natural moisture content	>1% for igneous and metamorphic rocks >5 – 15% for clayey rocks	Santi and Doyle, 1997
Dearman weathering classification (described below)	≥ category 4	Santi, 1995
CSIR rock mass rating (Bieniawski, 1976)	< 60	Santi, 1995
Norwegian Geotechnical Institute (NGI) "Q" rating	< 2	Santi, 1995
Hammer rebound (described below)	≥ category 4	Santi, 1995

6. Does it apply to rock masses, considering fractures and block-in-matrix structure?
7. Does it evaluate strength?
8. Does it evaluate durability?
9. Does it suggest a range of quantitative engineering properties?
10. If not quantitative, does it suggest at least a qualitative range of engineering properties?

While no single system addresses all the issues queried above, Santi (1997) suggests that modifications to Dearman (1976), Geological Society (1995), and Palicki (1997) may be incorporated and the systems used in tandem to provide a thorough and useful field classification system. Santi's (1997) system has been further modified and consolidated and is presented in Tables 3 and 4.

Once a material has been classified using Tables 3 and 4, additional engineering characteristics can be deduced using previously-published correlations. Table 5 (modified from Dearman, et al, 1978; Krank and Watters, 1983; and Lee and de Freitas, 1989) shows expected engineering behavior based on the Weathering Grade determined in Table 3. Quantitative values shown in Table 5 were determined for weathered granite, so they should only be used cautiously for other materials. Figure 2 (modified from Palicki, 1997) provides some interpretation of material behavior based on the S-D-R values from Table 4. Finally, Table 6 shows estimates of slake durability and long-term material behavior based on the R value from Table 4.

Table 3. Weathering Grades of Weak and Weathered Rock (modified from Dearman, 1976 and Geological Society, 1995).

Weathering Grade	General Description	Specific Description	
		Weakened Materials	Weathered Blocks
		Uniform weathered materials such as igneous and metamorphic rocks which show weakening and susceptibility to slaking upon weathering.	Materials which incorporate both matrix and mass features; weathering is enhanced along fractures, but most weathering occurs near the ground surface. Includes overconsolidated clays, shales, and mudstones.
VI – Residual Soil	The rock is completely changed to a soil in which the original rock texture has been completely destroyed.	Soil derived by in situ weathering but retaining none of the original texture or fabric.	Residual or Reworked. Matrix with occasional altered random or "apparent" lithorelics, bedding destroyed.
V – Completely Decomposed	The rock is changed to soil in which the original rock texture is (mainly) preserved.	Considerably weakened, slakes, and the original texture is apparent.	Destructured. Greatly weakened, mottled, ordered lithorelicts in matrix becoming weakened and disordered, bedding disturbed.
IV – Highly Decomposed	50 – 100% soil from decomposition of the rock mass.	Large pieces can be broken by hand and does not readily slake when dry sample immersed in water.	Partially or Distinctly Weathered. Weakened, close fracture spacing, weathering penetrating in from fractures, brown oxidation.
III – Moderately Decomposed	Up to 50% soil from decomposition of the rock mass.	Considerably weakened, penetrative discoloring, and large pieces cannot by broken by hand.	Unweathered. Original strength, color, and fracture spacing.
II – Slightly Decomposed	100% rock; discontinuity surfaces or rock material may be discolored.	Slight discoloration and slight weakening.	----
I – Fresh	100% rock; no discoloration, decomposition, or other change.	Unchanged from original state.	----

Table 4. Corestone (C), Strength (S), Discontinuity (D), and Change of Strength (R) Values for Weak and Weathered Rock (modified from Geological Society, 1995 and Palicki, 1997).

Corestone (C) Value	% of Each Weathering Grade		Typical Characteristics
1	100% I – III	0% IV - VI	Behaves as rock; apply rock mechanics principles to mass assessment and design.
2	>90% I – III	<10% IV - VI	Weak materials along discontinuities. Shear strength, stiffness and permeability affected.
3	50 – 90% I – III	10 – 50% IV - VI	Rock framework still locked and controls strength and stiffness, matrix controls permeability.
4	30 – 50% I – III	50 – 70% IV – VI	Rock framework contributes to strength; matrix or weathering products control stiffness and permeability
5	<30% I – III	>70% IV – VI	Weak grades will control behavior. Corestones may be significant for investigation and construction.
6	0% I – III	100% IV - VI	May behave as soil although relict fabric may be still significant.

Strength (S) Value	Field Estimation	Unconf. Compr. Strength (MPa)
0	Soil extrudes between fingers when squeezed in hand	<0.04
1	Soil easily molded with fingers	0.04 – 0.08
2	Soil can only be molded by strong pressure of fingers	0.08 – 0.15
3	Soil cannot be molded by fingers	0.15 – 0.30
4	Soil can be indented by fingernail	0.30 – 0.60
5	Brittle or tough, may be broken by hand with difficulty	0.60 – 1.25
6	Very soft rock, material crumbles under firm blows with sharp end of a geological pick	1.25 – 5.0
7	Too hard to cut by hand into a cylindrical lab specimen	5.0 – 12.5
8	Soft rock, 5mm indentations with sharp end of a rock pick	12.5 – 50
9	Hard rock, hand held specimen can be broken with single blow of a geological hammer	50 – 100
10	Very hard rock, more than one blow of a geological hammer required to break specimen	>100

Discontinuity (D) Value	Spacing		Aperture		Roughness
0	<6mm	Very narrow	>200mm	Wide	Polished
1	6 – 20mm	Narrow	60 – 200mm	Moderately wide	Slickensided
2	20 – 60mm	Mod. narrow	20 – 60mm	Mod. narrow	Smooth
3	60 – 200mm	Moderately wide	6 – 20mm	Narrow	Rough
4	200– 600mm	Wide	2 – 6mm	Very narrow	Defined ridges
5	600mm – 2m	Very wide	>0 – 2mm	Extremely narrow	Small steps
6	>2m	Extremely wide	0	Tight	Very rough

Note: select the lowest "D" value from the three factors (spacing, aperture, or roughness)

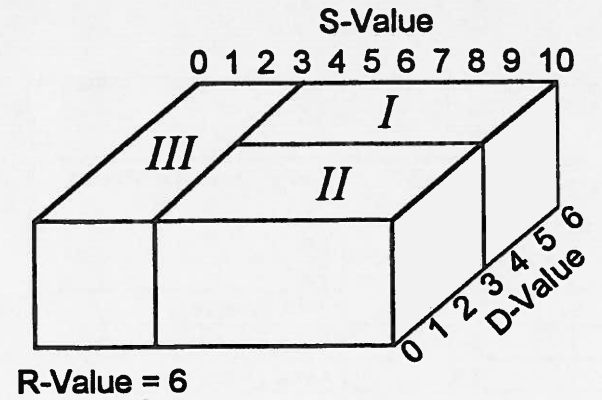
Rate of Change of Strength (R) Value ("Jar Slake" Value)	Term	Description
1	Mud	Degrades to a mud-like consistency.
2	Many chips, flakes	Sample reduced to chips. Original outline of sample not discernible.
3	Few chips	Chips of material fall from the sides of the sample. Sample may also be fractured. Original outline of sample is barely discernible.
4	Many fractures	Sample fractures throughout, creating a chunky appearance.
5	Few fractures, slabs	Sample parts along a few relatively planar surfaces.
6	No reaction	No discernible effect.

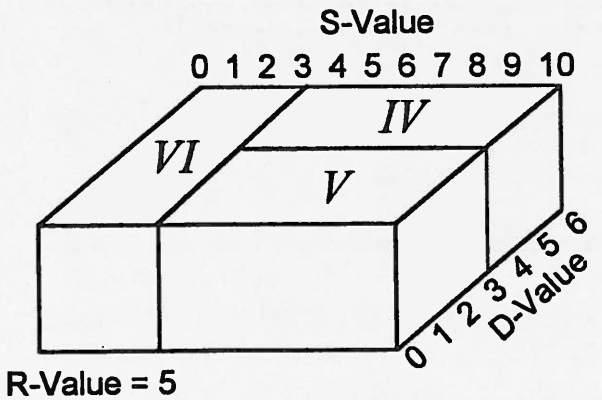
Note: these values are based on reactions after a 30-minute soaking in distilled water

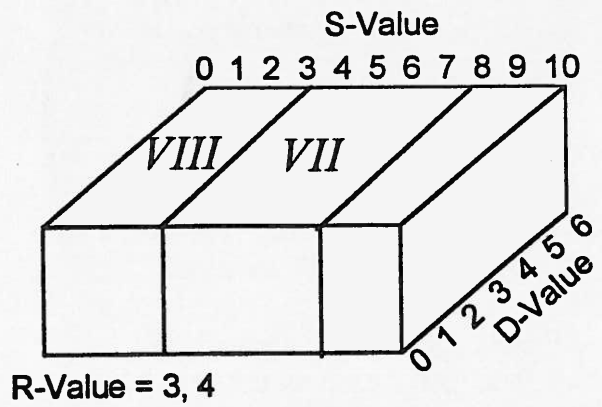
Table 5. Engineering Behavior from Weathering Grade (modified from Dearman, et al, 1978; Krank and Watters, 1983; Lee and de Freitas, 1989).

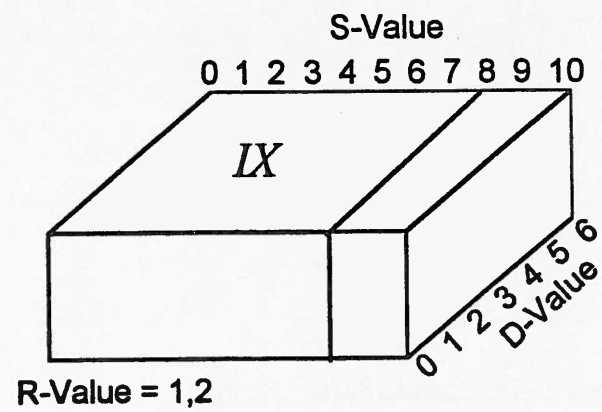
Engineering Property	Fresh, I	Slightly Weathered, II	Mod. Weathered, III	Highly Weathered, IV	Completely Weath., V	Residual Soil, VI
Foundation conditions	Suitable for concrete and earthfill dams	Suitable for concrete and earthfill dams	Suitable for small concrete structures, earthfill dams	Suitable for earthfill dams	Suitable for low earthfill dams	Generally unsuitable
Excavatability	In general, blasting necessary	In general, blasting necessary	Generally blasting needed, but ripping may be possible depending upon the jointing intensity	Generally ripping and/or scraping necessary	Scraping	Scraping
Building Material	Very durable, difficult to shape, excellent aesthetic properties, good aggregate	Very durable, but less aesthetically pleasing due to discoloration, good aggregate	Poor, subject to deterioration, not suitable as an aggregate	Unsuitable, not usable as an aggregate	Too angular and poorly graded to serve as an aggregate by itself	Too poorly graded to be used as an aggregate by itself, but may serve as a sand filler if screened
Slope stability (Krank and Watters, 1983)	Excellent – can stand vertically unsupported	Excellent – can stand vertically unsupported	Very Good – can stand vertically unsupported, but subject to deterioration	Good – can stand vertically unsupported up to 25 feet high, greater heights should be 1:1 H:V with benches, deteriorates with time	Moderate – can stand unsupported at 1:1 H:V with benches to catch detritus	Very Poor – unstable due to low cohesion, greatly influenced by ground water, best removed
Slope stability (Dearman, et al, 1978)	1/4:1 H:V	1/2:1 to 1:1 H:V	1:1 H:V	1:1 to 1.5:1 H:V	1.5:1 to 2:1 H:V	1.5:1 to 2:1 H:V
	Benches and surface protection structures are advisable, particularly for more highly weathered material. The presence of through-going adversely oriented structures is not taken into account.					
Tunnel support	Not required unless joints are closely spaced or adversely oriented	Not required unless joints are closely spaced or adversely oriented	Light steel sets on 0.6- to 1.2-m centers	Steel sets, partial lagging, 0.6- to 0.9-m centers	Heavy steel sets, complete lagging on 0.6- to 0.9-m centers. If tunneling below water table, possibility of soil flow into tunnel	Heavy steel sets, complete lagging on 0.6- to 0.9-m centers. If tunneling below water table, possibility of soil flow into tunnel
Point load strength (MPa)	9 – 18	5 – 12.5	2 – 6	0.3 – 0.9	--	--
Schmidt hammer value (MPa)	59 – 62	51 – 56	37 – 48	12 – 21	--	--
Moisture content (%)	0.06 – 0.30	0.15 – 0.29	0.25 – 0.49	0.37 – 3.80	7.84 – 21.00	12.24 – 22.1
Unconfined compressive strength (MPa)	125 – 260	100 – 175	60 – 120	35 – 55	--	--
Drilling RQD (%)	75, usually 90	75, usually 90	50-75	0-50	0 or does not apply	0 or does not apply
Core recovery (%)(NX)	90	90	90	Up to 70, if a high % of core stones, as low as 15 if not	15 as sand	15 as sand
Drilling rates (m/hr)						
(Diamond NX)	2-4	2-4	8-10	8-10	10-13	10-13
2 1/2 " percussion	5-7	8	12-15	12-15	17	17
Permeability	Low to medium	Medium to high	Medium to high	High	Medium	Low
Seismic velocity (m/sec)	3050-5500	2500-4000	1500-3000	1000-2000	500-1000	500-1000
Resistivity (ohm-m)	340	240-540	180-240	180-240	180	180
	Tends to be determined by joint openness and water-table depth					

Note: these values were determined for weathered granite. They should be used cautiously with other materials.









Region I
Strength controls properties

Region II
Fractures control properties

Region III
Soil-like properties

Region IV
Strength controls properties, but may experience slightly accelerated weathering

Region V
Fractures control properties, but may experience slightly accelerated weathering

Region VI
Soil-like properties, susceptible to minor reduction in strength

Region VII
Significant loss in strength may occur, but user should remain aware of original strength and fracturing

Region VIII
Soil-like properties, susceptible to a significant reduction in strength over time

Region IX
Readily slakes to soil-like material

Figure 2. Interpretation of S-D-R Values from Table 4 (modified from Palicki, 1997).

Table 6. Estimates of slake durability and long-term material behavior based on the R value (from Santi and Higgins, 1998 and Santi, 1995).

Rate of Change of Strength (R) Value ("Jar Slake" Category)	Slake Durability	Slake Index	Long Term Treatment
1	0-15	75-100	Soil
2	15-25	40-90	Soil
3	25-40	25-70	Very poor rock
4	40-55	5-30	Poor, yet durable rock
5	55-70	5-15	Good to fair rock
6	70-100	0-10	Good to fair rock

Note: "Long Term Treatment" is the expected behavior of the material following long term exposure to weathering.

FIELD TESTING OF WEAK ROCK

In addition to the estimates of engineering properties which may be made using Tables 4, 5, and 6, several field tests may be correlated to laboratory tests. For example, Smith (1997) proposes a relationship between Unconfined Compressive Strength (UCS) and the field-derived Point Load Index (I_s). For stronger rock the UCS is generally 20 to 25 times the I_s (ISRM, 1985). Smith (1997) recognizes that this factor is usually less for weaker materials, so provided the test can be run successfully (that is, with the test failure plane passing through both loading platens), his equations may be rearranged to give the following (UCS and I_s are both reported in psi units):

$$UCS = \frac{7.9 I_s}{1 - 0.001725 I_s} \qquad (1)$$

Smith (1997) also suggests using flat platens over the conical platens on the point load device so that loads will be more broadly distributed and weak materials can be successfully tested.

Slake durability may also be predicted by field tests. Santi and Higgins (1998) provide the following predictive equations for second cycle slake durability, $I_d(2)$, based on tests of 100 samples of Colorado shale units:

$$I_d(2) = 16.0(\text{jar slake}) - 1.7 \qquad R^2 = 0.79 \qquad (2)$$

$$I_d(2) = 66.5 - 3.7(Q) - 3.7(\text{hammer rebound}) + 7.2(\text{jar slake}) - 0.5(\text{slake index}) \qquad R^2 = 0.85 \qquad (3)$$

where:

jar slake = rate of change of strength (R) value shown on Table 4.

Q = Norwegian Geotechnical Institute rock mass classification (Barton, et al, 1974). The system integrates both block and discontinuity properties and requires estimates of

number of joint sets, joint roughness, joint alteration, joint water, and a stress reduction factor.

hammer rebound = number value indicating resistance to blows from a geologic hammer (NAVFAC, 1982):

1 – extremely strong (many blows required to break)
2 – very strong (more than one blow required to break)
3 – strong (one moderate blow required to break)
4 – moderately strong (indents 1 – 3 mm with one blow)
5 – moderately weak (crumbles with one blow)
6 – weak (disintegrates with one blow).

slake index = percentage material lost through a #10 sieve after oven drying and soaking in water (Deo, 1972; Chapman, et al, 1976).

Santi (1995) also provides a correlation equation to estimate one-cycle slake index in the field:

$$\text{slake index} = 106.0 - 18.6(\text{jar slake}) \qquad R^2 = 0.85 \qquad (4)$$

Although the jar slake test is convenient to perform in the field, the descriptions for each category are very vague (see Table 4), and as the test is normally performed, readings should be completed at both 30 minutes and 24 hours (Wood and Deo, 1975). This long time period reduces the test's utility as a quick field test. Santi (in review) showed that for fifty samples classified by thirteen different technicians, the 30-minute and 24-hour classifications showed no statistical difference, and the 30-minute test is sufficient. Furthermore, he showed that by more carefully defining the test categories and providing drawings of samples in each category, included as Figure 3, the consistency between different technicians is greatly improved. Use of the jar slake method shown in Figure 3 produced an average of the standard deviations of results from each sample of 0.39. Using the original jar slake descriptions shown on Table 4 produced an average standard deviation of 0.57. This is a reduction in the "error" of the test by one-third.

Lee and de Freitas (1989) provide a simple, rough field test to estimate permeability, shown on Table 7. The test is based on observing the penetration of a few drops of water from a dropper onto a hand specimen, over a period of one minute. The test was designed for weathered granite blocks, but should also apply to shales and other weak materials.

Table 7. Field Estimate of Permeability (from Lee and de Freitas, 1989).

Class	Description	Estimated Permeability
1	Specimen is almost impermeable, most of the water remains on the surface.	$\leq 10^{-7}$ cm/sec
2	Specimen is slightly permeable, some water is absorbed.	10^{-5} to 10^{-7} cm/sec
3	Specimen is moderately permeable, more than half of the water is absorbed.	10^{-3} to 10^{-5} cm/sec
4	Specimen is highly permeable, most of the water is absorbed.	$\geq 10^{-3}$ cm/sec



1. MUD - degrades to a mud-like consistency.



2. FLAKES - sample totally reduced to flakes.
Original outline of sample not discernible.



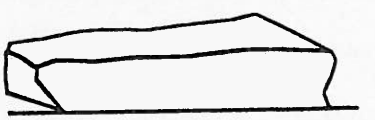
3. CHIPS - chips of material fall from the sides of the sample. Sample may also be fractured. Original outline of sample is barely discernible.



4. FRACTURES - sample fractures throughout, creating a chunky appearance.



5. SLABS - sample parts along a few planar surfaces.



6. NO REACTION - no discernible effect.

Figure 3. Modified Jar Slake Test Categories and Descriptions (from Santi, in review).

CONCLUSIONS

Adequate engineering of shales, weak rock, and weathered rock begins with adequate recognition of potential problems in the field. The tools presented in this paper are intended to help the practitioner recognize potential slaking and reactivity to water, low strength, expected strength loss over time, influence of strong corestones and weak matrix, and expected behavior for various engineering uses. The classification system presented is based on a broad review of existing systems, and is intended to produce a thorough overview of a particular material's properties. Finally, several predictive equations will provide the user with a field estimate of strength and durability properties.

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UNCONVENTIONAL EARTHWORK CONSTRUCTION MINIMIZES UNDERCUTTING

by

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ABSTRACT: A major expansion of a paper mill in the Southeast U.S. in 1996 and 1997 included construction of a Waste Fuel Storage Area (WFSA) and an Activated Sludge Aeration Basin (ASAB). Both were constructed in areas including wetlands containing compressible organic and soft clayey soils and having a groundwater table near the ground surface. The WFSA was constructed without any undercutting and utilized a spoil pile located on the site. The ASAB was constructed with minimal dewatering and undercutting by constructing a rock berm water cutoff prior to site clearing and undercutting. The unconventional site grading and earthwork design criteria and construction procedures minimized undercutting, dewatering, and backfilling, resulting in cost savings of approximately $400,000 at the WFSA and $100,000 at the ASAB, as well as saving several weeks of construction time for each facility.

INTRODUCTION

A Waste Fuel Storage Area (WFSA) and an Activated Sludge Aeration Basin (ASAB) were constructed as part of a paper mill expansion in areas including wetlands containing compressible organic and soft clayey soils and having a groundwater table near the ground surface. Spoil materials were used for fill in wetlands without grubbing or undercutting, and a water cutoff barrier was constructed in a water-covered wetlands without undercutting by placing granular fill directly on the vegetation and organic soils. This paper describes the site and subsurface conditions and unconventional site grading and earthwork design criteria and construction procedures that significantly reduced the cost and time of construction of the WFSA and ASAB.

PROJECT INFORMATION

The WFSA was designed to support a conical shaped waste fuel (bark and sawdust) pile surrounding a stacker/reclaimer. The waste fuel pile is formed by a 270^{o} arc with a 240 ft (73.1 m) diameter and has a maximum stack height of 53 ft (16.2 m). The bark and sawdust has a unit weight of 34± pcf (5.3± kN/m^3), so the maximum contact pressure under the waste fuel pile would be about 1,800 psf (86 kN) with an estimated average contact pressure less than 900 psf (43 kN). No surface treatment was planned since no vehicle or equipment traffic is required. The waste fuel is placed and retrieved by the articulating arm of the stacker/reclaimer, which is supported on a large mat foundation bearing on crusher run gravel backfill overlying stiff residual clayey soils. Besides the waste fuel, only a buried fire protection pipeline and small hose houses are located around the perimeter of the WFSA. The surface of the waste fuel pile has a maximum elevation 13± ft (4.0± m) higher than the surrounding wetlands. The entire WFSA has a plan area of 2.1± acres (0.8± ha).

The ASAB has a water surface area a little larger than 1 acre (0.4 ha) and water depth of 26 ft (7.9 m). Generally, the exterior slope of the dike is 3 horizontal to 1 vertical (3H:1V) and the interior slope is 2H:1V, except for the northern portion of the dike which was an existing waste treatment aerated lagoon dike with both interior and exterior slopes of 3H:1V. The bottom and inside slopes of the basin are double lined with a 80 mil HDPE top liner, 60 mil HDPE bottom liner and intermediate geocomposite drainage layer. The double lined basin bottom is underlain by a free draining coarse filter layer that drains to a leachate sump. The bottom and lower area of the inside slopes support lightly loaded pipes for the aeration system. The dike has a 15 ft (4.6 m) wide crest and maximum height of 31± ft (9.4± m) above original grade. The overall plan area of the basin, including the new dike, is 3+ acres (1.2+ ha).

WASTE FUEL STORAGE AREA

Site Conditions

A spoil pile, constructed over 20 years ago during original site grading and foundation construction for the existing mill, covered 1.3± acres (0.5± ha) within the area of the proposed WFSA. The spoil pile varied in height from 12 to 35 ft (3.7 to 10.7 m) above the natural ground surface and wetlands permitted for construction and was covered with trees and underbrush. The area surrounding the spoil pile was a low marshy wetlands containing a heavy growth of vegetation, including numerous large hardwood trees with extensive root systems. The wetlands, which comprises about 0.8 acre (0.3 ha) of the WFSA, often floods during heavy rainfall due to runoff from the mill.

Geotechnical Exploration

Soil test borings, hand auger borings, and trackhoe test pit excavations were performed within or adjacent to the proposed WFSA. The borings and test pits explored both the virgin soils beneath and outside the limits of the spoil pile, as well as within the spoil pile. Laboratory testing conducted on samples of both the virgin soils and spoil materials included Atterberg limits, grain size analysis, natural water content, compaction (Proctor), organic loss on ignition, triaxial shear, and consolidation tests.

Subsurface Conditions

The spoil pile was composed primarily of slightly to moderately plastic clayey soils (USCS symbol CL) and topsoil. Some areas of plastic clays (CH) were also present. The soils were typically slightly wet of the Proctor optimum moisture content. Except for an occasional localized concentration of highly organic material, the soils contained organics well below the normally acceptable limit of 3 to 5%.

The virgin soils underlying the spoil pile and the root mat and topsoil layer around the spoil pile were primarily soft to very stiff plastic clays (CH) containing variable quantities of chert fragments. Limestone bedrock was encountered at a depth of 75 ft (22.9 m) below the natural ground surface. The groundwater level was measured between 1 and 2 ft (0.3 to 0.6 m) below the ground surface during the geotechnical exploration.

Evaluation

Initially, after confirming that much of the WFSA contained a spoil pile, plans were to excavate and haul the material to an on-site landfill. At that time, it was assumed that the materials had a high moisture content and were highly organic and highly plastic. Initial plans also included clearing and grubbing the site, undercutting the soft soils in the wetlands area, and hauling most of those materials to the landfill. Dewatering of the undercut excavation by pumping from sumps would be required prior to placing a bridge lift of stone or layer of heavy geotextile fabric. Then the excavation would be backfilled and the embankment for the WFSA constructed utilizing soil from an on-site borrow area. Construction of the WFSA utilizing the above procedures was going to be very expensive. Besides cost, there were major potential problems associated with undercutting in the wetlands area. It would be very difficult to excavate the material due to the inability of the ground to support equipment. Timber mats would have to be used to support trackhoes or draglines to grub and undercut the area.

It was concluded that neither total nor differential settlement of the WFSA was critical, as long as the embankment did not have a bearing capacity or slope failure. Much of the WFSA had been pre-loaded for 20+ years by the weight of the spoil pile. The main area of concern was the wetlands where several feet of fill would be required to construct the embankment.

In 1971, prior to the common use of geotextiles, the author had been involved with a project where similar site development conditions existed, except that the upper soils consisted of several feet of a much more compressible highly organic peat. The project included a paved parking lot which was cleared and filled without grubbing, undercutting, or surcharging with additional temporary fill. Even after several years of use, the parking lot had not required significant repairs.

Considering the function of the WFSA and previous experience with similar site conditions, the following design criteria and construction procedures were developed:

1. The spoil pile materials, except for highly plastic clays and significant organics [stumps, logs, limbs, and roots larger than 2 in. (51 mm) in diameter], and other borrow materials satisfying the same criteria would be acceptable for the embankment fill.

2. All embankment fill placed in the wetlands area, except the lower 1.5± ft (0.5± m), would be compacted to at least 92% of the standard Proctor (ASTM D 698) maximum dry density. There was no compaction criteria for the lower 1.5 ft (0.5 m) of fill.

3. The perimeter of the embankment would have a final slope of 3H:1V. The contact slope of the spoil pile and the new embankment fill would not exceed 4H:1V.

4. All vegetation would be cleared and stumps and roots grubbed and removed from the surface of the spoil pile.

5. In the wetlands area, all trees and brush would be cut to within 6 in. (152 mm) of the ground surface. Grubbing of stumps and roots and operation of equipment in the wetlands area would be prohibited, resulting in minimal disturbance to the root mat.

6. An initial 1.5± ft (0.5± m) thick lift of fill material from the spoil pile would be spread over the wetlands area by low contact pressure equipment pushing the material ahead of it. No high contact pressure equipment would be allowed on this initial lift. If the initial lift of fill would not support the construction equipment, either a bridge lift of stone or a geotextile fabric would be placed over the wetlands area prior to proceeding with fill placement.

7. After the initial lift of fill was placed, the embankment would be constructed in lifts not exceeding 8 in. (203 mm). Material from the spoil pile would be pushed, spread, and compacted by dozers and self-propelled sheepsfoot rollers with push blades.

Construction

Initially, all vegetation was cleared and stumps and roots grubbed from the surface of the spoil pile utilizing track-hoes and dozers, as shown in Figures 1 and 2. In the wetlands area, manual labor was used to cut, pile, and burn the smaller trees and underbrush. The larger trees were also cut down by manual labor using chain saws, but the tree trunks were winched to the spoil pile by timber skidders and disposed of in the on-site landfill. As shown in Figure 3, all stumps were cut to within 6± in. (152± mm) of the ground surface.



Figure 1. Clearing the Spoil Pile at the WFSA



Figure 2. Spoil Pile Cleared and Grubbed



Figure 3. Cutting Stumps Close to Ground Surface in Wetlands

An initial 1.5± ft (0.5± m) thick lift of fill was placed over the wetlands area by dozers pushing material from the spoil pile. No stone bridge lift or geotextile fabric was required. After the initial lift, self-propelled sheepsfoot rollers with push blades were used in addition to the dozers to push, spread, and compact the spoil material in thin lifts until the embankment was completed. Embankment construction is shown in Figures 4, 5 and 6. Most of the spoil material was acceptable for the embankment fill and readily compactible. Occasional pockets of wet or highly plastic clayey soils and highly organic soils and large pieces of trees or roots were removed and spoiled in the landfill. Some of the more clayey spoil material having a water content wet of the optimum moisture content was sensitive to excessive compaction and would pump under the compactive effort of the large sheepsfoot rollers. When this occurred, either the pumping material was excavated and spoiled or allowed to dry before proceeding with the next lift. Approximately half of the fill was placed in November 1996 for the embankment in the wetlands area, and the embankment was essentially completed, except for final grading, in May 1997. The WFSA received waste fuel and started operating in November 1997. Figures 7 and 8 show the operational WFSA.

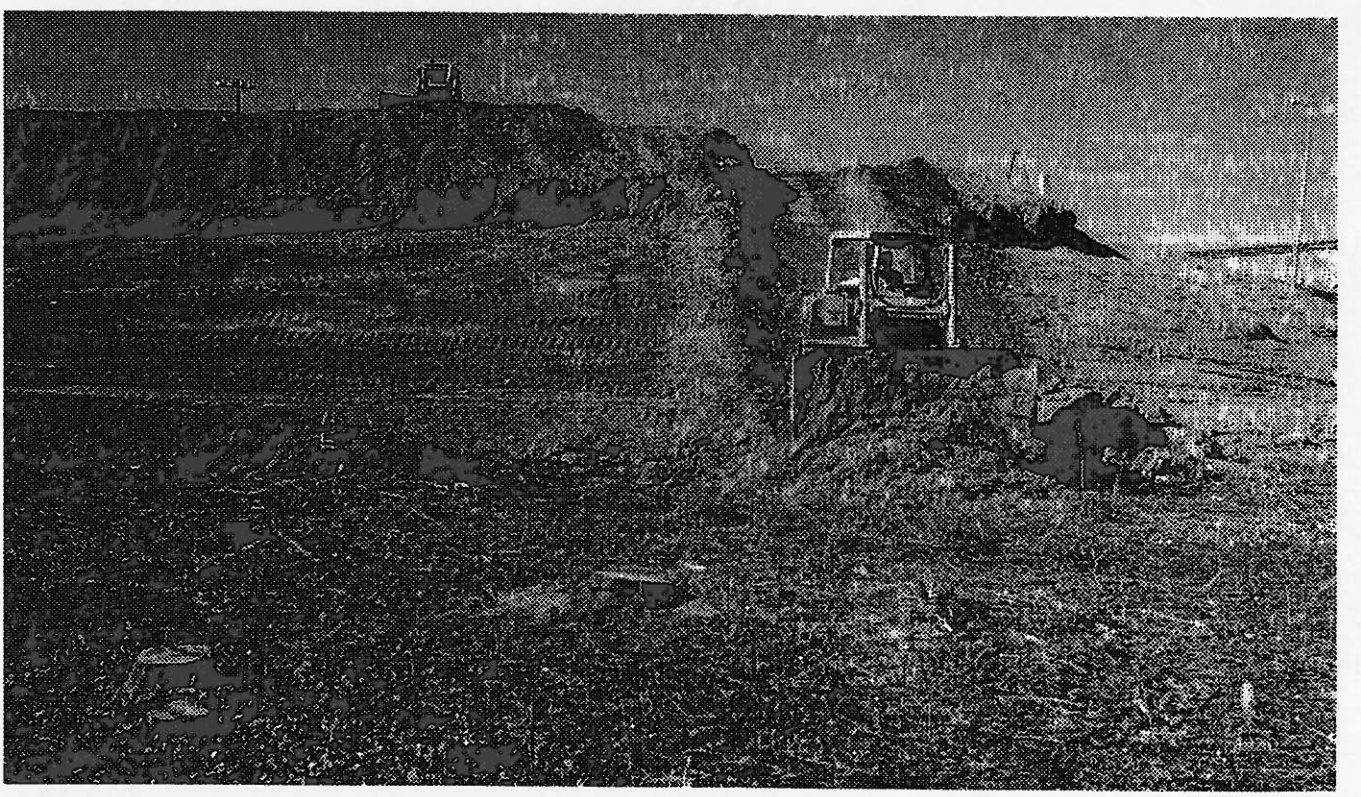


Figure 4. Spreading Initial Lift of Spoil Material in Wetlands



Figure 5. Spreading and Compacting Spoil Material in Wetlands



Figure 6. Completed Spoil Material Embankment in Wetlands



Figure 7. Grassed Slope of Spoil Material Embankment in Wetlands

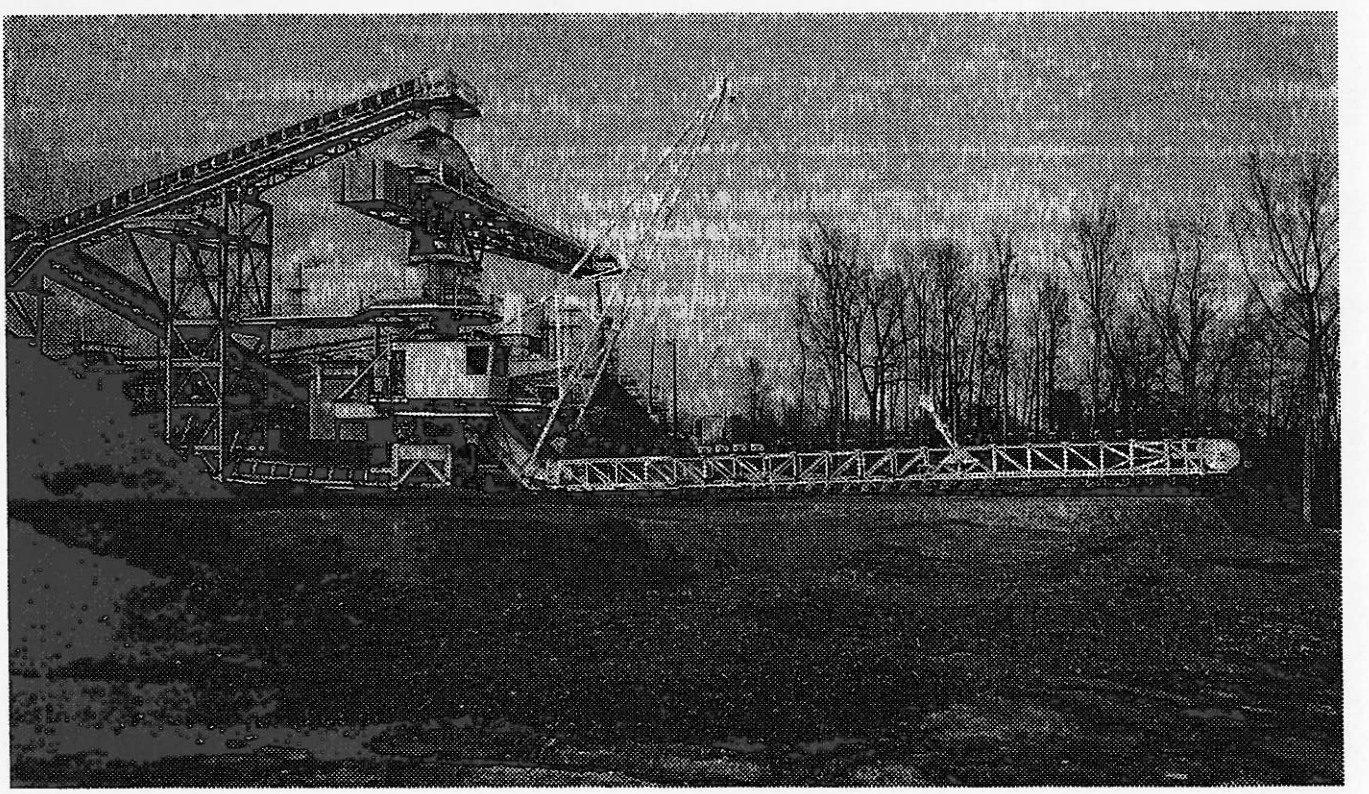


Figure 8. WFSA Operational Showing Low Waste Fuel Pile, Stacker/Reclaimer, and Conveyor System

No settlement measurements were made of the embankment fill either during or after construction. However, no visual signs of distress were noted in the partially completed embankment, such as dishing due to excessive settlement, cracking, sloughing, slumping, or sliding during the idle period over fall 1996 and winter 1997; during or after completion of the embankment; or since operation of the WFSA began. Except for some minor erosion on the surface of the slope, the embankment fill, as well as the portion of the embankment in cut in the spoil pile, has performed well.

ACTIVATED SLUDGE AERATION BASIN

Site Conditions

About 1.7 acres (0.7 ha) of the 3± acre (1.2± ha) plan area of the proposed ASAB was a wetlands permitted for construction. The wetlands area was covered with up to 8 in. (203 mm) of water, thick growth of aquatic vegetation, and scattered large hardwood trees. Another 0.6± acre (0.2± ha) was a slightly higher non-wetlands area covered with large pine and hardwood trees and underbrush. The remaining area was the outside slope of the dike of an existing waste treatment aerated lagoon constructed in 1974. All of the land, except the existing dike slope, often floods from rainfall runoff from the mill.

Geotechnical Exploration

Soil test borings and trackhoe test pit excavations were performed within or adjacent to the proposed ASAB. The borings and test pits explored the virgin soils within both the wetlands and non-wetlands areas. The borings were drilled on dry ground; however, the test pits were excavated both on dry ground and in standing water. Figure 9 shows a test pit being excavated by a trackhoe in the wetlands. Laboratory soil testing included Atterberg limits, grain size analysis, natural water content, and compaction (Proctor) tests.



Figure 9. Excavating Test Pit in Wetlands at the ASAB

Subsurface Conditions

The wetlands area contained 1 to 1.5 ft (0.3 to 0.5 m) of very soft organic moderately plastic silty clay (OH) at the surface, underlain by 6 to 12 in. (152 to 305 mm) of soft plastic clay (CH). The non-wetlands area was covered with 6 to 9 in. (152 to 229 mm) of topsoil. Below the soft plastic clay in the wetlands area and the topsoil in the non-wetlands area, the soils were primarily firm to hard plastic clays (CH), containing variable quantities of chert fragments, which extended to the top of limestone encountered at depths ranging from 13 to 19 ft (4.0 to 5.8 m) below grade. During the geotechnical exploration, the groundwater level was as much as 8 in. (203 mm) above the ground surface in the wetlands area and typically within 1 ft (0.3 m) below the ground surface in the non-wetlands area.

Evaluation

Groundwater and surface water control and minimizing undercutting were major considerations during construction of the ASAB. Not only was the groundwater level near the ground surface over most of the plan area, but during the wet season and during periods of heavy rainfall, surface water runoff which is directed across this area from the mill could flood both the wetlands and non-wetlands areas. This flooding could cause rapid rises in the water surface elevation over the site.

Another consideration was the limited work area outside of the ASAB plan area. The wetlands area permitted for construction left only 10 ft (3.0 m) between the boundary limit and toe of the dike. Any structure installed outside of the dike toe to control the groundwater and surface water during undercutting and to limit the lateral extent of undercutting would be restricted to a 10 ft (3.0 m) width. The primary conventional options considered and their drawbacks are presented below.

1. Excavate a trench around the outside toe of the dike and backfill with impermeable material to a height above the anticipated water level in the wetlands area during construction. This would have required that all excavation and backfilling in the trench be done "in the wet". That would result in disturbance and softening of otherwise acceptable soils which would also have to be excavated. All of the excavated soils, most of which would be quite fluid, would have to be hauled to the on-site landfill. The backfilled cutoff trench would have to be stable and wide enough to support large off-road dump trucks. There was also a real potential of undermining or otherwise disturbing the silt fence and compromising the adjacent wetlands that was not permitted for construction. This option was considered prohibitively costly, as well as technically questionable. There were also construction schedule problems with this option.

2. Partial undercutting in the wet in localized areas that would be isolated and separated by pushing up a temporary berm of spoil material and then dewatering, undercutting, and backfilling the local area to a level above the surface water and groundwater level. Then the temporary berm would be removed and another localized area undercut and backfilled in the same manner. This operation would proceed until the entire ASAB area had been undercut and backfilled. Because a lot of work would be done in the wet, there would be excessive undercutting of soft wet soils which would also require replacement with backfill. The wet conditions would also require more extensive use of stone as a bridge lift. This option was also considered prohibitively costly.

3. Construct a driven steel sheet piling or heavy flat timber piling cutoff wall outside of the dike toe. The main purpose of the cutoff wall would be to control the groundwater and surface water during construction. The wall would have to be constructed by equipment working on timber mats which would be moved with the equipment. There was concern about being able to drive or vibrate the piling to the required depth to allow the wall to act as a cantilever resisting both water and soil pressure when undercutting and dewatering was

performed inside the cutoff wall. [As it turned out, rock within 2 to 3 ft (0.6 to 0.9 m) of the ground surface was encountered at more than one location near the dike toe during undercutting.] The cutoff wall would either be left in place or removed after undercutting and backfilling above the water surface adjacent to and along the entire length of the wall. Again, this option was considered to be too costly and also had some potential serious technical problems.

Since the ASAB was the last major facility to be designed and constructed, considerable earthwork experience had been gained during site grading for some of the other facilities. Surge stone [blast rock grading from 15 in. (381 mm) to silt and clay size particles] had been used as a bridge lift in soft wet undercut areas and to widen a road out into an active sludge pond. It was believed that if the surge stone was placed directly on top of the wetlands surface, the stone would work its way into the soft organic clay under the weight of additional stone and the site grading equipment. The clay would tend to work its way into any voids around the larger rock, resulting in a relatively impermeable cutoff around the perimeter of the area that was to be undercut.

To accommodate small construction equipment and to stay within the 10 ft (3.0 m) wide zone outside of the proposed dike toe, an 8± ft (2.4± m) wide rock berm was designed with its outside edge 1± ft (0.3± m) inside the work limits of the permitted wetlands construction area. The top of the berm was to have an elevation at least 1 ft (0.3 m) above existing grade.

Construction

Surge stone was stockpiled at both ends of the proposed rock berm. Then the surge stone was transported in the bucket of a small track-mounted front-end loader and placed on the ground surface. As the berm was extended around the proposed toe of the dike, subsequent passes of the loader tracked in the stone. Periodically, a rubber-tired backhoe working on top of the berm would pick up the larger rocks that had rolled outside of the construction limits and shape the edges of the berm, maintaining a width of 8 to 9 ft (2.5 to 2.7 m). Additional stone was placed and tracked in until the berm was completed. Then the silt fence was constructed much more easily and efficiently by working off of the berm. A ditch was excavated through the organic soils and soft clay adjacent to and along the inside of the berm using a rubber-tired backhoe. A trackhoe excavated lateral ditches across the wet areas. Since the berm was relatively impermeable, it was possible to pump out the surface water and groundwater from inside of the berm until the groundwater was lowered below the required depth of undercutting. The proposed dike and pond area was cleared with small tracked equipment and manual labor using chain saws. Then the organic soils and soft clay were undercut and spoiled using large trackhoes, dozers, and off-road dump trucks. The undercut areas were backfilled with cherty clay transported in scrapers and off-road dump trucks from an on-site borrow area. The backfill was spread and compacted with dozers, graders, and self-propelled sheepsfoot rollers. A few localized wet areas were bridged with 1± ft (0.3± m) of surge stone. The dike was also constructed of the cherty clay. Figures 10 through 14 show earthwork construction at the ASAB. Construction of the rock berm and clearing of the site began in May 1997, and earthwork was essentially complete by August. After construction of the liner and aeration system, the ASAB was placed in service in November 1997. The outside slope of the completed dike and the adjacent wetlands are shown in Figures 15 and 16.



Figure 10. Constructing Rock Berm in Wetlands



Figure 11. Rock Berm Under Construction in Wetlands



Figure 12. Rock Berm and Inside Perimeter Drainage Ditch in Wetlands

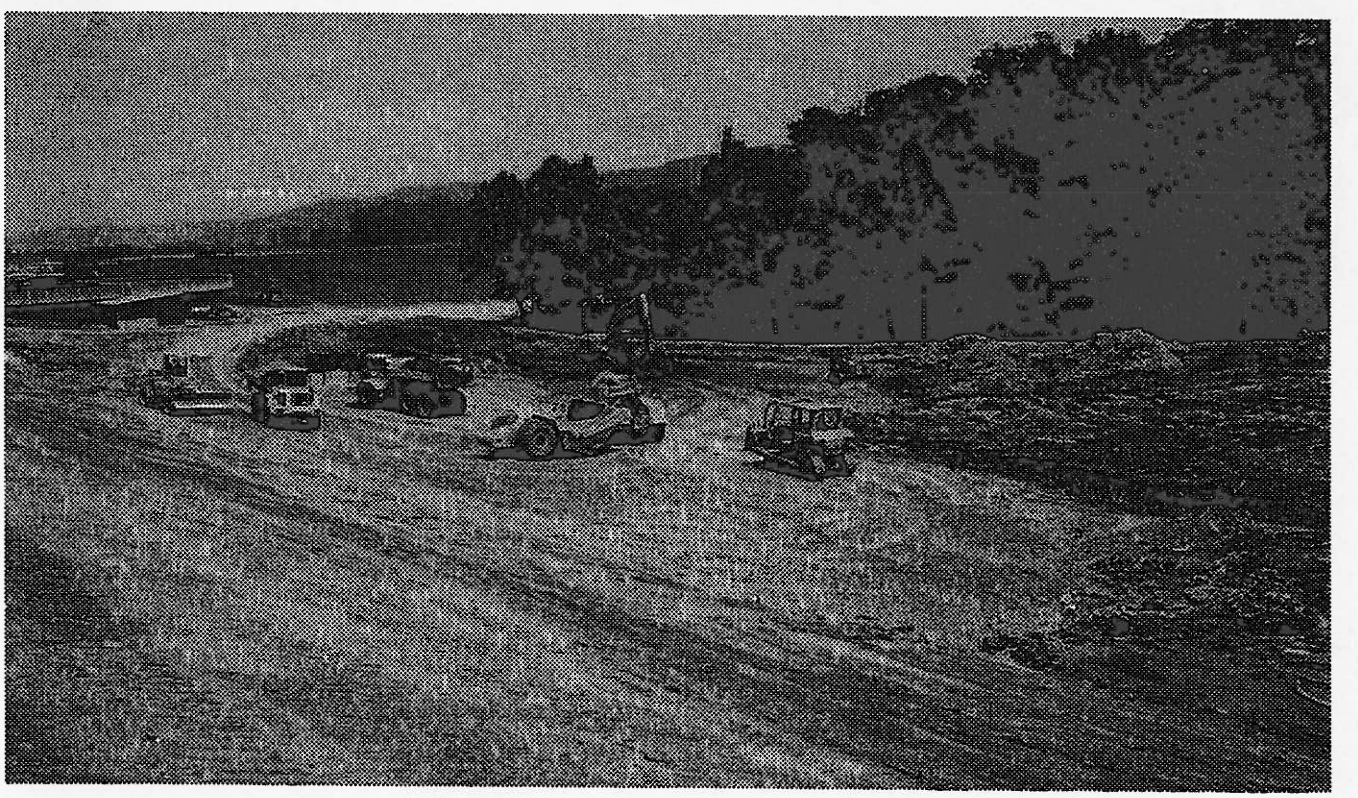


Figure 13. Undercutting and Backfilling "in the Dry"



Figure 14. First Lift of Backfill in Wetlands

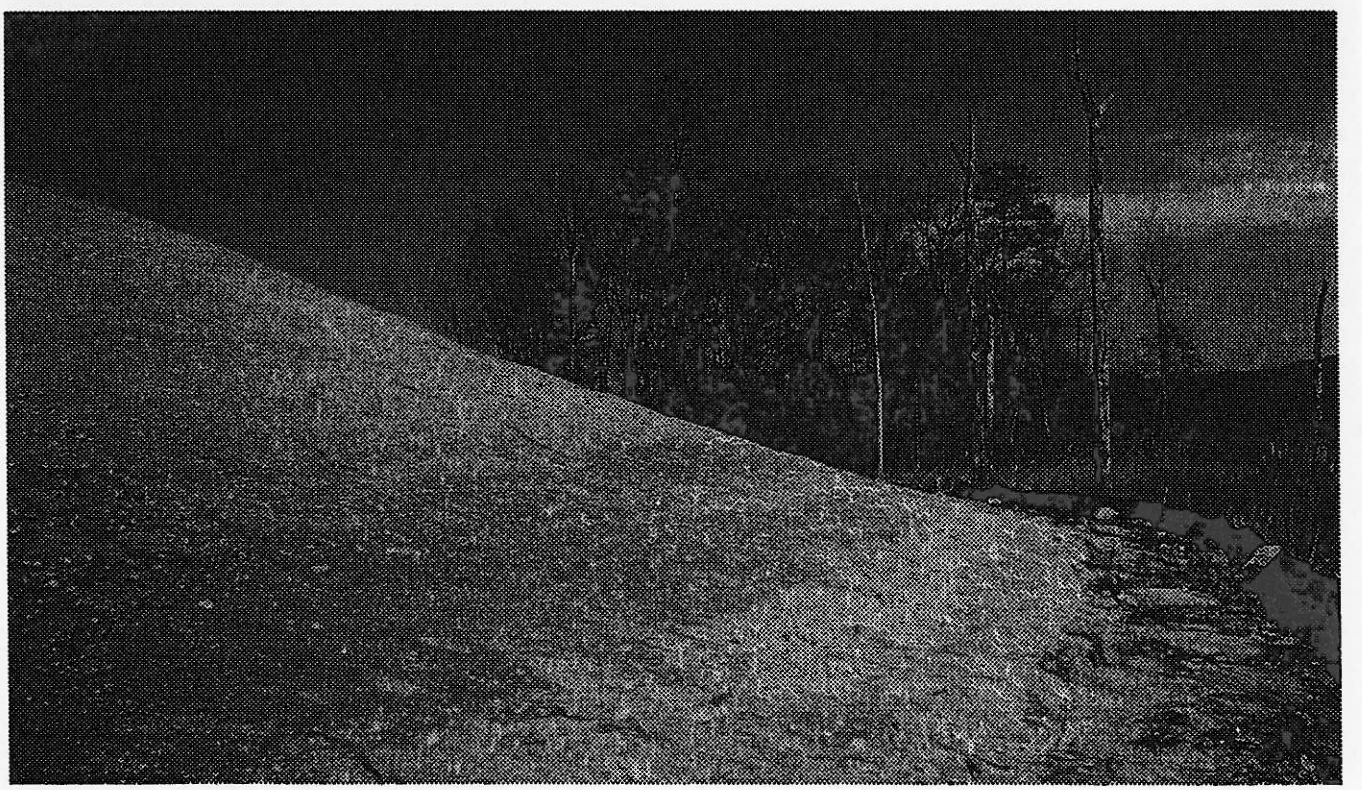


Figure 15. Completed Dike and Rock Berm in Wetlands

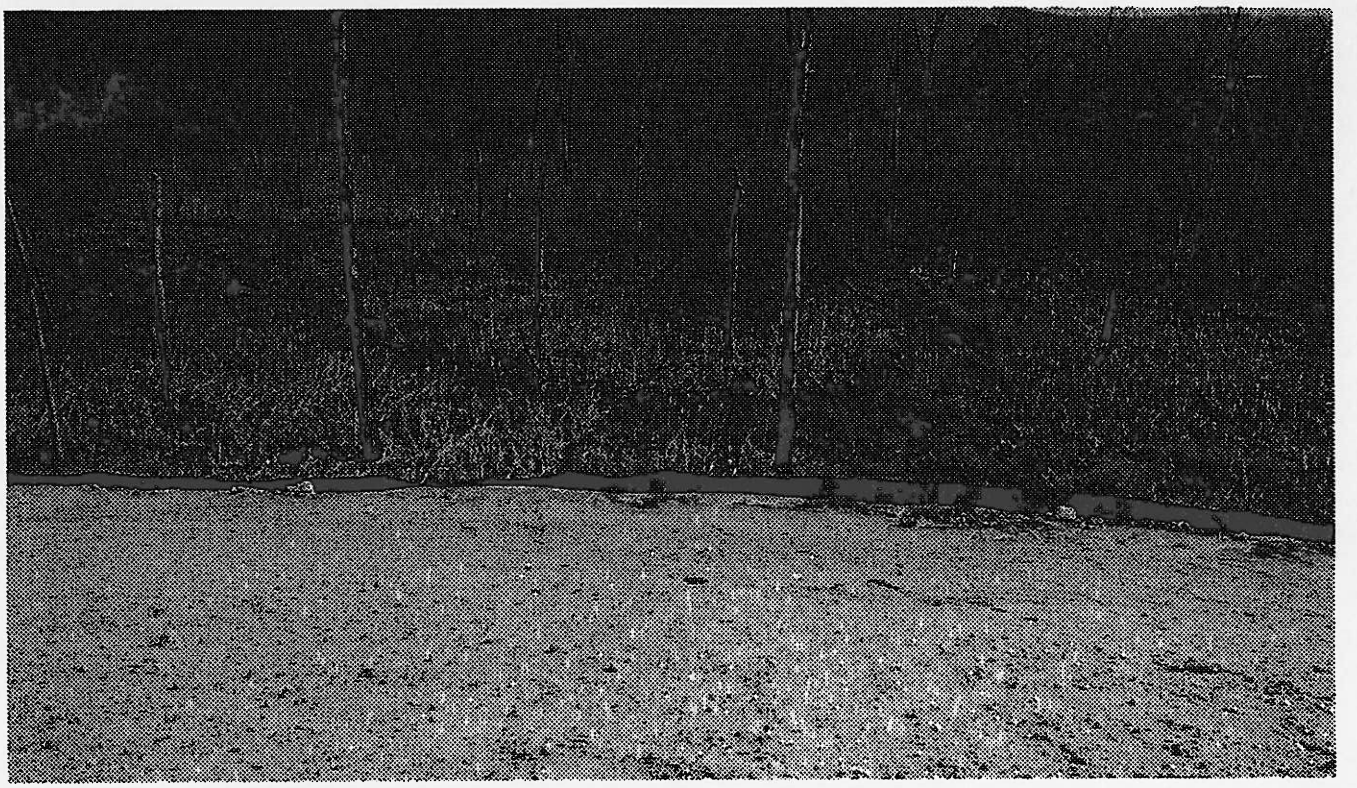


Figure 16. Looking Down Slope of Completed Dike Towards Wetlands

The rock berm worked well as a surface water and groundwater cutoff. Any localized seepage was easily controlled by pumping from a small temporary sump placed in the ditch excavated along the inside edge of the berm. All undercutting and backfilling operations were performed essentially "in the dry". Even though it was not a design consideration, the rock berm also provides some additional stability to the dike toe, as well as erosion protection during heavy flooding.

CONCLUSIONS

The following site grading and earthwork construction was performed that would be considered unconventional by most engineers:

1. Clearing only, without grubbing and undercutting, a soft wetlands area that would support an embankment fill several feet thick.

2. Using spoil material, including topsoil and plastic clay, as embankment fill.

3. Constructing a surface and groundwater cutoff by dumping granular material in water and forcing it into organic and soft clayey soils with only the compaction of the tracked equipment.

Using unconventional site grading and earthwork design criteria and construction procedures minimized undercutting, dewatering, and backfilling, resulting in substantial cost savings and construction time for both the WFSA and the ASAB. It is estimated that approximately $400,000 was saved at the WFSA and $100,000 at the ASAB, as well as several weeks of construction time for each facility. However, besides savings in construction cost and

time, the site grading and earthwork construction procedures used, although considered unconventional, probably had fewer potential problems that could have resulted in substantial cost and schedule overruns. Conventional site grading and undercutting in the wetlands areas at the WFSA and ASAB could have resulted in serious cost increases and delays if heavy rainfalls had occurred during construction. In the ASAB, the alternative system of a sheet pile wall to isolate and allow dewatering in the area of undercut likely would have encountered problems installing the piles deep enough to perform as a cantilevered wall.

Engineers designing earthwork should always consider the function of the earthwork. For example, embankment fill supporting spread foundations should consider bearing capacity and settlement of the fill, as well as the overall stability of the embankment. However, for fill supporting a yard or storage area, such as the waste fuel pile, the main concern is the stability of the embankment; the amount of settlement is of little concern. After determining the requirements or design criteria for the earthwork, then the engineer should attempt to design the most economical embankment. This usually results in minimizing improvement of the subgrade soils supporting the embankment and constructing the embankment of on-site or locally available materials that the local earthwork contractors have experience with, and the equipment to transport, spread, and compact to the required specifications.

PAPA JOHN'S CARDINAL STADIUM
FOUNDATION CONSTRUCTION OVER 100 YEARS
OF RAILROAD REPAIR SHOP DEBRIS

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ABSTRACT

The site of the University of Louisville Papa John's Cardinal Stadium was formerly the "South Louisville Shops" of the L&N Railroad; now part of the CSX system. Ground cover for the 43-acre parcel included floor slabs, pit slabs, basement slabs, railbeds, foundations, pit walls, tunnels, and surface paving resulting in about 85 percent of the area comprised of "hard cover."

Concurrent with the geotechnical exploration and design phase were: the crafting of the real estate transaction agreement known as the "land-swap agreement;" the completion of the Environmental Site Characterization and Risk Assessment; and initial work on the Remedial Action Plan, the Risk Management Plan, the Environmental Health and Safety Plan, and the Oil Recovery Plan.

This paper discusses the impact of these documents and the actual site conditions on the three critical geotechnical and construction issues:

- Site development and demolition of existing hard cover
- Foundation system selection
- Estimating and managing settlement of grade-supported seating

INTRODUCTION

The purpose of most geotechnical explorations is to obtain site specific subsurface data and to develop foundation and site preparation recommendations. The geotechnical exploration for the new University of Louisville Papa John's Cardinal Stadium was no different. However, the former use of the site resulted in site contamination, the occurrence of man-made structures, and the occurrence of man-made site materials that can be called "problematic geotechnical materials." This presentation focuses on the impact to foundation selection and settlement management due to site contamination and man-made structures and materials.

BACKGROUND

In 1992, during the search for a location for a new stadium for the University of Louisville (U of L), the former "South Louisville Shops" site of the L&N railroad, now CSX Transportation System, was considered. In February 1993, when this site became one of the preferred sites, Law Engineering and Environmental Services, Inc. (LAW) was retained to provide environmental oversight consulting services to U of L. Due to the complexities of construction on "contaminated" sites those services also relied on LAW's understanding of the construction process. During the course of the stadium project, these services included reviewing environmental documents dated as early as a 1989 Phase I Environmental Site Assessment to the 1998 Remedial Action Progress Report.

Environmental documents developed which impacted design and construction decisions included:

Environmental Site Characterization (ESC)
Environmental Risk Assessment (RA)
Remedial Action Plan (RAP)
Environmental Health and Safety Plan (EHASP)
Oil Recovery Plan

These and some companion environmental documents were contractually bound to the design and construction project by the real estate transaction document known as the "land-swap agreement."

The civil works design team consisting of:

Luckett & Farley, Inc. – Architect / Engineer
Rosser International – Stadium Consultant
U of L Department of Design, Planning & Construction – Owner
U of L Department of Environmental Health & Safety – Environmental Oversight
Huber Hunt & Nichols – Construction Manager

Law Engineering and Environmental Services, Inc. – Geotechnical/Environmental Consulting was required to design and develop a construction process within the environmental and site parameters.

A wealth of subsurface data was obtained during the development of the ESC: the overburden composition was described, the depth to rock was mapped; the rock type was determined; and the groundwater surface was mapped. Direct sampling and monitoring as well as geophysical methods were used.

Rock was measured at depths ranging from 34 to 65 feet or more, and a deep clay-filled trough in the rock surface was mapped. In addition, it was believed that the clay layer prevented downward migration of site contaminants and that any penetrations of this layer must be sealed to prevent creating a vertical conduit for contaminant migration.

The land-swap agreement required all materials, with little exception, be kept on site. This requirement was also echoed in the RAP. The RAP, RA, and EHASP set forth worker protection, material handling, dust suppression, surface and groundwater management, site control, and environment safety training requirements. Additionally, a vapor management system and an oil recovery system were mandated.

CHARACTERIZATION INFORMATION

The proposed stadium site is located at the Old South Louisville Shops of the CSX (formerly L&N) railroad. The site subsurface characteristics have been environmentally assessed by Resource Consultants, Inc. (RCI) under contract to CSX. LAW provided environmental oversight to the university during RCI's assessment. Some geotechnical data was obtained during the environmental characterization; this data consists of:

- Depth to rock (Figure 4)
- Penetration resistance of soil
- Soil description
- Dutch cone soundings
- Electromagnetic and magnetic surveys
- Seismic refraction survey
- Depth to groundwater
- Groundwater flow direction

This data was gathered to determine the general characteristics of the site. The location and frequency of testing as it relates to the stadium structure was somewhat limited with the exception of depth to rock and thickness of a lower level clay layer. The environmental site studies characterized the overburden into three strata based on statistical distributions of contaminants. A geotechnical review of this data characterized the site subsoils into two different stratagraphics:

Stratagraphy One: 6 feet of fill underlain by various layers of clay and sands to refusal depths of 40 feet of less.

Stratagraphy Two: 6 feet of fill, underlain by various layers of clay and sand to about 40 feet, underlain by plastic clay to refusal.

Refusal depths varied from 34 to 64 feet.

SITE GEOLOGY

A review of published geologic data for this area indicates sections of the site are underlain by glacial outwash consisting primarily of sand and gravel with silt and clay mixed. The Ohio River marks the southern boundary of the Pleistocene Glaciation, which resulted in outwash deposits by a slow moving river following the glacial melt. The elevation of the outwash plain generally lies between 450 and 470 feet National Geodetic Vertical Datum (NGVC). The sand and gravel are light brownish gray to light reddish brown. Varying elevations of the Ohio River and recurring flooding have deposited fine-grained soils on the majority of the river valley. Generally, the upper portions of the soil deposited in the river valley consist of the clays and silty clays.

The site is underlain by the Sellersburg and Jeffersonville Limestones of the Devonian age. The Sellersburg Limestone is light gray in color, and consists of coarse to very coarse fossil fragments in a fine-grained matrix. The Jeffersonville Limestone is light brownish gray to light gray in color, and consists of medium–to fine-grained fossil fragments in a matrix of calcite and calcareous mudstone. These formations are typically susceptible to differential weathering sometimes resulting in rock pinnacles and soil-filled slots.

DESIGN INFORMATION

Initial structural information provided at the time of the exploration indicated site development would include a playing field elevation of about 455 feet NGVD or about 6 feet below, existing average site grade (Figure 2).

The stadium will consist of two separate seating level structures. The lower portion of the seating structure will have 20 rows of seats supported at grade. The remaining 40 rows of seats will be structurally supported on three rows of columns and a grade beam at column line B (Figure 3). The lower seats will be supported on a soil berm about 280 feet wide at its base and about 15 feet tall.

The initial construction will include a second level only for the west stands. However, the design includes second levels for the south and the east stands. The initial construction of the second level will include only the press, club, and private box levels and a roof.

Structural loading conditions for the project site were provided by Luckett & Farley, Inc., and included the following:

Stadium Columns	Maximum design loads on the E, F, G column lines are 3,500 kips; maximum design loads on other column lines are 2,400 kips.
Soil-Supported Stands	The lower 20 rows of seats will be soil-supported with the higher; elevations supported on the adjacent stadium structure.
Athletics Building	Maximum column loads of up to 350 kips are expected and the structure will be somewhat settlement sensitive.

EXPLORATION APPROACH

Based on this information and preliminary design decisions, an exploration approach was developed to address two critical issues:

1) The compressibility of the existing site soils of the grade-supported seating and concession area

2) The rock characteristics and selection of feasible rock-supported foundation alternatives

The exploration included 24 soil test borings, five of which included sampling the refusal materials using rock coring methods (Figure 1). For environmental reasons, all borings were backfilled with cement-bentonite grout mixture.

The data obtained during LAW's geotechnical exploration combined with the previous information concluded that two different stratagraphics must be used to analyze the settlement potential of grade-supported structures (the stands being the most critical), and three different rock conditions must be evaluated for rock-supported foundations. The variation in subsurface stratagraphy was due to depth to rock and thickness of compressible, deep clay layer. The variation in the rock conditions was based on depth to rock and rock competency. Generally, where rock was encountered at depths of less than 36 feet and greater than 55 feet, rock quality was good. Where rock was encountered between 36 and 55 feet, rock quality was variable and difficult to predict.

Experts from the "Report of Geotechnical Exploration, University of Louisville Stadium" which describe the soil layers are as follows:

Beneath a varying surface cover of weeds, asphalt, or concrete, our borings encountered a layer of fill material (Stratum I). This fill material consisted of cinders and/or ash from past railroad activities. This fill material was encountered from 0 to 6 feet in depth and was stiff to very stiff in consistency. Standard penetration resistances (N-values) varied from 4 to 30 blows per foot (bpf), with an average of about 13 bpf.

Beneath this fill material, Stratum II soils were encountered in our borings. These soils consisted of brown and gray with green mottles, silty, lean clays and clayey silts with oxide nodules and occasional sand lenses. The consistency for this stratum varied widely from very soft to very stiff. Stratum II soils generally extended to depths of approximately 10 to 20 feet. N-values varied from 1 to 23 bpf, with an average of about 7 bpf.

Stratum III soils consisted of clayey sands and fine sands with fine gravels of firm to very firm consistency. Occasional pieces of weathered shale were encountered in this stratum. Stratum III generally extended to auger refusal, where rock was encountered at depths of less than 40 feet, or to depths of about 40 feet where rock was deeper. N-values ranged from 2 to 49 bpf, with an average of about 17 bpf.

Stratum IV soils were encountered in some of the athletic building and northern stadium borings and consisted of gray, fat clay with fine sand lenses at various depths. Stratum IV soils were encountered from approximately 40 feet to refusal. The consistency of these soils varied from firm to very stiff. N-values ranged from 1 to 25 bpf, with an average of about 11 bpf.

Refusal materials were sampled in five borings and consisted of hard, slightly weathered, gray limestone. No shale refusal material was observed during the exploration. However, shale fragments were noted in the soils during our soil test borings. We believe that shale may have been present on the site during geology history. The shale appears to have eroded away during geologic time. The base of the New Albany Shale Formation may be present in limited areas of the site. Sample recovery ranged from 60 to 100 percent. The Rock Quality Designations (RQD) ranged from 33 to 90 percent indicating poor to excellent rock quality from a geotechnical standpoint. The depth to refusal was consistent with RCI's top of rock data. In general, refusal depths were deeper on the northern half of the proposed stadium site due to the presence of a trough traversing northeast to southwest under the project site. Rock quality was generally good to excellent in the relatively shallow or deep areas to refusal; however, poor rock quality was observed (RQD 33 percent) in the sloped or "transition" areas from shallow to deep refusal depths.

The attached appendix contains four test boring records and other select information from that report.

ENVIRONMENTAL RESTRICTIONS

The ESC identified at least 47 contaminants of concern. Further evaluations in the RA and the RMP concluded that capping of the site by the stadium construction and maintaining an oil recovery system would protect the environment and the facility users. The RMP and EHASP concluded that the construction workers should take certain precautions to guard against dermal contact with diesel oil and incidental ingestion and inhalation of lead-or arsenic-contaminated soil or dust.

The land-swap agreement and the RAP contained four critical requirements:

- Reuse all materials on site
- Protect the lower clay layer from the formation of preferential vertical flow paths from new foundation construction
- Protect the lower clay while disturbing old foundations
- Allow for the installation and operation of an oil recovery system concurrent with stadium construction and operation

SITE DEVELOPMENT

At least 85% of the surface cover within the stadium footprint was comprised of concrete slabs of unknown thickness, pits and pit walls, basements, utility tunnels, piping, sanitary and stormsewer systems, and asphalt paving. In the authors' experience, these materials, are typically removed from structural areas due to their unknown characteristics. However, on this site, due to the environmental concerns and the construction of a 15-foot-high soil berm, it was decided not to remove these old structures from beneath the berm or fill areas. A requirement to break these structures to a maximum horizontal dimension of 4 feet was established in an effort to remove any temporary bridging capabilities these systems may have.

Using these time and cost saving measures, over 40,000 cubic yards of concrete was crushed and reused on site, and an additional 40,000 cubic yards or more were broken into useable pieces and used as fill in parking areas.

FOUNDATION SELECTION

The expected heavy column loads (up to 3,500 kips) required the use of rock-supported foundations. Feasible foundations alternatives had to satisfy four critical geotechnical and environmental criteria:

- High individual unit capacity (at least 100 tons)
- Installation process that protected the lower clay layer
- Flexible design length due to rock surface elevation and rock quality variations
- Reliable and timely installation process

In addition to these criteria, all feasible foundation alternatives must also be cost effective and must have an installation process that can penetrate upper elevation obstructions (known or unknown foundations or slabs).

Steel H-piles, drilled shafts (caissons) and auger-cast piles were considered technically feasible. Steel piles were eliminated due to the owner's experienced with uncertain availability and environmental uncertainty with the creation of a preferential vertical flowpath (protection of the lower clay layer) along the web of the H-section. The

authors' experience indicated a high likelihood for a "soil plug" to form between the web and the flange, creating a void in the soil mass.

Drilled shafts (caissons) supported on the underlying limestone at a relatively high contact pressure (150 kips per square foot) looked to be very economical. However, the unknown cost of handling potential contaminated inflow water and the uncertain environmental risk and inherent danger of having workers in the excavation which may contain vapor coupled with the potential rock removal cost, overruled their used.

Rock-supported, end-bearing auger-cast piles with a design capacity of 159 tons were selected. An economic analysis indicated that piles of this capacity would be slightly less costly than the drilled shafts. The volume of auger cuttings, while greater than the drilled shaft cuttings, would be easier to handle. In the authors' experience, the use of end-bearing auger-cast piles in the Louisville area is not wide-spread. In addition, the occurrence of poor quality rock in the transition zone and the uncertain "downdrag" forces along the piles demanded judicious use of a load testing program. Three or more test piles: one in the shallow rock area; one in the deep rock area; and one in the transitional rock area were specified. The Kentucky Building Code requires loading test in accordance with ASTM D-1143. Due to the high unit load capacity, we mandated test loading of 3 times the design load… "to ensure design assumptions are valid."

SETTLEMENT PREDICTIONS

Using the two generalized subsurface stratagraphies shown on Figures 8 and 9 and the results of laboratory consolidation tests, settlements of the top of earth berm, edge of earth berm and for a soil-supported footings were calculated using consolidation theory for the clay layers and Schmertmann approximations for the sand layers. Our calculations indicated as much as 5.5 inches of settlement to the lower clay layer due to the weight of the soil berm and grade-supported stands. The edge of the berm would produce about 1 inch of settlement in the clay layer.

Those calculations were considered somewhat suspect because the distribution of natural materials was variable and because it was unclear what impact the old foundation systems, pit walls, slabs, utility tunnels and other man-made objects would have on the distribution of stresses. It was determined that even if the evaluations were "off" by 50 percent, the resulting differential settlement between the grade-supported stands and structurally-supported stands could not be tolerable.
A settlement reduction program was developed which included partial demolition of existing structures, proper backfilling of pits and trenches, and preloading the embankment (top and seating side). A preload of at least 450 pounds per square foot (psf) and a detailed monitoring system was recommended. A preload period of 2 to 6 months was expected to reduce post-construction settlement to less than 1 inch.

CONSTRUCTION PROCESS

The site work contractor, RAM Engineering and Construction, used special truck-mounted guillotines to break the existing concrete structures to maximum horizontal dimension of 4 feet or to aid in excavation and crushing or excavation and reuse. The concrete crushing operations provided 40,000 cubic yards of high quality fill. However, the unusual size and quantity of reinforcing steel (occasional railcar axials) caused considerable damage to the crushers. Although considerable surface water problems were experienced, little groundwater problems were encountered.

The pile load test program proved beneficial and problematical. A total of 4 load tests were conducted. The initial "transition" pile tested "failed" at a load of 400 tons, or 2.5 times the design load. The top of the second transition pile broke at a load of 464 tons, or 2.9 times the design load. A reaction pile failed at a test load of 400 tons, or 2.5 times the design load, on the deep test pile. Due to the apparent limiting capacity of the load test system and the structural capacity of the piles, the load test of the shallow rock test pile was terminated at 400 tons.

Additional rock core borings were drilled around the failed pile to determine the method of failure. The actual failure mechanism was unclear and was thought to be either failure in the pile shaft (considered most likely) or shear at the rock contact. It was decided that additional redundancy was necessary, and all "two-pile" groups in the transition zone were changed to "three-pile" groups, and all groups with more than 5 feet difference between pile tip elevations would be reviewed by the geotechnical engineer and structural engineer. The review process resulted in the addition of piles in about 8 instances.

During the construction of the earth berm, activities were hampered with extended periods of wet weather. The site development contractor used on-site crushed concrete and cinders as well as imported soils. Mostly coarse-grained soils were used due to availability and weather conditions. Due to construction sequencing requirements the berm was built to the bottom of "A.8" line foundation elevation, auger-cast piles installed, the berm completed, and the preload placed.

The settlement monitoring system consisted of eight arrays of Kentucky Method settlement monitoring plates: some placed on the natural soils, some at mid-height of the berm and some at finished berm elevation. Preloading originally consisted of more than 5 feet of crushed stone. Berm construction and preloading sequenced from the northwest stadium corner through the south stadium (end-zone seating) to the northeast stadium corner.

Settlement monitoring results were very erratic with both positive and negative readings. After about 10 weeks of preloading, it became critical to initiate construction of the columns on "A.8" line. Partial preload excavation was allowed, and the contractor reshaped the preload to place 8 to 10 feet or more of preload along the critical juncture of soil-supported stands and structurally-supported stands (Figure 7).

Since the settlement monitoring results were erratic and interpretation problematical, the decision to remove preload was based on the initial analyses and the depth to rock. Preloading was maintained for a minimum of 12 weeks in the deep rock areas and 10 weeks in the shallow rock areas. Some sections were subject to preloading for more than 4 months.

CONCLUSIONS

Based on the authors experience man-made structures can be left in-place. Determining their impact on settlement calculations and predictions are difficult, particularly if their distribution is uncertain. Destroying the general integrity of the structures may be beneficial.

Preloading of complex subsurface stratragraphies reduces post-construction settlement. Settlement monitoring is more difficult that it initially appears, and when the predicted settlement period is short (less than 6 months), monitoring is problematical.

Auger-cast piles, end-bearing on rock, can be designed to carry high individual unit loads. However, until the local use is more widespread, extraordinary load testing programs, detailed monitoring, and design redundancies are prudent.

Projects with difficult site conditions, heavy loads, and environmental conditions are best served when the geotechnical and geoenvironmental engineer is an integral member of the design team. Similar benefits are derived when the design team includes a large project construction manger.

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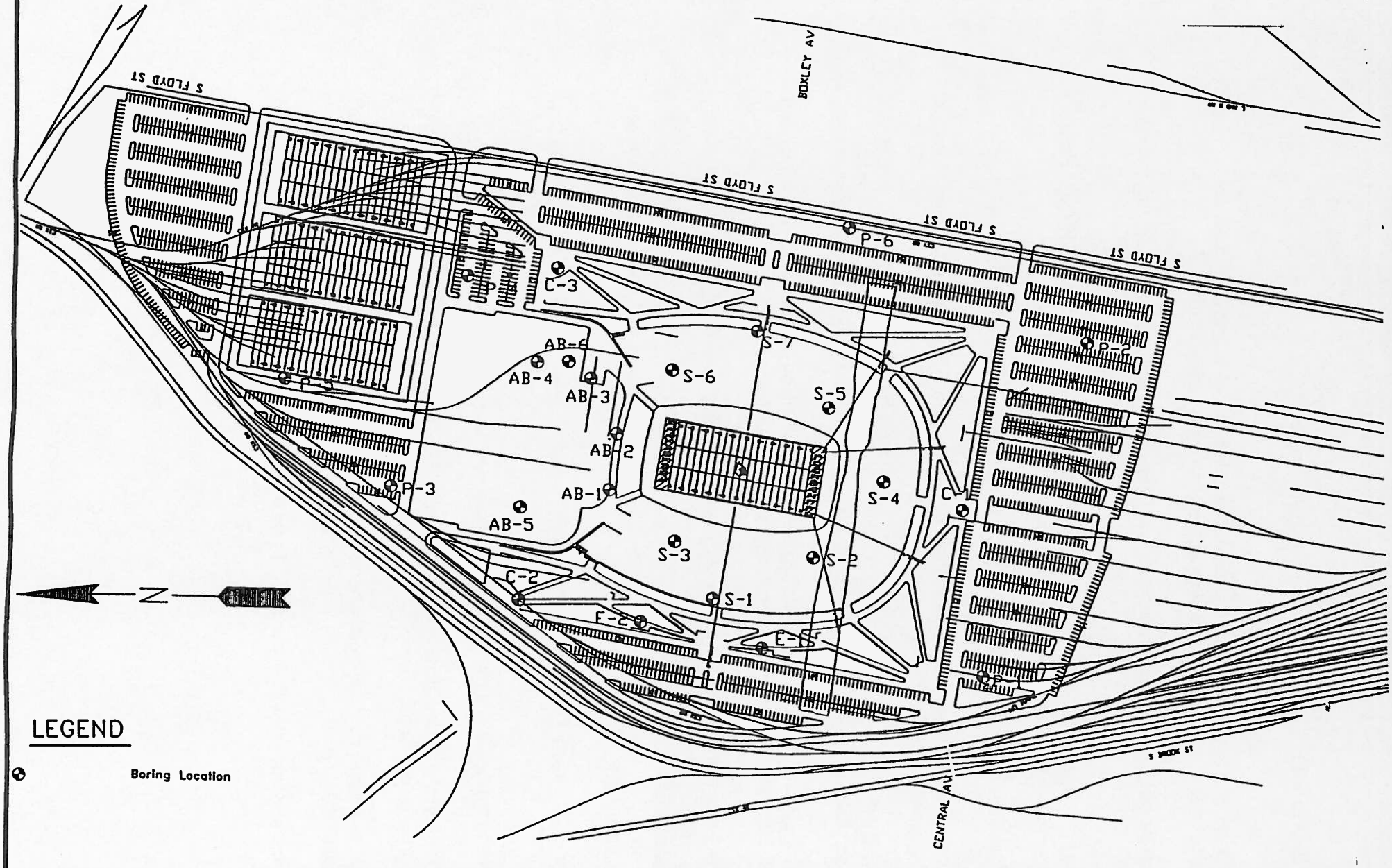
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SOURCE: LUCKETT AND FARLEY ARCHITECTS



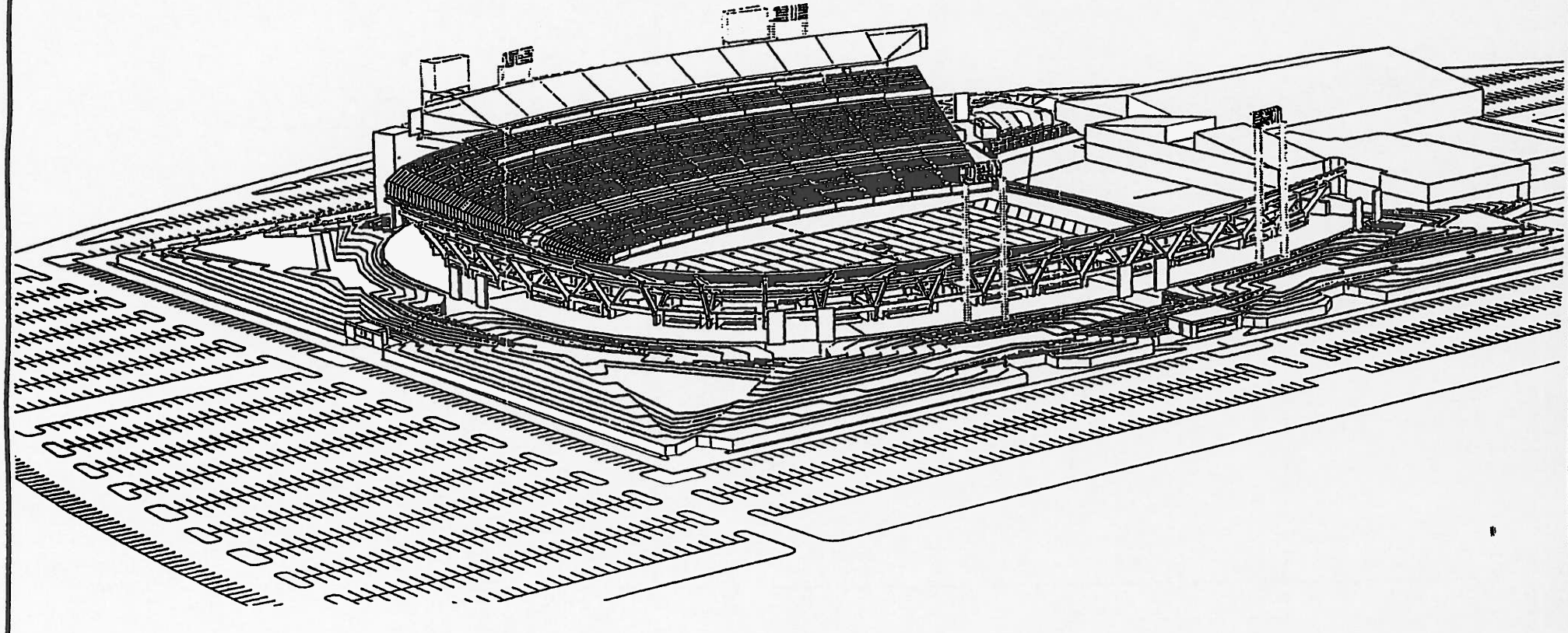
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ORVSS XXIX PRESENTATION
BORING LOCATION PLAN

FIGURE 1

SOURCE: LUCKETT AND FARLEY ARCHITECTS



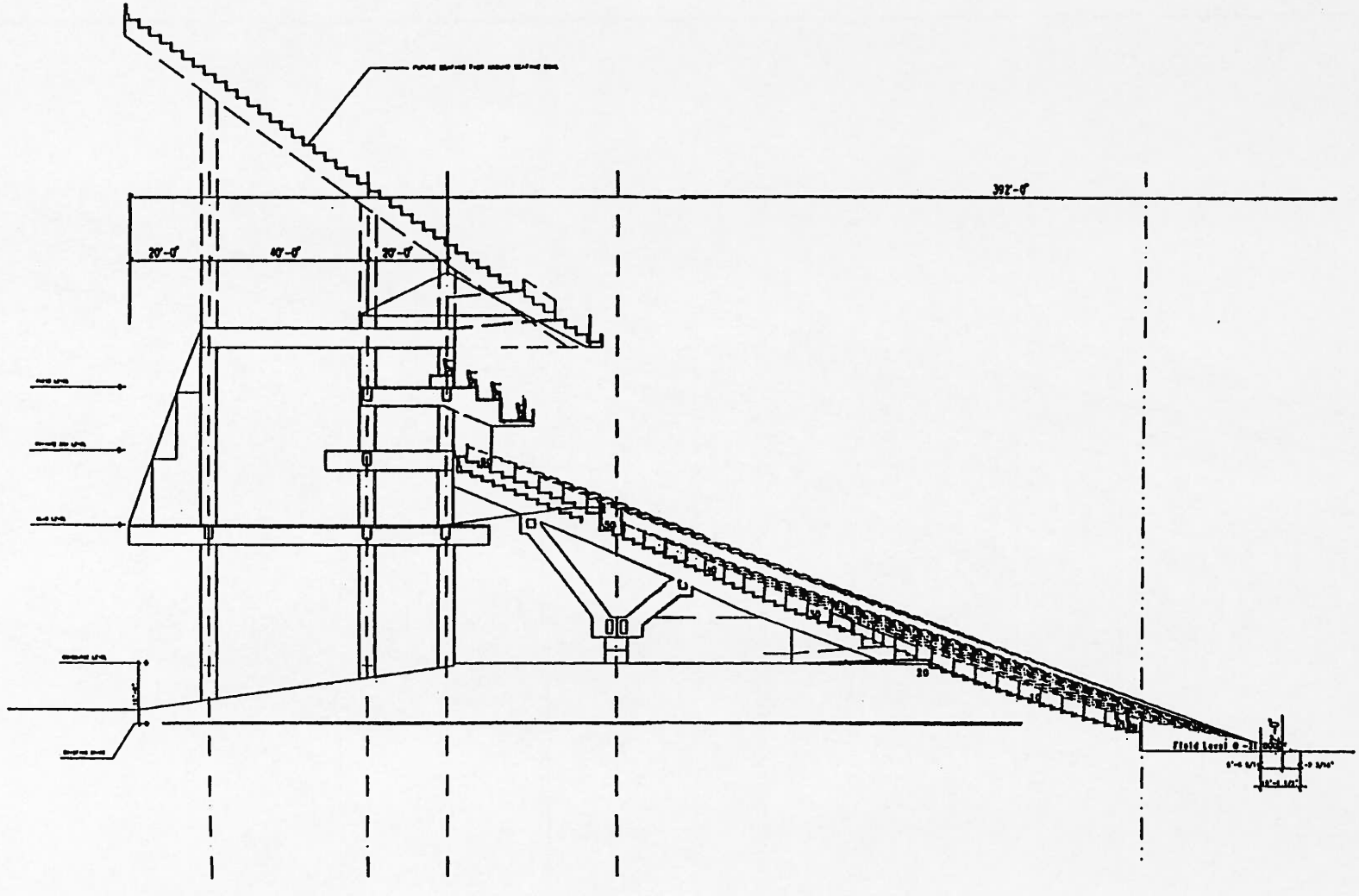
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ORVSS XXIX PRESENTATION
ISOMETRIC VIEW OF STADIUM

FIGURE 2

SOURCE: LUCKETT AND FARLEY ARCHITECTS

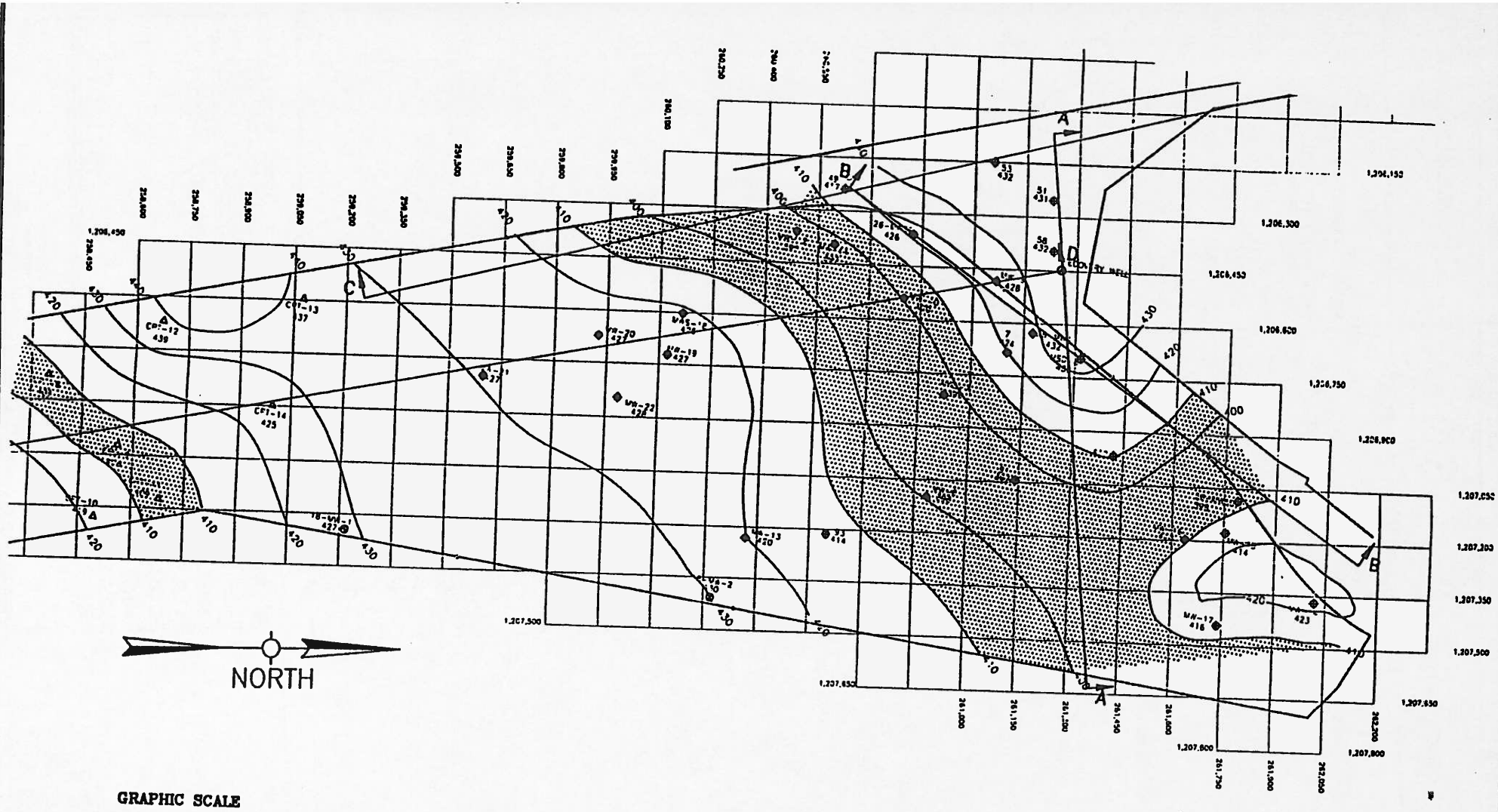


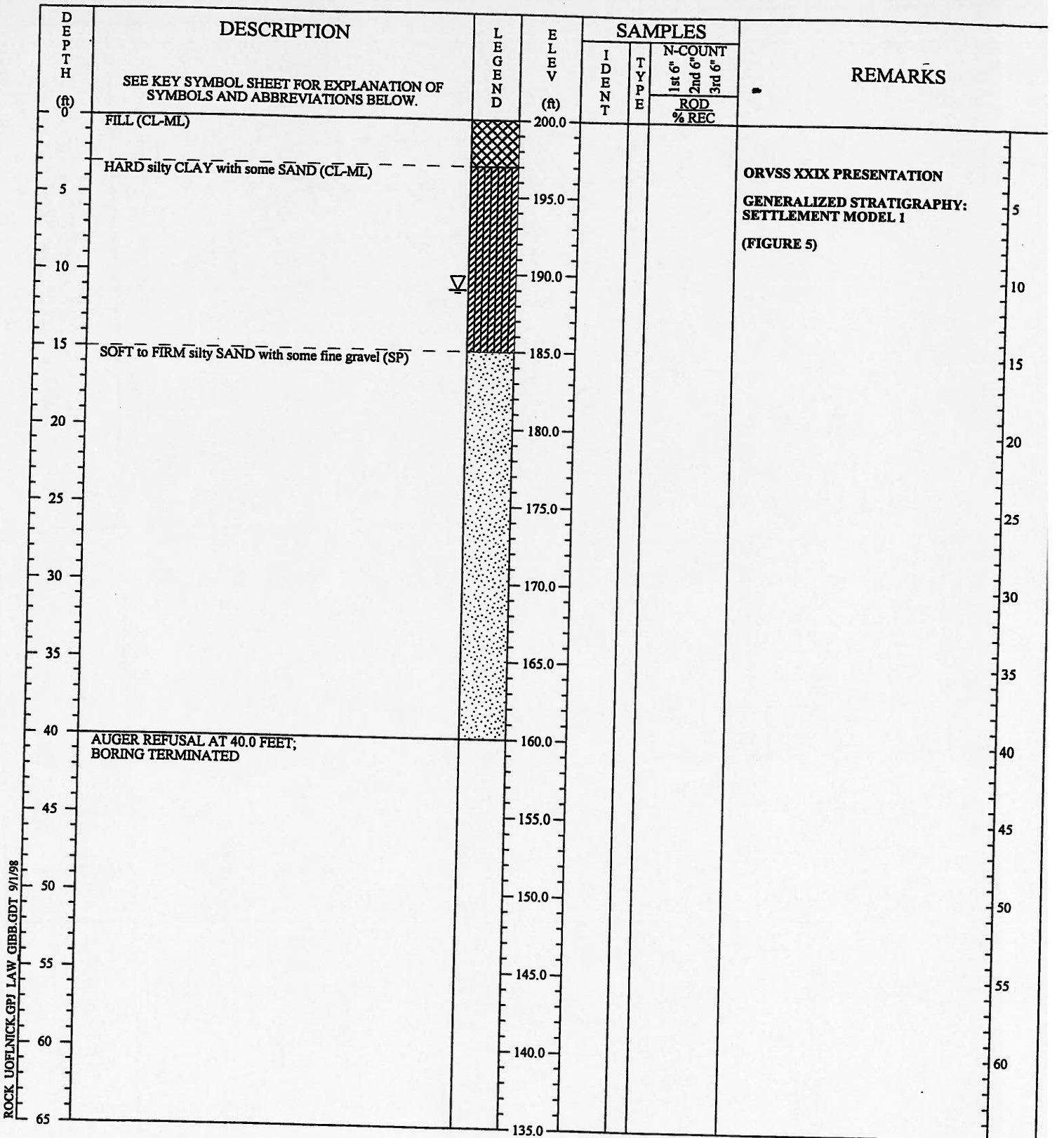
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ORVSS XXIX PRESENTATION
STADIUM SECTION

FIGURE 3


NORTH
A
B
C
D
CPT-10 429
CPT-11 427
CPT-12 439
CPT-13 437
CPT-14 425
MW-1 427
MW-2 430
MW-3 420
MW-4 428
MW-5 414
MW-6 423
MW-7 416
MW-8 428
MW-9 432
MW-10 432
MW-11 427
MW-12 428
MW-13 420
MW-14 414
MW-15 414
MW-16 423
MW-17 416
MW-18 427
MW-19 427
MW-20 428
MW-21 427
MW-22 428
MW-23 428
MW-24 428
MW-25 428
MW-26 426
MW-27 426
MW-28 428
MW-29 428
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MW-37 432
MW-38 432
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MW-40 432
MW-41 432
MW-42 432
MW-43 432
MW-44 432
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MW-50 432
MW-51 432



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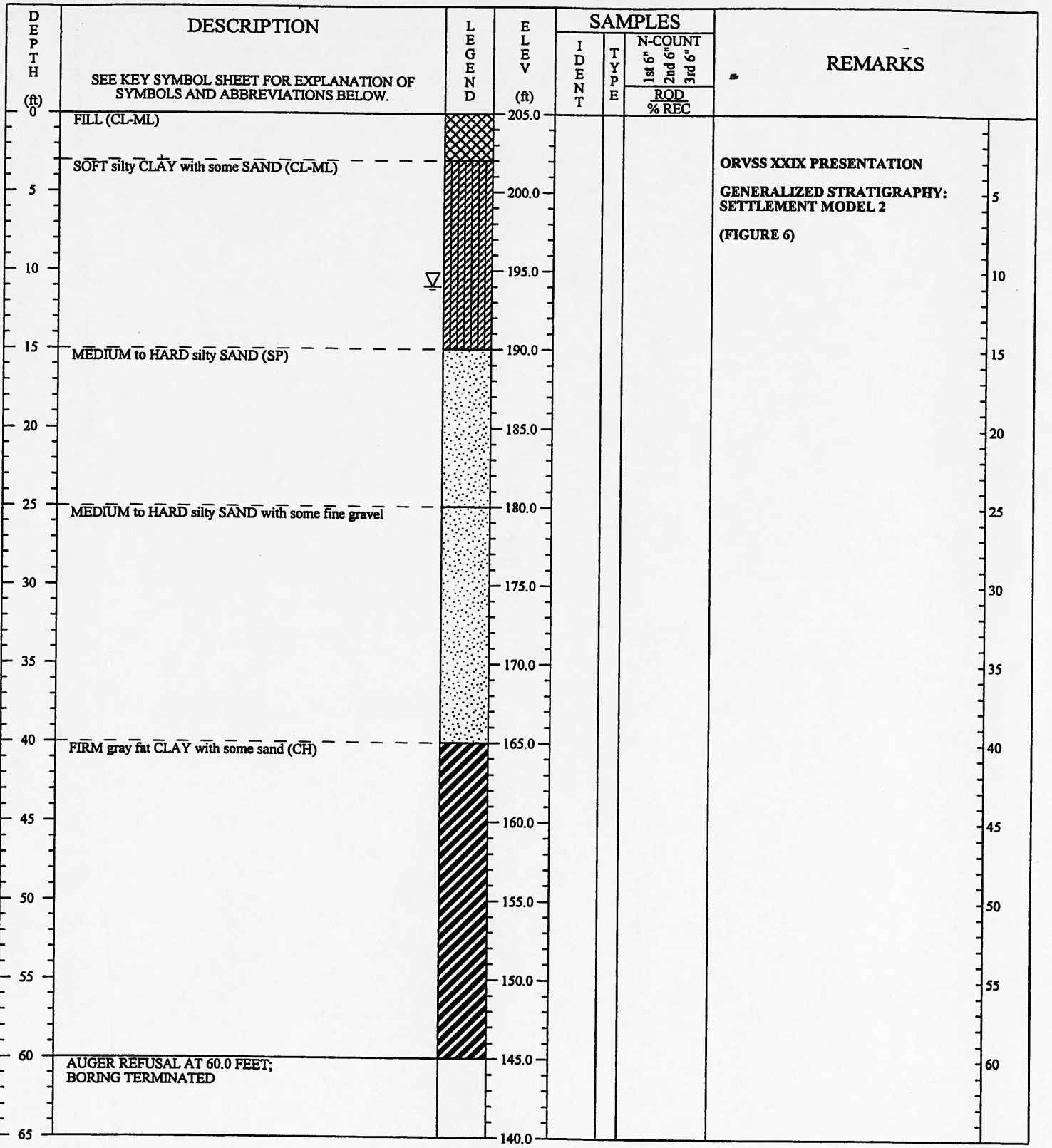
DRILLER:	LAW ENGINEERING
EQUIPMENT:	BME-75
METHOD:	HSA
HOLE DIA.:	4.5
REMARKS:	MANUAL AHAMMER

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

TEST BORING RECORD

PROJECT: UofL STADIUM **BORING NO.:** MODEL 1
COORD N:
COORD E:
DRILLED: August 30, 1998
PROJ. NO.: 50700-8-9210 **PAGE 1 OF 1**

LAW
LAWGIBB Group Member



KUCK_UOFLNICK.GPJ LAW_GIBB.GDT 9/1/98

DRILLER: LAW ENGINEERING
EQUIPMENT: BME-75
METHOD: HSA
HOLE DIA.: 4.5
REMARKS: MANUAL AHAMMER

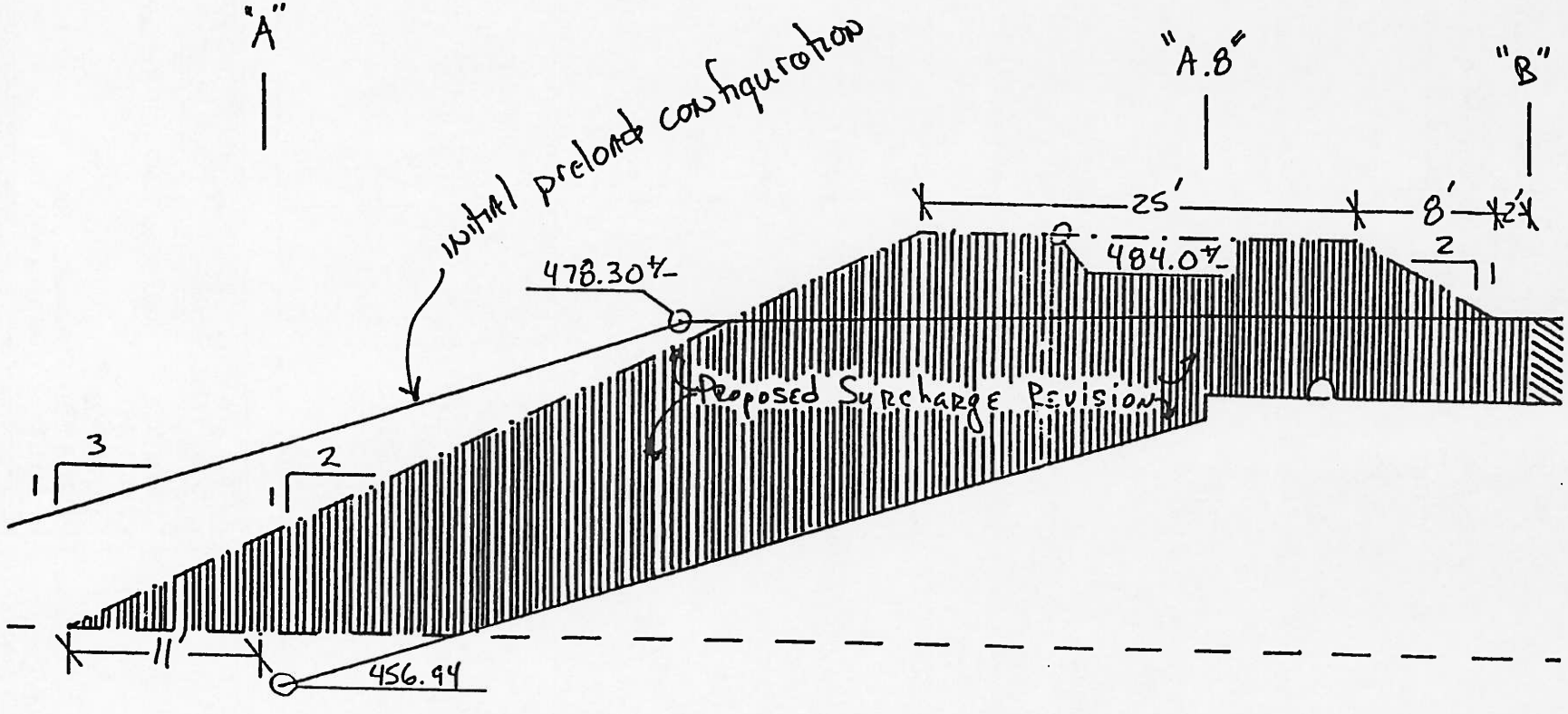
THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

TEST BORING RECORD

PROJECT: UofL STADIUM **BORING NO.:** MODEL 2
COORD N:
COORD E:
DRILLED: August 30, 1998
PROJ. NO.: 50700-8-9210 **PAGE 1 OF 1**

LAW
LAWGIBB Group Member

SOURCE: RAM ENGINEERING



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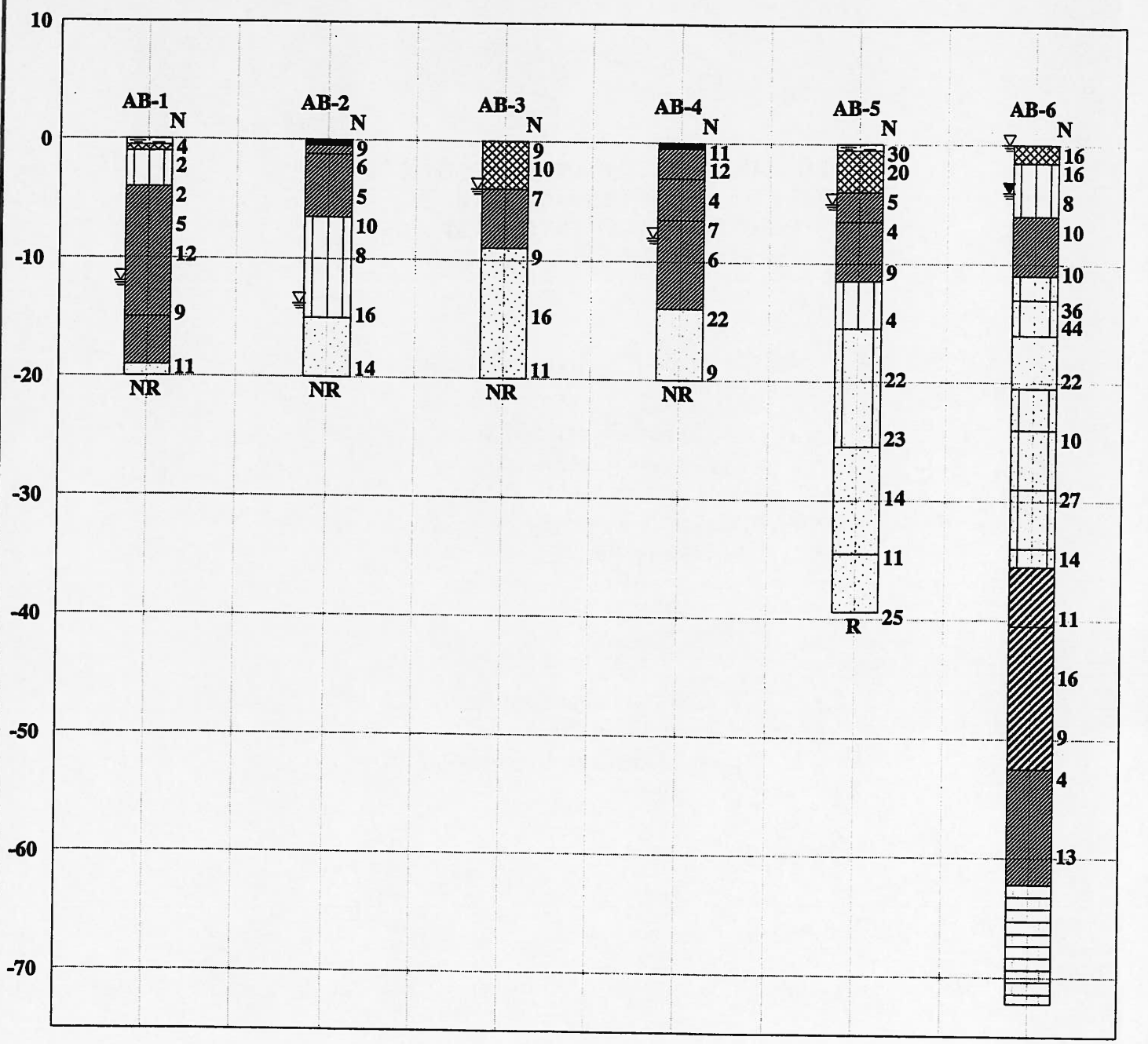
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ORVSS XXIX PRESENTATION
SURCHARGE TEMPLATE

PORTIONS OF THE APPENDIX OF THE REPORT OF GEOTECHNICAL EXPLORATION UNIVERSITY OF LOUISVILLE STADIUM LAW PROJECT NUMBER 50500-5-8301

Includes:

- Key to Soil Classifications
- Key to Rock Classifications
- Point to Point Profile – Figure 1
- Point to Pont Profile – Figure 2
- Test Boring Records
    - * AB-6
    - * S-1
    - * S-2
    - * S-3
- Summary of Laboratory Test Data
- Plasticity Index Graph
- Consolidation Test Report (Figure No. 3)



NOTE: Horizontal spacing is not to scale.

LEGEND: N - Standard penetration resistance.
▽ - Groundwater at time of boring.
▼ - Groundwater at 24-hours or 48-hours.
R - Refusal
NR - No Refusal

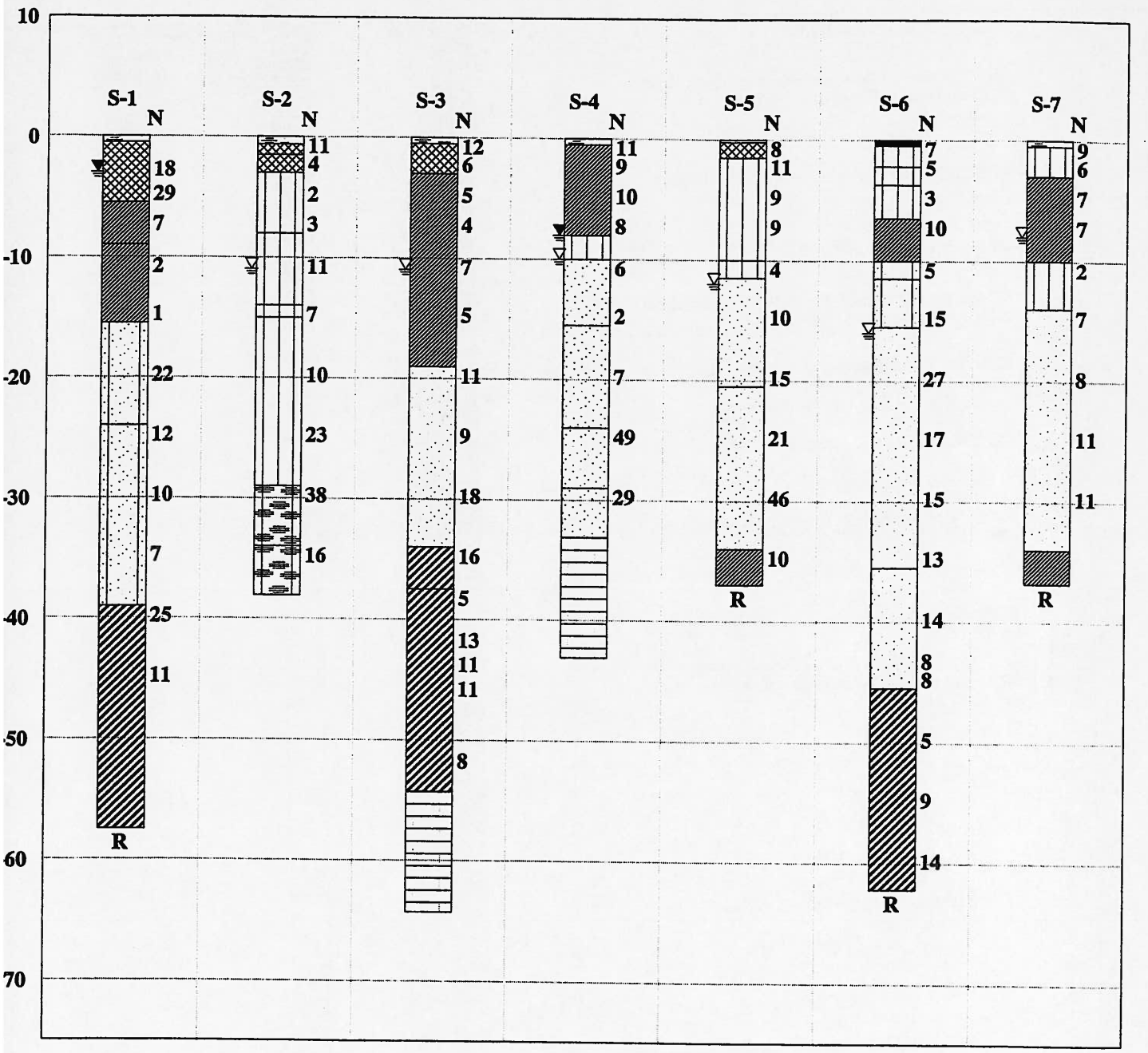
POINT TO POINT PROFILE

FIGURE 1

PROJECT U OF L STADIUM

PROJECT NO. 50500-5-8301

LAW ENGINEERING, INC.



NOTE: Horizontal spacing is not to scale.

LEGEND: N - Standard penetration resistance.
▽ - Groundwater at time of boring.
▼ - Groundwater at 24-hours or 48-hours.
R - Refusal
NR - No Refusal

POINT TO POINT PROFILE

FIGURE 2

PROJECT U OF L STADIUM

PROJECT NO. 50500-5-8301

LAW ENGINEERING, INC.

ELEV.	STRATUM DEPTH	VISUAL SOIL DESCRIPTION	D	SR	K	N	CR	RQD	REMARKS
	0.0								
	1.5	Black ash and cinders (FILL).	1.0	18		16			Surface Cover: Gravel Driller: T. Arthur Drill Crew: D. Vuker
		VERY STIFF to FIRM light brown and gray clayey SILT with black mottling and slight petroleum staining (ML).	2.5	16		16			
	6.0		5.0	18		8			
		STIFF light brown silty CLAY with gray mottling (CL).	7.5	18		10			
	11.0		9.0						
	13.0	FIRM greenish gray silty SAND (SM).	11.0	18		10			
		DENSE gray gravel and silty SAND with petroleum staining and hydrocarbon odor (SM).	12.5						
	16.0		14.0	18		36			
		DENSE to VERY FIRM gray fine silty SAND (SM).	15.5	6		44			
	20.5		20.0	18		22			C
		VERY FIRM gray fine silty SAND (SM).							
	24.0		25.0	11		10			
		LOOSE gray fine silty SAND with rounded gravel (SM).							
	29.0		30.0	18		27			
		VERY FIRM gray fine silty SAND with rounded gravel (SM).							
	34.0								
	35.5	FIRM gray fine silty SAND with rounded gravel and silty clay (SM).	35.0	18		14			
		STIFF gray fat CLAY (CH).							
	40.5		40.0	16		11			
		VERY STIFF to STIFF gray fat CLAY (CH).	45.0	15		16			
			50.0	16		9			
	52.5		51.5						
			53.5	18		4			

D - Sample depth (Ft.)
SR - Sample recovery (In.)
K - Soil Symbols
N - Penetration (Blows/Ft.)
CR - Core Recovery (percent)
RQD - Rock Quality Designation

5	18		10	
10	15		23	
				100
				80

SYMBOLS:
- Standard Pen. Test
- Undisturbed Sample
- Water level, time of drilling
- Water level
- C Caved depth of boring
- Rock Core
- Loss of Water

TEST BORING RECORD

BORING NUMBER	AB-6
DATE DRILLED	August 7, 1995
PROJECT NUMBER	50500-5-8301
PROJECT	U OF L STADIUM
PAGE 1 OF 2	

LAW ENG./ENV. SERVICES

ELEV.	STRATUM DEPTH	VISUAL SOIL DESCRIPTION	D	SR	K	N	CR	RQD	REMARKS
		SOFT to STIFF gray sandy lean CLAY with gray sand (CL).	60.0	18		13			
	62.2	AUGER REFUSAL AT 62.2 FEET, BEGIN CORING.							
		HARD, slightly weathered, gray LIMESTONE, with medium bedding and fine grain, fossiliferous.	67.0	100			97	79	
	72.2	CORING TERMINATED AT 72.2 FEET.							

D - Sample depth (Ft.)
SR - Sample recovery (In.)
K - Soil Symbols
N - Penetration (Blows/Ft.)
CR - Core Recovery (percent)
RQD - Rock Quality Designation

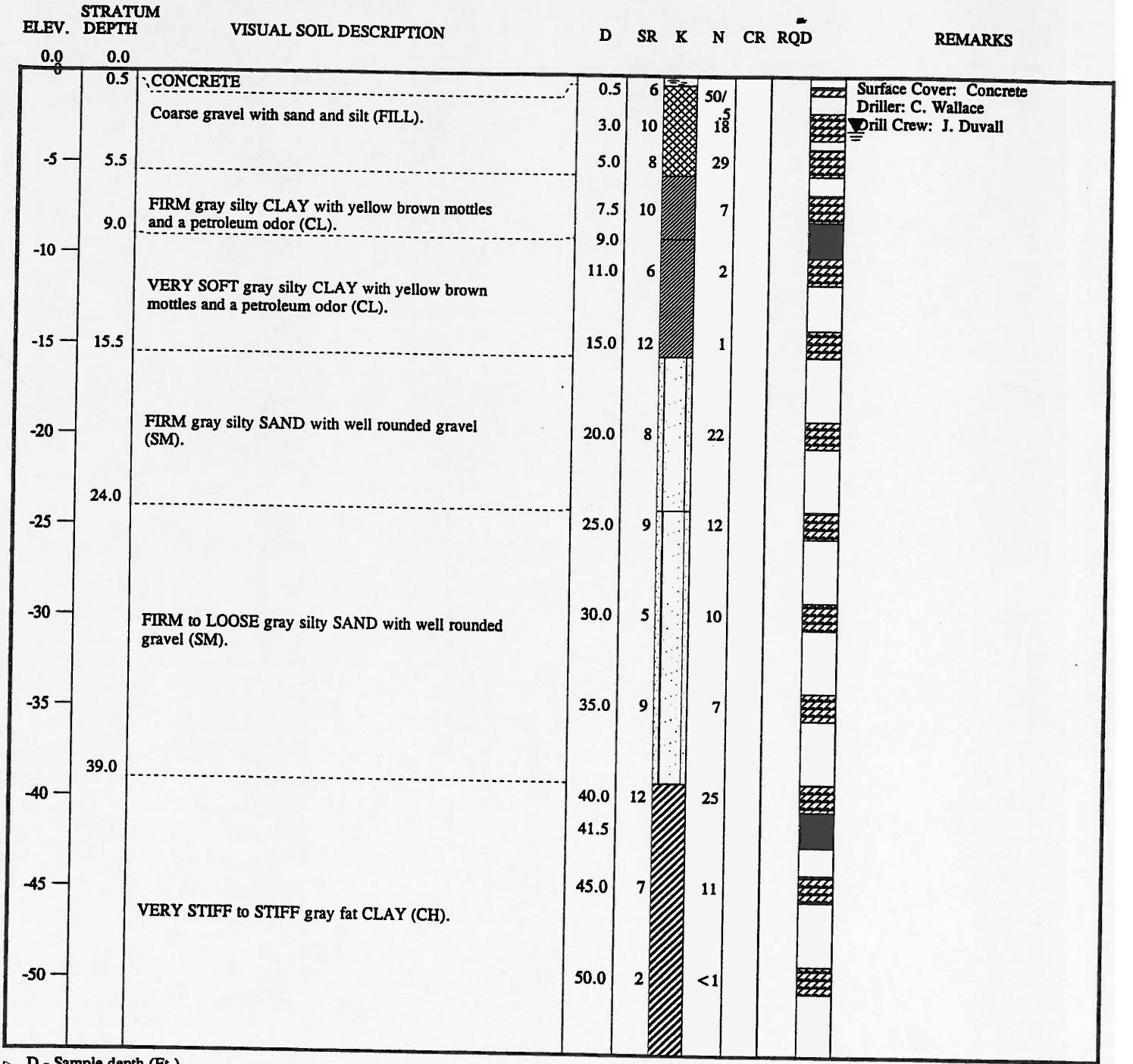
D	SR	N	CR	RQD
5	18	10		
10	15	23		
			100	80

SYMBOLS:
- Standard Pen. Test
- Undisturbed Sample
- Water level, time of drilling
- Water level
- C Caved depth of boring
- Rock Core
- Loss of Water

TEST BORING RECORD

BORING NUMBER	AB-6
DATE DRILLED	August 7, 1995
PROJECT NUMBER	50500-5-8301
PROJECT	U OF L STADIUM
PAGE 2 OF 2	

LAW ENG./ENV. SERVICES



D - Sample depth (Ft.)
SR - Sample recovery (In.)
K - Soil Symbols
N - Penetration (Blows/Ft.)
CR - Core Recovery (percent)
RQD - Rock Quality Designation

5 18 10
10 15 23
100 80

SYMBOLS:
- Standard Pen. Test
- Undisturbed Sample
- Water level, time of drilling
- Water level
- C Caved depth of boring
- Rock Core
- Loss of Water

TEST BORING RECORD

BORING NUMBER	S-1
DATE DRILLED	August 18, 1995
PROJECT NUMBER	50500-5-8301
PROJECT	U OF L STADIUM
PAGE 1 OF 2	

LAW ENG./ENV. SERVICES

ELEV.	STRATUM DEPTH	VISUAL SOIL DESCRIPTION	D	SR	K	N	CR	RQD	REMARKS
-55			55.0	3		2			
	57.5	AUGER REFUSAL AT 57.6 FEET, BORING TERMINATED.							
-60									
-65									
-70									
-75									
-80									
-85									
-90									
-95									
-100									
-105									

D - Sample depth (Ft.)
SR - Sample recovery (In.)
K - Soil Symbols
N - Penetration (Blows/Ft.)
CR - Core Recovery (percent)
RQD - Rock Quality Designation

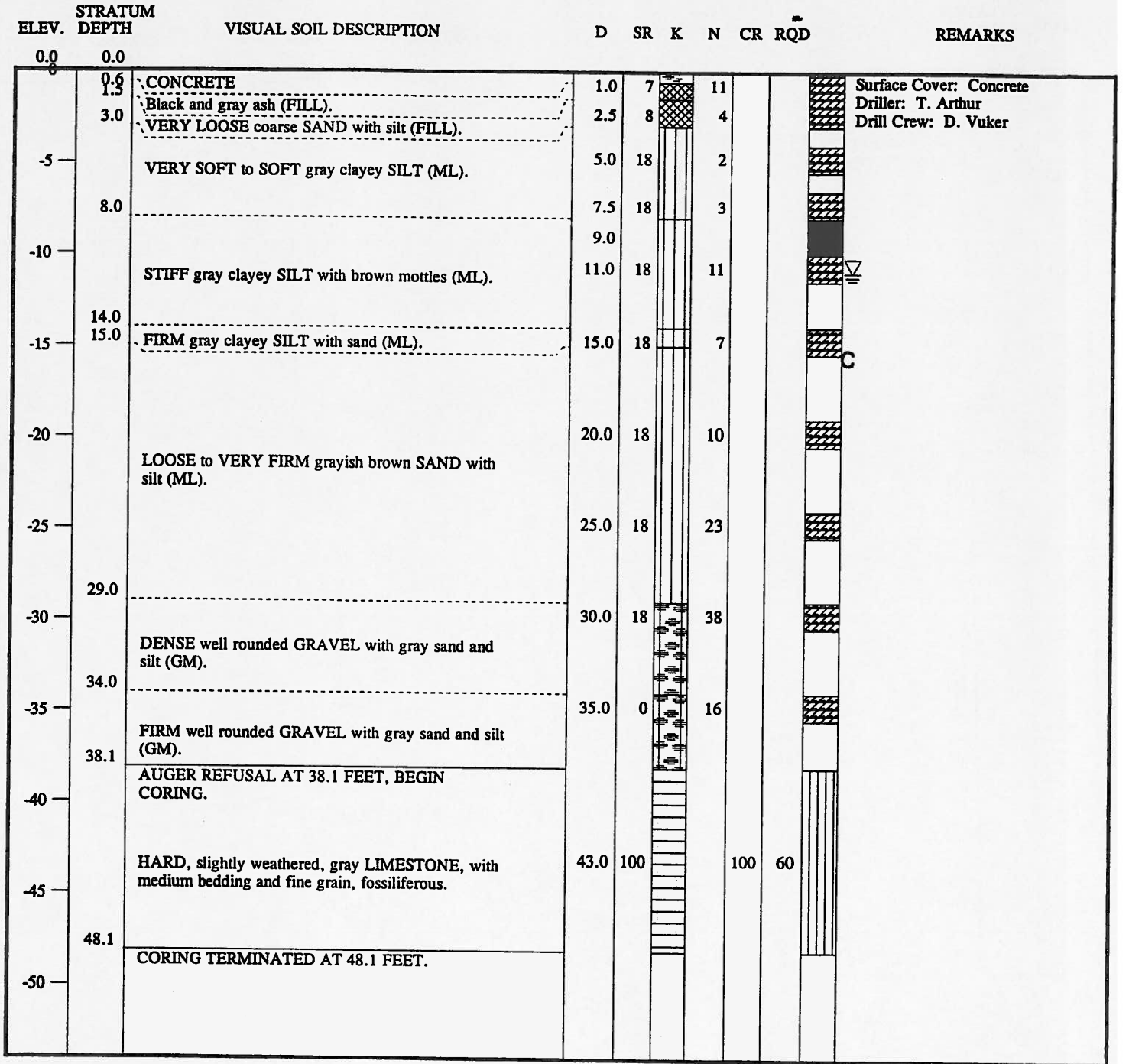
D	SR	K	N	CR	RQD
5	18		10		
10	15		23		
				100	80

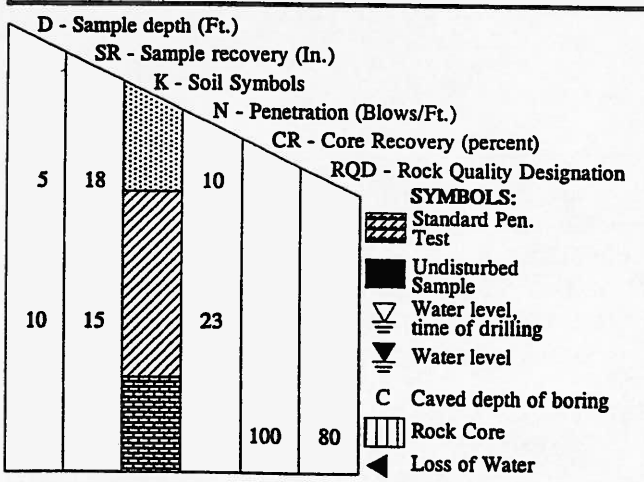
SYMBOLS:
- Standard Pen. Test
- Undisturbed Sample
- Water level, time of drilling
- Water level
- C Caved depth of boring
- Rock Core
- Loss of Water

TEST BORING RECORD

BORING NUMBER	S-1
DATE DRILLED	August 18, 1995
PROJECT NUMBER	50500-5-8301
PROJECT	U OF L STADIUM
PAGE 2 OF 2	

LAW ENG./ENV. SERVICES





TEST BORING RECORD

BORING NUMBER	S-2
DATE DRILLED	August 17, 1995
PROJECT NUMBER	50500-5-8301
PROJECT	U OF L STADIUM
PAGE 1 OF 1	

LAW ENG./ENV. SERVICES

ELEV.	STRATUM DEPTH	VISUAL SOIL DESCRIPTION	D	SR	K	N	CR	RQD	REMARKS
0.0	0.0								
	0.5	Gravel	1.0	10		12			Surface Cover: Concrete
	3.0	Gravel and black ash (FILL).	2.5	16		6			Driller: T. Arthur
-5			5.0	15		5			Drill Crew: D. Vuker
			7.5	18		4			
-10			9.0	18					
		SOFT to FIRM brownish gray silty CLAY with green and brown mottles (CL).	11.0	18		7			▽
-15			15.0	18		5			
	19.0								
-20			20.0	18		11			
-25		LOOSE to FIRM fine gray SAND with well rounded gravel (SP).	25.0	11		9			
-30			30.0	14		18			
	34.0								
-35			35.0	14		16			
	37.5	VERY STIFF gray fat CLAY (CH).							
			38.5	18		5			
-40			40.0	14					
			42.0	9		13			
			44.0	11		11			
-45			46.0	6		11			
		FIRM to STIFF gray fat CLAY (CH).							
-50			50.0	9					
			52.0	9		8			

D - Sample depth (Ft.)
SR - Sample recovery (In.)
K - Soil Symbols
N - Penetration (Blows/Ft.)
CR - Core Recovery (percent)
RQD - Rock Quality Designation

D	SR	N	CR	RQD
5	18	10		
10	15	23		
			100	80

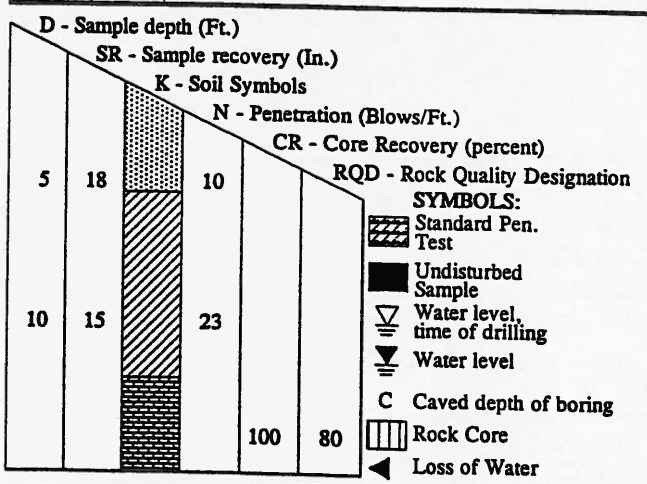
SYMBOLS:
- Standard Pen. Test
- Undisturbed Sample
- Water level, time of drilling
- Water level
- C Caved depth of boring
- Rock Core
- Loss of Water

TEST BORING RECORD

BORING NUMBER	S-3
DATE DRILLED	August 18, 1995
PROJECT NUMBER	50500-5-8301
PROJECT	U OF L STADIUM
PAGE 1 OF 2	

LAW ENG./ENV. SERVICES

ELEV.	STRATUM DEPTH	VISUAL SOIL DESCRIPTION	D	SR	K	N	CR	RQD	REMARKS
-55	54.3	AUGER REFUSAL AT 54.3 FEET, BEGIN CORING.							
-60		HARD, slightly weathered, gray LIMESTONE, with medium bedding and fine grain, fossiliferous.	60.0	100			95	33	
-65	64.3	CORING TERMINATED AT 64.3 FEET.							
-70									
-75									
-80									
-85									
-90									
-95									
-100									
-105									



TEST BORING RECORD

BORING NUMBER	S-3
DATE DRILLED	August 18, 1995
PROJECT NUMBER	50500-5-8301
PROJECT	U OF L STADIUM
PAGE 2 OF 2	

LAW ENG./ENV. SERVICES

SUMMARY OF LABORATORY TEST DATA

BORING NO.	SAMPLE DEPTH, FT.	SAMPLE TYPE**	USCS CLASSIFICATION	MAX. DRY DENSITY PCF	UNIT WEIGHT PCF		UNCONFINED COMPRESSIVE STRENGTH KSF	%FINER NO. 200	SPECIFIC GRAVITY	NATURAL MOISTURE CONTENT, PERCENT	ATTERBERG LIMITS			
					WET	DRY					L.L.	P.L.	P.I.	
AB-6	0-1.5	SS								20.9				
AB-6	1.5-3	SS								22.7				
AB-6	4-5.5	SS	ML							25.9	26	26	NP	
AB-6	6.5-8	SS								24.7				
AB-6	8-10	UD	ML		124.1	99.8				24.5	26	22	4	
AB-6	10-11.5	SS	ML							24.5	NP	NP	NP	
AB-6	13-14.5	SS						11.0		7.9				
AB-6	14.5-16	SS								13.5				
AB-6	19-20.5	SS								15.4				
AB-6	24-25.5	SS						4.0		21.0				
AB-6	29-30.5	SS								16.1				
AB-6	34-35.5	SS								26.7				
AB-6	39-40.5	SS								30.6				
AB-6	44-45.5	SS	CL							30.7	42	21	21	
AB-6	49-50.5	SS								39.9				
AB-6	50.5-52.5	UD	CL							27.2	31	19	12	
AB-6	52.5-54	SS						73.4		35.1				
AB-6	59-60.5	SS								29.5				

Table Checked By: TSD

* Graphic Presentations of Results of Triaxial, Consolidation, CBR, Proctor, Grain Size, and other tests follow this summary

** SS = Split-Spoon Sample (ASTM D-1586); UD = Undisturbed Sample (ASTM D-1587); BG = Bulk Bag Sample

LAW ENGINEERING AND ENVIRONMENTAL SERVICES, INC.

Louisville, Kentucky
Project Name: U of L Stadium
Project Number: 50500-5-8301

SUMMARY OF LABORATORY TEST DATA

BORING NO.	SAMPLE DEPTH, FT.	SAMPLE TYPE**	USCS CLASSIFICATION	MAX. DRY DENSITY PCF	UNIT WEIGHT PCF		UNCONFINED COMPRESSIVE STRENGTH KSF	%FINER NO. 200	SPECIFIC GRAVITY	NATURAL MOISTURE CONTENT, PERCENT	ATTERBERG LIMITS			
					WET	DRY					L.L.	P.L.	P.I.	
E-1	10-12	UD	ML		124.9	99.9	1.6			24.9	30	22	8	
E-2	12-14	UD	ML		129.1	101.3	2.8			27.4	28	26	2	
S-3	0-1.5	SS								3.4				
S-3	1.5-3	SS								22.3				
S-3	4-5.5	SS								21.7				
S-3	6.5-8	SS	CH							30.3	50	25	25	
S-3	8-10	UD	CL		127.3	102.8				23.9	49	18	31	
S-3	10-11.5	SS								25.3				
S-3	14.15.5	SS								21.2				
S-3	19-20.5	SS						7.0		27.2				
S-3	24-25.5	SS								20.9				
S-3	29-30.5	SS								14.8				
S-3	34-35.5	SS								28.9				
S-3	37.5-39	SS	CL							34.6	39	19	20	
S-3	43-45	UD	CL		123.0	95.3		1.0		29.0	49	23	26	
S-5	0-1.5	SS								24.1				
S-5	1.5-3	SS	ML							24.4	NP	NP	NP	
S-5	4.5-5	SS								23.8				
S-5	6.5-8	SS								22.0				

Table Checked By: TSD

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					WET	DRY					L.L.	P.L.	P.I.	
S-5	10-11.5	SS								30.3				
S-5	14-15.5	SS						5.0		22.6				
S-5	19-20.5	SS								25.2				
S-5	24-25.5	SS								25.5				
S-5	29-30.5	SS						5.0		9.2				
S-5	34-35.5	SS	CH							36.2	62	27	35	

Table Checked By: TSO

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** SS = Split-Spoon Sample (ASTM D-1586); UD = Undisturbed Sample (ASTM D-1587); BG = Bulk Bag Sample

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RAILROAD TUNNEL CONSTRUCTION IN THE KOPE FORMATION

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Bernard H. Voor III, P.E.[2]

ABSTRACT

Railroad tunnel clearance improvements in the early 1990s involved floor undercutting and track lowering through two tunnels located between Louisville, Kentucky and Cincinnati, Ohio. The new tunnel floors were founded atop calcareous shale of the Kope Formation. The shale unit decomposed rapidly under rail traffic causing considerable subgrade instability and mud pumping into the ballast section. Remediation at one tunnel involved an open-cut by-pass to effectively eliminate the tunnel and reduce long-term maintenance of the track through this area. The other tunnel, which remains in service, continues plagued by subgrade instability and requires frequent maintenance. Physical constraints will cause open-cut or by-pass remediation to be costly. Undercut and replacement techniques, or asphalt paving of the floor, will require track removal which is considered unacceptable due to operational constraints and potential delays to scheduled freight delivery. In-situ, on-track remediation is desired so that rail traffic can be maintained. This paper presents the results of research on potential lime or cement treatment of decomposed Kope Formation shale to enhance performance of the floor in the remaining tunnel.

INTRODUCTION

The Kope Formation consists predominately of calcareous shale with irregular, thin limestone layers. The formation is prevalent throughout the Northern Kentucky – Cincinnati, Ohio area. Although hard and competent when first exposed, the shale weathers quickly and swells, particularly in the presence of standing water. As a result, the Kope Formation is notoriously unstable and the cause of most landslides in the Northern Kentucky – Cincinnati area. In fact, Cincinnati has one of the highest per capita costs due to landslide damage of any city in the United States due to this troublesome geologic unit (Hansen 1997).

Over the past 15 – 20 years, the railroad industry has been systematically increasing clearances along many major routes to accommodate larger, heavier freight. A variety of methods or techniques have been used to gain these clearance increases, particularly through tunnels. Typically, those methods have included: liner notching or mechanized grinding (roadheading); segmental roof removal and reconstruction; or floor undercutting

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and track lowering. In some instances, tunnels have been daylighted by open-cutting or by-passed using open-cut "run-arounds."

In the early 1990s, a major eastern U.S. railroad performed clearance improvements along a heavy tonnage mainline route between Louisville, Kentucky, and Cincinnati, Ohio. The program included enlargements through two tunnels which were originally driven within the Kope Formation at the turn of the century. The original tunnel floors were situated atop a thin, but very competent, limestone layer.

Clearance improvements through both tunnels were to be gained by backgrouting the brick liners and roadheading the arch and crown areas for the needed clear space. Excessive grout take prompted a change to the work program whereby the floor subgrade was undercut and the track was lowered through both tunnels. Unfortunately, the new floors were no longer situated atop limestone, but instead were founded in Kope Formation shale as shown in the photograph in Figure 1.



Figure 1: Photograph of the undercut tunnel wall. The floor originally beared upon the thin limestone layer shown in the photo.

CHARACTERISTICS OF THE KOPE FORMATION

The Kope Formation consists of 65 to 80 percent shale and is commonly 200 to 250 feet thick. Limestone beds less than 12-inches thick account for up to 35 percent of the rock

unit. Limestone layers become more prevalent and thick toward the upper part of the formation. The dominant shale unit weathers rapidly when wet and typically has a thicker soil overburden than similar shale units. Shales of the formation are poorly laminated and can swell rapidly in the presence of water. Soils derived from the Kope Formation are typically 40 to 50 percent clay and 50 to 60 percent silt, with a small percentage of coarse material. Dominant minerals are illite, quartz, dolomite, and chlorite with little vertical variation in mineralogy (Scotford 1965).

PROBLEM DESCRIPTION

As noted previously, clearance improvements through both tunnels were ultimately achieved by undercutting the tunnel floors and lowering the track structure. In the process, a thin limestone bed which formed the original floors at both sites was penetrated leaving the new floors atop weaker shale. Typical flat track grades (less than 0.5%), combined with the floor undercutting program, producing poor surface drainage conditions. Figure 2 shows a generalized cross-section of the tunnels and the track-lowering program.

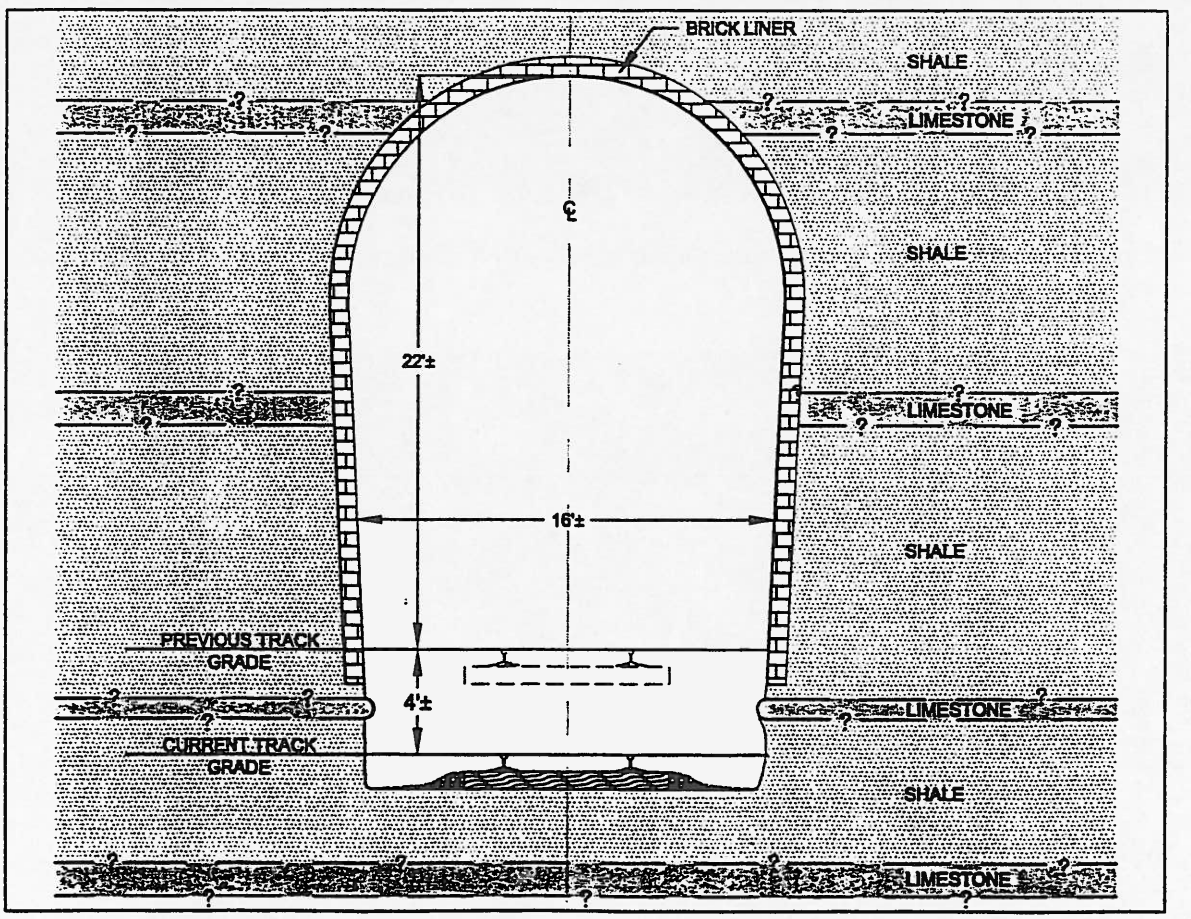


Figure 2: Generalized tunnel cross-section showing original and current track grades.

Generally wet subgrade conditions and dynamic loading under heavy rail traffic have caused rapid deterioration of the shale subgrade. The results are considerable pumping, ballast fouling and grade loss. On several occasions, mud has been observed to the top of rail within each tunnel. These conditions cause frequent track maintenance and almost constant speed restrictions along this important route. Figures 3 and 4 are photographs of the observed subgrade problems.



Figure 3: Photograph of subgrade pumping through ballast section covering railroad ties.

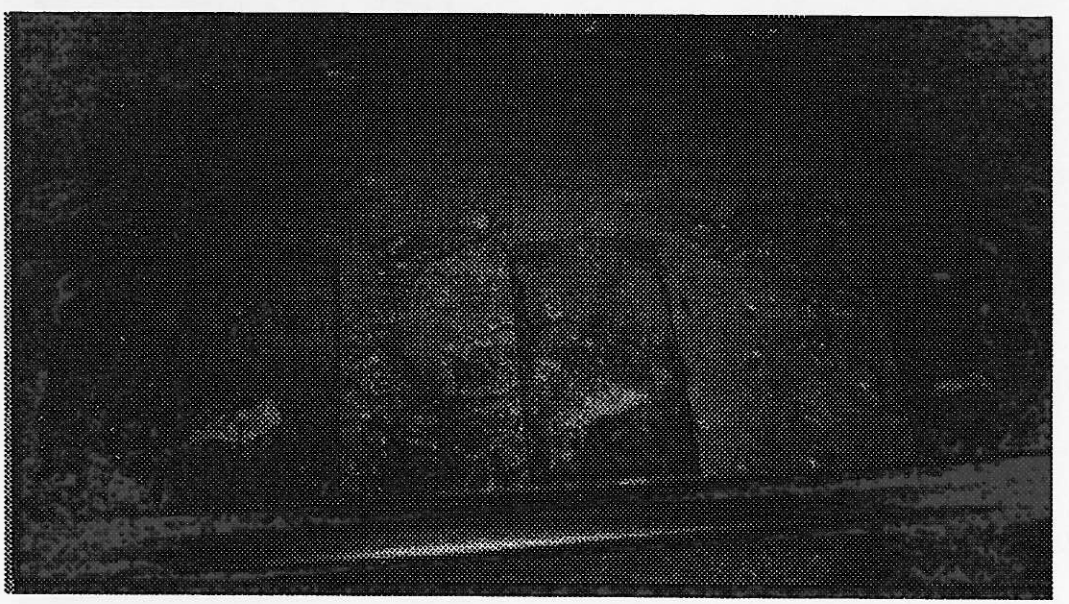


Figure 4: Photograph of fouled ditchlines along the head of ties.

REPAIR OPTIONS AND RAILROAD CONSTRAINTS

Typical remediation options to address subgrade instability such as that described can include: surface drainage improvements; undercut and replacement with engineered fill; invert paving; and in-situ stabilization or soil mixing using lime, cement or other chemical additives. Another form of remediation can include avoidance, whereby the track in this instance, is relocated away from the problem material or raised well above it to bridge over the problem material. Surface drainage improvements would undoubtedly improve conditions simply by "dewatering" the subgrade. Unfortunately, extremely flat existing grades within and beyond the tunnels will not permit substantial improvements to drainage, particularly since the floor undercutting process created a slight swale or low area through each tunnel.

At one tunnel site, remediation using an avoidance method was implemented whereby an open-cut by-pass or "run-around" was constructed. The open-cut was performed while traffic continued through the tunnel. Once the excavation was completed, new track was installed and connected to the active mainline at each end of the tunnel. Thereafter, traffic was routed through the new cut and the tunnel was abandoned in-place. The open-cut was made within the Kope Formation; however, better surface drainage was achieved and unrestricted headroom allowed the track to be raised as needed to bridge any soft subgrade areas. Although costly due to the required large excavation, the long-term benefits of reduced maintenance through an otherwise high-maintenance tunnel yielded a cost-effective solution.

The remaining in service tunnel, known as Eagle Tunnel, is located near Glencoe, Kentucky. The deteriorated subgrade continues requiring high levels of maintenance and traffic is forced to operate at restricted speed. Previous studies have considered open-cut alternatives for avoidance remediation; however, a county road and gas transmission line lying above the tunnel will result in extremely high open-cut construction costs. Undercut/replace methods as well as asphalt invert paving have also been considered for remediation. Unfortunately, these options would require a segmental track removal and replacement program in order to perform the work such that rail traffic could continue around construction periods. Although the actual cost of these repair techniques would be relatively low, the impact on railroad operations and the potential penalties for delayed freight delivery, should it occur, would be substantial. Accordingly, in-situ subgrade treatment techniques are preferable. In this manner, work could be performed between train traffic while the track remains in place. The balance of this paper considers in-situ lime and cement treatment of the Kope Formation shale subgrade within Eagle Tunnel.

IN-SITU SOIL STABILIZATION

General

The following sections describe the processes whereby lime and cement additives work to improve the strength of soils. These sections also provide general criteria on the applicability of such treatments based on the mineralogy of a given soil.

Lime Stabilization

Strength gain produced by lime treatment is derived from three reactions: dehydration of the soil, ion exchange, and pozzolanic reaction. Short-term strength gain is associated with dehydration and ion exchange of the soil particles. The drying action, or dehydration, is particularly beneficial in moist soil where a relatively small reduction in moisture content can increase shear strength and bearing capacity. Ion exchange causes the electrical forces on the clay surface to change. Changes of the soil structure begin through flocculation and coagulation of soil particles into larger-sized particles.

The long-term effectiveness of lime depends on the pozzolanic reactivity of the soil. Soils with high pozzolanic reactivity will typically achieve greater strengths. It has been observed that montmorillonite and kaolinitic clay soils contain effective pozzolanic agents when compared to clays which contain illite, chlorite, or vermiculite. The amount of secondary cementitious materials that are produced during pozzolanic reaction of the clay particles and lime depend on the clay mineralogy and presence of silica and alumina in the soil (Bergado 1996). The montmorillonite clay mineral will probably react more readily than the illites and kaolinites due to their poorly defined crystallinity. However, for non-plastic soils which are unresponsive to lime, a pozzolan is needed to produce the necessary lime-silica reaction. Fly ash is the most commonly used pozzolan substitute for lime stabilization.

There are several factors which influence the effectiveness of lime-treated soils, including: type of lime, lime content, curing time and temperature, soil type, soil mineralogy, and soil pH. The most important of these factors is probably soil mineralogy. The presence of silicates and aluminates in the soil has a substantial affect on the pozzolanic reactions in the soil. A high temperature and pH will enhance pozzolanic reactions because of the increase in solubility of the silicates and aluminates (pozzolans). Typically, a minimum pH of approximately 10.5 is necessary for pozzolanic reactions to take place. A summary of the main factors which control the hardening characteristics of lime-treated clay is provided in Table 1.

Typical percentages of lime additive are usually between 3 and 7 percent, depending on mineralogy. Hill and Davidson (Bergado 1996) developed a correlation showing that the amount of lime fixation is in proportion to the type and amount of clay present and independent of the ascribed cations present in the clays. The relationship is given as:

$$\text{Optimum Lime Content} = \frac{\text{Percent of clay}}{35} + 1.25$$

The type of lime is also a factor in judging treatment effectiveness. Although dangerous and expensive, quicklime is preferred over hydrated lime because of its affinity for water. Additionally, the hydration process produces an increase in temperature which is favorable to strength gain (National Lime Association 1991).

Table 1: Factors which affect the Hardening Properties of Lime-Treated Soil (Bergado 1996)

Factor	Effect on the Hardening Characteristics
Type of lime	Quicklime (CaO) generally more effective than hydrated lime; however, it requires special handling and is a more concentrated form of lime.
Lime Content	The strength of lime soil mixture increases as the lime content is increased; however, it may decrease when reaching a certain percentage. Typical lime content is 3 to 7%. There is typically a maximum strength limit that can be obtained at the optimum lime content.
Curing Time	Improvement is usually seen after 1 to 2 hours of mixing depending on the reactivity of the clay.
Type of Soil	The clay content of the soil should not be less than 20% and the sum of the silt and clay fractions should exceed 35%.
Clay Mineralogy	The presence of aluminates and silicates in the soil are essential for increasing shear strength through pozzolanic reactions.
Soil pH	The long-term chemical reactions in lime-stabilized soils are favored by a high pH-value (pH>12) since the reactions are accelerated due to the increased solubility of the silicates and aluminates present in the clays.
Curing Temperature	A high temperature favors chemical reactions in the soil; the effect of increasing curing temperature is to increase strength.

Cement Stabilization

Strength gain produced by cement treatment likewise occurs due to a number of phenomena. When calcium hydroxide [$Ca(OH)_2$] is introduced into clay soil a chemical reaction takes place between the calcium hydroxide and silica/alumina forming cementitious compounds (Pengelly 1998). The effectiveness of cement treatment, however, depends on a number of factors including the characteristics of the soil. Soils with high water content and organic content are not as responsive. The responsiveness increases with decreasing plasticity index. When the activity of a soil is high, strength gain using cement is generally low. A list of these factors and their effects are summarized in Table 2.

Table 2: Factors which affect the Hardening Properties of Cement Treated Soil (Bergado 1996)

Factor	Effect on the Hardening Characteristics
Type of Cement	Type III cement renders better improvement than Type I cement; however, Type I cement is the most popular cement used in soil stabilization.
Cement Content	The greater cement content renders greater increase in shear strength.
Curing Time	The rate of increase of strength is generally rapid in the early stages of the curing period. The rate of increase for cement-treated clay is greater than of lime-treated clay in the early stages.
Type of Soil	Soils with low plasticity are improved drastically with cement.
Clay Mineralogy	Greater strength is obtained from the soil with higher pozzolanic reactivity; Montmorillonitic and kaolinitic soils were found to be effective pozzolanic agents compared to illitic, chloritic, or vermiculitic soil.
Soil pH	The long-term pozzolanic reactions are favored by higher pH values.
Curing Temperature	An increase in temperature accelerates the chemical reactions and solubility of the silicates and aluminates.

The strength of the cement-treated soil generally increases with cement content. In the early stages, the rate of increase for cement-treated clay is typically greater than that of lime-treated clay (Bergado 1996). The type of cement is also a factor. Although Type III Portland cement renders greater strength gain, Type I is the most popular cement for soil stabilization because it is the cheapest and most readily available.

SAMPLING AND INDEX TESTING

Three large bag samples of the deteriorated subgrade material were collected from within Eagle Tunnel in order to perform index testing and assess treatability. Testing included: natural moisture content determinations, Atterberg Limits, and grain-size distribution. Clay mineralogy was determined by x-ray diffraction. Test results are summarized in Table 3.

Table 3: Summary of Index Testing

Test Type	Results
Natural Moisture Content	28.5% - 38.6%
Atterberg Limits	Liquid Limit: 32 – 36 Plasticity Index: 12 - 14
Grain-size distribution (with hydrometer)	Silt: 44% Clay: 55%
Clay Mineralogy (x-ray diffraction)	Illite: 80.5% - 81% Chlorite: 19.0% - 19.5%
Maximum Dry Density (Standard *Proctor*)	120 lb/ft^3
Optimum Moisture Content	12.0%

Index test results suggest that the effectiveness of lime treatment may be questionable due to the presence of illite and chlorite. Conversely, cement treatment may be quite effective due to low plasticity and sufficiently high moisture for long-term hydration.

LABORATORY RESEARCH

Laboratory-controlled treatability testing was performed on remolded samples of the silty clay subgrade material with various admixtures. These included: Type I cement, quicklime, hydrated lime, and fly ash. Initially, six samples were prepared at water contents of 18% and 20% (3 samples each) with no additives in order to provide baseline data for comparison to the treated samples. At a moisture content of 18%, the unconfined compressive strength ranged from 1.08 – 1.26 ksf; at 20% moisture, unconfined strength ranged from 0.58 – 0.63 ksf.

To assess relative strength gain with different admixtures and varying percentages of admixtures (by dry weight), additional samples were remolded and subjected to unconfined compressive strength testing. Each sample was remolded using Harvard Miniature Molds and subsequently tested with a constant strain load frame. Without an undisturbed sample or an in-place field density, it was extremely difficult to remold a sample to model field conditions. Therefore, each sample was prepared to the highest water content at which a sample could be extracted from the mold without significant disturbance or collapse. Each sample was prepared at approximately 20 percent moisture content prior to introducing the admixture. Each sample was prepared using similar compactive effort. Upon removal from the mold, each specimen was sealed to avoid moisture loss. Testing was performed approximately 12 hours later. Test results are presented graphically in Figure 5.

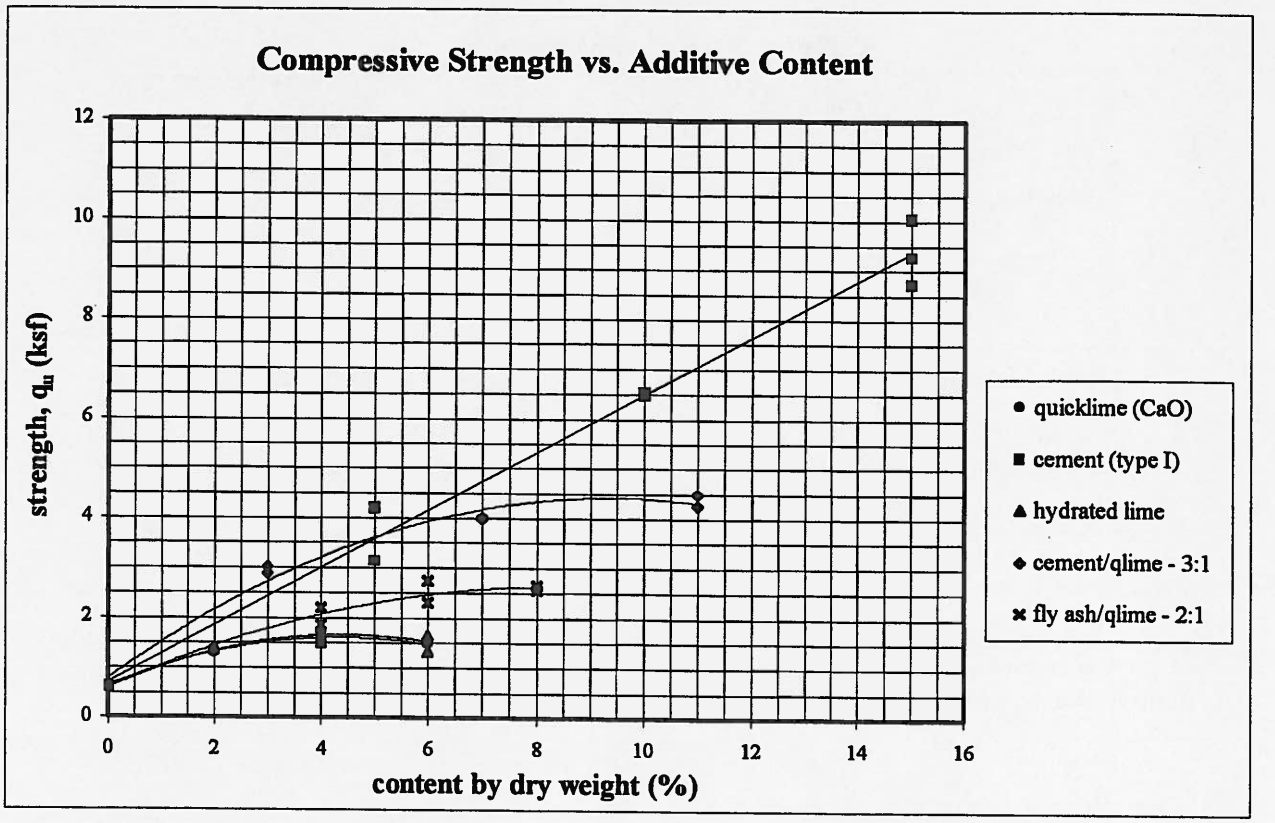


Figure 5: Graphical presentation of compressive strength with versus percent admixture.

The laboratory test results indicate that substantial strength gain can be achieved using a cement additive. Within the range of percent cement additive examined, strength gain increases linearly with increased cement content. Confirming published literature and the index test results presented earlier herein, only minor strength gain was observed with lime treatment likely due to the absence of pozzolans. The observed strength gain with lime was probably due to sample dehydration. Combined treatment using cement-lime admixtures achieved greater strength than the lime-only treated samples but not as much as the cement-only samples. Likewise, the lime/fly ash-treated soil experienced greater strength gain than the lime-treated soil probably due to the introduction of a pozzolanic material. Even though the fly ash additive yielded only a small increase in strength in 12 hours, additional improvement over time is likely because of its pozzolanic behavior.

ESTIMATED COSTS

Eagle Tunnel is about 640 feet long and 16 feet wide at its base. Average cover is about 50 feet; maximum cover is about 80 feet. Using this basic information, Table 4 was developed to present comparative costs for the remedial options mentioned herein.

Table 4: Cost comparison for various remedial options

Repair Method	Assumptions/Parameters	Estimated Cost
1. Open-cut by-pass or daylighting	• General excavation 100' x 50' x 640' • Bridge or detour for county road • Relocation of gas line	$2,000,000 to $2,500,000
2. Asphalt invert paving	• Railroad forces are required • 10-hour curfews to perform work • Disruptive to operations • Requires track removal/replacement	$200,000 to $250,000
3. In-situ cement treatment	• Treatment zone 3' x 16' x 640' • Cement content 8 – 10% • Maximum 4-hour daily work window	$300,000 to $350,000

As shown above, an open-cut repair will be costly even though it provides good long-term benefit due to reduced maintenance. Invert paving is relatively inexpensive; however, the impact on operations due to track removal and the potential penalties for delayed freight delivery are considered unacceptable at this time. While more expensive than paving, in-situ treatment with cement appears to constitute a cost-effective remediation option provided such a program can be implemented in short work windows around frequent rail traffic.

CONCLUSIONS

The Kope Formation shale is a problematic material which is responsible for numerous landslides in the Northern Kentucky – Cincinnati, Ohio area. When required to serve as subgrade for heavy tonnage railroad traffic, the formation must be well drained and protected from the effects of weathering. If poorly drained and unprotected, the shale component will decompose rapidly and swell. Beneath mainline track, these conditions will severely foul the ballast section and disrupt rail service. Although a variety of remedial repair options are available under normal circumstances, the unique operating requirements of the railroad industry make in-situ treatment desirable.

The research presented herein indicates that the mineralogy and characteristics of deteriorated Kope Formation shale are conducive to treatment using Type I cement. It appears that the addition of cement at rates of 6 – 10% may yield suitable subgrade

improvement. Lime treatment of the Kope Formation shale does not appear favorable due to the absence of constituents necessary for pozzolanic reaction. Although cement treatment appears appropriate, additional research is needed to assess the long-term durability of this in-situ stabilization technique.

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Landslide Repair at Mission Viejo Mall

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Synopsis

In 1990, a major landslide occurred at Mission Viejo Mall in Mission Viejo, California. Mission Viejo is located just south of Los Angeles. The mall was owned by the Edward J. Debartolo Corporation. The landslide occurred along an approximate 2(H):1(V) slope between the Mall and Interstate 5, the San Diego Freeway. The slide mass was about 350 long and 90 feet high, encompassing almost 2 acres in area. After a rapid geotechnical evaluation by Woodward-Clyde Consultants, the slide was temporarily halted by grading and dewatering. The initial design concept for remediation of the slide consisted of three rows of four foot diameter drilled piers capped by a reinforced earth retaining wall. Through the use of the design/build contracting method, a tieback anchored wall was installed which included deep concrete caissons, drilled and grouted tieback anchors, horizontal drains, and a deep well pumping system.

Project Background

Staff at Mission Viejo Mall noted broken irrigation lines associated with soil cracking and slumping near the edge of the parking lot on April 20, 1990. A large headscarp formed near the top of the slope shortly thereafter. In addition, the slide began encroaching on Interstate highway right-of-way. The California Department of Transportation (CALTRANS) noted tilting of the Crown Valley Parkway exit sign on Interstate 5 at the base of the slope. The sign was removed and discussions between Woodward-Clyde, the owner, and CALTRANS were held to establish field investigation objectives. Optical survey points were established at the top of the slope and inclinometers were installed to monitor slope movement. Initially, the slope movements remained steady, but showed no indications of slowing down. Plans were made for interim grading operations at the top of the slope to reduce the movements. However, during the week of May 21, 1990, the field data indicated that the rate of movement was

increasing. At that time, the geotechnical consultant recommended to the owner that dewatering wells be immediately installed and pumped to reduce groundwater levels.

The well installation began over the Memorial Day weekend. The deepest well screen was set between depths of 130 to 170 feet below the crest of the slope. An additional well screen was set in a boring at the bottom of the slope at a depth of 30 to 50 feet. Pumping began on Monday, May 20, in conjunction with grading works at the top of the slope. Groundwater levels were observed to drop approximately 30 feet in response to the pumping. Heavy rains fell that weekend, prompting CALTRANS to temporarily close the Crown Valley exit ramp. During the next 3 days, slope movements stopped in response to the grading and pumping efforts on the site. Approximately 8000 cubic yards of material were excavated and hauled off site. The graded area at the top of the slope was covered in plastic and weighted with sand bags to seal out rainwater. CALTRANS reopened the exit ramp on June 1.

Site Subsurface Conditions

After the emergency activities concluded, a geotechnical investigation was initiated to develop concepts for a permanent repair of the slope. In addition to the borings originally drilled for the mall, nine additional borings were drilled for the remedial work. In 5 of the borings samples were taken and slope indicator casing were installed. The remaining 4 borings were drilled as extraction wells, and were logged from the cuttings. Review of the original project records and extensive field mapping were performed.

The landslide site had been extensively reworked during the construction of the San Diego Freeway and the mall. The predominant "bedrock" at the site is the Capistrano Formation. The Capistrano formation consists of gray to black siltstone and clayey siltstone. The siltstones are weakly indurated and exhibit swelling and collapse during drilling. Numerous angled fractures, joints and slickensides were noted within this formation during previous excavations at the site.

The slope contained 20 to 60 feet of stiff clay fill material with plasticity indices ranging from 16 to 32. Unconfined compressive strengths of the fill were estimated by pocket penetrometer to be about 2.5 to 3 tons per square foot (tsf). Observed moisture contents of the fill were 15 to 31 percent.

Under the fill, 7 to 33 feet of weathered Capistrano Formation consisting of clay soils were identified. Pocket penetrometer results were 2.5 to 4 tsf. The weathered Capistrano exhibited plasticity indices of 36 to 48. In place moisture contents were 12 to 44 percent. Numerous remolded clay seams, presumed to be associated with subsurface movements, were noted in this material.

The underlying unweathered Capistrano Formation was observed in the borings as hard clay with plasticity indices ranging from 34 to 43. Water contents observed ranged from 26 to 35 percent. Pocket penetrometer results were greater than 4.5 tsf. No remolded zones were noted in the unweathered Capistrano, however bedding planes were recognized to be locally weak and were observed to be at unfavorable angles.

Assessment of Slope Movements

Please refer to Figure 1 for a plan illustration of the landslide. Slope movements were generally due West, towards the San Diego Freeway. Measured inclinometer movements were rapid and observed as deep as 90 to 110 feet below the crest of the slope. Toe bulging was observed at the base of the slope, above the level of the Freeway. Based on the observed bulging and inclinometer readings at the toe, it was concluded that the slope failure did not encompass the Freeway.

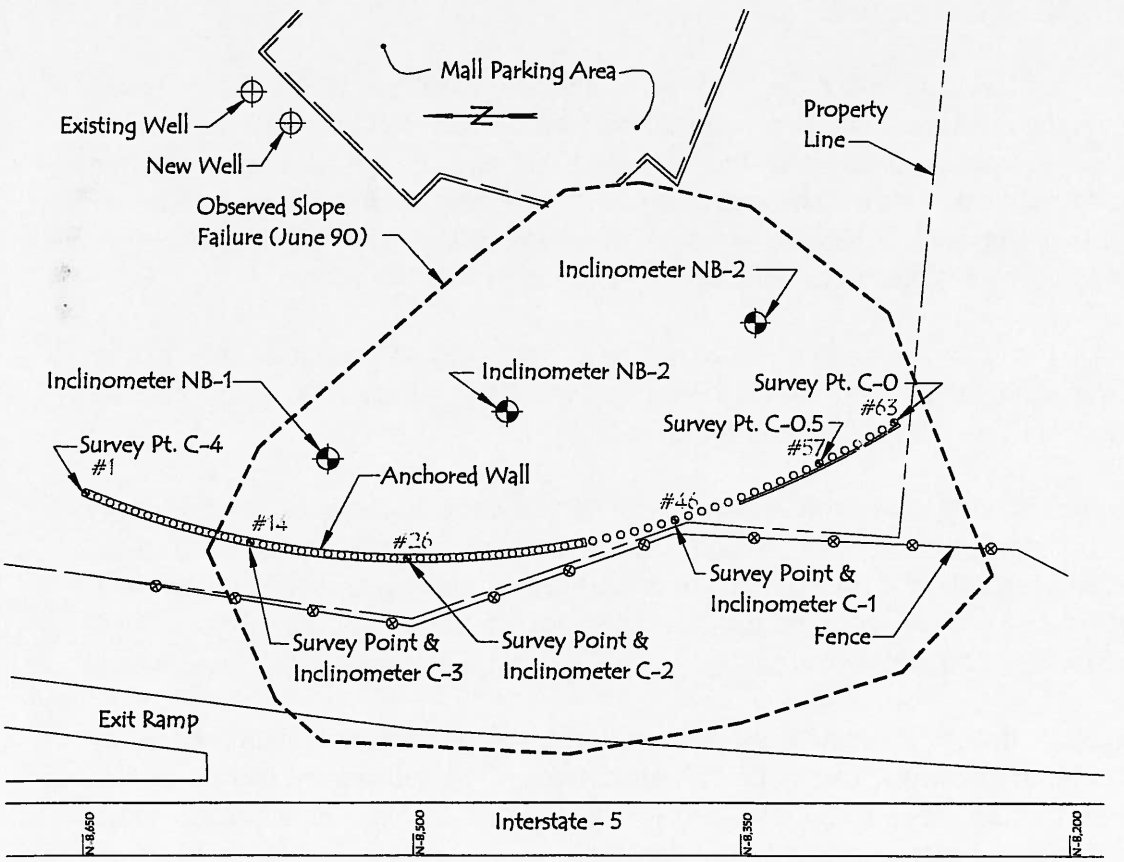


Figure 1 - Project Plan View

Several factors were concluded to correlate with the observed slope movements.

- Piezometric levels were measured up to 20 higher near the slope since the original construction of the mall
- Remolded clay zones within the Weathered Capistrano formation provided likely avenues for slippage. These zones may have been present from previous landsliding or may have been associated with prior folding, uplifting, or faulting activities
- Irrigation and drainage utilities were broken near the slope. Measured water contents of the fill materials were 5 to 9 percent higher than the optimum 17 percent achieved during placement
- Creep of the slope, associated with the increase in moisture due to irrigation, may have separated irrigation and drainage utilities, thereby activating the slide with large quantities of water

Slope Stability Evaluation

The geometry of the slide was defined by the headscarp, the depth to the failure plane indicated by the inclinometers, toe bulging, and associated surface cracking. This information was used, along with the observed subsurface soil and groundwater conditions, to back calculate the shear strength of the failure plane, assuming a factor of safety of 1.0. The back calculated average shear strength of the failure plane was determined to have a residual strength of about 15 degrees using this process.

The effect of the interim grading and dewatering work was evaluated based on this residual shear strength and the assumed failure surface. It was found that an increase of almost 20 percent was achieved by this work.

It was recommended by the geotechnical consultant that the final repair for the slope include consideration of elevated groundwater levels and seismic loadings. It was therefore established that the final design conditions provide for a rise in the groundwater level to the ground surface. A target factor of safety of 1.5 was established for the slope under static loading conditions.

Seismic stability design parameters included a horizontal ground acceleration of 0.32, associated with an assumed magnitude 7.0 earthquake. The seismic evaluation of the slope was performed using a Makdisi-Seed (1978) type of analysis, as a pseudo-static approach was deemed inappropriate for this application

Wall Design

The wall design was produced as a cooperative effort between members of Gannett Fleming, Inc., the wall structural designer, Nicholson Construction, and Woodward Clyde

Consultants. The wall design as presented in Figure 2 consisted of 63 each 42 inch diameter reinforced concrete drilled piles spaced at 6 to 6.5 feet on center supported at the top with three rows of tieback anchors.

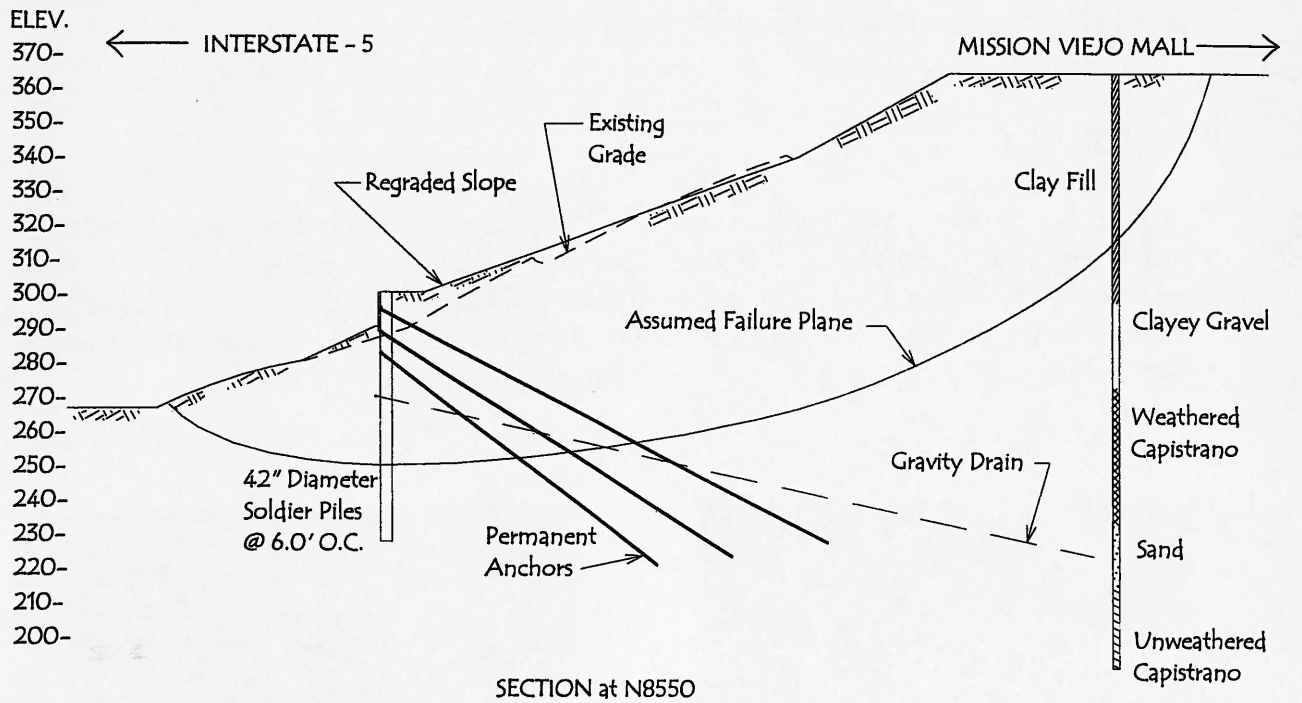


Figure 2 - Anchored Wall Section View

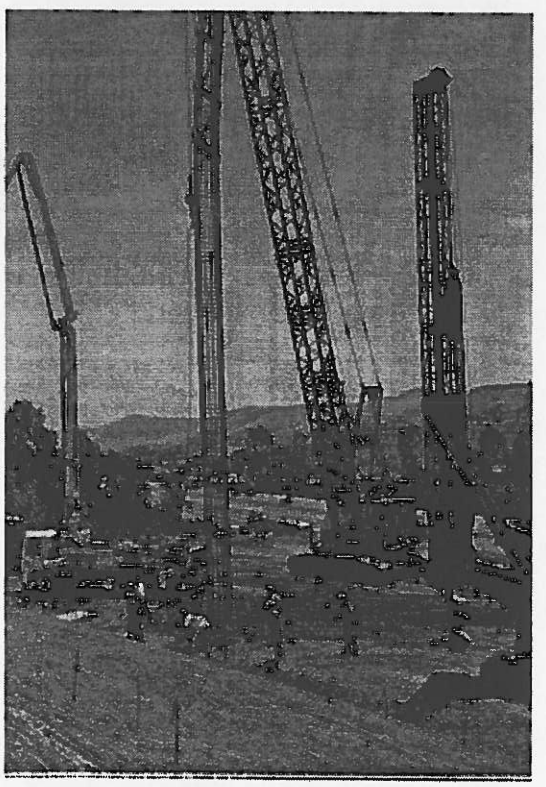
The depth to the failure plane at the wall location was approximately 50 feet below the top of wall. The drilled piles extend an additional 23 feet below the slide plane to provide passive support for the base of the pile. The tieback anchors have a design capacity of 250 kips which is developed in a 40 to 50 foot long bond length. An anchor "unbonded" length ranging in length from 54 to 118 feet was required to found the anchor beyond the slide mass.

The wall resisting force required to provide a slope stability factor of safety of 1.5 was determined through the stability analysis developed by Woodward Clyde. The capacity of the three anchors plus the shear strength of the pile at the failure plane interface provide this resisting force. The required force distribution acting on the wall for the pile structural analysis was determined by Gannett Fleming. The piles were also analyzed for the stresses and deflections induced by the application of the anchor forces ignoring the presence of the slide forces. This was accomplished using a beam on elastic foundation analysis method.

A key consideration to the wall design was to locate the anchors in the upper portion of the pile, reducing the amount of excavation and backfill required for anchor installation. This provided for a more cost-effective design but increased the structural demand on the piles. The pile reinforcing cage, which weighed approximately 100 pound per foot, included grouping of the bars towards the front and back of the pile to optimize the bending strength provided. Steel pipe blockouts were tied into the cages to provide an access points through the piles for anchor installation.

Construction

A test anchor program was conducted as the first order of work. Five anchors were installed and load tested to their ultimate capacities. Due to the creep sensitivity of the clay soils, several long term creep holds up to 24 hours in duration were conducted. Based on the results of the test program, the initial production anchors were installed with a 40 foot long bond length to provide the 250 kip design capacity.



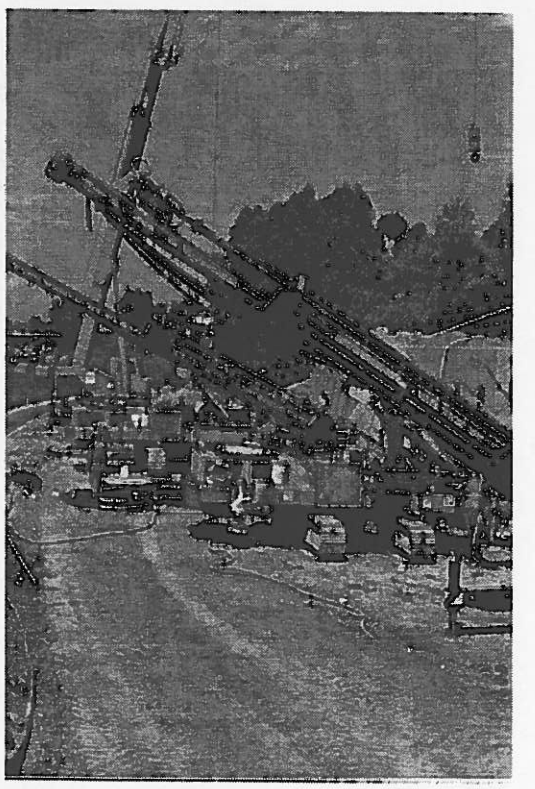


Photo 1 & 2 - Drilled Pile & Tieback Anchor Installation

The drilled piles were installed using a crawler mounted auger drill as shown in Photo 1. The pile holes were stable, with no caving and minimal water problems experienced. The top section of the northern portion of the wall which extended above existing grade was formed by pouring a continuous 4 foot thick grade beam. A concrete facing was placed over the less exposed southern portion of the wall. The soil was considered to arch between the piles below final grade.

The tieback anchors were installed using cased hole drilling techniques as shown in Photo 2. Water and/or air were used as a flush medium for transporting the soils out of the anchor holes. To further isolate the bonded length of the anchors from the slide mass, a bentonite slurry was inject into the anchor at the top of the bond length after cement grouting. This provided for a non-structural compressible zone which allowed movement of the bond length grout plug if anchor failure occurred.

Additional components of the construction included installation of drilled horizontal gravity drains, installation of a surface drainage collection system, filling of the major surface cracks with grout, and installation of an instrumentation system as described below.

Instrumentation

To monitor performance of the wall during and after construction, an instrumentation system was installed which included three inclinometers installed midslope above the wall alignment. The inclinometers were installed in three of the drilled piles. Load cells for monitoring the load for the anchors were also included at these three piles. Survey points were also installed along the top of the wall. A plot of the load cell readings for the anchors at one of the piles is included as Figure 3. A plot of the readings from one of the midslope inclinometers is attached as Figure 4.

Construction Difficulties

Anchor geotechnical failure was experienced in some of the anchors in the first and a portion of the second rows where 40 foot long bond lengths were installed. After these failures, the bond length for the remainder of the anchors was increased to 50 feet. The slope stability was reanalyzed taking into account the reduced capacity of the failed anchors. The problem was addressed through the installation of four additional anchors connected to the piles with concrete wales.

The second main difficulty involved movement of the piles during and after installation of the anchors. The slope behind the wall was disturbed from the previous slope movements. The air used during anchor drilling increased the air pressure in the cracks which resulted in increased slope movement and increase of the loading on the piles. The

deflection at the tops of the piles reached a maximum of approximately 2.5 inches in the downhill (Eastern) direction, with the pile curvature starting up from the failure plane location. As the anchors were tested and locked off, the piles gradually moved back to near their initial alignment.

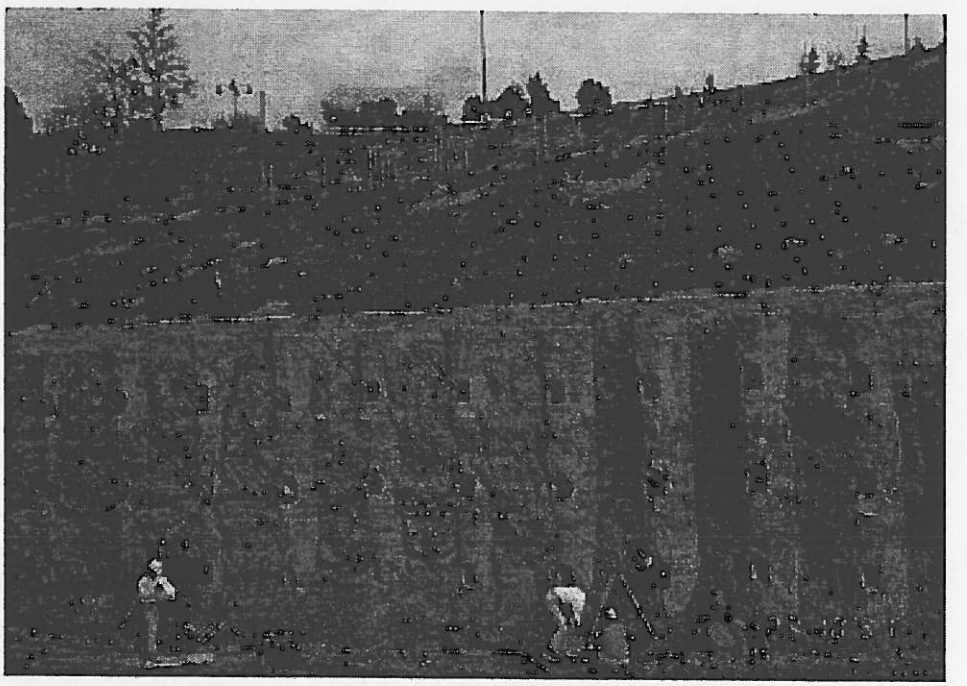


Photo 3 - Tieback Anchor Stressing

The piles deflecting back upslope resulted in relaxation of the anchor loading. It was necessary to verify that the reduction in the anchor loading was a result of this movement and not from anchor bond creep. To accomplish this, the elastic shortening of the anchor that was necessary to produce the measured drop in load was calculated and compared to the wall deflections measured through the surveys and inclinometer monitoring. The correlation between the data for all three of the instrumented piles was very good. An example of this comparison is included as Figure 5.

Conclusions

The last reported readings from the wall instrumentation were in 1995, which showed a continued decrease in the rate of wall movement and anchor load reduction. Visual inspections after the usual California catastrophic events (earthquake, heavy rains, famine) have revealed no signs of stress in the wall system.

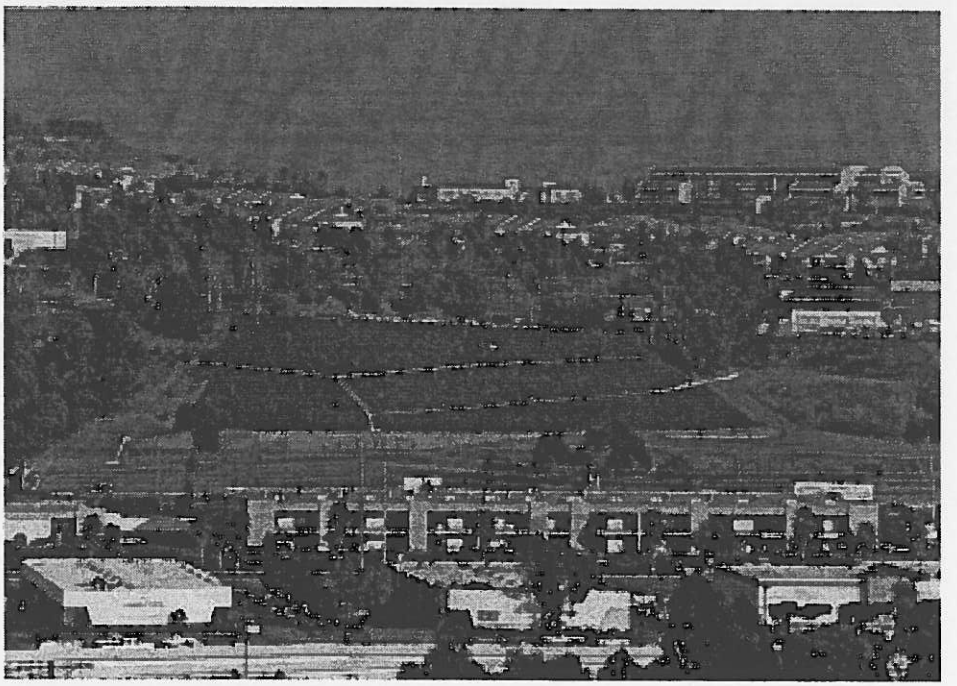


Photo 4 - View of the Completed Project

Acknowledgments

The owner of the project was the Edward J. Debartolo Corporation, Youngstown, Ohio. The design/build contractor was Nicholson Construction Company, Riverside, California. The authors were in the employ of Nicholson at the time of the project. The geotechnical consultant was Woodward-Clyde Consultants, Santa Ana, California. Mr. Dennis Glaser was the project manager for Nicholson. Mr. Jim Sebastian was the Project Representative for the Debartolo Corporation. Ms Paulette Alderfer prepared the drawings for this paper.

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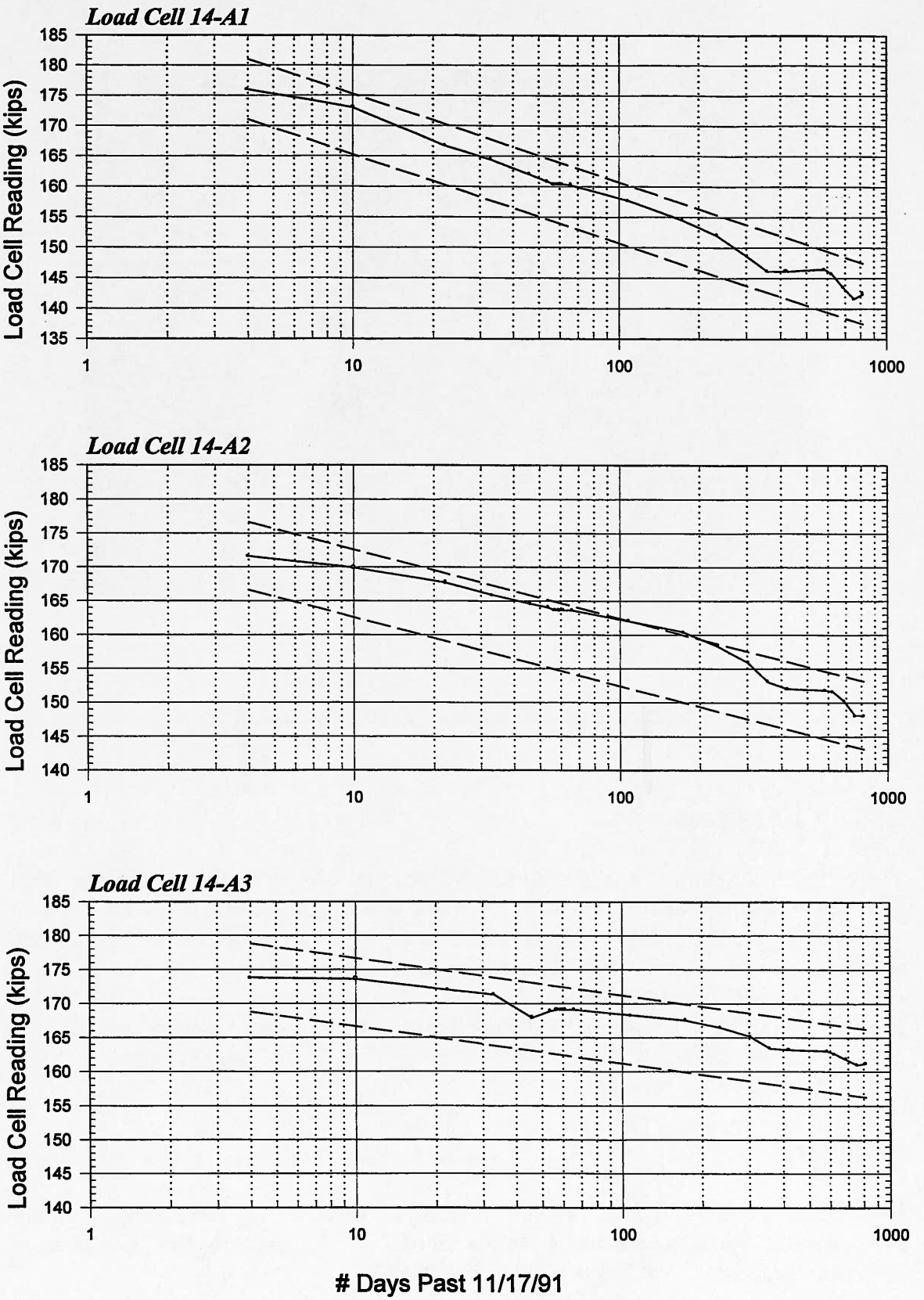


Figure 3 - Anchor Load Cell Readings - Pile # 14

Inclinometer NB-2
Middle Midslope Inclinometer
Monitoring Period
6/13-11/05/91

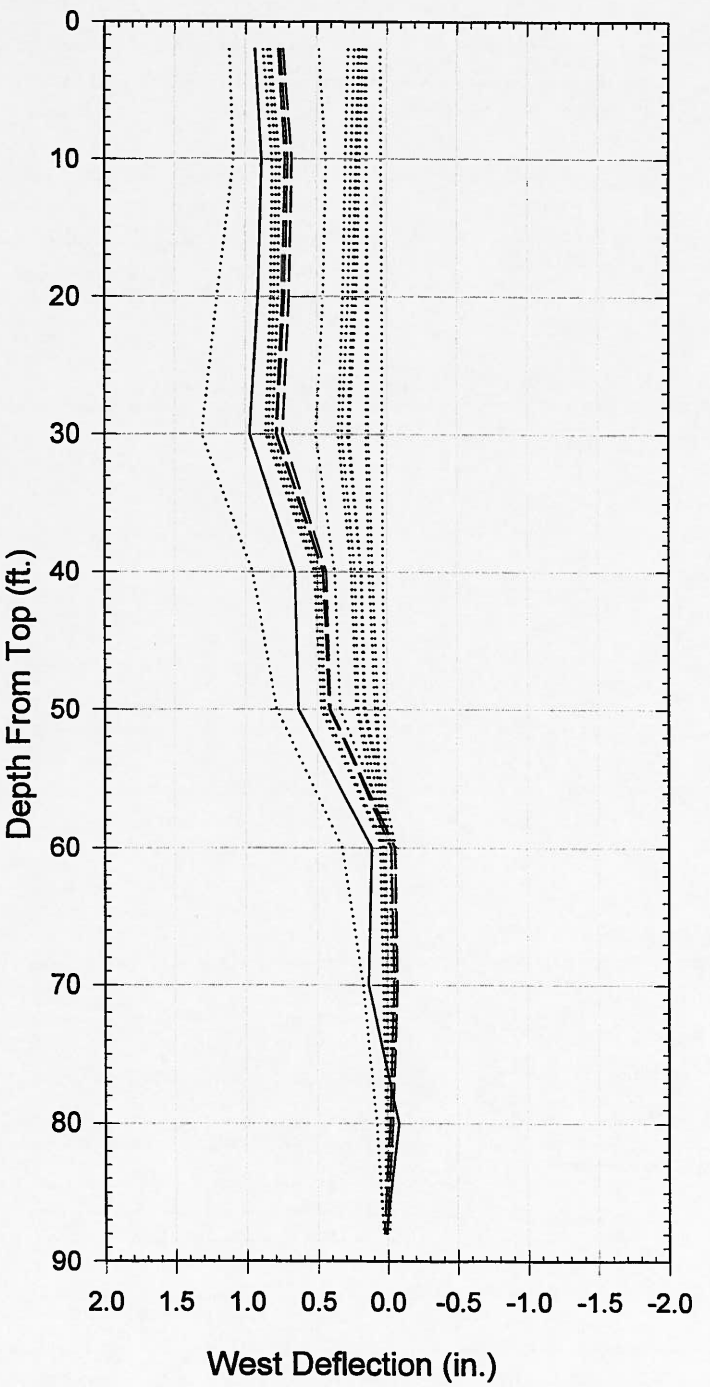
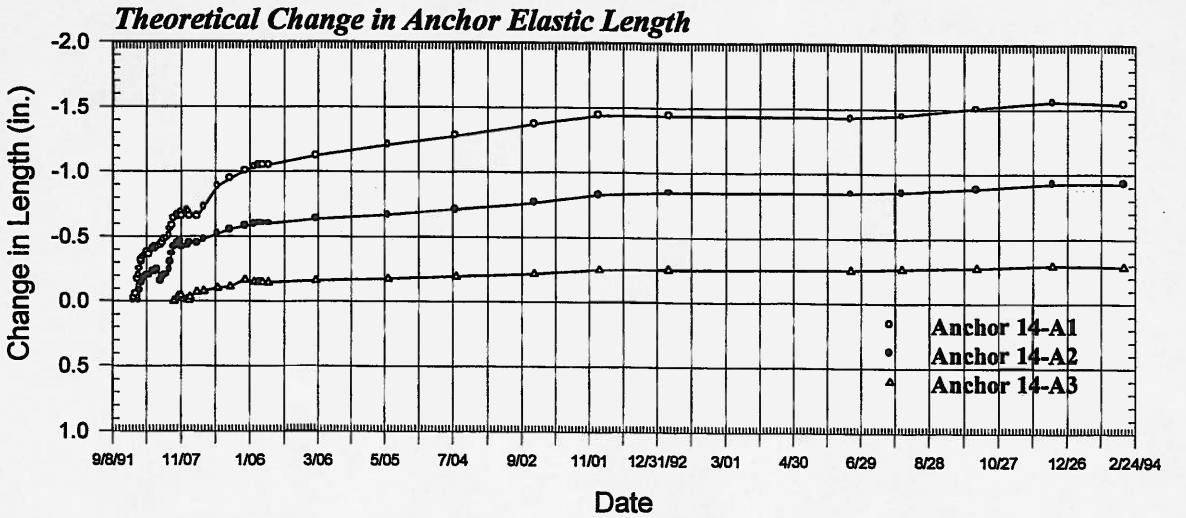
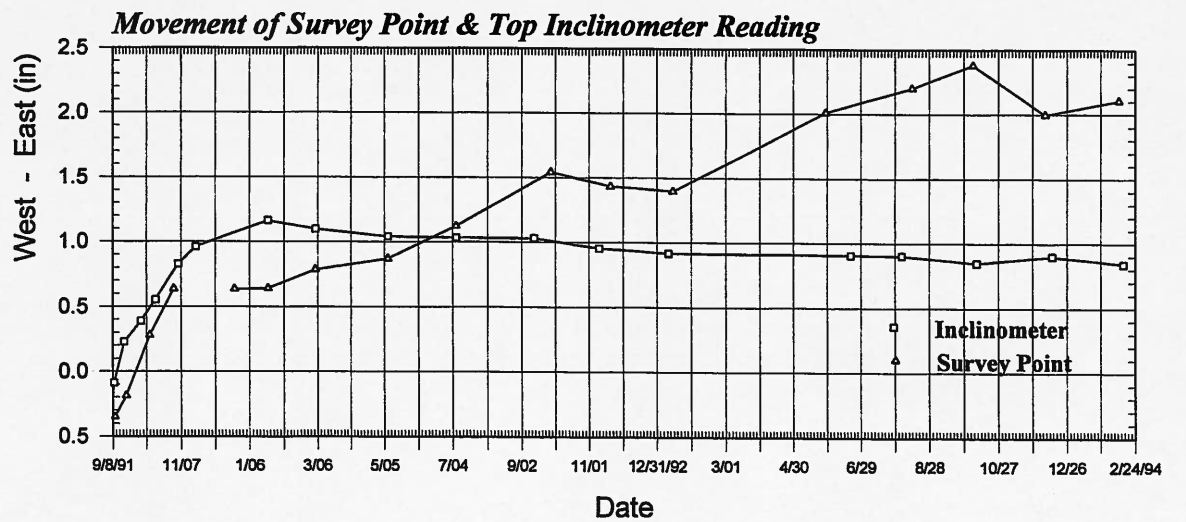


Figure 4 - Plot of Inclinometer Readings – Mid Slope Inclinometer





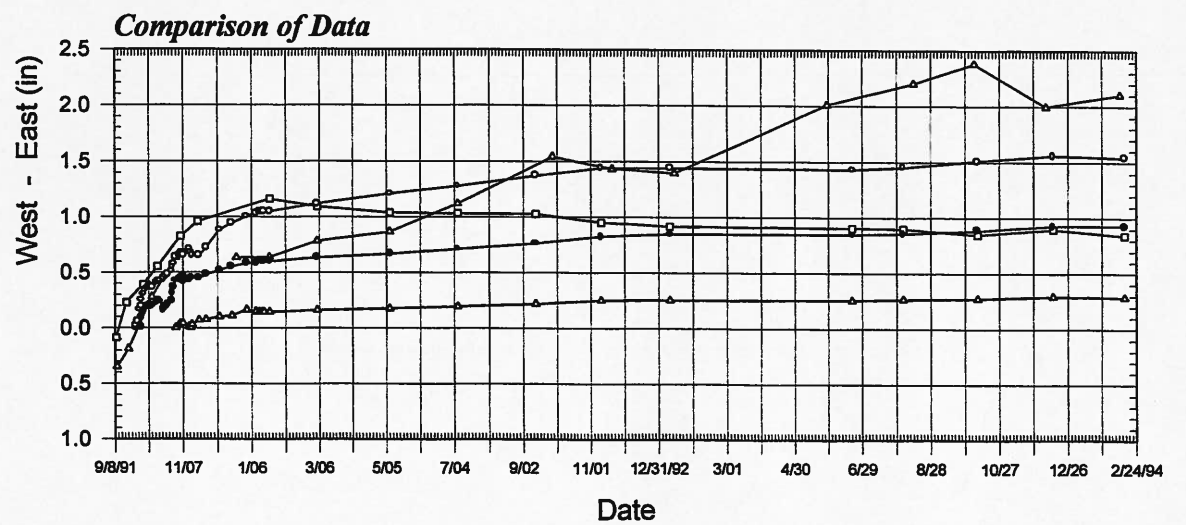


Figure 5 - Analysis of Anchor Performance - Pile #14

DESIGN AND PERFORMANCE OF FOUNDATIONS IN EXPANSIVE SHALE

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ABSTRACT

Expansive shale has been documented in the eastern United States, parts of Canada, and western Europe. Although the expansion process in black shale is well published, less attention has been given to the engineering evaluation of the expansive potential and application of preventative measures in engineering practice. This paper describes the engineering techniques used by the authors in the expansive shale of the upper east Tennessee and southwest Virginia region. Study of structural damage due to expanding shale, including the first documented case in the upper east Tennessee region, indicates that the expansive potential increases inverse proportionally to the shale's degree of weathering and proportionally to the shale's sulfide sulfate content. Detailed review of case histories of structural distress further indicates that relatively light structural components are the most at risk. That is, the authors have observed uplift of floor slabs and lightly loaded spread foundations while heavily loaded foundations such as rock bearing drilled shafts, are very rarely observed to experience uplift. This indicates that either the actual foundation loads, supplemental loads applied by pre-stressed anchors, or a combination of both of these, may be used to resist the uplift pressures generated during expansion. The authors have employed this approach successfully in a variety of ways.

INTRODUCTION

Black, unweathered, pyritic shale tends to undergo an increase in volume when exposed to air and moisture. This volume increase can result in damaging heave to overlying structures. Practicing geotechnical engineers need to be able to recognize and evaluate potentially expansive shale geologies. Provided the expansive shale is identified, there are several construction and design approaches that may be incorporated into the project to reduce the risk of damage due to heave. Published literature and the authors' experience indicate that it is lightly loaded structures that are most susceptible. Unfortunately, lightly loaded structures have smaller budgets available for geotechnical exploration than more heavily loaded, larger projects. This realization, coupled with the fact that very little can be done for in-place structures founded on heaving shale, emphasizes the importance of early recognition and proper interpretation of the shale condition by the geotechnical engineer during the design exploration.

LITERATURE REVIEW

Expansive pyritic shale has been reported in Ohio (ENR, 1960); West Virginia (Dubbe *et al.*, 1984); Pennsylvania (Spanovich, 1969); Missouri and Kansas (Coveney and Parizek, 1977); Tennessee (Long and Williams, 1990; Goodman, 1993); Ontario and Quebec, Canada (Penner *et al.*, 1970; Quigley and Vogan, 1970; Gillott *et al.*, 1974; and Bérubé *et al.*, 1986); England (Hawkins and Pinches, 1987; Nixon, 1978); Oslo, Norway (Moum and Rosenquist, 1959); and Östersund, Sweden (Jangdal, 1971). Table 1 summarizes available information concerning shale geologies that have been documented to be expansive. Nixon (1978) presents a summary of geologic formations in England that have a potential for causing expansion through pyritic oxidation.

In summary, most of the engineering studies of black pyritic shale conclude that heave is caused by oxidation of pyrite (FeS_2) and the formation of new minerals. The new minerals have a significantly lower density and greater volume than pyrite and therefore, uplift pressures and heave result. The pyrite contents of expansive shales have been reported to range from a minimum of 0.1 percent to over 20 percent (by weight). There has been some disagreement between researchers with respect to the exact oxidation process and the resulting heave-causing minerals. Spanovich (1969) identifes seven oxidation products from pyrite including melanterite ($FeSO_4.7H_2O$), rozenite ($FeSO_4.4H_2O$), coquimbite ($Fe_2(SO_4)_3.8H_2O$), jarosite ($K.Fe_3(SO_4)_2(OH)_6$), limonite (variable composition), gypsum ($CaSO_4.2H_2O$), and kaolinite [$Al_2Si_2O_5(OH_4)$]. Hawkins and Pinches (1987) conclude that heave in the Westbury Formation was caused by the growth of secondary minerals, primarily gypsum along the bedding partings which is consistent with the conclusions made by several others (Dougherty and Barsotti, 1972; Dubbe *et al.*, 1984; Lutenegger *et al.*, 1979; Quigley and Vogan, 1970).

Table 1: Shale Geologies Documented to be Expansive

Geology	Heave (cm)	Heave Rate (cm/year)	Uplift Pressure (kPa)	Reference
Appalachian carbonaceous shale	37-50	NR	475-575+	Spanovich, 1969
Billings Shale	9.5	0.6-2.2	NR	Penner *et al.*, 1970
Conemaugh/Monongahela Form.	38 (projected)	NR	190-475	Dougherty and Barsotti, 1972 Dubbe *et al.*, 1984
Eastview Formation	6	NR	NR	Grattan-Bellew and Eden, 1975
Hushpuckney Shale	15-20	NR	NR	Coveney and Parizek, 1977
Lias Clay	NR	NR	NR	Chandler, 1972
Lorraine Formation	10	0.25-2	72	Quigley and Vogan, 1970
Rome Formation	NR	NR	NR	Goodman, 1993
Sainte-Foy Formation	10-20+	>2	NR	Bérubé, *et al.*, 1986
Sevier Shale Formation	11+	1.5	<75	Long and Williams, 1990
Westbury Formation	6	NR	NR	Hawkins and Pinches, 1987
Whitbian Shale (fill)	NR	NR	NR	Nixon, 1978

NR – Not Reported

Several researchers attribute all or some of the oxidation process to activity of bacteria (Hawkins and Pinches, 1987; Penner *et al.*, 1970; Quigley and Vogan, 1970).

The shale expansion process begins with the oxidation of pyrite, when fresh, unweathered shale is exposed to a humid and oxidizing environment. As pyrite is oxidized, sulfuric acid is created as described in the following chemical equation:

FeS_2	+	$2H_2O$	+	$7O_2$	→	$2FeSO_4$	+	$2H_2SO_4$
pyrite		water		oxygen		ferrous sulfate		sulfuric acid

The sulfuric acid then reacts with available calcium carbonate to form sulfate-rich solutions which tend to migrate to the surface by capillary action (Bérubé, *et al.*,

1986); driving forces from temperature gradients (Bérard, 1970); and by the pressure gradient within discontinuities due to the fissuring near the rock surface (Penner *et al.*, 1973a; Grattan-Bellew and Eden, 1975). As the pressure within the discontinuities is reduced, the sulfate-rich solutions precipitate as sulfate minerals, primarily gypsum.

$CaCO_3$	+	H_2SO4	+	H_2O	→	$CaSO_4.2H_2O$ (and other sulfate minerals)	+	H_2CO_3
calcium carbonate		sulfuric acid		water		gypsum		carbonic acid

The preceding reactions can be self-conducting; however, it is expected that the continuation of the process may be difficult in low pH conditions. Several researchers (Hawkins and Pinches, 1987; Penner *et al.*, 1970; Quigley and Vogan, 1970) have reported that oxidizing bacteria, such as Ferrobacillus and Thiobacillus ferrooxidans, will assist in the continuing creation of sulfuric acid (even in low pH conditions) as follows:

$4FeSO_4$	+	O_2	+	$2H_2SO_4$	→	$2Fe_2(SO_4)_2$	+	H_2O
ferrous sulfate		oxygen		sulfuric acid		ferric sulfate		water

and,

$7FeSO_4$+	FeS_2	+	$8H_2O$	→	$15FeSO_4$	+	$8H_2SO_4$	
ferric sulfate	pyrite		water		ferrous sulfate		sulfuric acid	

As summarized in Table 1, the heave associated with pyritic shale has approached 50 cm. Observations of weathered and unweathered shale indicate that it is only the upper 1.5 to 2 meters of the rock surface that experiences the formation of gypsum. Therefore, the volume change is estimated to be on the order of 25 to 33 percent. Comparisons between theoretical and measured values of heave indicate that the gypsum crystals do not completely fill discontinuities and openings in the rock, but that the growth of gypsum crystals at points along the discontinuities is sufficient to lift overlying rock layers and structures.

The onset of the shale heave has been observed to typically begin a few years after construction (Penner *et al*, 1973a), and the reported rate of heave ranges from .25 cm/year to over 2 cm/year. The duration of the heaving process has been observed to continue over a 40-year span (ENR, 1960; Dougherty and Barsotti, 1972). Laboratory tests indicate that the uplift pressure could be as high as 500 kPa (Lutenegger *et al*, 1979). Comparison of the loads on distressed and non-distressed portions of structures indicates that in many cases the uplift pressures are significantly less. We note that because the pyrite content is variable, conclusions based on different locations within the same formation may not be appropriate.

ENGINEERING PRACTICES TO MITIGATE HEAVE RISK

The goal of the geotechnical engineer is to identify the expansive shale and then recommend methods of risk mitigation to the owner and design team. There is, of course, the recommendation of avoiding the pyrite-bearing material either by selecting a location with more favorable conditions or designing the structure above the unweathered/ shale portion. The former recommendation can be applicable, but cannot always be the solution as better available sites become more scarce. The latter recommendation can be considered the first approach to developing an expansive shale site; however, project requirements often dictate the structure's location and elevation. As a consequence, the geotechnical engineer, after identifying the expansive shale, will need to be able to evaluate the influence of the expansive shale on overlying structures and to prepare effective design approaches.

The remaining portions in this paper present the engineering practices of the authors and their experience in the Sevier Shale formation of the Valley and Ridge Physiographic Province. The Valley and Ridge can be characterized by elongated, northeasterly-trending ridges formed on more resistant sandstones and shales. Between ridges, broad valleys and rolling hills are formed on less resistant limestones, dolomites, and shales. The Sevier Shale formation is part of the Middle Ordovician System and in its unweathered state, is predominately a dark blue and black, calcareous shale with calcite streaks. In undisturbed areas, the Sevier Shale at the ground surface is typically weathered to a stiff clayey silt that contains greater amounts of shale as the depth increases. At moderate depths of 3 to 5 meters, the uppermost portion of the unweathered shale is encountered.

Evaluation

Identification and subsequent evaluation of the Sevier Shale begins with review of published geologic information and field exploration. The general near-surface locations of the Sevier Shale in the northeast Tennessee Region have been mapped by the State of Tennessee Department of Geology (DeBuchananne and Richardson, 1956). Once the site has been identified as underlain by the Sevier Shale formation, then observation of outcrops and collection of rock core samples is specifically included in the scope of the field exploration. The purpose of rock outcrop observation is to identify evidences of pyrite oxidation (most notably the presence of gypsum crystals and yellow jarositic alteration products). Rock core samples of the shale are also obtained using conventional coring methods (ASTM D2113). The rock cores are examined for weathering, the presence of calcite streaks, and evidences of pyrite oxidation. While the presence of calcium carbonate is necessary for the formation gypsum crystals, the authors believe that sufficient calcium carbonate is available in the shale and groundwater so that additional sources such as calcite streaks are not particularly necessary for heave.

In addition to the visual examination, the rock core is tested to determine its sulfide sulfur content, sulfate sulfur content, and total sulfur. Sulfide sulfur is considered the primary indicator of the shale's heave potential. Using published values of sulfide sulfur and heave, the authors have constructed the plot shown in Figure 1. A portion of the plot is based on the authors' experience with the Sevier Shale. The sulfide sulfur of unweathered Sevier Shale has been observed to range up to 1.5 percent, which puts it in the high range with respect to heave potential. However, tests performed on material obtained below heave-distressed structures confirm that shale with even lower sulfide sulfur contents can undergo heave. Particularly, portions of the Sevier Shale with sulfide sulfur contents in the range of 0.1 to 0.5 percent or greater have caused heave in overlying structures.

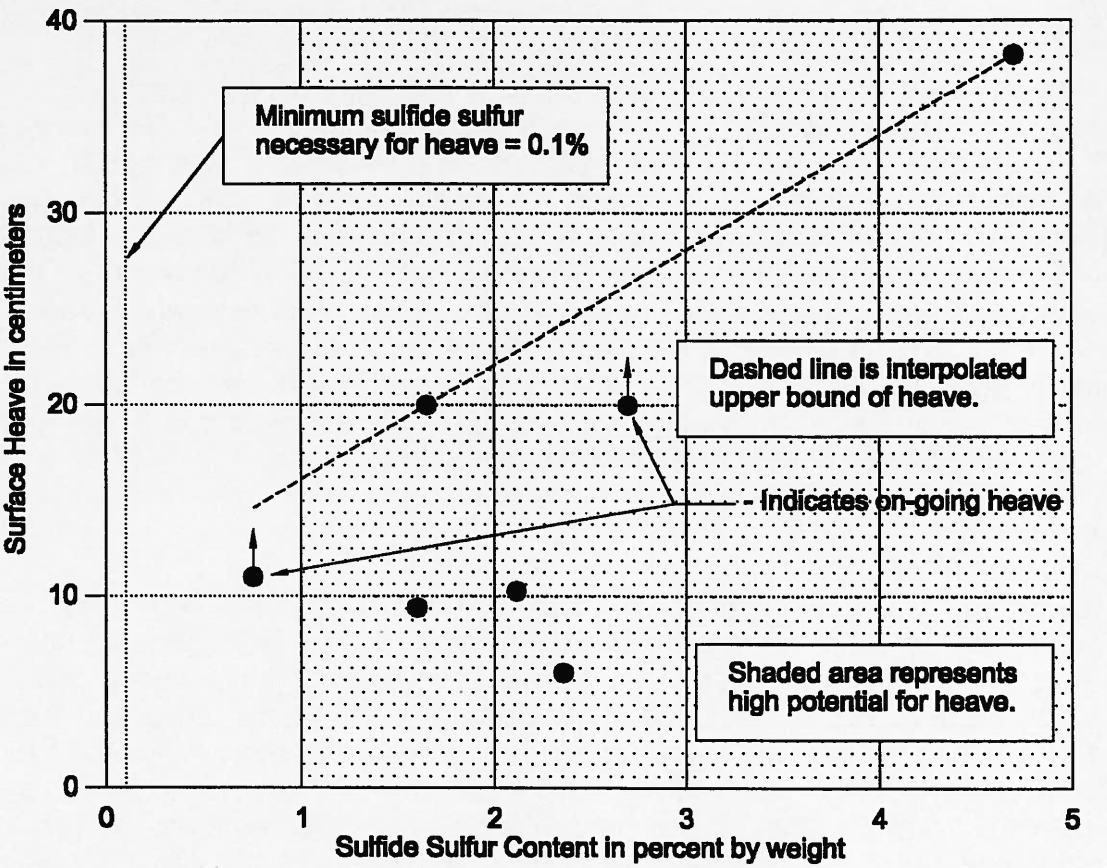


Figure 1: Plot of Sulfide Sulfur Content versus Heave

Though less important than the sulfide sulfur content, the sulfate sulfur content and total sulfur can also provide information into the shale heave potential. Because the sulfate sulfur content is a product of pyrite oxidation, the ratio of the sulfate sulfur to the total sulfur can be especially helpful. For example, a sulfate sulfur-to-total sulfur ratio near 100 percent indicates significantly oxidized or weathered portion of shale. Conversely, a low ratio indicates an unweathered or unaltered shale.

Design Approaches

Several design approaches to mitigate risks associated with expansive shale have been suggested in literature; however, the level of success that each approach achieved is not known. The design approaches include:

- placement of an impermeable coating (e.g. bitumen, gunite, concrete, etc.) on the unweathered shale surface to create an anaerobic environment in the underlying shale;
- rock bolting to offset expansion pressure;
- structural floors supported on foundations on a stable bearing material. (Typically, a space is provided between the subgrade and non-bearing components to prevent shale heave from causing movement in the overlying structure.);
- placement of a limestone aggregate or lime layer directly on the unweathered surface to reduce the pH in the underlying shale;
- permanent adjustment of the groundwater level to create an anaerobic environment, and;
- saturation of the near-surface shale with potassium hydroxide to reduce the pH.

Of the aforementioned design approaches, the authors consider the latter three undesirable. The use of a limestone aggregate or lime layer is a reasonable approach for reducing the pH in contact with the structure and therefore reducing acid attack. However, with respect to the expansion process, the limestone aggregate or lime layer probably does not significantly reduce the pH within the shale discontinuities. Furthermore, the limestone aggregate or lime layer may leach calcium carbonate, a necessary component in the formation of gypsum, and actually increase the potential for shale heave. Because permanent adjustment of the groundwater level or saturation of the near-surface shale with potassium hydroxide would require installing a permanent source of water or potassium hydroxide, respectively, these two approaches are not very practical at most sites in the upper east Tennessee and southwest Virginia area. There also is the question of whether the potassium hydroxide will be successfully delivered to all of the rock discontinuities.

The authors have applied the other three options in practice with success. Because the impermeable coating is inherently a less sure solution, this approach is reserved for less critical structures such as pavements. The reasoning is that this approach is less direct in that it intends to seal the discontinuities in the rock. With even the most conscientious applicator, there is no confirmation or verification that all susceptible discontinuities have been successfully sealed. In fact, the only verification is the performance of the overlying structure.

The authors have used rock bolting primarily for floor slabs and other lightly-loaded structures, specifically structures with a contact pressure of less than 475 kPa (about 10 kips per square foot). Typically, these rock bolts are pre-tensioned so that the total sustained pressure on the shale subgrade is at least 475 kPa. There are two reasons for prestressing these anchors. The primary reason for prestressing is to keep near-surface discontinuities closed and thus inhibit the formation of gypsum. The second reason is that a non-prestressed (passive) anchor would only mobilize tension during heave, which may cause cracking and other distress. The authors' experience have used a rock anchor spacing of about one meter and anchor length of about 3 meters to achieve the design prestress in floor slabs. The actual spacing for any specific project should be a function of the actual rock anchor and floor slab design.

For a variety of reasons (some unrelated to the potential for heave), the authors have readily used rock-bearing drilled shafts (i.e. deep foundations) for support of structures overlying the Sevier Shale. One advantage of drilled shafts is that applied loads are typical high and small contact areas may be maintained. Therefore, achieving a sustained end bearing pressure of 475 kPa or greater without the use of rock anchors is not difficult. A second advantage is that the water table is typical located within the few feet into the hard shale, and therefore, anaerobic conditions are maintained in the subgrade of the drilled shaft. The authors have observed placement of several thousand drilled shafts in the Sevier Shale over the last 20 years and are not aware of any failures due to heave in the bearing stratum.

In areas of shallow rock, spread foundations have been designed in a manner similar to drilled shafts. That is, the minimum sustained bearing pressure is at least 475 kPa. For relatively light structures, the sustained load on spread foundations may need to be augmented using rock anchors. For projects that have shallow groundwater (e.g., a bridge over a river), the anaerobic conditions allow even lightly-loaded foundations to bear directly on the unweathered shale without the minimum bearing pressure requirement.

CASE HISTORIES IN EXPANSIVE SEVIER SHALE

The following two case histories help illustrate the importance of proper foundation design in expansive shale. Both case histories involve construction over the Sevier Shale formation of the Valley and Ridge Physiographic Province. In review of numerous projects, a sulfide sulfur content range of 0.4 to 0.9 is typical for unweathered Sevier Shale; however, the sulfide sulfur content in portions of the Sevier Shale has been measured to be as high as 1.5 percent.

Library Damaged by the Sevier Shale

The cause of distress at the Johnson City, Tennessee Public Library was determined to be heave of pyritic shale (Long and Williams, 1990). The authors believe that this is the first documented case of structural distress due to secondary mineral (i.e. gypsum) growth in black, pyritic shale of East Tennessee. Structurally, the two-story library has a plan area of about 6,000 square meters and is slab-on-grade/spread foundation construction. The library was constructed in 1979. By 1982, significant cracking of floor slabs and buckling of partition walls had developed, particularly in areas underlain by fill. Prior to any geotechnical investigation, some repairs of distressed areas were performed in 1982. The repaired areas continued to experience distress and other areas began experiencing similar distress after a few more years. Due to the continuation of the distress, a thorough geotechnical exploration was performed to explore the building subgrade conditions, to identify likely causes of the lift, and to evaluate possible repair options.

The results of the geotechnical exploration revealed that the building subgrade consisted of undisturbed shale in the southern portion and shale fill in the northern portion. In the undisturbed shale areas, excavation of up to 8 meters of overburden was performed during construction. The fill encountered in the exploration ranged up to 1.5 meters in thickness. Laboratory tests indicate that the sulfide sulfur content in the unweathered shale typically ranged from 0.59 to 0.71 percent. Vertical displacement of the floor slab at construction joints and cracks ranged between 2 and 3 cm and differential vertical displacement within a given room approached 11 cm. The fill-supported portion of the library experienced heave within 3 years after construction as compared to 6 to 8 years in the portions supported by undisturbed shale. This is consistent with observations by others and further supports the conclusion that the larger surface area present in shale fill is more susceptible to formation of gypsum. We note that gypsum was also present in a layer of dense-graded crushed limestone base immediately below the floor slab. Comparison of estimated dead loads and column heave did not reveal any significant trend. The lack of trend between the two can be explained by the large variation of pyrite content that is characteristic of the Sevier Shale.

Convention Center Designed for Support on Sevier Shale

A 60,000 square meter convention center was constructed over the Sevier Shale formation in 1995. The design geotechnical exploration indicated that the soil overburden was relatively thin and that samples of the shale bedrock had a sulfide sulfur content between 0.44 percent and 1.43 percent. The potential for heave was recognized, and as a consequence, the design intended to support the foundations either in fill or relatively shallow excavations in the undisturbed soil. However, an escalator pit added during construction required an excavation of roughly 7 to 8 meters into undisturbed material. At this depth, the Sevier Shale was black and very hard.

The original design of the escalator pit called for a retaining wall on four sides each supported by a continuous foundation. The design bearing pressure for this structure was less than 144 kPa . Prestressed rock anchors were considered as one design solution. Because of the nature of the retaining wall, in that it represented a relatively deep beam situation, a redesign of the retaining wall was also considered. The redesign essentially consisted of alternating bearing and non-bearing portions of wall foundation. This "tip-toe" foundation is shown in Figure 2. In the bearing areas, the foundation pressure was maintained between 475 kPa and 950 kPa. In non-bearing areas, a space of roughly 30 cm was created using cardboard forms between the shale subgrade and the cast-in-place concrete. Due to its cost-effectiveness, the "tip-toe" foundation was constructed.

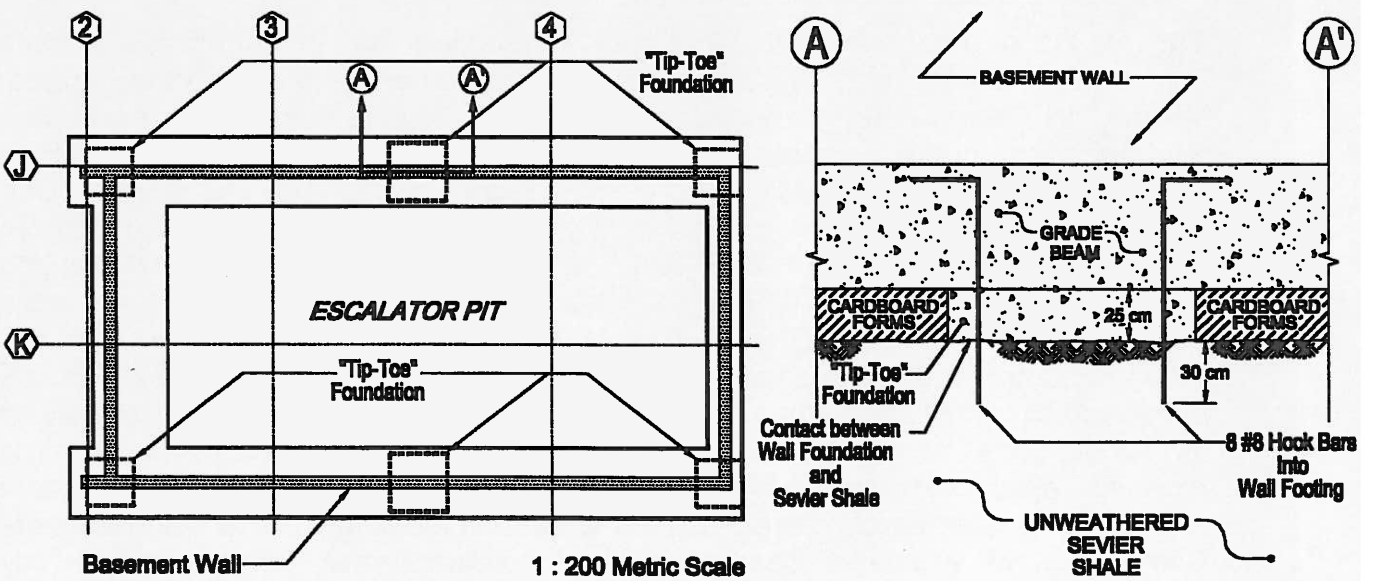


Figure 2: Schematic of "Tip-Toe" Foundation

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PROBLEMATIC SOILS IMPROVED BY COMPACTION GROUTING

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ABSTRACT

Compaction grouting is becoming an increasingly versatile procedure for the *in situ* improvement of soils with specific geotechnical deficiencies or configurations. Originally developed and refined for the primary purpose of reducing settlement potential by compacting low-density soil beneath existing structures, a wide variety of proven applications for compaction grouting has evolved to keep pace with the changing demands and needs of the geotechnical community regarding problematic soil types or deficient foundation systems. In addition, the limited-access capabilities that have necessarily accompanied the development of compaction grouting have allowed soils in unique locations and configurations to be improved *in situ*, regardless of the nature of their deficiency.

Structures today are being increasingly threatened by the inclusion and adverse performance of problematic soils in site development. This trend will continue and accelerate as the amount of undeveloped "prime" property shrinks in area due to ongoing development. Also, the relatively recent recognition of some soils as being "liquefiable" or "collapsible" results in either the potential or the actual poor performance as an adequate foundation material.

It is the intent of this paper to present several summary case histories of projects where compaction grouting was used to correct deficient foundation conditions not strictly describable as "a mass of soil that is subject to vertical settlement in response to existing or future static vertical stresses." Projects which illustrate the versatility of compaction grouting in reaching and upgrading relatively inaccessible zones of foundation soil are also presented. Finally, the ongoing research programs and possible future applications of the technique of compaction grouting for upgrading problematic soils are also discussed.

INTRODUCTION

The Origin of Problematic Soils

Statement: No problematic soil deposits have existed for longer than 10,000 years (pre-Holocene) anywhere on the planet Earth. True or false?

Despite the typical first reaction to this statement, the answer is "true." Although most natural soil deposits and rock formations have existed in virtually their current conditions far longer than this brief geologic time span, they did not represent any "problems" until we still-evolving humans began to rely upon soil stability when constructing man-made structures -- for example, that vertical round stone tower in Pisa, Italy. And any significant artificial fill deposits that might create "problems" for the engineering community are, by definition, younger than 10,000 years old.

The Variety of Problematic Soils

Problematic soil types are as varied as the designs of the engineers who want to build structures on, adjacent to, or through a soil or rock mass. The vertical downdrop of a ground surface over collapsible soils, which are subject to the breaking of weak interparticle bonds upon wetting or vibration, might go undetected if buildings or subsurface utilities did not rely on an unchanging ground elevation for adequate long-term performance. The same is true for karst terrain, with its subterranean voids and highly weathered soil horizons; clean low-density sands beneath a shallow groundwater table in earthquake country, with their potential for liquefaction; uncontrolled debris-landfill sites, with their possible macroscopic voids, organic material, and often a minimum of successful excavations during geotechnical investigations; and tunnel overburden, subject to settlement as a subsurface excavation proceeds along a horizontal path below the ground surface.

The Improvement of Problematic Soils with Compaction Grout

Compaction grouting is a technique being increasingly applied to resolve geotechnical deficiencies of problematic soils. Once thought of primarily as a procedure for densifying regional mixed-grain soil zones below depths of 10 to 15 feet, the specific site conditions which would preclude compaction grout from consideration are rapidly shrinking in number. Project economics have played a significant role in the variability of these applications, due to *in situ* soil improvement often being more economical than previous mitigation measures involving deepened foundation systems.

Briefly stated, compaction grouting is a process of slowly injecting 2 cubic feet or less per minute of a true low-slump soil-cement grout -- averaging 1.5 in., but in no case to exceed 2.0 in., as measured by ASTM C-143-78 -- capable of high internal friction and of remaining in an injected homogeneous mass, thereby displacing and compacting the in-situ soil in a controlled manner (Graf, 1969). Particular care and experience need to be applied in the selection of an appropriate grout mixture for the prevailing *in situ* soil properties, in order to minimize the Travel Index of the grout during injection. Specifically, a relatively harsh mixture is required to limit the radial grout propagation in fine-grained soils, whereas a mixture with higher fines content and non-plastic clay fraction is required in very coarse-grained soils. Referring to Figure 1, note that the "C" grout mixture had the least amount of fines in the aggregate, the "A" mixture had a higher percentage passing the #200 sieve, and the "B" mixture had been supplemented with a small amount of highly plastic clay.

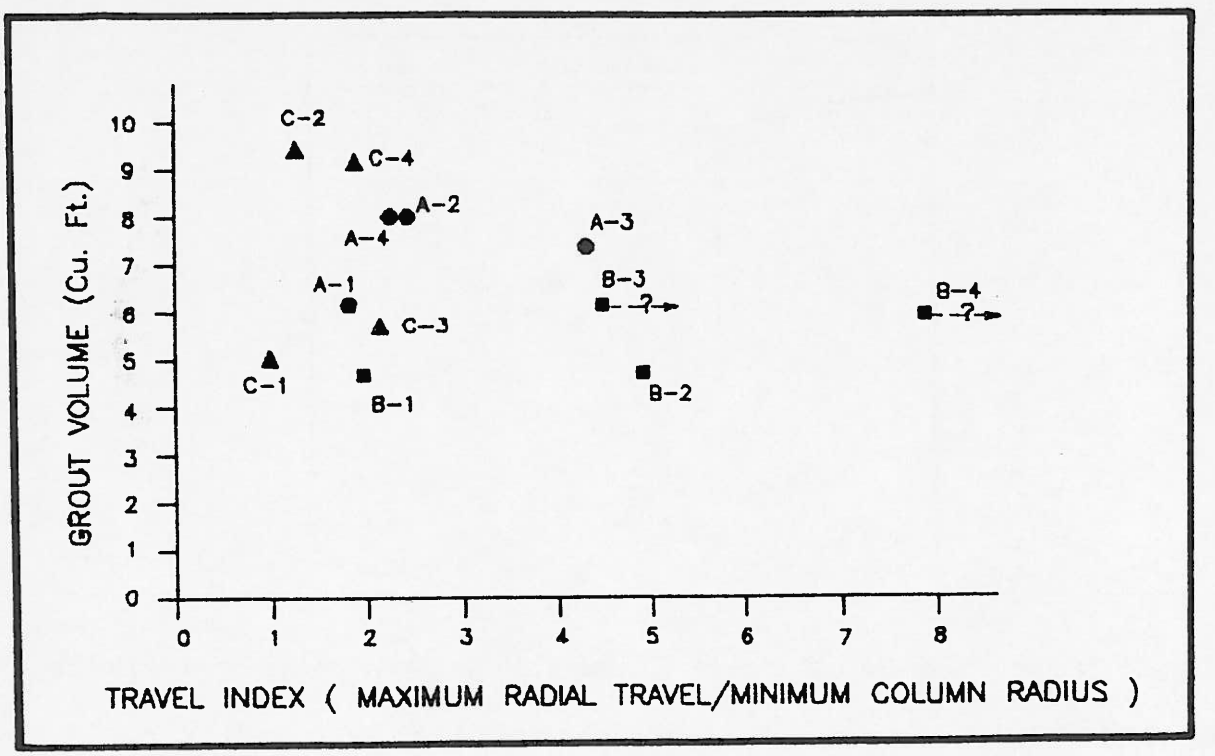


Figure 1: Travel Index for Three Different Compaction-Grout Mixtures (reprinted from Warner et al, 1992)

CASE HISTORIES

This paper presents summaries of case histories for ten different problematic soil zones or restricted-access environments, to help illustrate the variety of possible applications for controlled compaction grout to mitigate adverse geotechnical conditions.

Case History No. 1 Problematic Soil: **COLLAPSIBLE SANDS**

Project Name: *Municipal Building*
Location: *Rangley, Colorado*

THE PROBLEM: In 1982 an 8500-sq.-ft. addition was added to the existing structure. The addition was constructed on shallow spread footings with a slab-on-grade floor, while the existing structure was founded on deep foundations and a structural floor system. The site is located in a semi-arid environment, with the foundation soils being a 30-ft.-thick deposit of clayey silt (loess). Differential settlements of the new addition, due to collapse and consolidation of the loess, ranged from 2 to 11 inches.

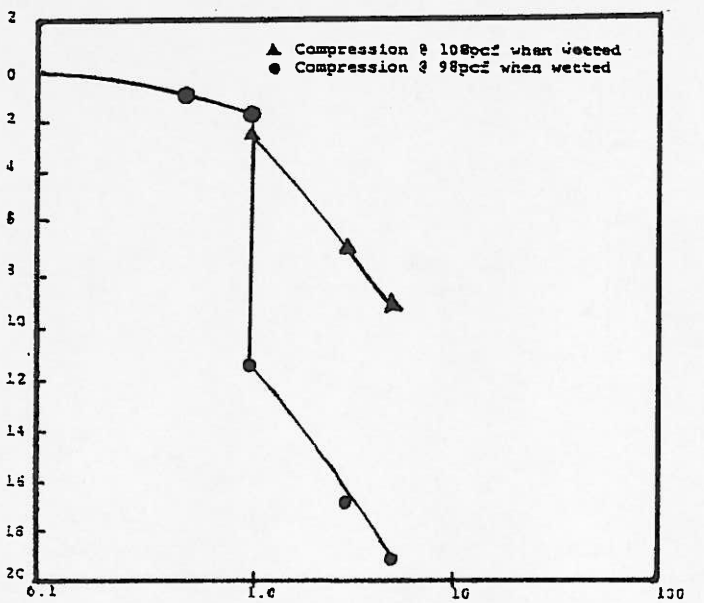


Figure 2: Consolidation Curve for Collapsible Sands at Municipal Building

THE SOLUTION: Laboratory tests showed that settlements could be arrested by increasing the *in situ* density of the entire loess deposit by 5 percent, from 98 pcf to 103 pcf (see Figure 2). A compaction grouting program was developed to: a.) increase *in situ* density, and b.) relevel the structure. Grout hole spacing was optimized at 8-ft. x 8-ft. centers, for a total of 133 grout holes. 12,750 ft.^{3} of grout was required to increase the density by the required 5 percent. This corresponded to 96 ft.^{3}/ hole, or 3.2 ft.^{3}/ stage (1 stage = 1 ft.). The grouting criteria of 3.2 ft.^{3}/ stage in each grout hole was established to provide uniformity of treatment throughout the soil horizon. Density increases were verified by down-hole nuclear densometer. At the conclusion of the compaction phase of the grouting program, the structure was releveled to within 0.5 in. of original elevation.

Case History No. 2 Problematic Soil: **MIXED-FACE TUNNEL**

Project Name: *Skinker-McCausland Tunnel*
Location: *St. Louis, Missouri*

THE PROBLEM: Affholder Inc. was contracted to bore a 10-ft.-diameter tunnel for the Metropolitan St. Louis Sewer District. The tunnel alignment was located 60 ft. below ground surface and was presumed to be in competent limestone for its entire alignment. However, during tunnel advancement an 80 LF section of tunnel was determined to be in mixed face conditions. Approximately 40 LF had a cinder & brick fill from the crown to spring line with hard limestone throughout the remainder of the face (Section A). About 30 LF was entirely within the cinder & brick fill with the invert only on hard limestone (Section B), while another 10 LF was located entirely within the cinder & brick fill with the invert also founded in the cinder & brick formation (Section C). These mixed conditions required pretunneling improvement to provide uniform soil conditions for the Tunnel Boring Machine (TBM) to maintain alignment and grade.

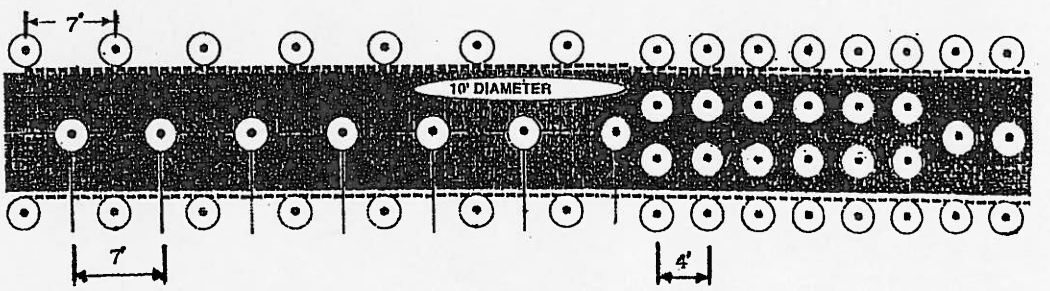


Figure 3: Plan Layout of Grout Holes for Skinker-McCausland Tunnel

THE SOLUTION: An *in situ* improvement program utilizing compaction grouting was performed to densify the cinder & brick fill materials as well as provide for uniform face consistency to allow the TBM to maintain alignment and grade. All grout holes were drilled and grouted from the ground surface. The program generally consisted of three rows of grout holes, one at centerline, and one each along both outside edges of the tunnel (see Figure 3). Grout hole spacing varied depending on the face condition. The closest spacing was contained in Section C where both the crown and the invert were located within the cinder fill. A fourth row of grout holes was also added in Section C. All 58 grout holes terminated on competent rock. The grouted zone extended from rock to 15 ft. (1.5 x tunnel diameter) above the tunnel crown (see Figure 4). The grouting criteria included: 1.) a maximum grout pressure of 600 psi, 2.) grout injection of 5 $ft.^3$ per stage (stage = 1 ft.), and 3.) detection of undesirable ground movement. The grouting program was successfully completed in 22 days and permitted the tunnel to be advanced within specified tolerances.

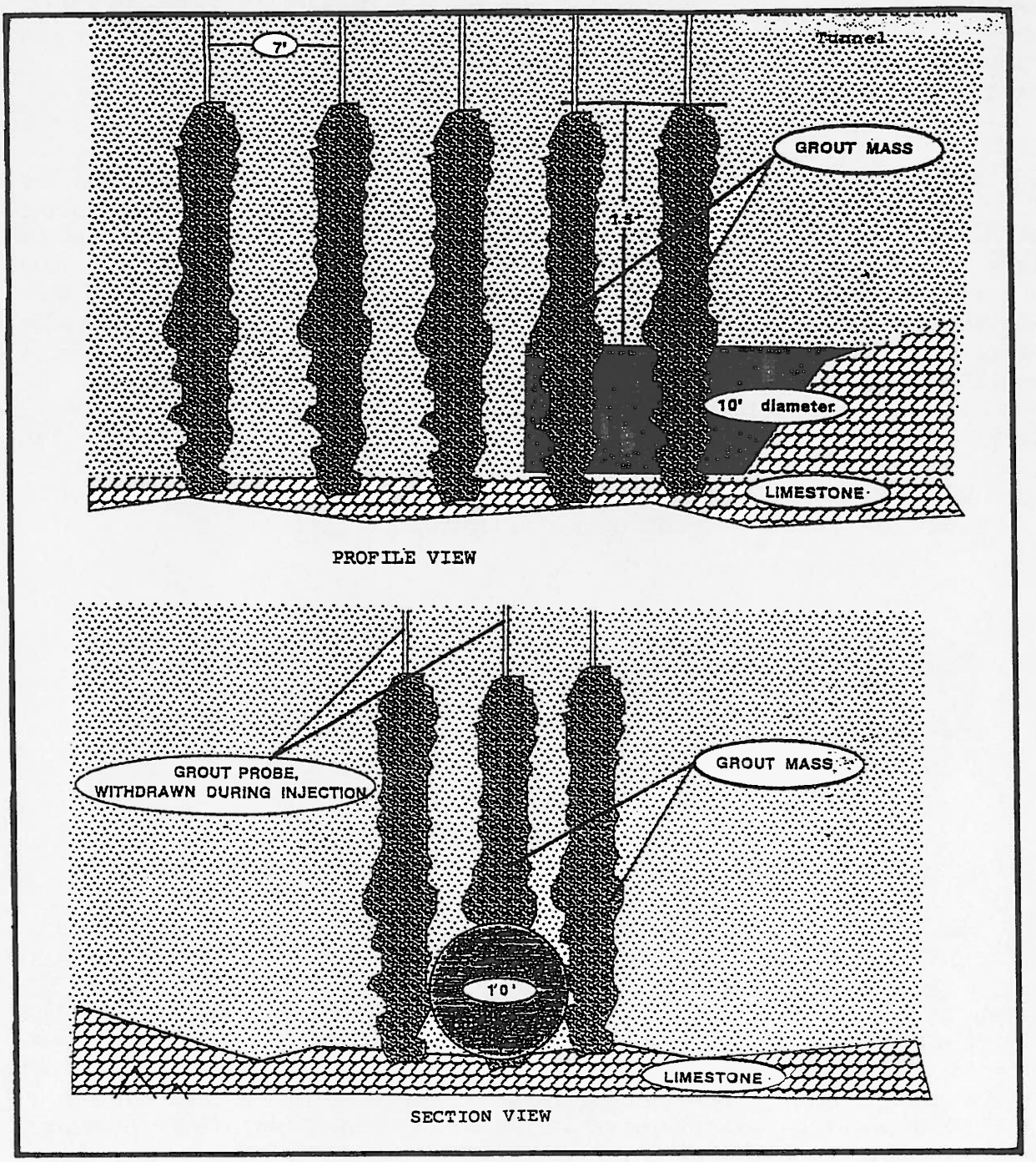


Figure 4: Profile and Section Schematic for Skinker-McCausland Tunnel

Case History No. 3 Problematic Soil: **DEBRIS LANDFILL**

Project Name: *Regency Villas*
Location: *San Diego, California*

THE PROBLEM: A new three-story apartment complex and its concrete parking garage were separated by a single-story recreational building. Two of the structures were exhibiting signs of differential settlement, including a severe 13-inch tilt of the recreation building's concrete floor slab over a horizontal distance of less than 30 feet. The recreation building had to be condemned by structural engineers even before the geotechnical investigation into the cause of the problem had been completed.

Site evaluation methods included review of historical aerial photographs, and drilling of geotechnical borings that all terminated at "refusal" in fill. The evidence suggested the distressed portions of the site to be underlain at a shallow depth by a structural fill mass, but at greater depths by an uncompacted to poorly compacted, debris-ridden, canyon dump fill. Before reaching drilling refusal, several of the borings revealed macroscopic voids and end-dumped soil debris between and beneath rubber tires, large chunks of asphalt paving and curbing, and 4-foot x 4-foot pieces of concrete slabs.

Even though not a single exploratory flight-auger excavation was able to penetrate and establish the full depth of the old canyon dump fill, all of the shallower findings and floor-level surveys indicated the ongoing structure settlement to be due to collapse and/or consolidation of the voids and loose soils that were prevalent throughout the exposed portion of the old "impenetrable" debris fill.

THE SOLUTION: Compaction Grouting was selected as the optimal method of simultaneously stabilizing the debris fill and performing the final phase of the site investigation, using typical low-mobility grout mixtures and several types of portable hand-held self-drilling grout casings. The low-slump soil-cement mixture would not only fill any actual macroscopic voids in the debris dump at depth, but due to the high internal friction of the grout itself, would also densify any compressible soil within the effective compaction radius of each grout probe. Where possible, grout casings were placed to the bedrock below the canyon fill. (Smaller-diameter grout casings are often able to be successfully advanced through zones of very rocky or debris-laden soil, even when conventional rotary flight-auger drills have consistently encountered practical refusal.)

Less than 100% of the ideal grid of grout casings were able to completely penetrate through the void-ridden debris fill, so typical volume/pressure termination criteria were not used during grout injection. Instead, while maintaining diligent monitoring of structure and ground movement, grout

injection would continue at the intermittent bottom stages until practical grout refusal was reached. In one case, this resulted in almost 60 cy^3 (cubic yards) of grout being injected over a single stage interval. Compaction-grout injection stabilized and releveled both structures at a fraction of the cost of other methods of repair (i.e. structural slabs on deepened foundations, if the necessary excavations were even possible at all). The condemned single-story recreation structure was releveled to within 1.5 inches, the design limit of releveling work desired by the project structural engineer. The condemned recreation building was restored to full use following post-grouting repairs to walls and floor slabs.

Case History No. 4 Problematic Soil: **LIQUEFIABLE SOILS SURROUNDING MOVEMENT-SENSITIVE UTILITIES**

Project Name: *The Century Freeway Storm Drain System*
Location: *Los Angeles, California*

THE PROBLEM: The deep storm drain system for the Century Freeway in Los Angeles began to show signs of significant distress within a few years of original construction. Several sinkholes and open depressions developed at on-ramps, off-ramps, overcrossings, slopes, and around a new pumping plant. Geotechnical analyses revealed erosion and piping of significant quantities of granular trench backfill soils into the deep storm drain system, primarily at unsealed pipe joints. The result was collapsible voids around and above the storm drains, low-density bedding and backfill soils subject to differential settlement, and seismic risk due to liquefiable fine sands below a very shallow groundwater table (see Figure 5). The problem was compounded by areas of severe pipe deformation and damage to the thin-walled corrugated steel pipe (CSP) that comprised the majority of the drain system. Any viable mitigation method to improve the prevailing trench backfill conditions would have to account for the very movement-intolerant condition of the deep storm drains.

Extensive subsurface exploration of the eastbound and westbound storm drain systems included 67 rotary mud borings to maximum depths greater than 110 ft., and over 330 CPT soundings. All of the borings included Standard Penetration Tests (SPT), which indicated very loose granular soils at several locations and depths along the storm drain (N-values ranging from 0 to 7). The actual extent of soil infiltration and pipe damage was investigated with two complete videotape drain surveys. In addition, several studies using ground-penetrating radar were performed from the freeway pavement over the drains.

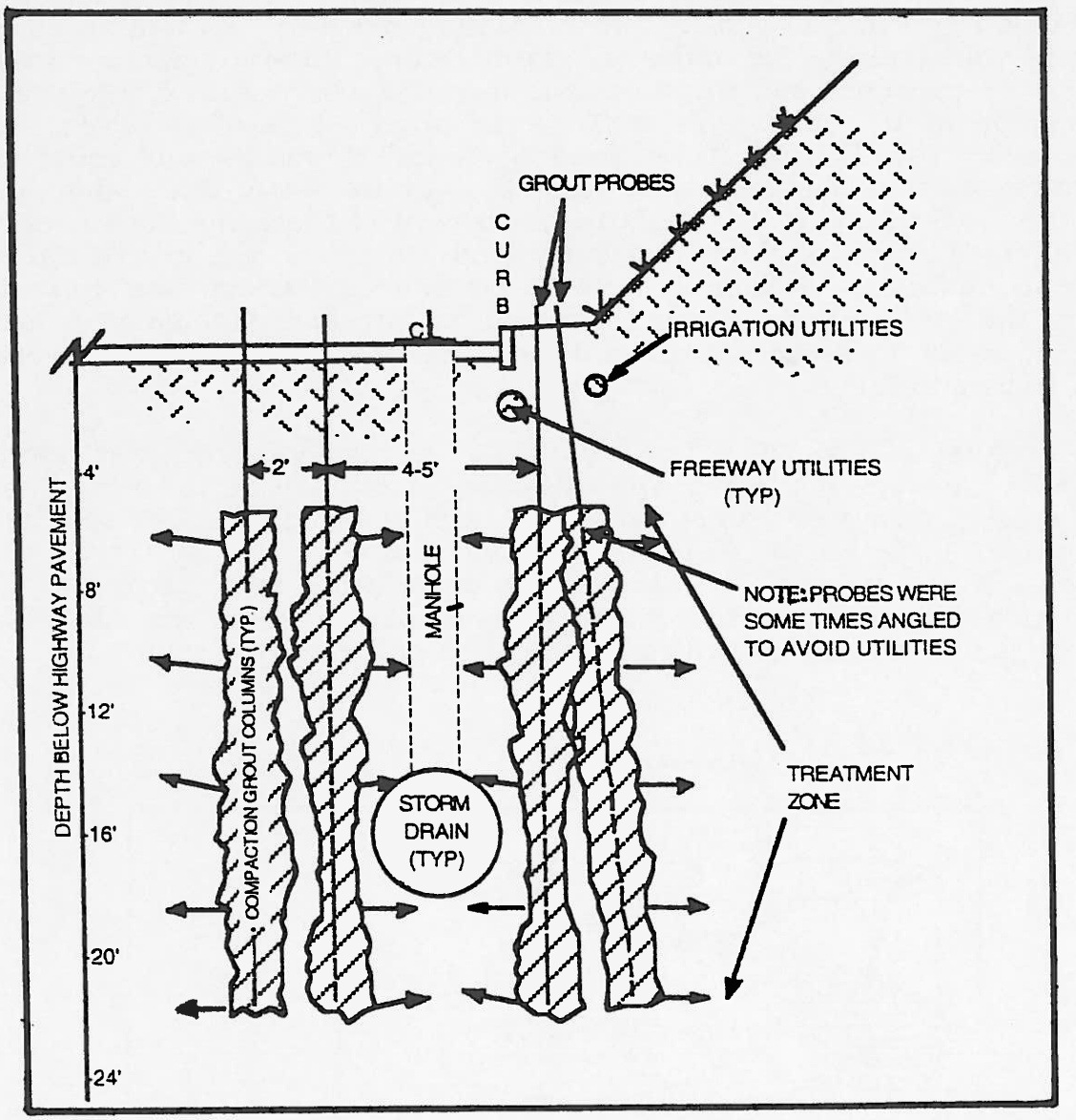


Figure 5: Cross Section of Grouting Plan for Century Freeway

The interim geotechnical findings confirmed a relatively widespread presence of either outright voids, or unacceptably loose granular deposits, surrounding or above the deeper storm drains. In response to these findings, Caltrans authorized a limited trial program of compaction grouting. The purposes of the 150-ft. test section were to evaluate the success of this technique in correcting the inadequate soil conditions along the storm-drain alignments, and to establish the necessary design parameters for an economical (less than $260 per lineal foot of storm drain) production program of compaction grouting.

THE SOLUTION: The success of the trial program resulted in a design for compaction grouting to densify "loose granular deposits", as well as fill any existing voids, along 3.2 miles of storm drain. Limiting factors to the amount of expected soil improvement included the required injection of compaction grout within only 2 ft. of the outer edges of the deep drain pipes, as well as the presence of freeway shoulders, ramps, and active-lane freeway pavement sections overlying the entire storm-drain alignment. Less than 0.5 in. of total cumulative movement of these nearby structures was specified, in an attempt to severely limit movement and related damage to these facilities. A comprehensive monitoring system was employed during the compaction grouting program to prevent damage to existing facilities, which included the storm drains, freeway slabs, and a wide variety of subsurface utilities.

A total volume of 176,120 ft.^{3} of low-mobility compaction grout was injected into 8,343 probes bordering approximately 3.2 miles of the deep storm drain system. Approximately 1,500 pre- and post-grouting CPT soundings were placed to depths of 26 ft. along either side of each deep storm drain, within 4 ft. of the outside edges of the drain pipes (see Figure 6). The verification data indicated that effective trench backfill had been achieved as shallow as 5 ft. without impairing the function of the storm drain.

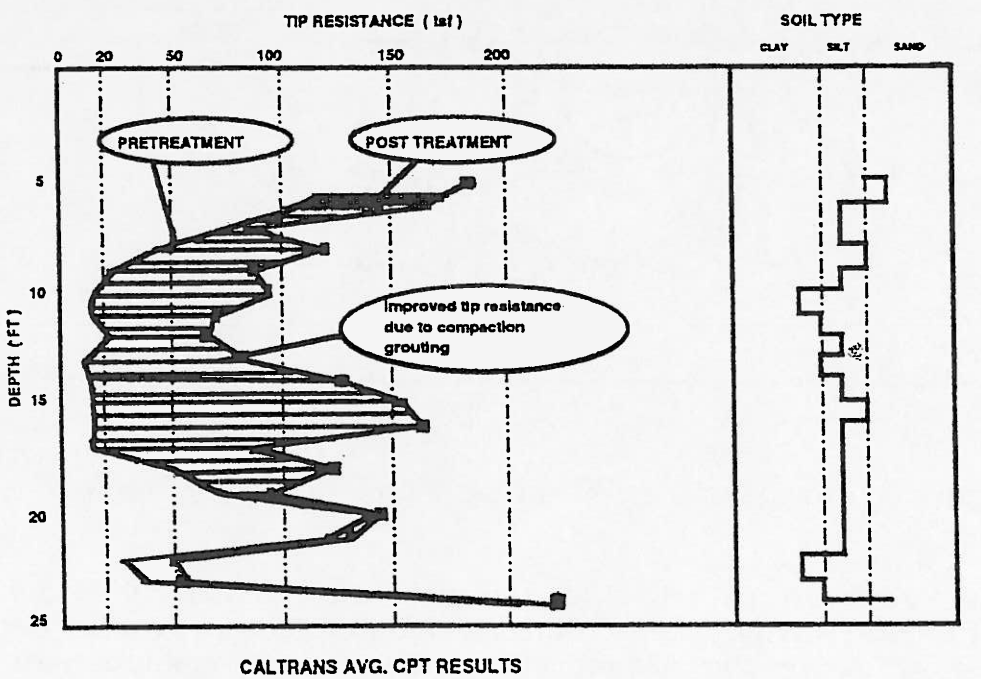


Figure 6: Schematic of Post-Grouting CPT Improvement at Century Freeway

Case History No. 5 Problematic Soil: **KARST TERRAIN**

Project Name: *Keystone Mercy Health System*
Location: *Conshohocken, Pennsylvania*

THE PROBLEM: The proposed development of a 4.2-acre sloping site included three large multi-story office buildings and concrete parking structures. Geotechnical borings indicated that the 5-story parking garage would be located in an area underlain at varying depths (up to at least 110 ft.) by karst terrain, a formational carbonate rock known to be susceptible to solutioning features (i.e. caverns and/or smaller voids in the limestone). The planned foundation system was comprised of widely separated continuous strip footings ranging from 4 ft. to 8 ft. in width, with a relatively high design bearing capacity of 6,000 psf. The karst terrain represented potential foundation settlement/collapse problems due to the suspected presence of open or infilled solutioning features at depth, including: highly variable bedrock interface, with pinnacles; isolated boulders; ledges, troughs and seams; and enhanced weathering zones of very soft soil. Given such suspected extreme variation of the subsurface conditions over very short distances, no attempt was made to accurately map and locate all significant solutioning features during the initial site investigation.

THE SOLUTION: Compaction grouting was selected over deep-foundation alternatives to upgrade the foundation conditions, due both to economic considerations, and to the fact that only the grouting approach would have the added benefit of improving lateral soils beneath proposed on-grade improvements. Approximately 580 grout probes were set in pre-drilled rows of holes along either side of the strip foundations, to depths that exceeded 110 ft. in some areas. The termination criteria for grouting of each stage included a pressure-established series of predetermined grout quantities, wherein the desired grout quantity was based on the necessary pressure to inject grout at any given stage. In addition, no heave of the ground surface in excess of 0.01 ft. was to result from the shallower injection attempts.

Case History No. 6 Problematic Soil: **RETAINING-WALL BACKFILL**

Project Name: *Denver International Airport*
Location: *Denver, Colorado*

THE PROBLEM: Significant settlement problems arose due to improperly compacted backfill soils in the 8- to 12-ft.-wide backfill zone around the basement walls of Concourses A & B. The basement walls extended 22 ft. below grade and had unrestrained distances of up to 500 ft. during initial backfilling operations. The consolidation of the backfill soils caused the overlying concrete aprons to settle 2 to 6 inches, resulting in the shearing of utilities and prevention of the airport opening. It was necessary to a.) compact the existing backfill soils to prevent further settlement, b.) relevel the aprons and surrounding utilities, and c.) not damage adjacent basement walls in the process (see Figure 7).

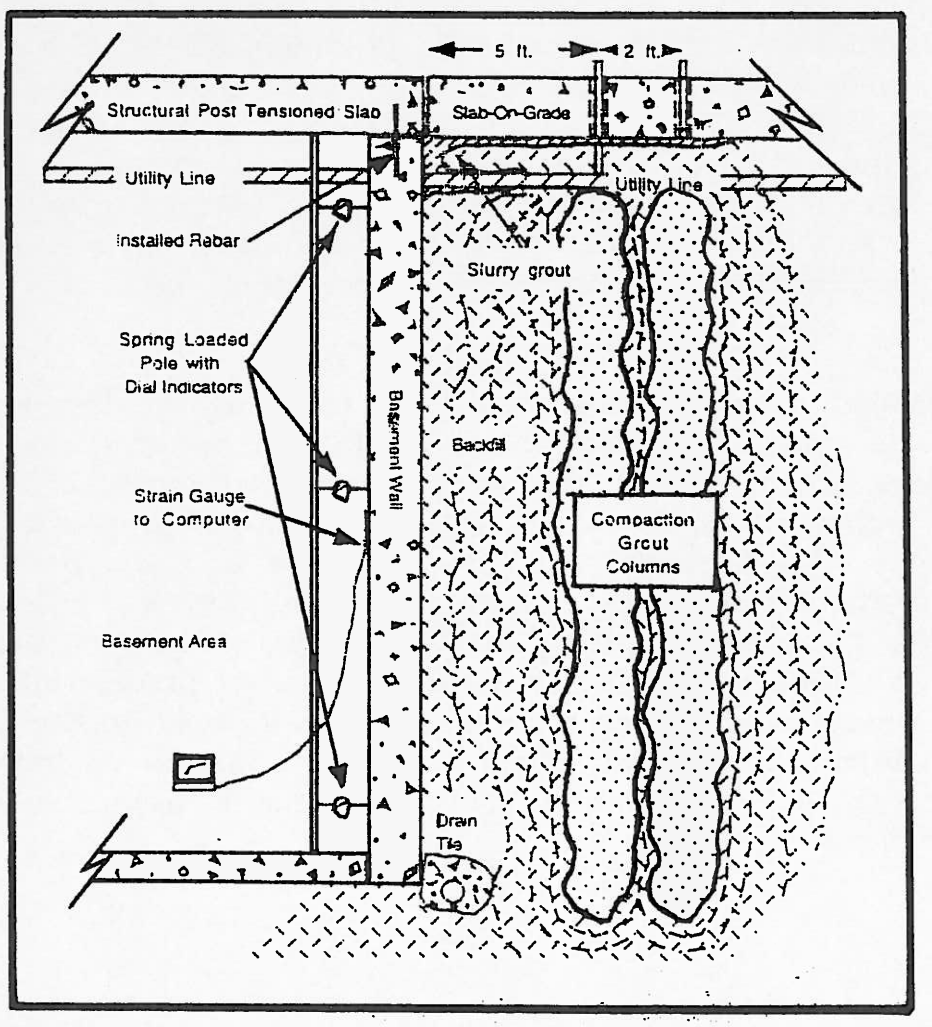


Figure 7: Cross Section of Backfill Grouting at Denver Int'l Airport

THE SOLUTION: Often (and incorrectly) regarded as a technique not to be applied near restrained walls, controlled low-mobility compaction grouting was selected to satisfy all three repair requirements. A total of 54, 320 ft.^{3} of compaction grout was injected in 1,860 grout holes around Concourses A & B, with an average of 29.2 ft.^{3} of grout per hole or approximately 1.5 ft.^{3} of grout per stage (stage = 1 ft.). Post-grouting CPT tests confirmed that the required 5% density increase was achieved. Basement walls and utilities were diligently monitored at all times during grout injection, to avoid any damage to the basement walls, and to confirm that releveling of slabs and utilities was accomplished.

Case History No. 7 Problematic Soil: **RUNNING SANDS**

Project Name: *Cherry Creek Shopping Center*
Location: *Denver, Colorado*

THE PROBLEM: Two new 4-story parking structures supported on 30-in.-diameter, 48-ft.-long caisson foundations constructed in saturated well-graded sands, exhibited excessive settlements during construction of the structures. Sonic echo-impulse testing of the caissons revealed that several of the caissons contained voids/discontinuities as much as 8 feet in height (avg. 6 ft.) caused by improper pulling of the temporary steel casings during concrete placement. The question was how to repair the caissons, *in situ*, to permit utilization of the existing foundations.

THE SOLUTION: A program of compaction grouting was employed to repair unacceptable caissons. The grouting program involved coring the caisson near centerline to encounter the sand in-fill zone(s). Grout casings were inserted in the cored holes and advanced through the sand discontinuity to firm concrete (see Figure 8). Compaction grout was then placed at a calculated volume equal to or greater than the volume of sand to be displaced in the discontinuity regardless of grout injection pressure achieved (e.g. at least the area of the caisson, multiplied by the discontinuity length). At the conclusion of the in-caisson repairs, four compaction grout holes were placed uniformly around the perimeter of each caisson and extended to bedrock. Compaction grouting was then performed the full length of the caissons to increase skin friction of the repaired caissons. Sonic echo-impulse testing was again performed at each caisson to confirm satisfactory repairs.

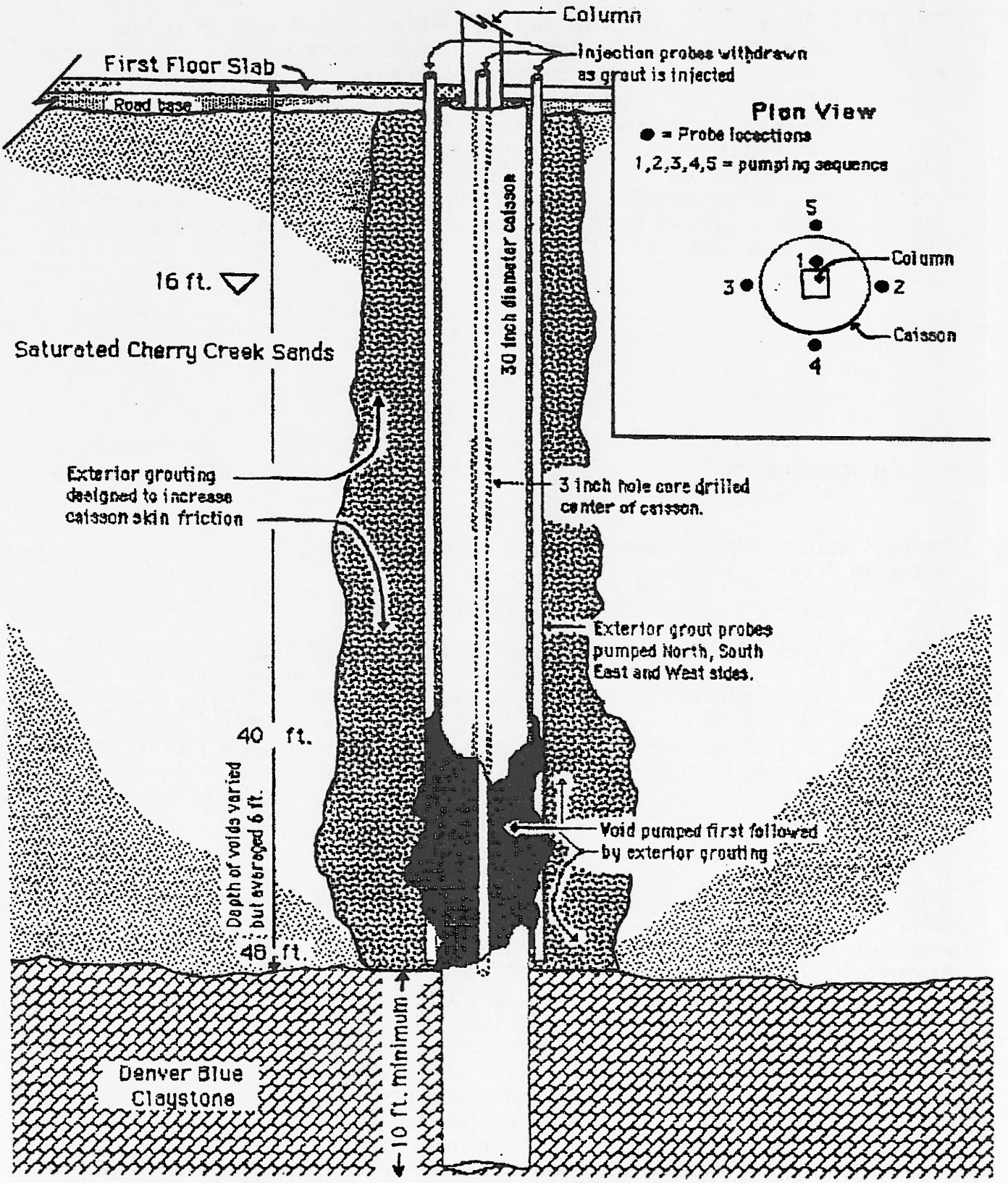


Figure 8: Cross Section of Caisson Restoration at Cherry Creek Shopping Center

Case History No. 8 Problematic Soil: **DAM FOUNDATION (Liquefaction)**

Project Name: *Chessman Dam*
Location: *Helena, Montana*

THE PROBLEM: The 80-year-old Chessman Dam in the Boulder Mountains to the southwest of Helena, Montana, was classified as "high hazard" due to its location within the seismically active Intermountain Seismic Belt. Richter Magnitude 7.0 to 7.5 was the assigned range of design earthquake values for the region during an evaluation of any dam deficiencies or safety problems. Test borings in 1983 and 1985 revealed the central concrete cutoff wall of the otherwise earthen dam to not completely extend down to competent bedrock material; rather, a 10-ft.-thick zone of very loose well-graded to silty sand existed beneath one-half of the earthen embankment, and also between the bottom of the concrete barrier and the underlying bedrock. N-values for the *in situ* sand layer generally ranged from about 2 to 10 when SPT tests were performed. Subsequent analyses indicated the intervening sandy layer to be potentially liquefiable, assuming the duration and intensity assigned to the design earthquake loads.

THE SOLUTION: Initial evaluation of methods to reduce the liquefaction potential included partial dam reconstruction, dynamic compaction, vibratory densification, compaction piles, and chemical grouting. Compaction grouting was ultimately selected over the other alternatives due to its a.) relative economy, b.) ability to control and limit embankment movement, and c.) ability to be injected while water flowed unimpeded through the soil zone requiring improvement.

Based on the results of a test grouting program where data on pre- and post-grouting SPT and relative sand density was obtained, selected areas of the concrete cutoff wall and the downstream embankment received compaction grout through primary, secondary, and tertiary probes spaced on a 5-ft. grid. A total of 133 grout probes were ultimately installed and grouted. These locations resulted in 4,753 linear ft. of 2-in. casing being installed, and over 5,630 $ft.^3$ of low-slump compaction grout being placed. Upon completion, the Engineer concluded that the loose sandy soils had been densified to an acceptable level of engineering confidence as a direct result of the compaction grouting program.

Case History No. 9 Problematic Soil: **CLAYSHALE (slough beneath existing caissons)**

Project Name: IBM Village Center
Location: Dallas, Texas

THE PROBLEM: Two 7-story office buildings connected by a 4-level parking garage exhibited excessive settlements during construction. The structures were supported on 14-ft.-long, 36-in.-diameter caisson foundations founded in the Woodbine (clayshale) Formation. Settlements were attributed to improper cleaning of the caisson bottom prior to concrete placement. It was necessary to determine how to arrest settlements and relevel the structures (if releveling was possible), *in situ*, in order for the complex to be completed and safe for occupation.

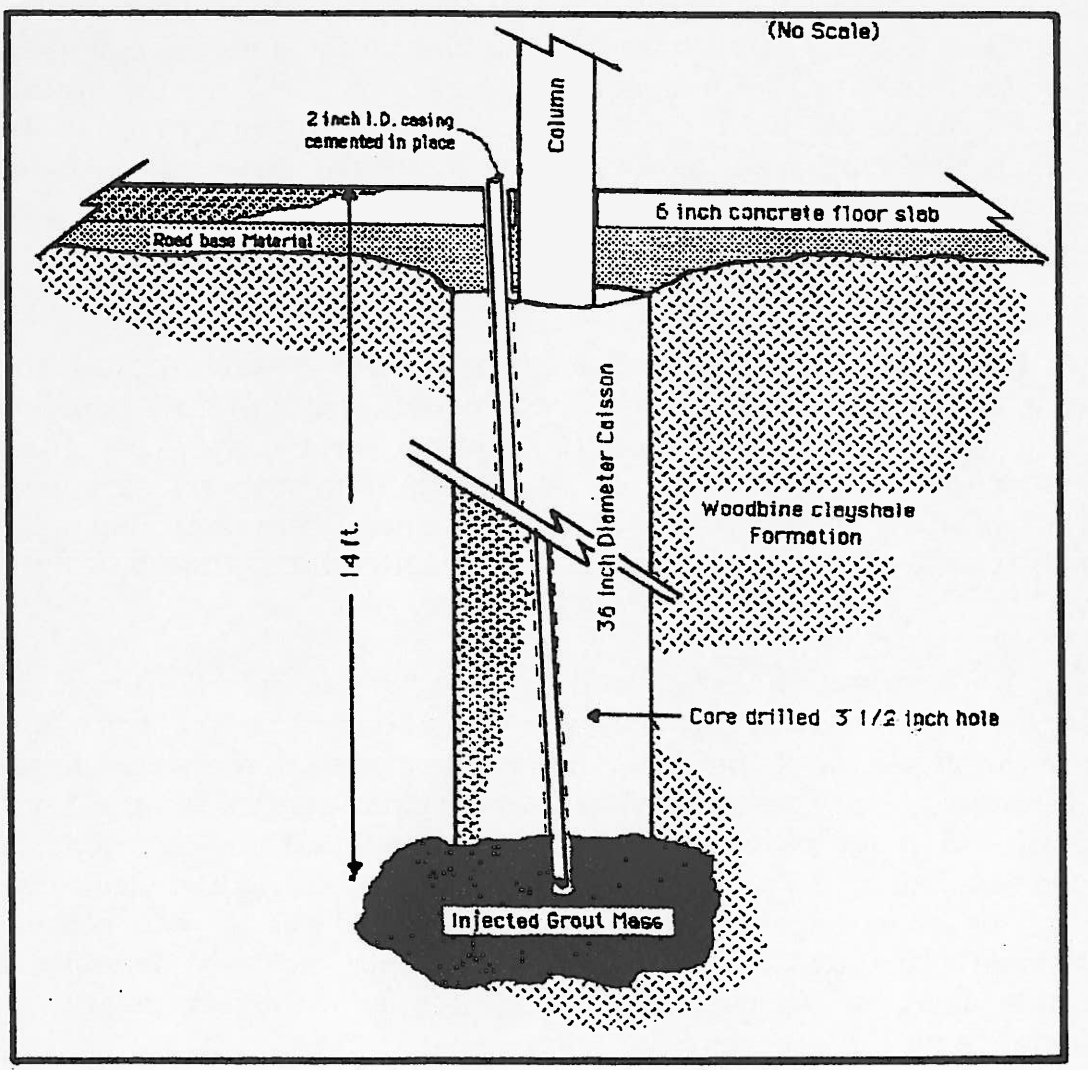


Figure. 9: Caisson Bearing Restoration at IBM Village Center

THE SOLUTION: Near centerline core holes were drilled to the caisson bedrock bearing level. Grout pipes were installed in the corehole and compaction grouting was performed at the interface to displace drill cuttings, provide competent bearing, and to relevel the caisson/structures (see Figure 9). Post-grouting full-scale load testing was performed to verify that the compaction grouted caissons were able to support design loads.

Case History No. 10 Problematic Soil: **SHALLOW GRAVELLY SANDS**

Project Name: *George Washington Way Sewer Project*
Location: *Richland, Washington*

THE PROBLEM: Settlement of the centerline street pavement along two major thoroughfares soon after open-cut installation of new large-diameter sewer lines. The settlement was due to erosion and consolidation of the very coarse-grained sewer-trench backfill soils beneath more than a mile of roadway. The initial excavation for the 5-year-old pipeline had been extensive, measuring 10 feet in width and ranging up to 23 feet in total depth. Standard Penetration Tests (SPTs) yielded blowcount values as low as 2 in the loose clean sandy backfill and voids directly above the sewer pipes, with values varying from 5 to 30 in the shallower sands and sandy gravels, up to the pavement subgrade.

THE SOLUTION: Compaction grouting was selected as the one-step procedure for restoring pavement stability, since any existing subsurface voids would be mitigated as a by-product of a comprehensive compaction-grout backfill densification program. All granular backfill soils from a depth of 5 feet down to the top of the sewer pipe had to be densified to minimum and average post-grouting SPT values of 15 and 20, respectively. A very low-mobility grout was injected along both sides of the pipe alignment, with probe spacings ranging from 4 to 8 feet in each row of probes.

A total of 65,000 cubic feet of controlled compaction grout was injected at 1,300 probe locations during the 80-day contract. Post-grouting SPT testing revealed the specified degree of soil improvement to have been generally achieved on a project-wide basis, with much higher values being recorded in several areas. This project demonstrated the ability to effectively densify shallow coarse-grained soils with the compaction grouting technique, but only if the rheological properties of the grout mixture are carefully matched to the grout-acceptance properties of the soil matrix.

CONCLUDING REMARKS

The anecdotal case histories presented in this paper do not begin to fully illustrate the variety of problematic soil conditions and environments that can be mitigated with controlled compaction grout, or the flexibility of this technique in responding to unanticipated site conditions that might be revealed once the site upgrading procedures have already begun. Continuing research into the variables that dictate specific grout behavior in specific environments, as well as further development of customized equipment and monitoring devices, will only add to the list of potential problems for which a program of controlled compaction-grout injection is a viable and cost-effective solution. However, it is already appropriate to at least consider the use of this soil-modification technique wherever the problematic soil is deficient in vertical static load-bearing, lateral support, or under seismic loading.

ACKNOWLEDGEMENTS

We would like to take this opportunity to thank and recognize Mr. Sam Bandimere, founder and President of Denver Grouting Services, Inc., from 1976 to 1996. His unwavering efforts to advance the technology and applications of controlled compaction grouting have produced not only the types of case histories summarized in this paper, but also the continuation of research into equipment and grout rheology that will promote further advances in the understanding and versatility of this technique to assist the geotechnical community.

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SOIL-DISPLACEMENT PILES IN COASTAL DEPOSITS: A CASE STUDY

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ABSTRACT

Soil-displacement piles are a relatively new deep foundation system being used in the southeastern United States. The construction procedure is similar to that of augercast piles, with the major difference being that soil cuttings are forced into the sides of the drilled hole rather than brought to the surface. The application of soil-displacement piles for a coastal site in Savannah, Georgia is presented in this paper. Approximately 2300 piles were installed on the site and over a dozen compression load tests were performed. Site subsurface conditions, pile installation procedures, analytical procedures and results, and construction problems are discussed.

SITE DESCRIPTION

The site discussed in this paper is located on the south side of Hutchinson Island across the Savannah River from downtown Savannah, Georgia. The island is situated in the Savannah River channel with the back river to the north and the main shipping channel to the south. About 15 miles upstream from the Atlantic Ocean, the island is approximately 4½ miles long and .8 miles wide at its widest point. Historically, Hutchinson Island was a marsh island prior to development in the nineteenth century. Dredged materials from the intercoastal streams and the Savannah River harbor and shipping channel were used to build up land, primarily adjacent to the river.

The project site is situated between an active slip (No. 3) to the west and an old abandoned slip (No. 1) to the east. Another abandoned slip (No. 2) that has silted-in over the last 50 years exists in the center of the site. The site borders the Savannah River for 1300 feet and extends north from river for about 2000 feet. At the time of the initial geotechnical exploration, the site was covered with

trees and undergrowth but still contained remnants of abandoned structures built in the early 1900s. These structures included a brick warehouse, railroad tracks and timber piles in the abandoned slip.

Site elevations varied from approximately 10 feet (MSL) to -2 feet (MSL) with the majority of the site at the higher elevations. The lowest portion of the site was in Slips 1 and 2 where surface elevations ranged from about 0 feet at the north end to -3 feet at the mouth. At the time of the field exploration (May and June 1995), high tide reached about elevation +5 or +6 feet. Ground elevations over the remaining portions of the site ranged from about +9 feet at the southern end of the site to about +7 feet at the north end. Water covered low-lying areas at the northern portion of the site.

PROJECT INFORMATION

The project consisted of a large convention center and resort hotel (WSHR). The first level of he convention center is composed of a cast-in place concrete frame and a structurally supported floor slab. Portions of the building will include two additional floors, which will incorporate a structural steel frame. The building has dimensions of roughly 400 by 600 feet and column loads of 500 to 1500 kips. It is situated along the river between Slip No. 3 and Slip No. 2. With a portion of the northeast corner of the building extending into the area of the abandoned Slip No. 2. The finished floor elevation of the Convention Center is 14 feet.

The resort hotel is located east of the Convention Center between abandoned Slips 1 and 2. A portion of the western edge of the building extends into Slip 2. The reinforced-concrete structure consists of a one-story (low rise) section with column loads of 200 to 300 kips and a 16-story high-rise section with column loads between 1200 and 1500 kips. The hotel sits about 150 feet from the river and has a finished floor elevation of approximately 13 feet MSL. The Resort will be connected to the Convention Center by a concrete Plaza/Parking deck at elevation 14 feet. This deck has column loads of 500 kips.

AREA AND SITE GEOLOGY

The site is in the Coastal Plain Physiographic Province. Coastal Plain soils consist of ancient marine deposits including interbedded limestones, gravels, sands, silts, and clays. The Coastal Plain is a wedge-shaped deposit of Cretaceous and younger sediments. The thickness of these sediments ranges from near zero at the contact with the Piedmont Physiographic Province (Fall Line) to the northwest, to thousands of feet in Savannah, at the coast.

With increasing depth, the lower Savannah River area is generally underlain by recent sediments, followed by Pleistocene and then Miocene deposits. The Pleistocene deposits, generally about 50 to 55 feet thick in the Savannah area, consist of tan to gray, clayey sand and sand with interbedded tongues of dark brown, blue-gray clay. The clay contains abundant plant matter and shell fragments and is evidence of the rise and fall of the sea level during the Pleistocene Epoch as massive glaciers advanced and retreated over the northeastern North American Continent.

The base of the Pleistocene contains a moderate amount of olive-green sandy clay. This transitions into the upper Miocene Formation which consists of gray-green to olive-green clay and sand with variable amounts of silt. The formation becomes more phophatic and calcarous with depth. The green, slightly cemented sands and clays that extend from the base of the Pleistocene into the Miocene have been described locally as "marl".

TESTING PROGRAM

Field Exploration

The initial geotechnical site exploration included 31 soil test borings, eight dilatometer soundings and three Cone Penetration Tests (CPT). In addition 14 relatively undisturbed samples were obtained with Shelby tubes for consolidation, tube density and triaxial shear tests. Other laboratory testing included classification tests, modified Proctor tests, and California Bearing Ratio Tests (CBR).

A summary of soil test borings is shown below.

Table 1. Soil Test Boring Summary

Location	Date	Number of Borings	Depth (ft)	Purpose
Convention Center between Slips 2 and 3	June 1995	10	75	Initial exploration
Westin Resort between Slips 1 and 2	June/July 1995 March 1998	11 3	15 to 75 30 to 70	Initial exploration Evaluation of site after initial grading
Savannah River	June 1995	5	75	Initial exploration
Slip No.1	September 1995	5	53 to 63	Evaluation of Slip 1

Location	Date	Number of Borings	Depth (ft)	Purpose
Slip No. 2	June 1995	4	75	Initial exploration
	March 1998	4	40	Evaluation of Slip 2 after reclamation
Parking area north of Convention Center	June 1995	12	10	Initial Exploration

At each boring location Standard Penetration Testing was performed at regular intervals and split-barrel samples were obtained. All drilling and sampling operations were performed in general accordance with applicable ASTM designations.

During the initial investigation dilatometer and piezocone (CPT) soundings were conducted adjacent to selected borings. Dilatometer testing was conducted by advancing a steel DMT blade into the soil to depths varying from 29.5 to 49.5 feet. CPT soundings were pushed to depths of 36 to 50 feet using a conventional drilling rig. The cone penetrometer was equipped with a piezometer for measuring pore pressures and a accelerometer to measure shear wave velocities.

Over 100 additional CPT soundings were performed by the pile contractor during the pile load testing. These soundings were used by the contractor to evaluate pile installation conditions prior to mobilizing to the site. The CPT records were provided to LAW for use in evaluating the subsurface conditions. Seven additional CPT soundings and a consolidation test were performed in Slip 2 after completion of a surcharging program (described later in this report).

Laboratory Testing

A relatively extensive laboratory soil testing program was performed for the site. The program was intended to allow estimation of soil strength and compressibility and included classification, triaxial shear, consolidation, compaction and CBR tests.

Pile Load Tests

Two episodes of load tests were performed for this project. In addition, the results of numerous previous pile load tests performed on Hutchinson Island were reviewed. Historically, driven piles have been used on Hutchinson site.

Driven timber piles were used to support early structures (warehouses, wharfs, etc.). Later, driven precast concrete, steel H, and pipe piles were used.

During the phase of the project, augercast piles were considered because of potential cost-savings. This pile type had not been used on Hutchinson Island. Therefore, we recommended that a load test program be performed for augercast piles to evaluate their feasibility in the soft deposits of the island and to provide reliable design information. Two augercast piles were load-tested for this phase for both compressive and lateral capacity. In addition, four soil-displacement (DeWaal) piles were load-tested. These tests comprised the "design phase" tests. The design-phase test piles were instrumented with strain gauges to allow estimation of load transfer.

Soil-displacement piles were eventually selected as the lest expensive foundation system. Twelve additional load tests were performed during the "construction phase". These test piles were not instrumented.

SUBSUFACE CONDITIONS

The borings and soundings encountered soils representative of published geologic descriptions of the area. With the exception of the abandoned slip areas, the borings encountered four distinct layers.

Table 2. Subsurface Stratigraphy

Description	Standard Penetration Resistance (bpf)	Cone Tip Resistance (tsf)
Upper Sand – Typically very loose to medium dense brown sands are present from the ground surface to depths of 6 to 8 feet (approximate elevations +1 to +3 MSL). There is some variation in the depth of this layer, especially in the northern portion of the site. The sands contain varying amounts of silt and clay and occasionally contain debris and organic material. The upper sands likely represent fill from previous dredging and site grading operations.	4 to 15	25 to 75
Interbedded sands, silts and clays - Beneath the upper sands, very loose to loose sands and very soft clays and silts occur to depths of 17 to 27 feet (approximate elevations -8.5 to -22.5 feet MSL). The inter-bedded sub-layers of sand, silt and clay	Less than 5 Often 0 to 2	5 to 80

Description	Standard Penetration Resistance (bpf)	Cone Tip Resistance (tsf)
vary in thickness from 2 to 10 feet. The clays and silts generally exhibit high plasticity. This layer probably corresponds to recent marsh deposits.		
Sand - Below the interbedded layer, relatively uniform medium dense sand is present with very little silt and clay. This layer extends to depths of 52 to 63 (approximate elevations -50 to -58 feet) and is typical of the previously described Pleistocene deposits.	10 to 25 with isolated softer zones	50 to 200
Marl - Material described as marl lies beneath the sand to boring termination depths of 75 feet. The marl is typically described as gray-green silty, clayey sand and sandy silt. The marl likely defines the transition from the Pleistocene to the Miocene.	Range from 15 to 100 but mostly 20 to 40	Not Available

In the slip area, the upper 30 to 35 feet of the subsurface profile has been removed during the past dredging. The materials have been replaced by very soft silts and clays naturally deposited beginning 50 years ago when the slip was abandoned. These materials extend to depths of 33 to 36 feet (approximate elevations -31 to -34 feet MSL) where the medium dense sand is again present. The material is very weak with standard penetration resistances near 0 bpf. The material has a moisture content over 100%, void ratio from2 to 5 and a plasticity index typically greater than 50.

FOUNDATIONS

As previously mentioned, driven piles embedded in the marl had been used historically on Hutchinson Island and were initially considered as the logical foundation option for this project. As is often the case, cost led the designers and contractors to consider other foundation options. After a technical evaluation of both augercast and soil-displacement piles, the later was selected as the least expensive, technically feasible pile. The primary reason appears to be that the displacement process densifies soils adjacent to the piles, resulting in somewhat higher shear resistance. Consequently, displacement piles can provide equivalent capacities with shorter lengths. Limited CPT testing performed after pile installation seemed to support this conclusion.

Soil-Displacement Pile Construction Procedures

Between January and July of 1998, over 2,300 soil displacement piles were installed at the site by Morris Shea Bride Company. The pile lengths ranged from 45 to 62 feet with lengths mostly in the 55 to 60 foot range. Final recommendations called for 30 feet of embedment in the medium dense sand layer for 150-ton piles and 20 feet of embedment for 100-ton piles. Daily production ranged from 2 to 35 piles with an average of about 20 piles per day.

The soil displacement piles used at the site are called DeWaal piles. These piles are installed with a tool approximately 10 feet long that consists of steel flighting built around a thick-walled steel pipe. The first few feet of flighting penetrate the soil and remove cuttings upward. The cuttings are carried upward to a solid section called the bulb with an outside diameter equal to the diameter of the flighting. The bulb forces the cuttings out into the sides of the shaft. Above the bulb is another section of auger with reverse flighting to carry soil cuttings downward to the bulb. This tool is attached to the drilling stem which is a long thick-walled steel pipe with constant inside diameter of approximately 8 inches. The drilling tool is advance into the ground using a high-powered hydraulic rotary drilling table that exerts torques as high as 120,000 ft.-lbs. A winch and crane can provide can exert a pull down force of up to 33 tons. A steel end-plate or "shoe" is placed in the opening at the bottom of the drill stem just prior to pushing the tool into the ground. This shoe is left at the bottom of the pile when the auger is withdrawn.

Once the pile tip elevation is reached, a steel reinforcing bar extending the length of the pile is dropped into the hollow drill stem. Concrete with a slump of 6 to 8 inches is pumped into a hopper attached to the top of the drill stem. From the hopper the concrete free falls into the stem. When sufficient concrete has built up in the pipe to exert positive pressure head, the auger is slowly withdrawn.

Foundation Analysis

Initially, we looked at several different analysis procedures that predicted both ultimate capacity and load-deflection performance. It became evident early on that a deflection-compatibility analysis described by Montgomery (1) yielded reasonably accurate results. The following is a brief description of the analysis incorporates a numerical solution. The analysis incorporates a numerical solution. The pile is divided into segments. For each segment, a stress-deflection reflection relationship for the soil adjacent to that segment is assigned. For our analyses, the relationship was developed from the results of the instrumented load tests performed during the design phase. Another source for this relationship would be laboratory direct shear tests.

Figure 1 shows the stress deflection curves developed from the load test data. The deflection of the soil beneath the pile tip is modeled using a modulus of elasticity. Again, we used the results of the instrumented load tests to estimate the modulus of elasticity at the pile tip. We also experimented with several methods using CPT data to estimate this value.

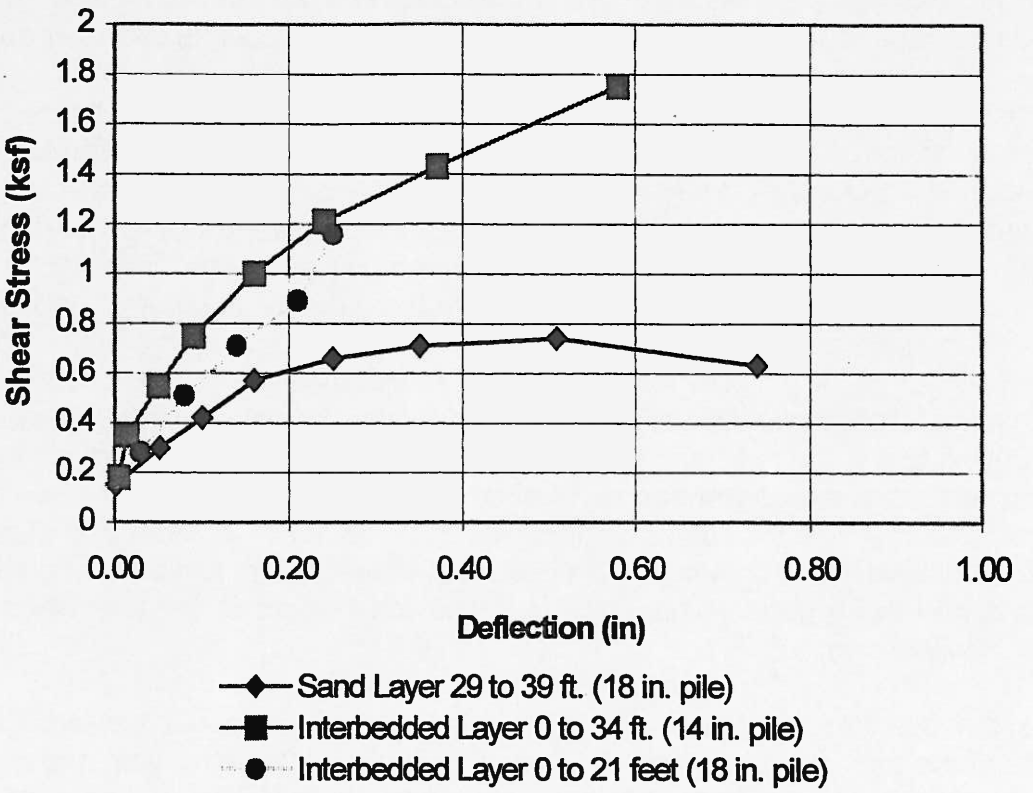


Figure 1 – Shear Stress vs. Deflection

The steps in the solution are as follows:

- An arbitrary pile tip deflection is chosen and corresponding tip load is determined using the input load-deflection relationship.
- An assumed load at the top of the segment is selected and the average force in the bottom segment is used to calculate the elastic shortening of the segment.
- The deflection at the midpoint of the bottom segment is calculated as ½ of the elastic shortening plus the assumed tip deflection.

- The relationship between shear stress and deflection for the soil round the bottom segment is used wit the midpoint deflection to determine the average shear stress along the bottom segment.

- The bottom segment should be in equilibrium when the top, bottom and side forces are considered. If not, a new value for the force at the top of the bottom segment is selected and the process is repeated. This iterative process is continued until convergence is achieved.

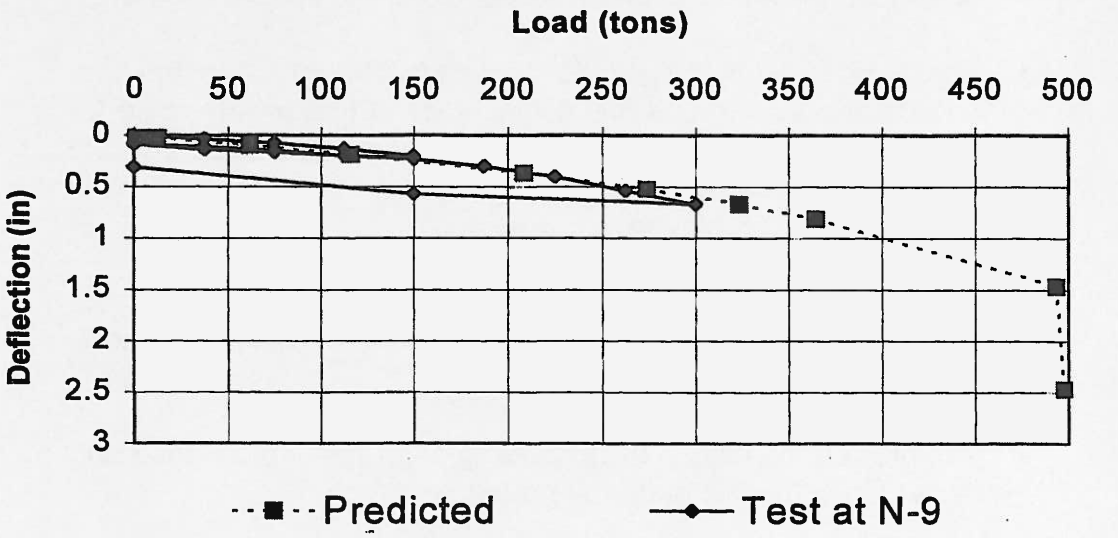
- The load at the top of the bottom segment is then taken as the bottom load for the next segment, and the iterative process is repeated for the second segment.

- The process continues through each segment until the load and deflection at the top of the pile are determined.

As previously mentioned, our analysis used shear stress-deflection relationships derived from instrumented load test made at the site. Therefore we believe this information adequately reflected the in-situ soil stress conditions. In many cases, this type of information would not be available in the design phase of a project. Alternatively, direct shear data can be used to develop the shear stress-deflection relationship.

In this case, a normal stress representative of the soil adjacent to the pile would be used to select stress-deflection curve from the direct shear data. In the past, we have assumed $\sigma'_n = K\sigma'_v$, where $\sigma'_n = \sigma'_n$ to the critical depth of 15 pile diameters. σ'_n is the normal effective vertical stress, K is the ratio of horizontal to vertical stress, σ'_v is the effective vertical stress, γ' is the buoyant soil density, and h is the depth. Below the critical, σ'_n is assumed to be constant.

Figures 2 shows the predicted and measured load-deflection relationship at the pile butt for two of the load tests performed. Using our predictive model that was validated by the load tests, we were able to develop a relationship between embedment in the "bearing sand" and capacity for a given design deflection. This allowed us to quickly evaluate piles that did not meet the recommended embedment criteria.



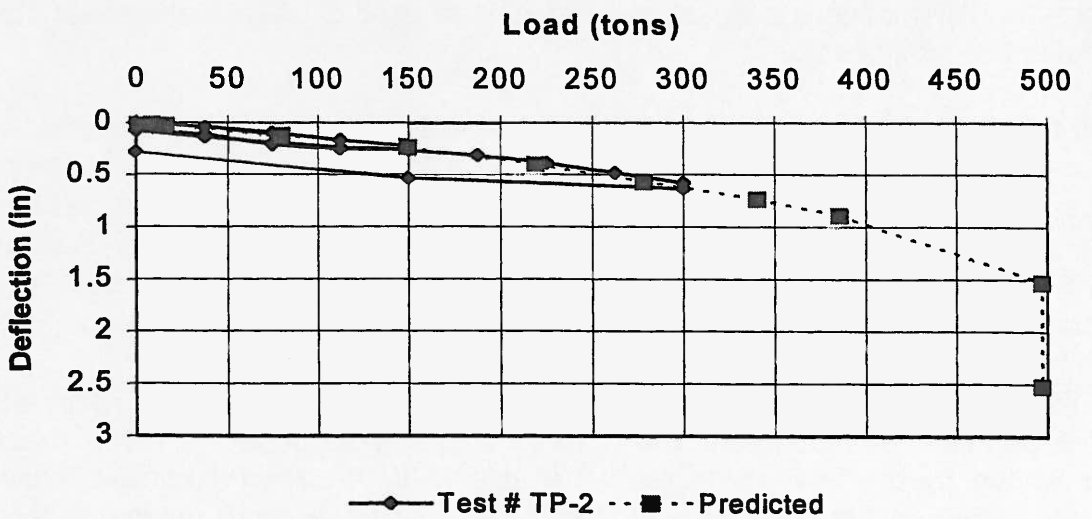


Figure 2 – Predicted and Measured Results

CONSTRUCTION PROBLEMS

- Initially, a pile spacing of four pile diameters was recommended due to concerns that soil displacement would make installation of closely spaced piles difficult. This larger spacing increased the cost significantly, so a spacing of three diameters was used once the contractor demonstrated that installation was possible at the smaller spacing. Occasionally, as more piles were installed in the larger pile caps (six or more piles) drilling became more difficult due to soil displacement. In some cases the last piles could not be drilled to the required termination depth.

- In augercast pile construction, a minimum spacing of six pile diameters is recommended to prevent heaving of piles drilled in the same day. For cast-in-place displacement piles this spacing should be greater. For the DeWal piles installed at this site this spacing was determined by trial and error to be eight to nine pile diameters. Piles installed in the same day at closer distances heaved as much as one foot mostly within the upper 10 feet. Measurements in the upper 10 feet of some piles indicated diameter reductions of up to two inches for 16-inch piles.

- Installation began using an 18-inch pile but drilling was slow and difficult. At the contractor's request, 16-inch piles were tested and then installed. The smaller piles improved production slightly but drilling in the medium dense sand layer was still sometimes difficult.

- In some cases the steel shoe did not come off until the drilling tool had been withdrawn some distance from the pile tip. These piles were given a capacity based on skin friction only.

- When two center bars were used for piles to support the shear walls, the concrete and bars had a tendency to clog the pipe preventing concrete from falling to the bottom of the pile. Tack welding the bars together nearly eliminated this problem. The bars were tension tested to verify that the bar strength was not affected by the welding.

- In the slip, isolated areas of silt remained after dredging and filling. Concrete for piles in these areas essentially displaced the silt resulting in extremely high concrete ratios. These piles were abandoned.

- When problems occurred during concrete placement the piles could not be redrilled as could be done with augercast piles. Redrilling was attempted early on but the auger was unable to penetrate through freshly placed concrete to reach the original pile tip elevation. As a result the contractor agreed to abandon piles where problems of this nature occurred and install a replacement pile at an offset location.

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APPENDIX
PAST OHIO RIVER VALLEY SOILS SEMINARS

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

ORVSS XV	*PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING* November 2, 1984, Fort Mitchell, KY
ORVSS XVI	*APPLIED SOIL DYNAMICS* October 11, 1985, Lexington, KY
ORVSS XVII	*NATURAL SLOPE STABILITY AND INSTRUMENTATION* October 17, 1986, Clarksville, IN
ORVSS XVIII	*LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION* November 6, 1987, Fort Mitchell, KY
ORVSS XIX	*CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES* October 21, 1988, Lexington, KY
ORVSS XX	*CONSTRUCTION IN AND ON ROCK* October 27, 1989, Louisville, KY
ORVSS XXI	*ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING* October 26, 1990, Fort Mitchell, KY
ORVSS XXII	*DESIGN AND CONSTRUCTION WITH SYNTHETICS* October 18, 1991, Lexington, KY
ORVSS XXIII	*IN-SITU SOIL MODIFICATION* October 16, 1992, Louisville, KY
ORVSS XXIV	*GEOTECHNICAL ASPECTS OF INFRASTRUCTURE RECONSTRUCTION* October 15, 1993, Fort Mitchell, KY
ORVSS XXV	*RECENT ADVANCES IN DEEP FOUNDATIONS* October 21, 1994, Lexington, KY
ORVSS XXVI	*SITE INVESTIGATIONS: GEOTECHNICAL AND ENVIRONMENTAL* October 20, 1995, Clarksville, IN
ORVSS XXVII	*FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING* October 11, 1996, Cincinnati, OH
ORVSS XXVIII	*UNCONVENTIONAL FILLS: DESIGN, CONSTRUCTION, AND PERFORMANCE* October 10, 1997, Lexington, KY