OHIO RIVER VALLEY SOILS SEMINAR XXXIX

URBAN CONSTRUCTION

October 17, 2008

Cincinnati, Ohio - Northern Kentucky

ORVSSS XXXIX
URBAN CONSTRUCTION

6:30-7:30 AM	Exhibitor Registration and Setup
7:30-8:15 AM	Registration
	<u>MORNING SESSION</u>
8:15-8:30 AM	Opening comments by Moderator, Ron Lech, P.E., H.C. Nutting Co.
8:30-9:05 AM	"**Mass Excavation**" by Scott Ludlow, Ph.D; P.E., Earth Exploration Inc.
9:05-9:40 AM	"**Earth Retention, Underpinning and Deep Foundations For An Entire City Block In Salt Lake City, Utah**" by Fred Tarquino, P.E., Nicholson Construction Co.
9:40-10:15 AM	"**Challenges to Successful Urban Soil Nail Earth Retention Systems**" by Frank Reinart, P.E., Kleinfelder West, Inc.
10:15-10:35 AM	Break
10:35-11:10 AM	"**Erskine Commons-Successful Urban Development on a Closed Landfill Site**" by Ron Ebelhar, P.E., H.C. Nutting Co.
11:10-11:45 AM	"**Geotechnical Construction at the World Trade Center in New York**" by Rick Deschamps, Ph.D; P.E., Nicholson Construction Co.
11:45-12:35 PM	Lunch
	<u>AFTERNOON SESSION</u>
12:35-12:45 PM	Keynote Introduction by David Keller, P.E., Richard Goettle, Inc.
12:45-1:40 PM	KEYNOTE: "**Current Landslide In An Urban Environment**", by Timothy Stark, Ph.D; P.E., University of Illinois
1:40-2:15 PM	"**Unstable Slopes In The Cuyahoga River Valley, Cleveland, Ohio: Their Effects on Urban Development, the Geologic History, and Planning for Future Geotechnical Investigations**" by Stephen Pasternack, P.E., BBC&M Engineering, Inc.
2:15-2:45 PM	Break
2:45-3:20 PM	"**Innovative Construction Techniques Used at North Shore Connector Project**", by Kanchan Sen, P.E., Nicholson Construction Co.
3:20-3:55 PM	"**Real Time Response to Landslide at the Lexington Apartments**", by Michael Marasa, P.E., Hayward Baker Inc.
3:55-4:30 PM	"**Which In-Situ Test Should I Use?**" by Roger Failmezger, P.E., In-Situ Soil Testing L.C.
4:30-4:45 PM	Closing Remarks

PROCEEDINGS OF THE THIRTY NINTH
OHIO RIVER VALLEY SOILS SEMINAR

URBAN CONSTRUCTION

October 17, 2008

Northern Kentucky Convention Center

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3 Mass Excavation

By

Scott J. Ludlow, Ph.D., P.E.[1]

ABSTRACT: Controlling ground movement in an urban setting as a result of excavation is an art. Often, many factors affect movements associated with excavations, including: site constraints (i.e., location of utilities and other subsurface appurtenances), relationships with adjacent property owners, subsurface conditions, support system details, overall scope of the project, contractual arrangements, and construction activities related to sequencing and workmanship. Navigating through these factors is necessary for a successful project. This paper presents a case history of such a project in downtown Indianapolis involving a 20-ft deep excavation for a 10-story condominium adjacent to two streets, two multi-story masonry brick buildings and a parking lot. Those sides of the excavation adjacent to the streets included traditional soldier piling and lagging with tiebacks to control ground movement, while those adjacent to the buildings included a tangent pile system with tiebacks. The side of the excavation adjacent to the parking lot included a vertically-prestressed cantilevered steel H-pile. Emphasis is placed on the results of performance monitoring via inclinometers and vibrating wire strain gages located on the piles of the tangent and cantilevered walls as related to system stiffness and ground movement.

INTRODUCTION

As part of an urban development project (i.e., 3 Mass) in downtown Indianapolis, consideration was given by others to construct a 10-story condominium on a vacant parcel of property adjacent to two streets, two multi-story masonry brick buildings and a parking lot. Improvements also included two levels of below-grade parking requiring a 20-ft deep excavation. Foundations for the adjacent buildings were supported by spread footings established at depths of 7 and 10 ft below the ground surface. Figure 1 shows a plan view of the site in relation to these elements as well as locations of the instrumentation and utilities.

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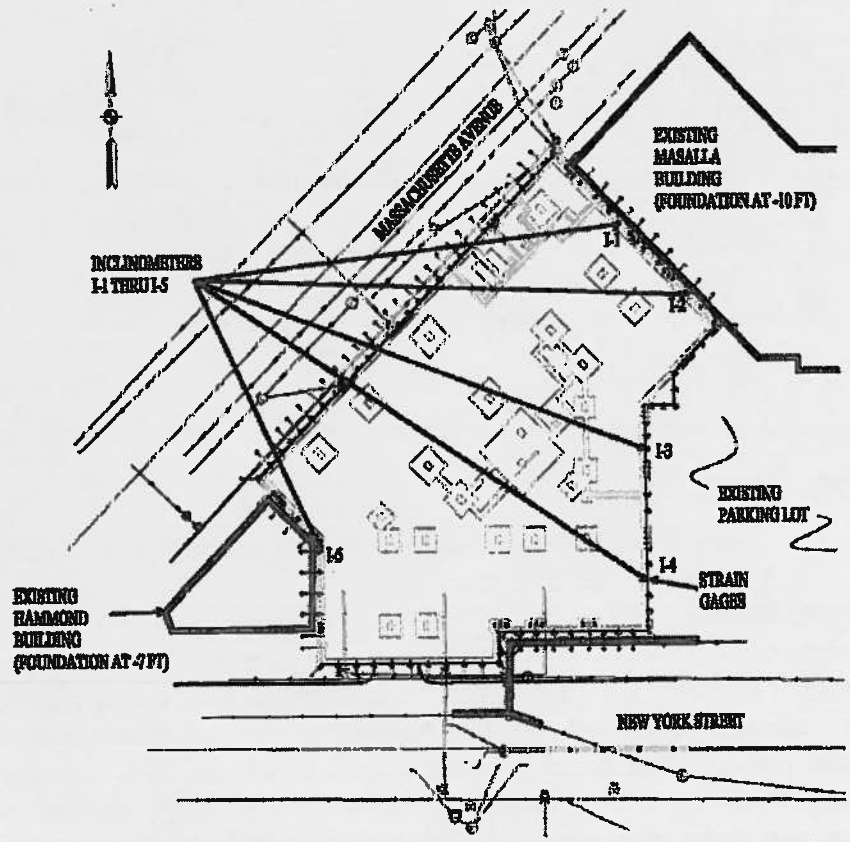

Fig. 1 Plan View of 3 Mass

Given the location, careful consideration was given by the project team to the type of excavation support systems required to control ground movement and maintain a high certainty of execution. This was especially important considering that the development was surrounded by law offices on three of the five sides. (Fear can be a good motivator.) Considering the previous needs, a design-bid-build method of delivery was chosen that included a performance-based specification for the tieback installation. To control ground movements and maintain economy with a system of adequate stiffness, consideration was initially given to non-gravity systems consisting of traditional soldier piling and lagging with tiebacks adjacent to the streets and parking lot, and a tangent pile system with tiebacks adjacent to the existing buildings. These systems had been used by the author on other projects where control of ground movement was essential. These systems were also discussed in detail with the project team and owner of the development and would require cooperation in the form of temporary easements from adjacent property owners to install tiebacks beneath their property. Through discussions with the owner, relationships with adjacent property owners were disclosed and, in one case (an attorney who owned/occupied one of the buildings), were not favorable. Nevertheless, the owner was encouraged to make peace with this owner and meet with other owners to discuss the project concepts while design of the previous systems progressed. During this process, the author was contacted by the insurance agent of the attorney whose relationship with the owner of the development was less than favorable. This was a coincidence since the insurance agent also happened to represent the author's company which proved to be fruitful. However, another owner (also an attorney) came forward after construction had started and would not make a concession for a temporary easement to install tiebacks beneath their property (i.e., the parking lot). This presented a new challenge considering that work was in progress. At this time, consideration was given to either internally bracing this side of the excavation or using a series of vertically-prestressed cantilevered steel H-piles – all within the means of the earth retention contractor. Additional discussions with the project team suggested that the latter method of support would be the most economical and allow the current contractor to continue with their work while maintaining a division of responsibility.

SUBSURFACE CONDITIONS

The city of Indianapolis is located near the western boundary of the Indiana Physiographic unit known as the New Castle Till Plains and Drainageways, which is part of the Central Till Plain Region. This unit is typified by nearly flat to gently rolling terrain that is dissected by generally southwest trending valleys. Natural Indianapolis surface features result from the most recent glaciation (i.e., Wisconsin age), which is believed to have crossed Indiana approximately 20,000 years ago. While most of the Indianapolis area is covered by a relatively thick layer of glacial till, major valleys (e.g., those of the White River and Fall Creek) were formed by melt water streams during glacial recession. Outwash deposits within these melt water valleys, which generally coincide with current stream channels but are much wider, are predominantly granular soils consisting of sands and gravels, sometimes overlying cobbles and boulders near the interface with the till.

The location of the project is located on a terrace above the White River flood plain. The majority of the natural unconsolidated deposits in the immediate vicinity of the site consist of outwash sand and gravel that was deposited by melt waters. The outwash is often interrupted by layers of glacial till that vary in thickness and appear to be somewhat random in occurrence. Geologic mapping indicates that the upper bedrock in this area is limestone that was deposited on the order of 400,000 years ago during the Devonian Age. Published geologic mapping indicates that the bedrock surface underlying downtown Indianapolis varies from Elevation 590 to 640 (i.e., 70 to 120 ft below the ground surface).

Exploratory test borings performed for the project revealed a surficial layer of urban fill that varied in character and thickness (i.e., ranging from 7 to 10 ft below the ground surface). The fill was primarily granular in nature with varying amounts of debris including brick, concrete, wood, and metal. Conditions beneath the fill were characterized as granular outwash soils. These soils were classified as sand and gravel, and sand with varying proportions of silt and were predominantly medium dense to dense.

RATIONALE FOR DESIGN

Anchored Wall

An approach described by Clough and O'Rourke (1990), which relates deformations to system stiffness and basal stability, was used for the design of the anchored system. This approach allows key elements of the design, such as wall stiffness (EI), embedment of the vertical structural element below the line of excavation, and spacing of horizontal supports (i.e., struts or tiebacks), to be specified for controlling deformations within tolerable limits. Once these elements have been specified, the completion of the design becomes a relatively simple problem of structural analysis satisfying shear, moment and compatibility.

Clough and O'Rourke's approach is also derived from case histories, aided by a series of non-linear finite element analyses, whereby two categories are considered:

1. Stiff clays/residual soils/sands
2. Soft to medium clays

For Category 1, the maximum lateral and vertical wall movements were observed to be approximately 0.2 and 0.15 percent of the retained height, respectively. Category 2 requires consideration of the factor of safety against basal stability and also the system stiffness, which is defined by Clough et al. (1989) as:

$$(EI)/(\gamma_w s^4)$$

where: E is the modulus of elasticity of the wall; I is the moment of inertia of a unit length of wall; γ_w is the unit weight of water (to provide a dimensionless system stiffness parameter); and s is the average spacing between horizontal supports. Figure 1 illustrates the effect of system stiffness, basal stability and predicted lateral wall movement.

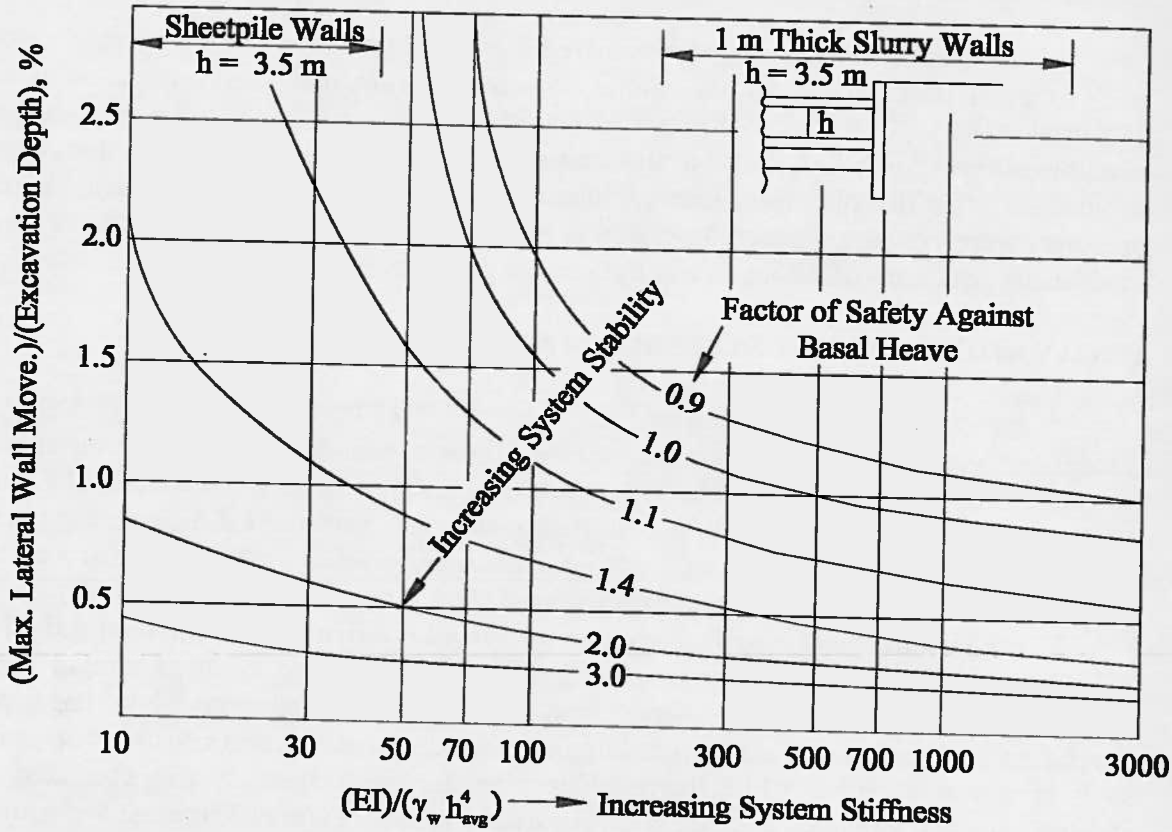

Fig. 2. Chart for predicting maximum lateral wall movements (Clough et al. 1989)

Figure 2 allows an estimation of maximum lateral deformations as a percentage of the excavation depth, once the system stiffness has been selected and the factor of safety against basal heave has been estimated. The factor of safety calculated as proposed by Clough et al. (1989) is not a true factor of safety, because it ignores the embedment of the wall, soil anisotropy, and other factors. It should be viewed as simply an "index" factor of safety. Clough et al. (1989) also provide guidelines for determining the approximate profile of lateral deformations, the distribution of lateral deformations with distance from the excavation, and the profile of normalized settlements behind the excavation. Clough and O'Rourke (1990) also presented guidelines for estimating settlement profiles behind excavations. Using the procedures and guidelines included in the previous references, the shape of the lateral deformations of the wall can be estimated. The next step is to estimate the settlement profile behind the wall consistent with the estimated deflection profile of the wall. Simple geometric relations can be used to estimate settlements and lateral deformations behind the wall on the assumption that the areas under the lateral deformation and settlement profiles must be equal. The assumption about equality of the areas under the deformation and settlement profiles is based on the premise that the deformations caused by excavations in soft clays take place under undrained conditions.

Vertically-Prestressed Cantilevered Wall

The design of this wall system utilized a combination of equations of mechanics and empirical observations by the author of other instrumented cantilevered walls or anchored walls in a cantilevered phase in downtown Indianapolis. For this approach, a bending moment was induced on the earth-side of the pile via tendons that were prestressed after the pile was inserted into a predrilled grout fluid-filled hole. Earth pressures were computed based on classical earth pressure theory, and deflections were made using equations of mechanics in light of the prestressing effect on the pile.

EXCAVATION SUPPORT SYSTEM DETAILS

Considering the approach by Clough et al. (1989) for anchored walls, a dimensionless system stiffness parameter of 2,300 was estimated for: a limiting lateral movement of ⅛ in. adjacent to buildings and ¼ in. adjacent to sidewalks, streets and utilities; excavation depths of 10 to 20 ft below adjacent grades albeit the existing grade or footings; and a basal stability of 3. Using a maximum soldier beam spacing of 8 ft, an HP shape of 12x53 size was inserted into a pre-drilled grout fluid-filled hole to depths of 27 to 32 ft below the ground surface (i.e., about 7 to 10 ft [adjacent to buildings] and 12 ft [adjacent to streets] below the excavation grade). Tiebacks were installed using a thru-pile connection with a built-up section, and design loads varied from 34 to 102 kips, depending on location and surcharge. Lagging adjacent to the streets consisted of 3 in. by 8 in. nominal timber while intermediate piles for the tangent system consisted of grouted piles that extended 3 ft below the excavation grade. Figures 3 through 5 illustrate these details for various sections.

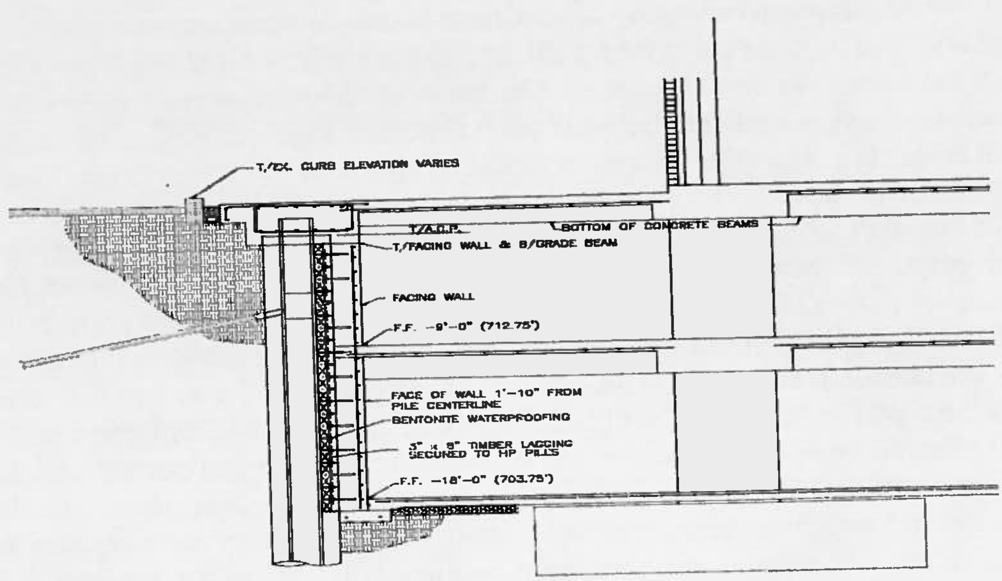

Fig. 3. Typical Section at Soldier Beam along Streets

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EARTH RETENTION, UNDERPINNING AND DEEP FOUNDATIONS FOR AN ENTIRE CITY BLOCK IN SALT LAKE CITY, UTAH

ABSTRACT

The City Creek Center Block 76 Project includes the demolition and redevelopment of an entire city block for mixed use retail, office and residential properties directly adjacent to Historic Temple Square in Salt Lake City, Utah. Prior to construction, the Block 76 Project footprint was completely occupied by an existing shopping mall, subsurface parking structure and a twenty two story office building. Demolition of these structures, including the implosion of the office tower was required to make way for the new construction on the project. However an adjacent light rail system and four existing buildings between 9 and 23 stories remained at the corners of the project.

Excavation was carried out 65ft below street grade and by as much as 50ft directly below adjacent buildings supported by shallow foundations. Movements of the earth retention system and adjacent buildings were monitored in real time using automated equipment which complemented the traditional suite of geotechnical instrumentation. Due to the high profile nature of this project and taking into consideration the requirement of deep excavations through granular soils and the adjacent high rise structures on shallow foundations, careful consideration was applied to evaluate the proposed support system. In the areas immediately adjacent to the existing buildings greater than 10 stories, anchored diaphragm walls were employed. This is believed to be the first anchored diaphragm wall for earth retention in Salt Lake City, Utah.

Areas of the site not supported by diaphragm walls employed a soil nail wall system. The location of soil nail walls was generally adjacent to streets where larger wall movements were acceptable to the project team. Soil nail walls were also employed directly below existing structures, but were enhanced by an underpinning system consisting of micropiles with an integral cap beam to limit anticipated movements to 1".

Jet grouting was employed as an alternate for water cutoff in lieu of dewatering to mitigate water inflow along the North and East sides of project. Prior to commencing production jet grouting, a field test program was instituted to optimize injection parameters and to confirm jet grout element geometric, mechanical properties. The test program involved the construction of test elements followed by a 3 day cure time. Jet grout test elements were exposed by excavation and geometric properties obtained by direct measurement. The jet grout wall consisted of overlapping secant columns and was constructed along two alignments, providing a connection between diaphragm walls adjacent to buildings along three corners of the project.

In total, Nichoson's earth retention scope included 45,000 SqFt of Diaphragm Wall, 104,000 SqFt of Soil Nail Wall, Micropile underpinning of 3 buildings, and jet grouting for water control where soil nails walls were below the ground water table. This paper will provide an overall description of the Block 76 project and discuss the approach for the selection of diaphragm walls versus soil nail walls as well as the necessity of the underpinning and cutoff wall systems. A detailed account of the installation of the earth retention, underpinning and water cutoff systems will be provided with the soil conditions encountered at the site.

A discussion will be provided comparing predicted versus actual earth retention wall movements. Finally a few words and photos will be shared on the augercast piles (over one thousand of them!) currently underway for the new structure that is taking shape.

INTRODUCTION

City Creek Center is a $1 billion Project that will transform the downtown area of Salt Lake City, Utah. The project is spread over 3 city blocks encompassing 20 acres and involves the demolition and redevelopment for mixed use retail, office and residential properties directly adjacent to Historic Temple Square, (Figure 1).

FIGURE 1 – City Creek Center Project

Prior to construction, the Block 76 Project footprint was completely occupied by an existing shopping mall, subsurface parking structure and office buildings. Demolition of these structures, including the implosion of the office tower, was required to make way for the new construction on the project. However an adjacent light rail system and four existing buildings between 9 and 23 stories remained at the corners of the project (Figure 2).

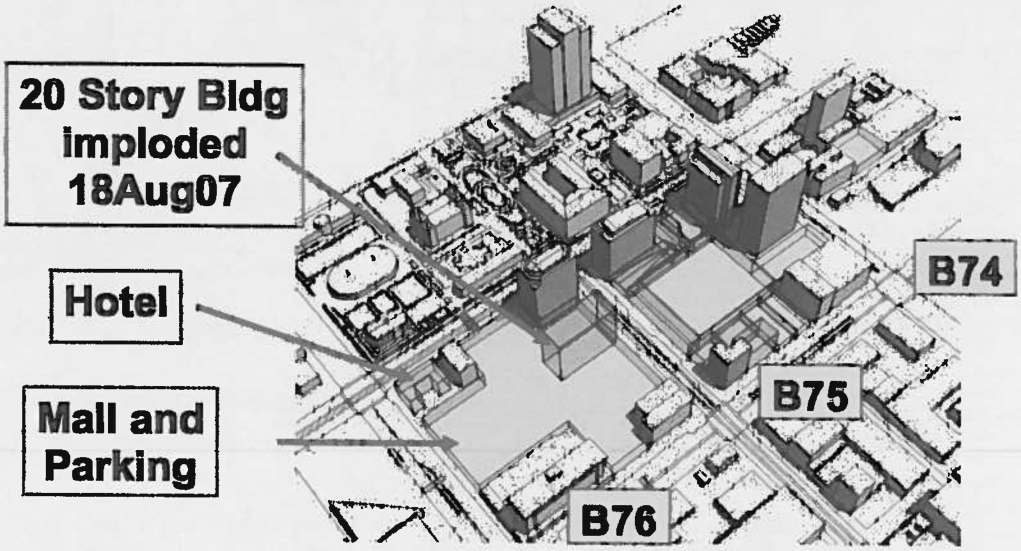

FIGURE 2 – Block 76 Demolition

DESIGN CONSIDERATIONS AND SOIL CONDITIONS

Excavation was to be carried out up to 65 ft below street grade and by as much as 45 ft directly below adjacent buildings supported by shallow foundations. Controlling movement of the earth retention system and adjacent buildings is of critical importance to the Project.

The soil stratigraphy at the site consisted of alluvial medium to very dense sand and gravel over medium stiff clay and silts, followed by a very dense layer of sand and gravel. The clay and lower gravel layer are deepest at the southwest corner of the site and rise towards the northeast corner of the site. Depth to existing groundwater is approximately 40-54 ft below grade with the highest groundwater at the northeast corner of the site. Figure 3 depicts the subsurface profile. (Image from Geotech report prepared by AGEC).

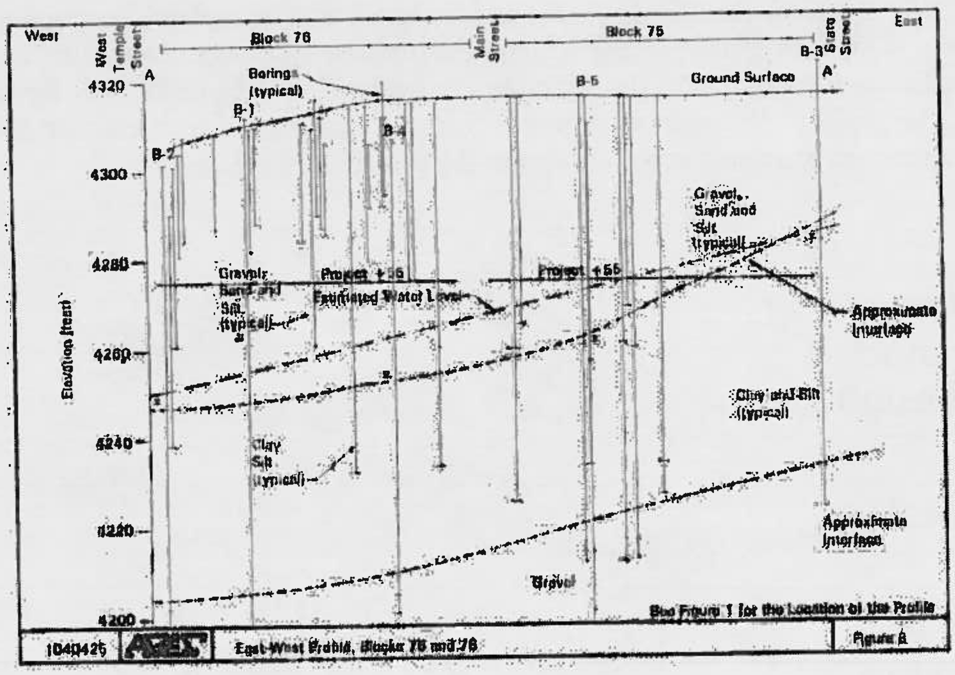

FIGURE 3 – Block 76 Subsurface Profile from Geotechnical Report

Due to the high profile nature of this project and taking into consideration the requirement of deep excavations through granular soils and the adjacent high rise structures on shallow foundations, careful consideration was applied to evaluate the proposed support system. Figure 4 depicts the location where different earth retention systems were incorporated at the site.

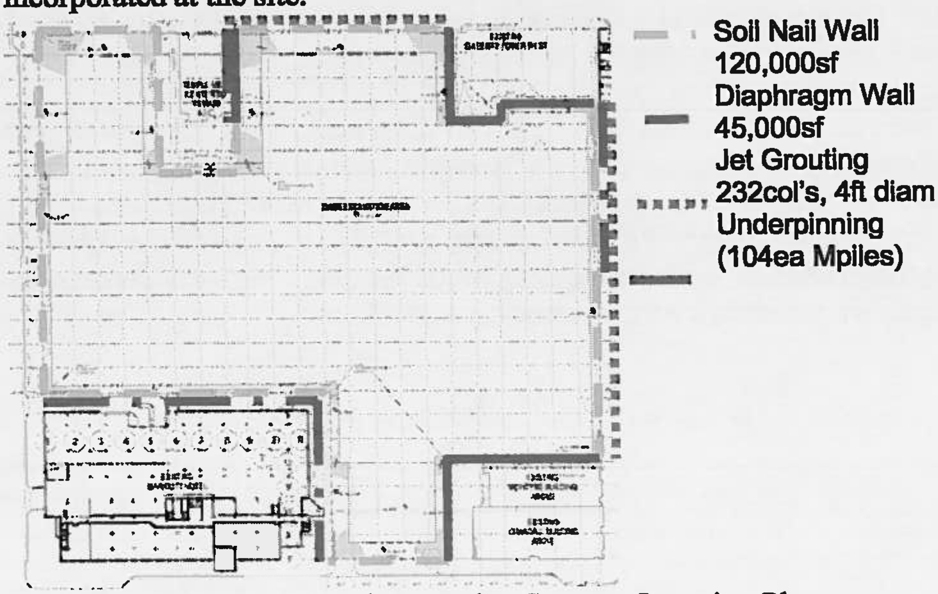

FIGURE 4 – Block 76 Earth Retention Systems Location Plan

In the areas immediately adjacent to the existing buildings greater than 9 stories, anchored diaphragm walls were employed. The diaphragm wall design incorporated 3 to 4 levels of 6 strand tie back anchors to control movement. A compromise was reached with the Owner in regards to diaphragm wall thickness and encroachment on precious retail space which led to the design of a 24" reinforced concrete thick wall versus the 30" wall originally envisioned.

Areas of the site not supported by diaphragm walls employed a soil nail wall system. The location of soil nail walls was generally adjacent to streets where larger wall movements (up to 2") were acceptable to the project team. Due to the temporary nature of the soil nail walls, self drilling hollow bars were utilized in the design. Shotcrete facing was employed with wire mesh reinforcement.

Soil nail walls were also employed directly below existing structures, but were enhanced by an underpinning system consisting of micropiles with an integral cap beam to limit anticipated settlement to 1". The underpinning system was designed to transfer the building loads to an elevation below the earth retention system. This allowed some of the building surcharge to be eliminated from the soil nail wall design. The micropile system consisted of both self drilling hollow bars and Nicholson cased Pin-Piles and were bonded in the dense lower gravel layer, generally 100 feet below working grade. Load was transferred from the existing structure through a doweled connection to the reinforced concrete cap beam into the micropiles and down to the bond zone in the lower gravel.

As seen in figure 3 Subsurface profile, the excavation to project elevation 36 would be below the existing ground water at the north east corner of the site. The soil nail wall requires a stable face for drilling and shotcreting, therefore a groundwater control system needed to be implemented. A dewatering system was evaluated and a proposal was reviewed by the General Contractor. The main concerns with the dewatering systems were two fold. 1) The dewatering system was to be located in a zone of fine material with low hydraulic conductivity, requiring many small diameter wells at a very close spacing and 2) the potential for removal of fines by the dewatering process could lead to settlement. Ultimately, jet grouting was employed as an alternate to dewatering to mitigate water inflow along the North and East sides of project. A series of overlapping columns was designed to create a cutoff wall in between diaphragm wall segments along the north east part of the site.

The entire earth retention system and all existing adjacent structures were monitored with Sol Data's Cyclops real time monitoring system. This system was a design requisite and an overall risk management tool for the project team. The Cyclops system utilizes robotic total stations that cycle through prism targets mounted on the earth retention systems and adjacent structures and transmits data through wireless connection to Sol Data where the proprietary software reduces and reports the data to project participants in real time. Complementing the

Cyclops system was an array of inclinometers in soil and in diaphragm walls, observation wells, and surface settlement points.

PRODUCTION WORK

Soil Nail Walls: The project began with a demolition phase that was supported by soil nail wall construction. As subsurface demolition occurred, the soil nail walls were sequentially installed as earth retention to support further demolition below grade. Figure 5 shows the concurrent demolition and soil nailing operations.

FIGURE 5 – Block 76 Concurrent Demolition and Soil Nail Wall Installation

The soil nail walls were constructed using excavator mounted, top drive rotary percussion drill masts. The excavator mount provided flexibility with drill setup and tracking over variable terrain. In addition, the drill mast has 360 degrees of movement in 2 directions making setup and installation at corner locations easier.

As demolition areas were completed, a more productive pace was realized and soil nail wall construction was largely governed by the excavation progress. Over 5000 soil nails were installed totaling over 150,000 linear feet drilling and over 104,000sf of shotcrete facing.

During this time, underpinning activities were underway along the Marriott Hotel (see figure 4). The use of hollow bar micropiles at this location was driven by the drill proximity to the existing building. Only a one foot clearance was available from the existing building to the backside of the new construction. As a construction aid, the drill rig also installed inclined grouted bars to mitigate potential ground loss for the first 3 lifts of soil nailing. Figure 6 details the conceptual sketch for the Marriott hotel underpinning and Figure 7 shows the actual installation. Micropiles were installed to a depth of 102 ft below grade and were bonded into the lower dense gravel formation. A total of 103 micropiles and 850 linear feet of cap beams were installed on the project at 3 locations; Marriott Hotel, Temple View Center, and at the Main Street Garage.

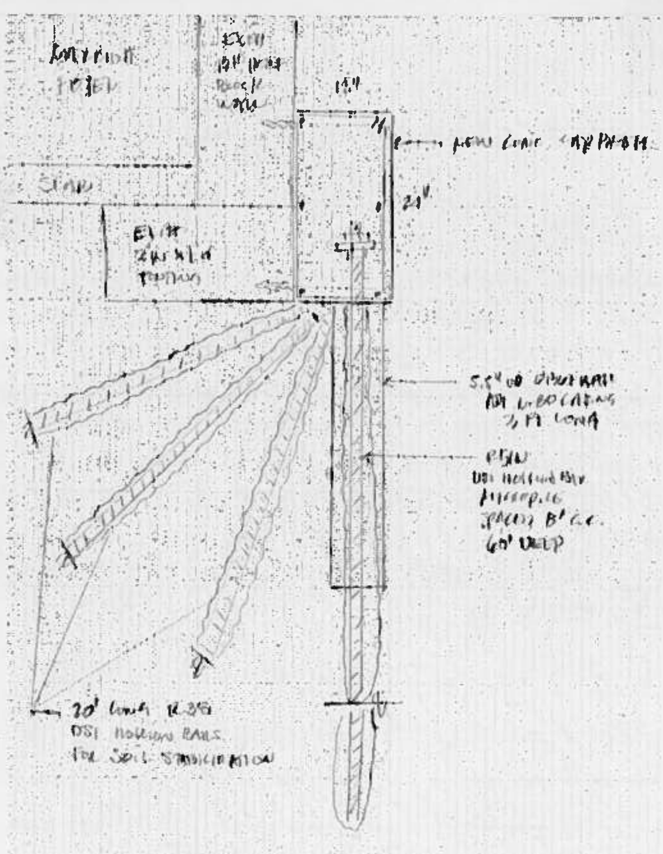

FIGURE 6 – Block 76 Marriott Hotel Underpinning Concept

FIGURE 7 – Block 76 Marriott Hotel Underpinning Micropile Installation

Jet Grouting:

As discussed above, jet grouting was selected as an alternate to dewatering along South Temple and Main Streets. Prior to commencing production jet grouting, a jet grout field test program was instituted to optimize injection parameters and to confirm jet grout element geometric and mechanical properties. The test program involved the construction of 4 sets of 2 test columns followed by a 3 day cure time. The injection times were varied for each set of test columns, while all other parameters remained constant. The grout mix included a small amount of bentonite (2% by weight) to stabilize the grout mix and provide additional water control properties to the grout mix. The target column diameter was 4ft using the single fluid technique. The test columns were then exposed by excavation and geometric properties obtained by direct measurement (Figure 8 Test Columns). All test sets achieved 4ft column diameters or greater and production work commenced.

FIGURE 8 – Block 76 Jet Grout Test Columns – Initial Excavation

The jet grout walls were located along South Temple and Main Street as shown in Figure 4. Each wall consisted of a single row of overlapping secant columns and connected the diaphragm wall segments between the Temple View Center Building and the Gateway Tower on South Temple Street and between the Gateway Tower and the McIntyre Building on Main Street. By connecting to the Diaphragm walls, the north east quadrant of the site was essentially cutoff from major water inflows.

The jet grout columns were installed so the bottom of column was 5-10ft below the base of the excavation and the top of the column was installed to 5ft above the known high water elevation determined from the observation well readings. The average drill length for each column was approximately 50 ft and the jet length was 30 ft. 236 columns were installed in a 2 month time frame to complete the water control activities enabling completion of soil nail walls below the ground water table. Figure 9 shows the jet grout wall with soil nails being prepared for shotcrete facing.

FIGURE 9 – Block 76 Jet Grout Wall South Temple Street

Diaphragm Wall (D-Wall):

D-Walls were constructed adjacent to the Temple View Center building (9 stories), the Gateway Tower (23 Stories) and the McIntyre and Crandall Buildings (9 Stories). Each building is on shallow foundations, and excavation was required to approximately 40ft below the foundations.

Each Building had its own challenges. The D-Wall adjacent to the Temple View Center needed to be installed within 1ft of the existing building facade and required removal of a portion of the existing footings that protruded into the dwall alignment. Prior to any footing removal, a micropile underpinning system was installed to replace the capacity of the existing footings that were to be removed. After the micropiles were installed, excavation for footing removal revealed a larger footing than expected and required a wire saw to remove. (Figure 10). D-Wall Panels in this area were installed as "single bites" meaning open D-Wall excavation trenches were limited to the length of the bucket, i.e. 9ft. This technique was employed because the TVC building was supported by independent spread footings and we did not want to induce any potential bearing failure by having open trenches greater than the length of the bucket.

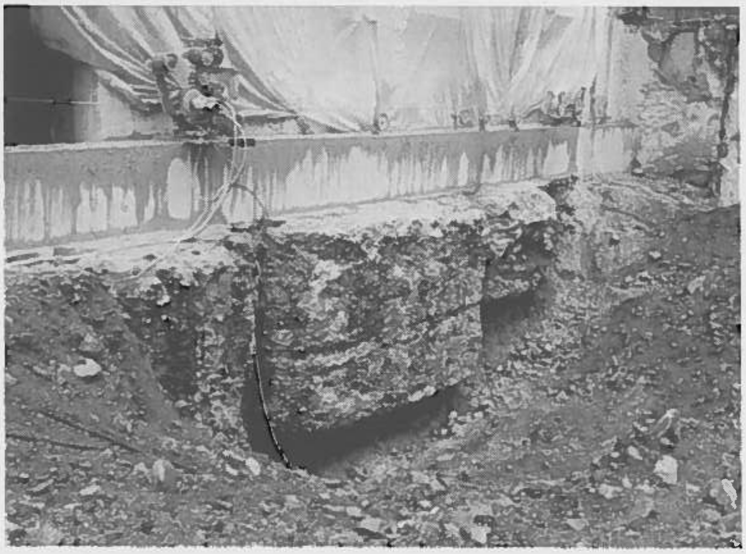

FIGURE 10 – Block 76 Temple View Center footing in Dwall Alignment

Along the Gateway Tower alignment, an existing building overhang was present approximately 86ft above our work platform elevation. This required careful consideration of crane requirements and rigging to ensure the reinforcing cages, when tripped would fit under the overhang without the added costs of cage splicing. The actual placement of cages required the crane boom tip to be within 3ft of the underside of the overhang (Figure 11). As a precautionary measure, the crane boom was fitted with a proximity sensor that warned the operator when he was too close to the building. On the plus side, the Gateway Tower was founded on an 8ft thick concrete mat, enabling us to construct the D-Wall with traditional 22ft long "three bite" panels.

FIGURE 11 – Block 76 Diaphragm Wall at Gateway Tower Cage Placement

At the McIntyre and Crandall Buildings, no as-built drawings were in existence. These building were constructed in the 1900's and are some of the oldest in Salt Lake City. We expected surprises, but only one surfaced which required adjusting the D-Wall alignment north to miss the existing McIntyre footing.

In total 43 panels were installed to an average depth of 60ft. Over 45,000sf of D-Wall was installed in two months, beating the project schedule and eliminating a demobilization and remobilization because all D-Walls were complete prior to the implosion of the Key Bank Tower, a significant project milestone. Figure 12 below is believed to be the excavation for the first anchored diaphragm wall panel for earth retention in Salt Lake City, Utah.

FIGURE 12 – Block 76 First Anchored Diaphragm Wall in Salt Lake City June 2007

MONITORING OF EARTH RETNTION SYSTEM DURING EXCATATION

Nicholson's overall scope included the monitoring, collection, and reporting of geotechnical instrumentation on the project. The Sol Data Cyclops system provided the real time monitoring of the shoring systems and building movements. Traditional inclinometers, observation wells and surface settlement points were reported on a weekly basis. Evaluation of earth retention system and building movements were done routinely by our Engineer, GEI Consultants, and Nicholson's project and home office staff. Figure 13 shows the inclinometer data and provides wall performance behavior of a typical Soil Nail Wall, in this case along West Temple Street.

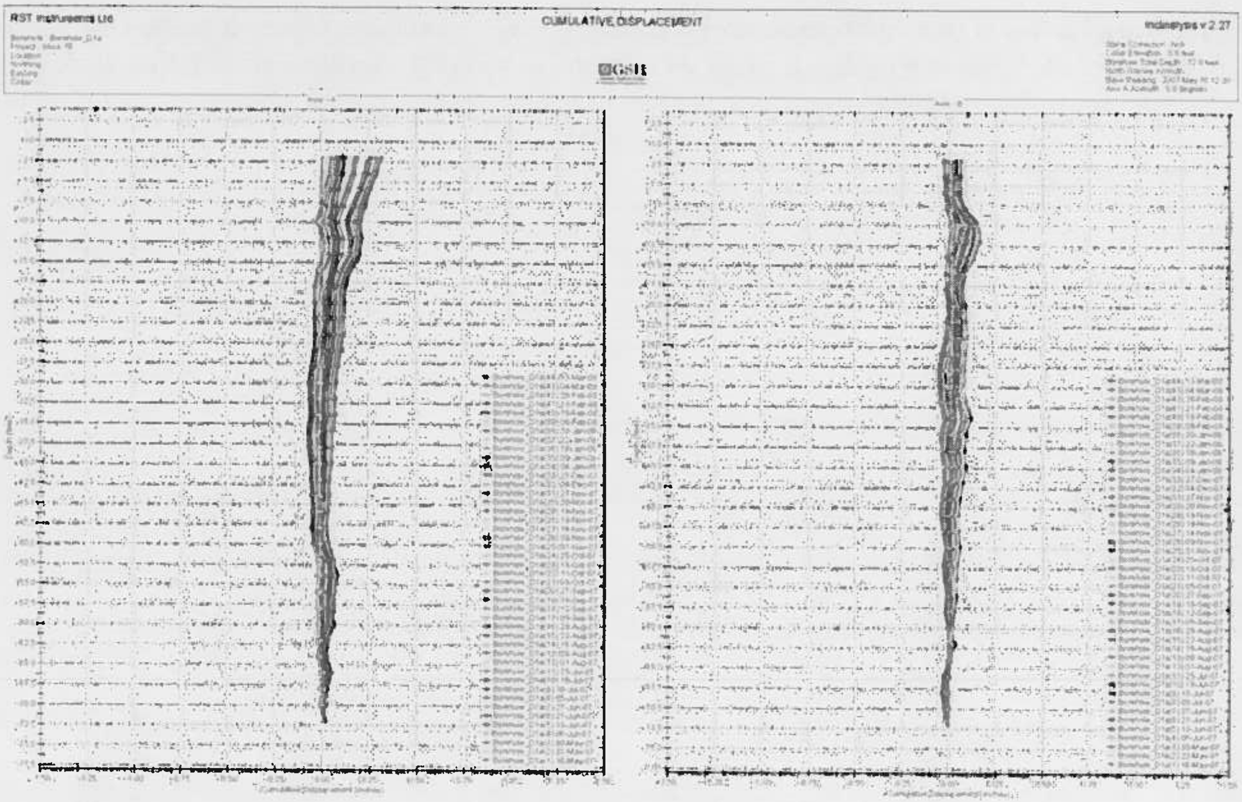

FIGURE 13 – Inclinometer Data of Soil Nail Wall Along West Temple Street

Movement of the soil nail walls was expected to follow the typical pattern of rotation about the toe, similar to a gravity or cantilevered retaining wall. Maximum movements were expected at the top of the walls, with magnitudes in the range of 1½ to 2 inches for the deepest cuts (65 feet). Observed movements were typically much less than expected (generally by more than 50%). We attribute the good performance of the soil nail walls to the predominance of the upper gravel stratum and its slightly cemented nature (which was conservatively neglected in the design). In isolated areas where the upper gravel stratum was disturbed, the observed deformations were closer in magnitude to the anticipated values.

Figure 14 shows the inclinometer data and provides wall performance behavior of a typical D-Wall, in this case along the McIntyre Building.

Movement of the diaphragm walls was expected to follow a pattern of increasing deflection with depth, corresponding to greater driving loads and the weaker clay and silt strata near the bottom of the cut. Maximum movements were expected between mid-height and the toe of the walls, with magnitudes of approximately 1 inch for the deepest cuts (40 feet). Observed movements were typically in line with expectations, although additional movement control measures were required in some portions of the work. Subgrade struts and supplemental anchors were used to control diaphragm wall deflections adjacent to the Gateway Tower, where the weaker clay & silt strata were found at a higher elevation than anticipated.

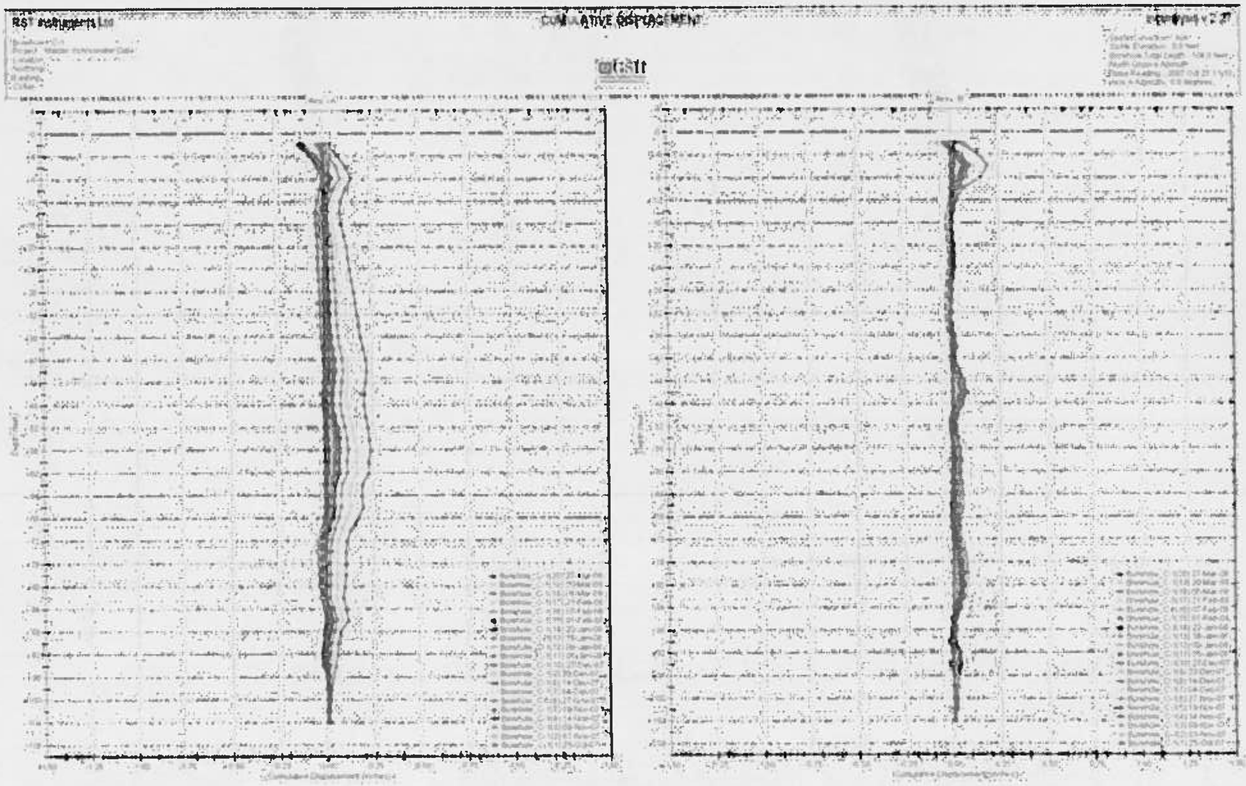

FIGURE 14 – Inclinometer Data of D-Wall at McIntyre Building

Closing:

In total, Nicholson's earth retention scope included 45,000 sf of Diaphragm Wall, 104,000 sf of Soil Nail Wall, Micropile underpinning of 3 buildings, and jet grouting for water control where soil nails walls were below the ground water table. Sol Data's Cyclops monitoring system provided real time data in a web based format for quick evaluation of earth retention systems and building deformations. Traditional geotechnical instrumentation, such as inclinometers, validated the Cyclops data and provided wall movement at incremental depths for subsequent wall behavior analyses.

The scope and magnitude of this project is unprecedented in Salt Lake City. The amount of earth retention for deep excavation accomplished in around and below existing buildings was truly unique and full of challenges. Figure 15 depicts a wide angle view of the project to give the reader some prospective.

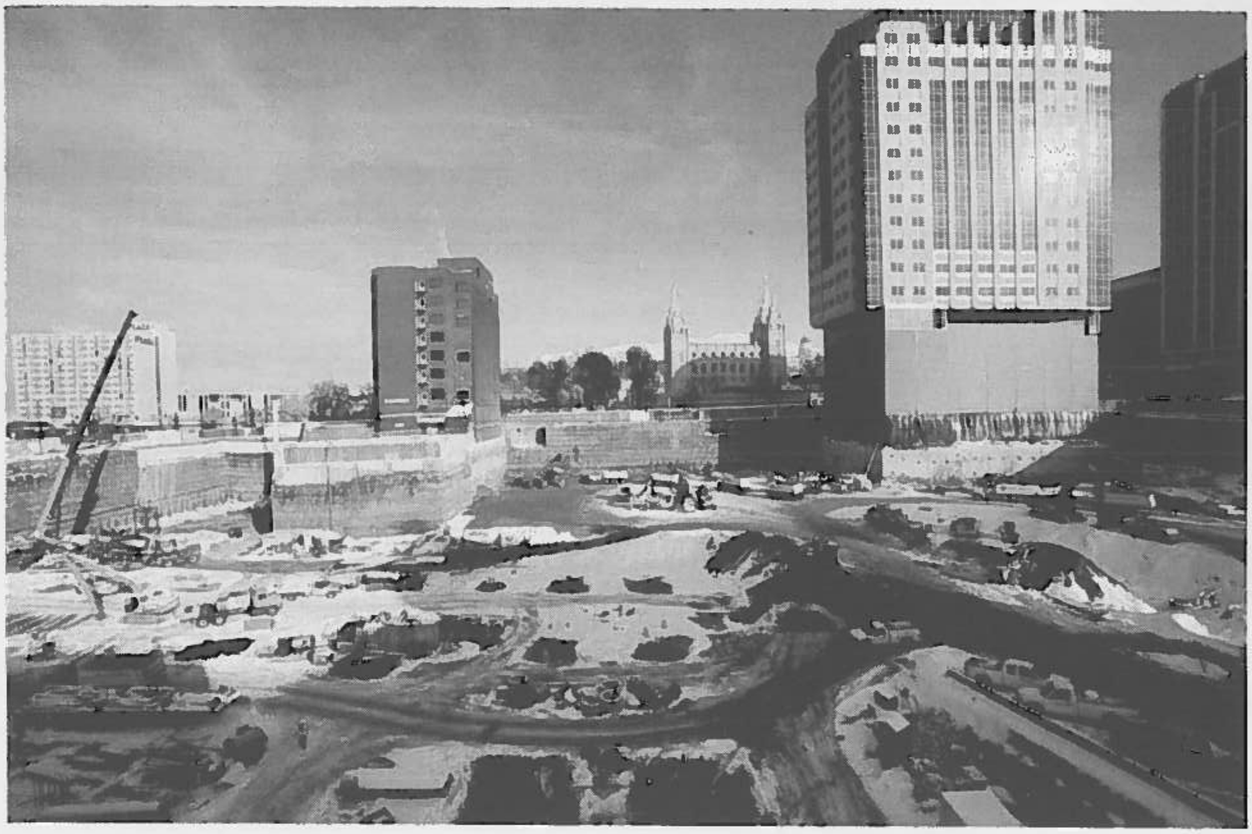

FIGURE 15 – Wide angle view of Block 76 Site

Currently, the new construction is under way and over 1000 – 24" diameter augercast piles are being installed beneath the entire footprint of the Block 76 site. Each pile has a #11 or #18 center bar with a rebar cage going to half the depth of the pile. The average pile length is 75 ft and some piles are over 100 ft. We are currently about 80% complete and leave you with a site photo of completed piles in the Tower 1 Area.

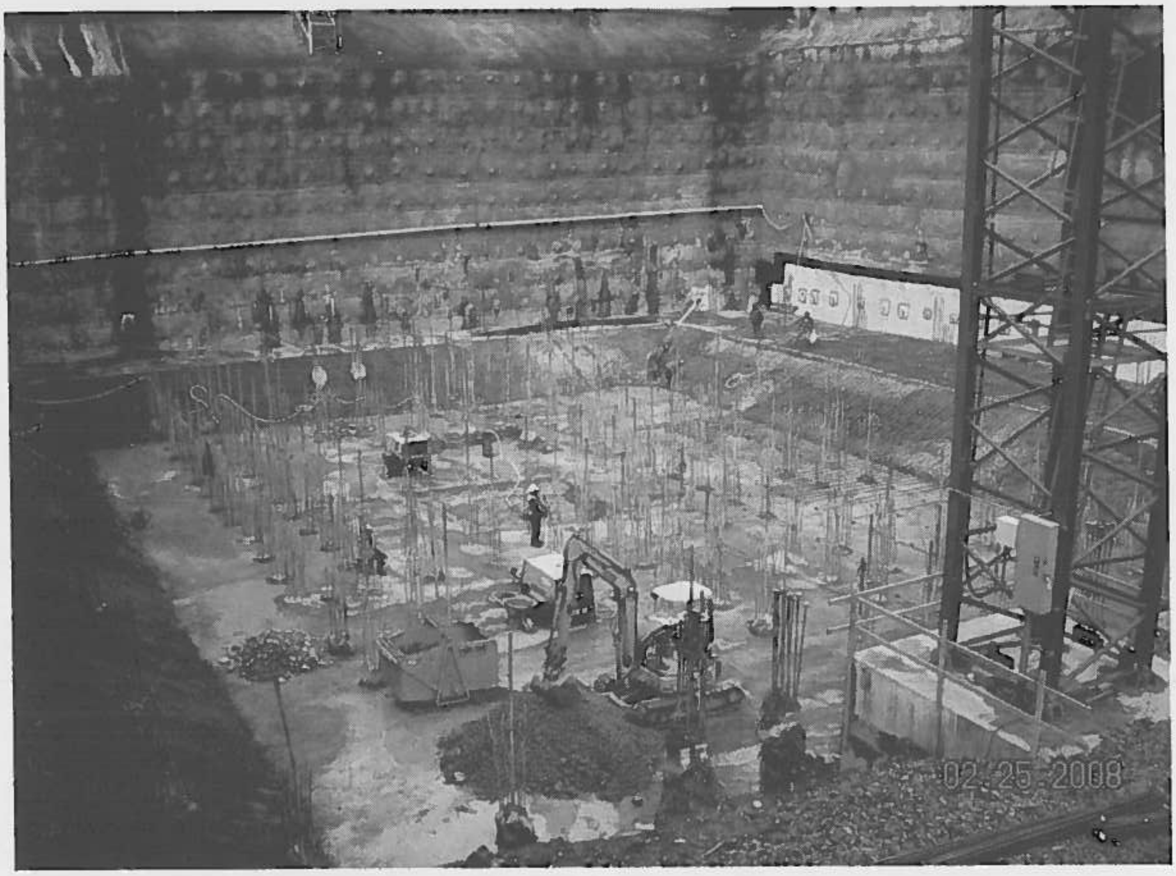

FIGURE 16 – Augercast Piles in Tower 1

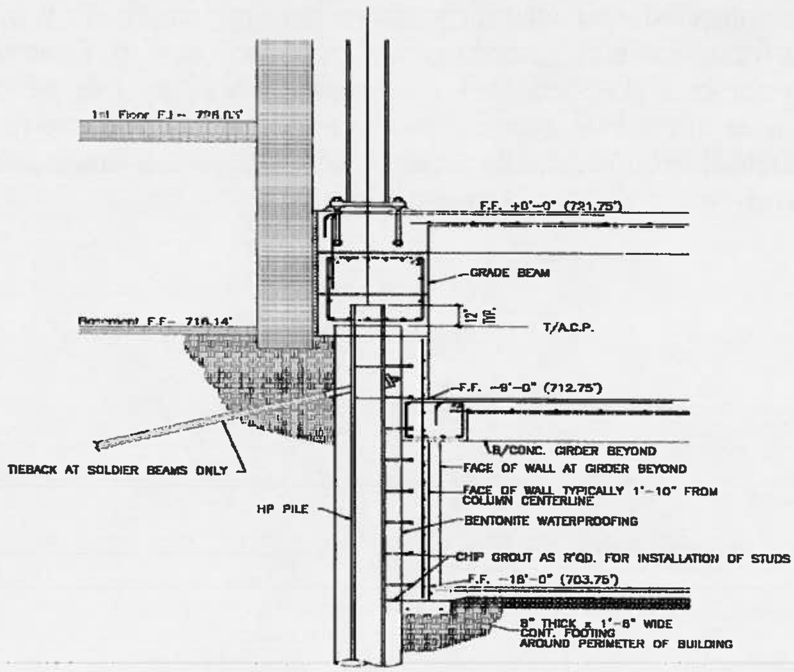

Fig. 4. Typical Section at Soldier Beam at Hammond Building

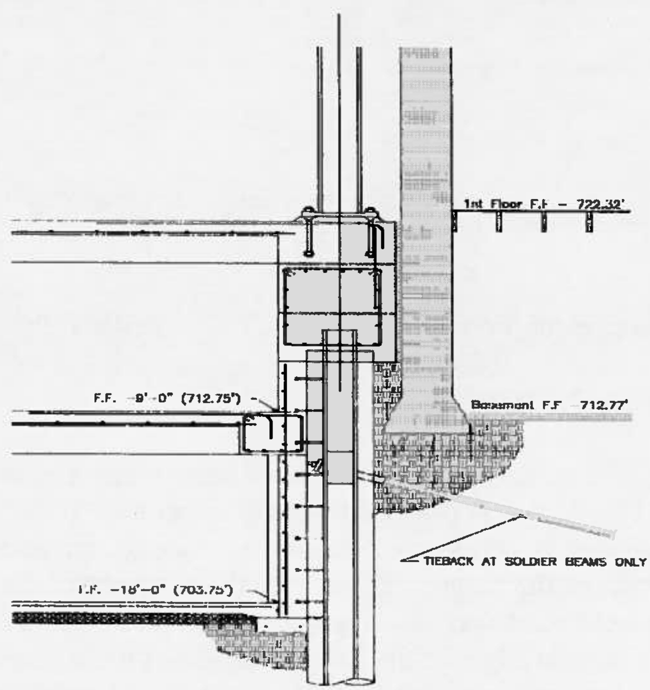

Fig. 5. Typical Section at Soldier Beam at Marsalla Building

For the cantilevered wall adjacent to the parking lot, an HP shape of 14x73 size was utilized on 6-ft centers limiting lateral ground movement to ¼ in. To accomplish this, two 1⅜-in. diameter bars (i.e., tendons) were located on either side of the web and adjacent to the inside of the back flange. Refer to Figure 6. The pile was inserted into a predrilled grout fluid-filled hole, and the tendons were prestressed simultaneously with a side-by-side jacking system to 200-kip stressing level.

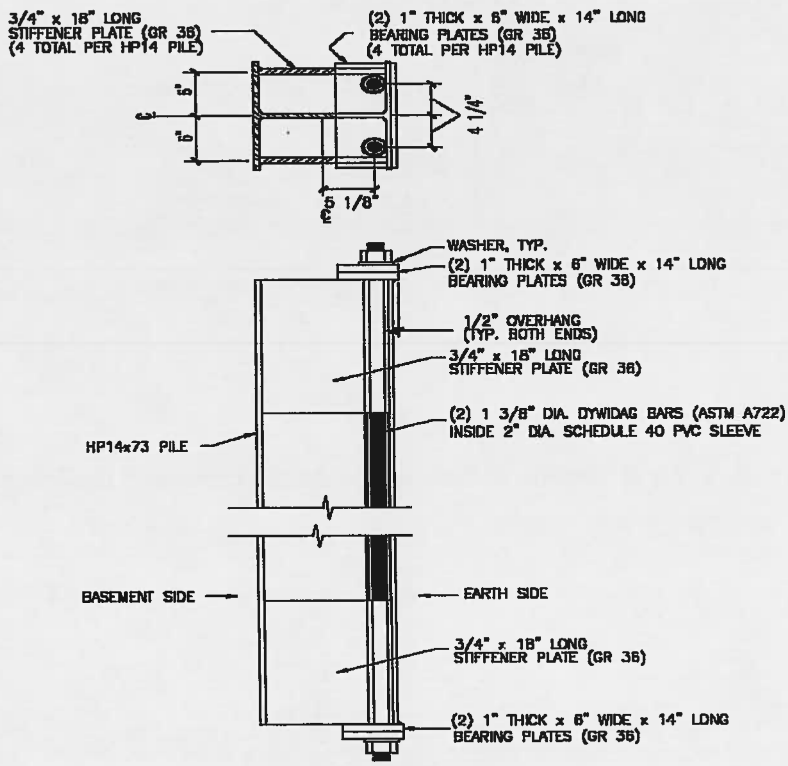

Fig. 6. Typical Section for Vertically-Prestressed Cantilevered Pile along Parking Lot

PERFORMANCE MONITORING RESULTS

Monitoring of a select number of soldier beams and cantilevered piles at those locations shown in Fig. 1. was performed to assess excavation performance and provide a means of further action, if necessary. Monitoring was accomplished via inclinometers installed on the outside of the flanges. To validate the accuracy of the instrumentation, one of the five instrumented piles was also equipped with four pairs of vibrating wire strain gages located on the outside edges of the flanges. Readings were generally taken on weekly intervals until the excavation reached a depth of 20 ft and monthly thereafter until the framing was in place. For purposes of discussion, results from two inclinometers are presented below.

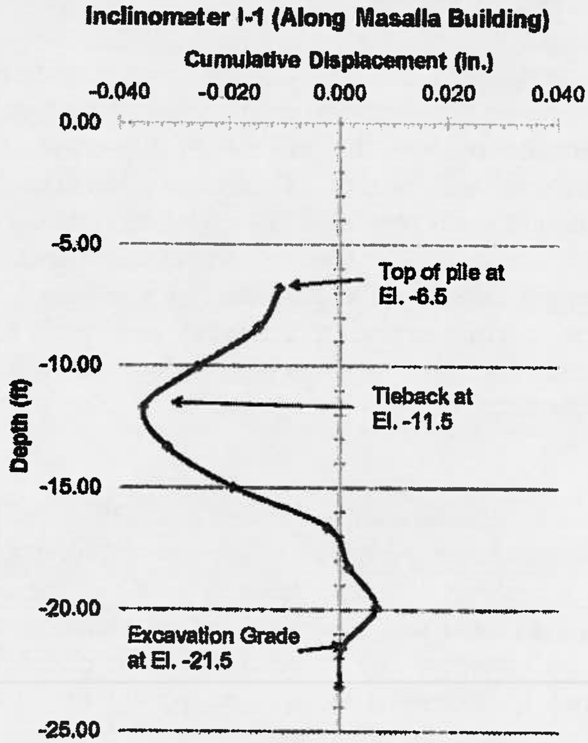

Fig. 7 Deflection Profile at Inclinometer I-1

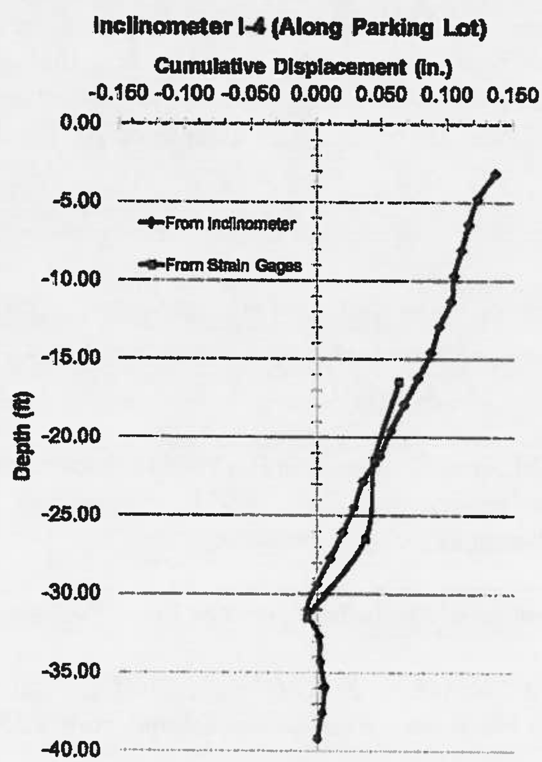

Fig. 8 Deflection Profile at Inclinometer I-4

DISCUSSION AND CONCLUSIONS

The key to understanding the behavior of earth retention systems lies in the deflection profile. Fig. 7 shows the results from an inclinometer located along the Marsalla building. This inclinometer reflects the effects of top-down construction including installation of the tieback, the effects of structural system stiffness/serviceability requirements, and surcharges/earth pressures. To understand this, a regression analysis of the deflection profile was performed in order to compute the curvature in the piling. From the curvature, the bending moment can be computed by knowing the flexural rigidity (EI). Based on the deflections, bending stresses in the piling were generally less than 500 lbs/sq in (psi). These stresses are relatively low when comparing them to the actual strength of the piling considering that the section of the piling was selected on a limiting deflection of ⅛ in.

Fig. 8 shows the results from an inclinometer and four pairs of strain gages. The strain gage locations were chosen based on where the maximum bending moment was anticipated to occur in the piling. It is important to note that these results reflect the net effect of the prestressing element and, therefore, do not include the prestressing bending moment. In this case, the curvature can be obtained as the difference between the strain gage readings measured at the same depth divided by the distance between them. Consequently, bending stresses in the piling were generally less than 1,000 psi as a result of the earth pressure. Back-calculating an average earth pressure distribution, this was less than what was used for design and a limiting movement of ¼ in. The reason for the lower than expected earth pressures is not know and could be attributed to a number of factors including shearing stresses in the soil that transfer part of the load to lagging or the presence of an existing element, e.g., an adjacent basement, that would tend to limit earth pressures. When considering the prestressing effects, the maximum bending moment in the pile after tensioning was about 700 in-kips near a depth of 20 ft and about 800 in-kips after excavating.

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CHALLENGES TO SUCCESSFUL URBAN SOIL NAIL EARTH RETENTION SYSTEMS

Abstract:
This paper will discuss a case history involving successfully meeting urban-environment challenges at a construction site in Seattle, Washington. Temporary excavation shoring was required for the construction of one to two levels of underground parking for a retail / residential development that will eventually encompass an entire city block. This project has been organized into four phases and, at the time of publication two of these phases have been completed.

Given the relatively high property values typical in urbanized areas, property owners and developers are choosing to dimension new building footprints so as to completely fill their properties. Additionally, there is an ever-increasing desire to incorporate underground levels into building construction, primarily to provide sufficient parking space for the building's tenants. This need to maximize the use of every square foot of property creates several physical hurdles with respect to both the design and construction of earth retention systems used to support temporary excavations to construct underground levels.

The project featured in this case study included both developing the maximum property area possible as well as incorporating one to two levels of underground parking. The physical hurdles the project team had to overcome in order to develop this project site included:

- Shallow utilities along the public right of ways;
- Settlement-sensitive, concrete masonry block buildings located alongside each phase of construction, which required both underpinning and lateral support;
- A significant tree that required preservation; and,
- A concrete retaining wall supporting an existing parking lot that had to be maintained during construction.

A combination of conventional and hybrid soil nail earth retention systems were designed to support the excavation sidewalls during the first two phases of this case history's project. Where needed to either support adjacent structures or wedges of fill soil associated with adjacent utilities, vertical elements were incorporated into the shoring system. Where lateral movement was not tolerable, the soil nail system incorporated horizontally-reinforced, stressed soil nails. Soil nailing was selected over

more conventional soldier pile and tieback earth retention for this project because of its flexibility both during design and construction, lower costs, and the desire to utilize the maximum possible area within the property boundaries.

INTRODUCTION

There is an ever-increasing demand in urban environments to maximize land usage within city limits for residential, retail, commercial, and transportation purposes. Cities in the United States continue to grow in both resident populations and urban employment. Medium- to high-rise buildings for commercial and corporate use necessitate comparable construction to provide accessible residences for the employees of those companies. Since increased commuter traffic continues to outstrip growth of the transportation infrastructure in many urban areas in the United States the demand for high-density residential buildings in proximity to urban business and commercial areas continues to rise.

The need for a high-rise building; whether intended for commercial, retail, corporate, and/or residential purposes, includes the need to provide adequate parking spaces for the anticipated number of tenant automobiles as well as possible public parking for surrounding businesses and residences. Since an ever-increasing number of these buildings will occupy the entire parcel of land bordered by city streets or other properties, the only viable locations for the necessary parking areas are underground. The anticipated number of automobiles associated with the building's tenants can necessitate two, three, or more levels of underground parking.

The construction of just one level of underground parking for a high-rise building can require excavations in excess of fifteen feet below the adjacent street levels. Multiple underground levels may require substantially deeper excavations. With building footprints now filling much, if not all, of the available land surface between streets and adjacent properties, adequate sloping of temporary construction excavations is typically not possible. Consequently, temporary, vertical or near-vertical earth retention options have to be considered. In areas where it is suitable, soil nail earth retention systems are frequently the most cost-effective means of achieving these tall, near-vertical excavation sidewalls.

The purpose of this paper is to discuss the use of soil nailing earth retention systems for a phased, retail / residential, medium-rise building project located in Seattle, Washington. In particular, this paper will address the challenges and special conditions encountered because of the urban environment this development was constructed in.

PROJECT DESCRIPTION

The overall development project (designated the Press Apartments) was divided into four phases. Each phase corresponded to one apartment building located within one city block. At the time this paper was published, two of the Press apartment buildings have been completed (designated in chronological construction sequence Press I and Press II for the purposes of this paper). The construction period for Press I and Press II were separated by approximately 4 years. Together, Press I and Press II fill the west half the city block bounded by East Pine Street to the south, Belmont Avenue to the west, and East Olive Street to the north, see Figure 1.

The east half of the city block is presently occupied by 2- to 3-story wood, masonry, brick, and concrete commercial and residential buildings; and a surface gravel parking lot. Development of this half of the city block with two future apartment buildings (Press III and Press IV) is still being considered.

Press I consists of a 6-story apartment building with two levels of underground parking. The building was constructed in the southwest quarter of the city block, as shown in Figure 1. Construction began in 2001 and was completed in 2002. Temporary shoring systems were required on all four sides of the building footprint for this phase. In addition to the retained earth, the shoring systems were designed to support the existing buildings to the north and east, and the existing streets and utility infrastructures to the west and south.

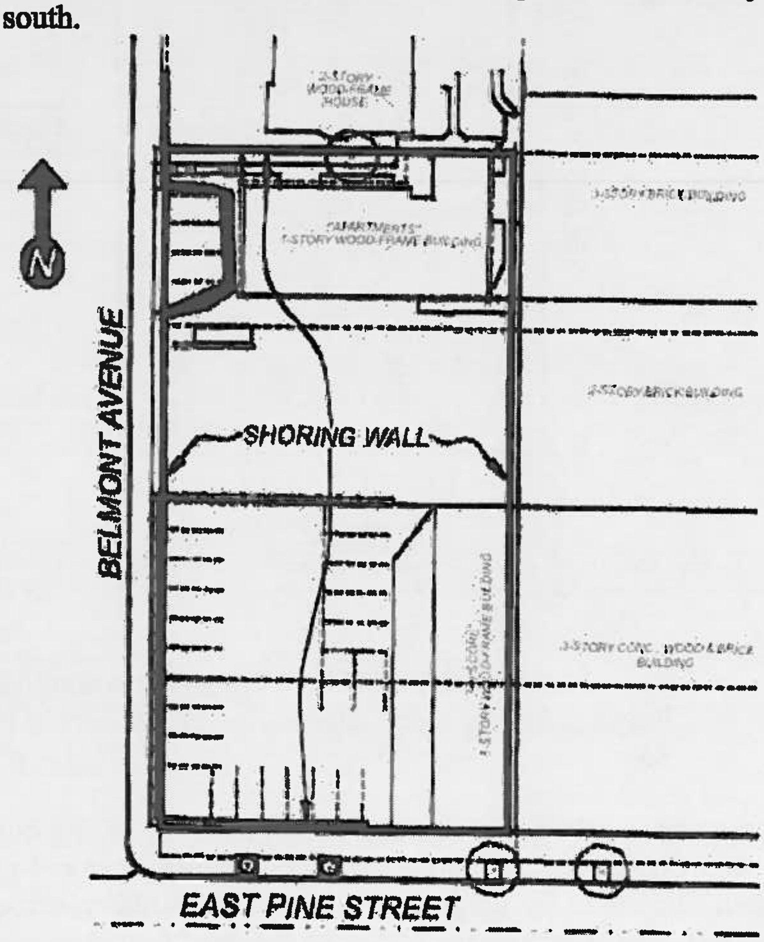

Figure 1: Footprint Of The Press I Excavation Shoring Wall. Existing Features Shown From The Topographical Survey Map Prepared For This Site

Press II consisted of a 6-story apartment building with one level of underground parking. The building was constructed in the northwest quarter of the city block, directly north of Press I, as shown in Figure 2. Construction for this building began in 2005 and was completed in 2006. Temporary shoring systems were required on three of the four sides of the building footprint for this phase. The Press I underground stem wall was exposed during excavation for Press II, and the temporary shoring system previously used for the north sidewall of Press I had to be demolished and removed. In addition to the retained earth, the shoring systems were designed to support the existing buildings and a concrete retaining wall to the east; and the existing streets and utility infrastructures to the west and north.

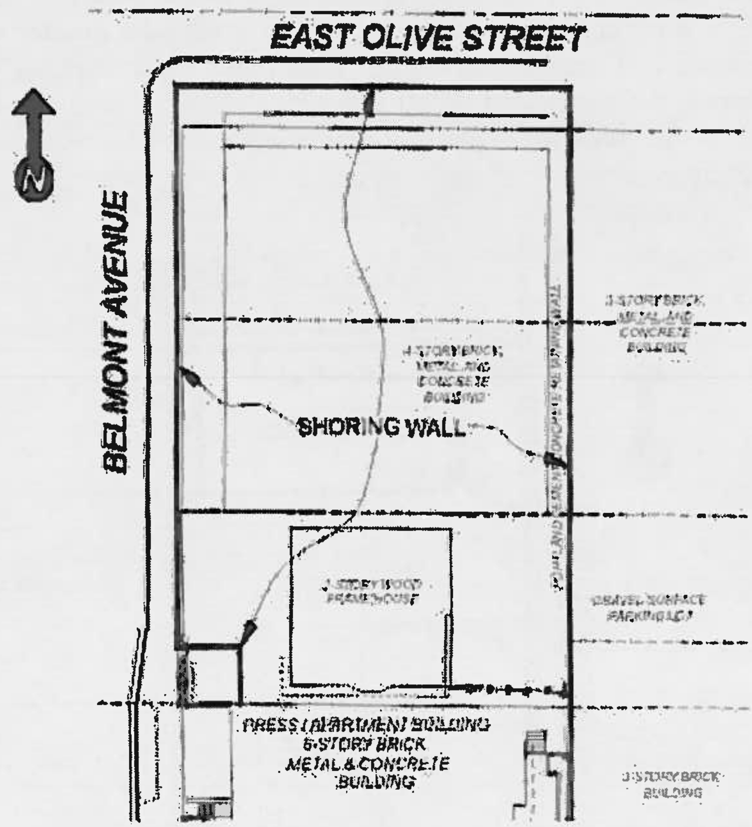

Figure 2: Footprint Of The Press II Excavation Shoring Wall. Existing Features Shown From The Topographical Survey Map Prepared For This Site

One additional site feature had to be preserved during both construction project phases. The City of Seattle required the preservation of an old-growth Japanese Maple tree located adjacent to the northwest and southwest building corners of Press I and Press II, respectively. Shoring systems had to be designed to preserve both the soil supporting the tree as well as the major root structure of the tree itself.

Soil nailing was selected for the temporary excavation shoring systems for both project phases because of its ease and flexibility of use during both design and construction, the presence of reasonably-ideal soil and groundwater conditions at the site, and a favorable evaluation of the expenses of soil nailing as contrasted with other vertical temporary shoring system options.

GEOTECHNICAL SITE CONDITIONS

Subsurface conditions for the Press I and Press II sites generally consisted of about 1 to 15 feet of sand and gravel fill overlying Quaternary-era colluvial and alluvial silty sand and sand. At depths on the order of 11 to 15 feet below the ground surface, glacially-consolidated silty sand till was encountered. The fills and Quaternary-era deposits were generally loose to medium dense, with dense to very dense conditions observed in the underlying till.

Static groundwater levels at the project site were deeper then the construction excavations. However, seams of perched groundwater were encountered during the geotechnical exploration and subsequent construction activities at both sites. These seams were of finite volume, and generally controlled by careful excavation and construction drainage-control practices during construction of the shoring systems.

DESIGN AND CONSTRUCTION CHALLENGES

The general design procedures for both project phases were performed in general accordance with the "Manual For Design & Construction Monitoring of Soil Nail Walls", published by the United States Department of Transportation, Federal Highway Administration in November 1996 and revised in October 1998.

In general, designing of temporary soil nail shoring systems for excavation sidewalls is a straightforward design process, provided the in-situ soil and groundwater conditions at a project site are appropriate for soil nail installation. These designs typically have simple geometries, and reasonable access can be provided and maintained for the construction of the design. However, the urban environment surrounding both Press apartment building project phases introduced additional challenges and hurdles to be overcome both during the design processes and subsequently during construction of each phase.

Press I Apartment Building

There were four, characteristically-urban, challenges to the design of the temporary shoring systems for Press I. Those challenges were:

1. In-place utilities beneath the city rights-of-way adjoining the site to the south and west;
2. The existing masonry and block buildings adjoining the site to the east;
3. The wood-framed house adjoining the site to the north; and,
4. The large Japanese Maple tree located adjacent to the northwest corner of the project site.

Each of these challenges arose because the Press I excavation sidewalls needed to be located as close to the property lines as possible. Each challenge, and the solutions implemented to address each challenge, are discussed in more detail as follows:

1. Generally speaking, city engineering departments are sensitive to how close a soil nail drillhole advances to an existing utility or other buried structure (such as a catchbasin or utility vault). Utilities are frequently variable in their burial depths, and city engineering records are often more precise about the alignments of utilities then their buried depths. As a result, the city engineering departments do not typically allow soil nails to extend within three lineal feet of the utility. In addition, city engineering departments do not typically allow soil nails to be installed over a utility alignment because of the maintenance issues.

   Consequently, a utility or other buried structure located horizontally within a couple feet of the embedded end of a soil nail can have profound implications on the design of the shoring wall. These close utilities often result in nails located at odd spacings or locations on the wall, variable inclinations or lengths, or, at worst, the need for another shoring method to be designed and constructed to address situations where installation of effective soil nails proves infeasible. Since utility alignments can create significant cost and feasibility implications during the design and construction phases of a project, it becomes very important to perform a detailed utility investigation during the initial stages of soil nail design at an urban project site.

   The soil nail design for Press I required investigation of utilities located under the rights-of-way on the south and west sides of the project site of the surrounding utilities. The investigation yielded records of utilities located a horizontal distance from the faces of the south and west sidewalls of the planned excavation within the length of the drilled nails (See Figure 3). On the west sidewall, this challenge was overcome by sloping back the upper three vertical feet of excavation sidewall using a 1H:1V (horizontal to vertical) cut slope. This cut slope allowed the first row of soil nails to be lowered to sufficient depth to pass below the existing utility alignments.

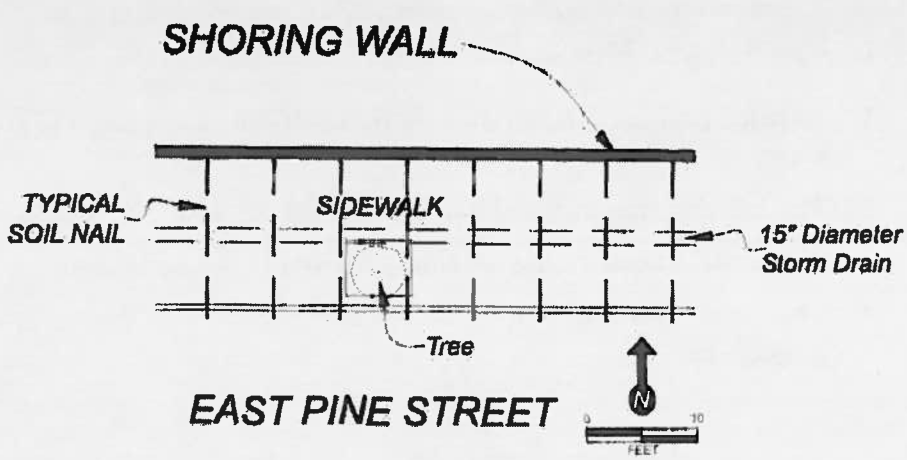

Figure 3: Typical Area Illustrating The Proximity Of The Soil Nail Wall Face To Existing Utility Alignments And Trees

However, a cut slope at the top of the excavation could not be performed for the south sidewall, as the City of Seattle building department mandated the preservation of a line of trees in the existing sidewalk on the south side of the project (illustrated in both Figures 1 and 3). After significant negotiations with the City of Seattle, an agreement was reached to allow temporary soil nails to pass over utility alignments. Figure 4 illustrates a typical example of the adjustments we made to a two-row soil nail wall section adjacent to Pine Street, where soil nails had to be adjusted around a 15-inch diameter storm sewer line. As illustrated, the declinations of the soil nails were changed from the typical 15-degree declination for soil nail design in order to increase the clearance distance between the soil nails and the utility.

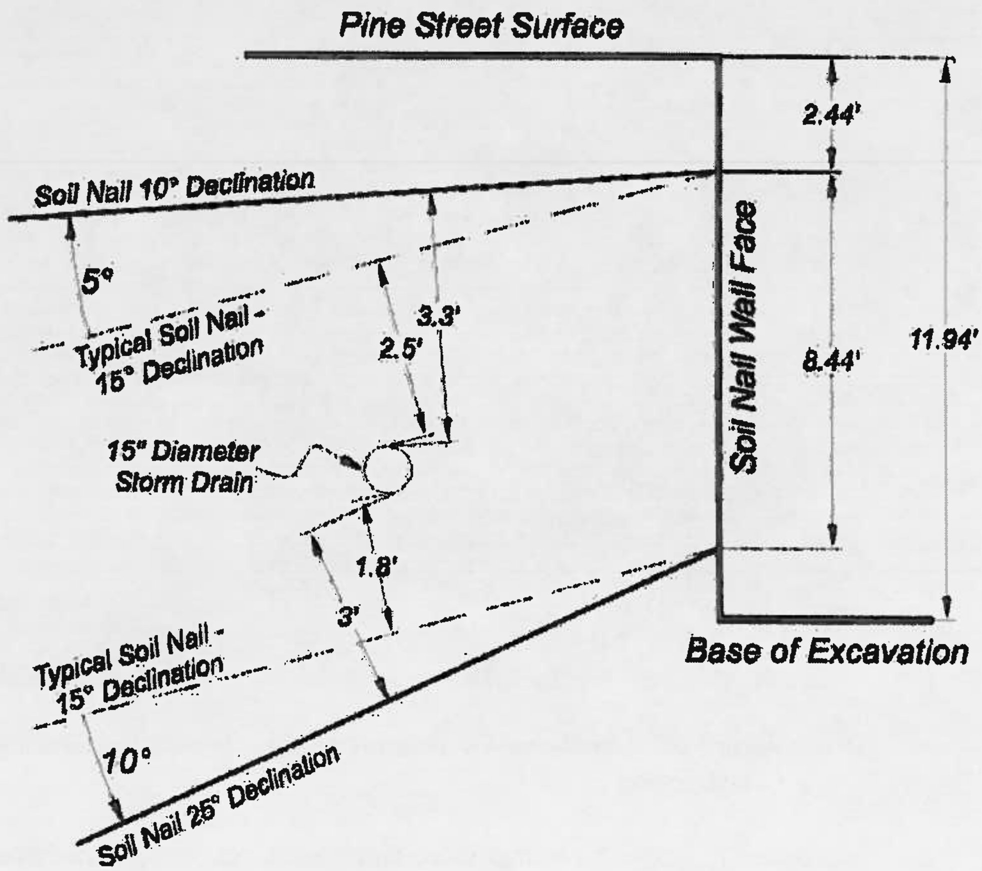

Figure 4: Typical Cross-Section Illustrating The Adjustments Performed On Soil Nails Extending Into Utility Alignments

This successful resolution was achieved because of the good relationship the geotechnical engineer maintained with the City of Seattle engineering department and the design team's willingness to investigate utility conflicts in advance and maintain flexibility regarding the soil nail design, layout, and orientation.

2. The existing buildings on the east sides of Press I represented another challenge to the design team. These buildings adjoining the east side of the excavation were relatively old, two-story, brick and concrete masonry structures.

   Existing cracks were observed on the floor slab and one south wall inside one of the building, which was utilized as a repair shop at the time of construction. The locations of these three cracks (designated Crack 1, Crack 2, and Crack 3) are shown in Figure 5.

Figure 5: Locations Of Interior Cracks Monitored During The Press I Construction

   Consequently, these buildings were considered susceptible to damage from both settlement and lateral movement. Since conventional soil nail shoring systems are passive systems and rely on wall movement in order to mobilize soil shear strength, a conventional soil nail shoring system was considered unsuitable to adequately support these buildings without adding an unfeasible level of building movement.

   Instead, a hybrid shoring system comprised of pre-stressed soil nails, horizontal steel walers, and steel-beam vertical elements was designed for the excavation wall adjacent to these buildings. This hybrid system, called a VENIS® system, permits the use of soil nail shoring without requiring the relatively small degree of wall movement necessary for a conventional soil nail wall.

Installation of the VENIS® system starts with drilling vertical holes along the line of the excavation and installing vertical elements. This work is performed before face excavation begins. The holes drilled for these vertical elements were 10 inches in diameter and extended to a depth four feet below the final excavation depth. The vertical elements consisted of W6x12 Grade 50 structural steel beams. Once the vertical element was installed in each hole, the hole was backfilled with concrete.

Pre-stressed soil nails are drilled, installed, and grouted in the same fashion as more conventional soil nails. The vertical excavation face cut for each row of nails is trimmed flush with the outside flange of the vertical elements, and the soil nails are installed. However, instead of a square steel plate for each nail, a 4-foot long, horizontal, steel tube waler is installed over each soil nail and rested on studs welded to the adjacent vertical elements, connecting the soil nails with the vertical elements. Steel thickness for the walers varied with height above the excavation base, with 1/2-inch thick steel used for the upper walers and 5/16-inch thick steel used for the lower. A hydraulic ram is attached to each soil nail and load applied to pre-stress the nail. The nut for each nail is then tightened down to the outside of the horizontal waler, firmly connecting it to the soil nail, and then the load placed on the nail is released.

Figures 6 and 7 together illustrate the relationship between the vertical elements, the soil nails, the horizontal walers, the shotcrete construction facing, and the drainage elements installed in spaces between vertical elements not occupied by soil nails.

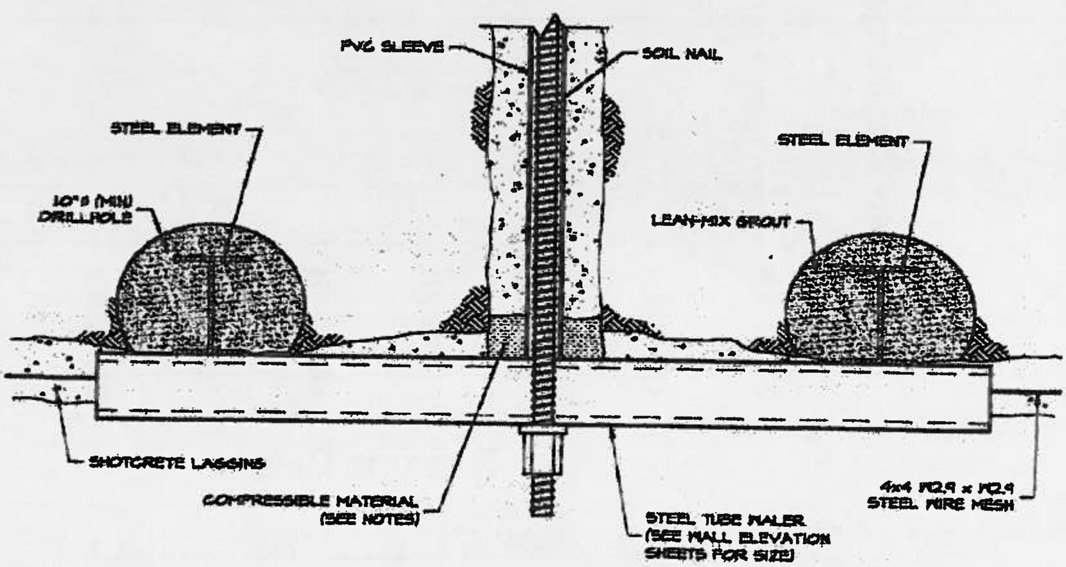

Figure 6: Horizontal Section Depicting Vertical Elements And Soil Nails

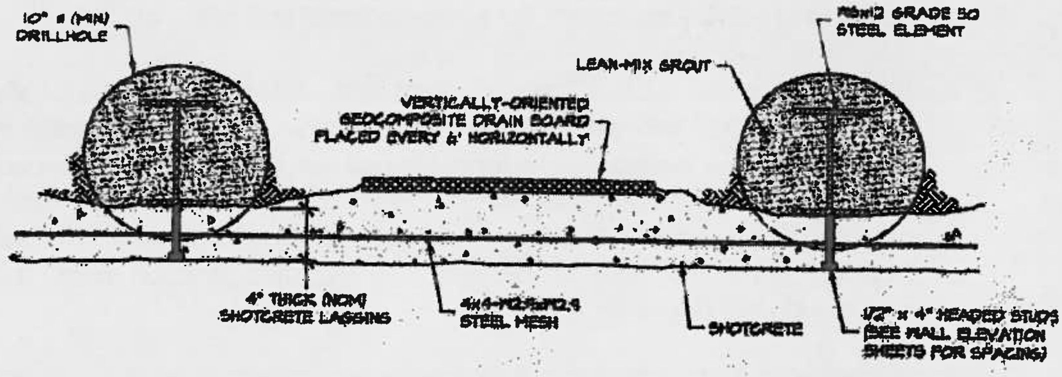

Figure 7: Horizontal Section Depicting Vertical Elements Without Soil Nails

To verify that there would not be further movement of the existing cracks during the construction of the underground portions of Press I, a monitoring program was implemented in this building to observe displacement changes in the cracks. This monitoring was performed between April and December, 2006.

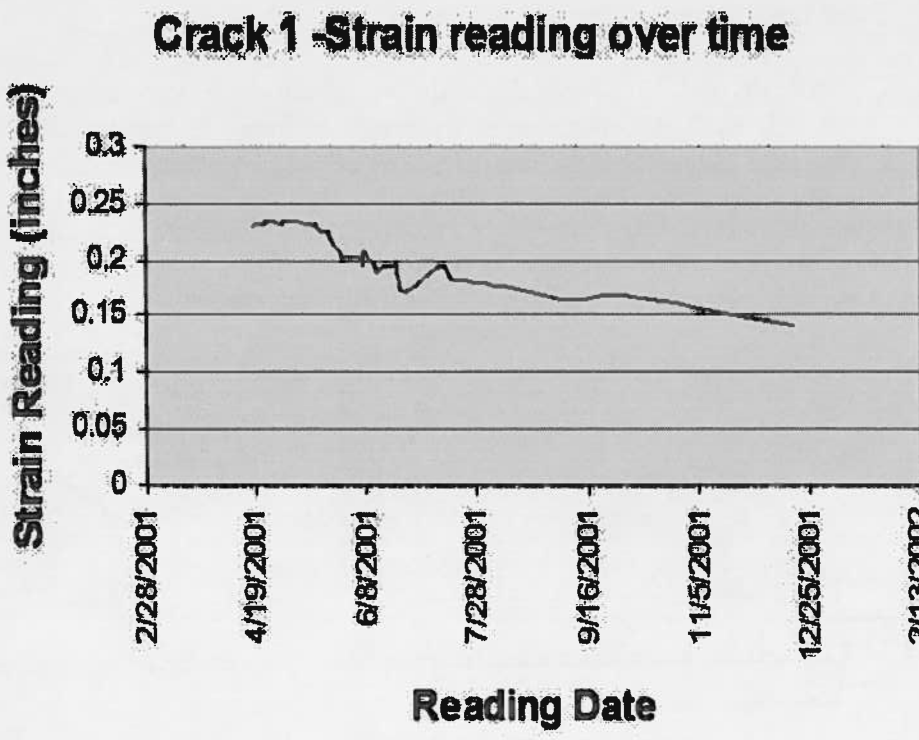

Figure 8: Crack Movement At The Location Of Crack 1

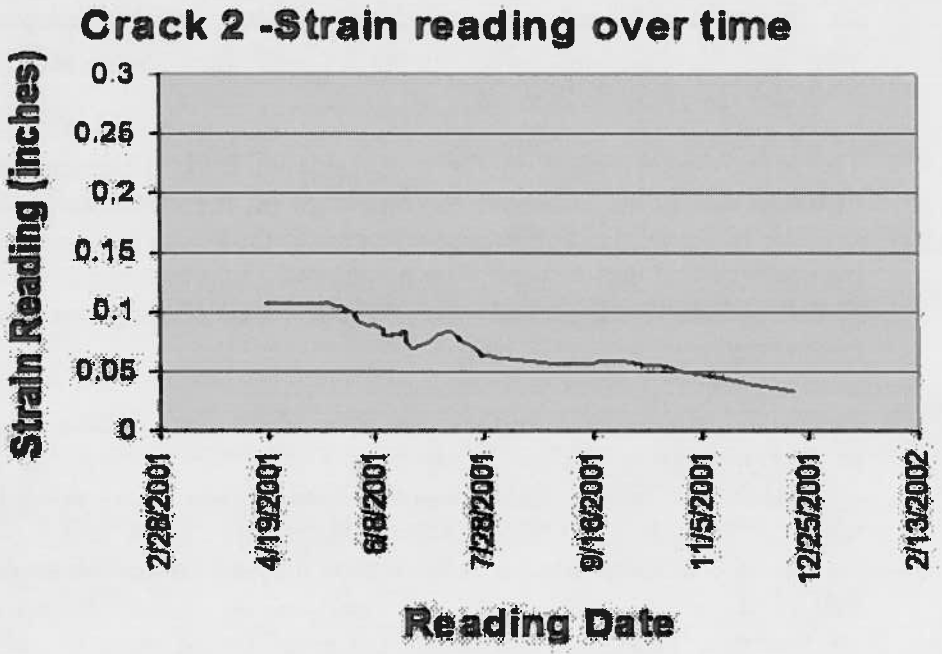

Figure 9: Crack Movement At The Location Of Crack 2

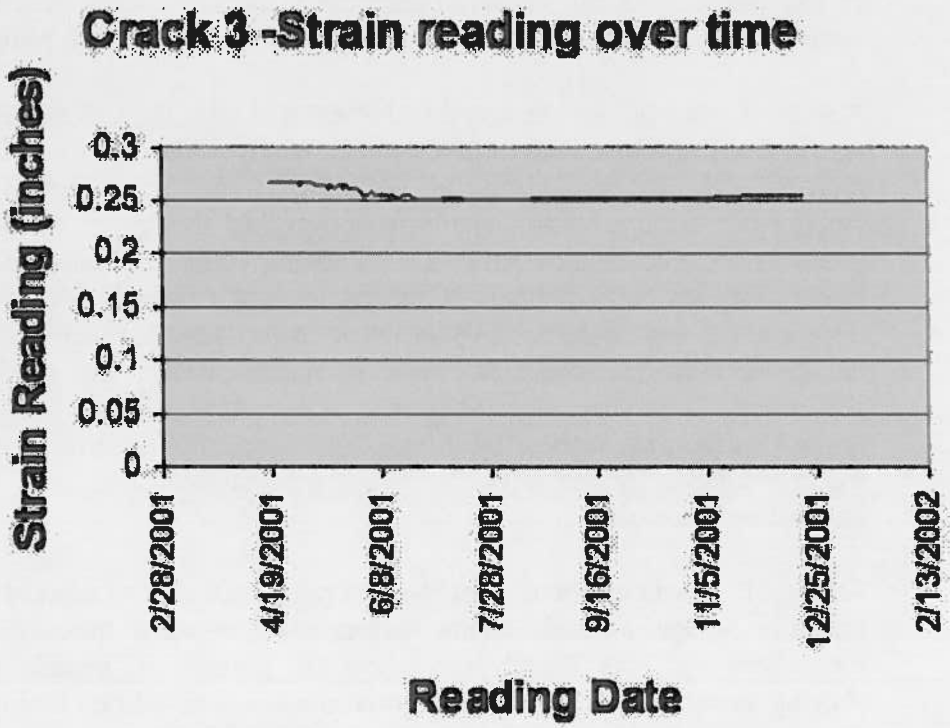

Figure 10: Crack Movement At The Location Of Crack 3

As illustrated in Figures 8 through 10 above, there was little displacement change observed at the three crack locations during or post construction of the underground portions of Press I.

3. The wood-frame house on the north side of Press I represented a somewhat different challenge. As with the buildings on the east side of the construction site, the soil qualities and properties beneath the house had to be assumed for the purposes of soil nail design. The house, including basement, was assumed to be founded directly on glacial till. The soil nail shoring system was designed accordingly.

   During the excavation activities, a portion of the north excavation sidewall began to slump, and material was lost from behind the shotcrete face of the upper approximately 15 feet of sloped-and-soil-nailed wall face. After first buttressing the slope with on-site soil to inhibit further slope failure and possible distress to the house, horizontal pot-hole exploration discovered the soil supporting the west half of the house was a much cleaner sand and gravel then was encountered elsewhere at the project site during the geotechnical study and site construction. It was concluded that further slope failure using the conventional soil nail system as already designed was likely as the sand and gravel soils would not stand vertical reliably for the soil nails to be adequately installed. Further, as a portion of the soil behind the already-constructed shoring system had already failed, continued slope failure would eventually undermine the house foundation.

   A solution was quickly designed and integrated with the soil nail shoring system. As further potholing determined that the sandy soils transitioned back into the more typical silty sand glacial till within a few feet of the house foundations, a tangent-pile earth retention system was designed to support the wedge of sandy soil directly beneath the house. This tangent-pile system consisted of a series of 6-inch vertical holes drilled 18 inches on-center, which had steel reinforcing installed and were subsequently grouted. Alternate piles were installed to allow the grout from the previous phase to harden before the second phase was performed. This supplemental shoring system was installed from the top of the ground surface adjacent to the house, and access by the drilling equipment was made available by the buttress of soil previously constructed to support the excavation sidewall.

   The result was a curtain of cement-grout piles sufficient to support the potentially unstable wedge of soil. This system was installed quickly, as it utilized equipment that was already onsite for the purpose of installing the soil nail shoring system. Further, the additional materials could be quickly mobilized to the site. Construction continued with only minor delays.

4. The City of Seattle required the project team to preserve an old Japanese Maple tree located adjacent to the northwest corner of the project site. This is not an uncommon situation when working in the urban environment today, as city

planners can be sensitive about preserving the existing aesthetics. However, this tree had not been depicted on the original civil drawings provided to the design team. Consequently, a redesign in this corner had to be performed quickly. Since the excavation sidewall directly adjoined this tree, the project team needed to discuss design and installation practices with an arborist to position soil nails so that neither the soil nails nor the soil nail installation procedures would impact the root system of the tree. The flexibility of the soil nail design process again proved itself, as this challenge was addressed with little overall cost impact to the project.

Press II Apartment Building

The soil nail design for Press II incorporated lessons learned during the design and construction of Press I. The characteristically urban, physical challenges were much the same as those encountered during Press I, and are summarized as follows:

1. In-place utilities beneath the city rights-of-way adjoining the site to the north and west;
2. The existing apartment building adjoining the site to the east;
3. The concrete retaining wall supporting a gravel parking area to the east;
4. The large Japanese Maple tree located in the southwest corner of the project site; and,
5. The soil nail shoring system left over from the construction of Press I.

The flexibility of soil nailing as a shoring process was demonstrated during Press II, as the shoring system could be quickly designed using the same design assumptions, soil parameters, and methodologies as were previously demonstrated to work for Press I. The challenges described above were addressed as follows:

1. In general, utilities along the north and west sides of the Press II excavation were designed in the same fashion as for Press I. Fortunately, none of the City of Seattle as-built records for these utilities indicated particularly deep burial depths, and so the soil nails were not required to extend over the utilities. The upper three vertical feet of the excavation sidewalls were slope-cut to a maximum steepness of 1.5H:1V to lower the first row of nails sufficiently to extend under the existing utilities. In addition, vertical elements (typically W8x31 or W6x9, Grade 50 steel beams) were installed, in order to temporarily support the soils beneath the city street rights-of-way. These measures permitted the first row of nails to be lowered deeper then the typical maximum 3-foot depth below top of excavation for soil nail design.

   As indicated previously, city engineering as-built records for utilities are typically more accurate in terms of horizontal alignment then vertical depth. However, it was found with Press II that this consideration is not always reliable. One water utility had been accounted for in the original soil nail design.

However, a second water line (not shown on the as-built drawings) extended the length of the wall face, and was located much closer to the wall face then the first (though at the same burial depth).

Consequently, the project team had to swiftly re-design the east shoring system to account for this problem. The soil nail design process demonstrated its flexibility, as individual nail locations and declinations could be adjusted to miss the second water utility by a margin acceptable to the City of Seattle engineering department without impacting the overall design of the wall and the project schedule.

2. The three-story apartment building adjoining the excavation to the east was not as settlement-sensitive as were the existing buildings further south that had adjoined the Press I excavation. It was still necessary, however, to design a shoring system that would support this building both vertically and laterally. Additionally, the first row of soil nails had to be located such that they extended below the footing of the apartment building. Furthermore, there was an existing grade change on the order of 8.5 feet between the ground surface at the existing apartment building and the ground surface for Press II. This grade change was supported by an existing concrete retaining wall aligned along the east property line.

   That concrete retaining wall provided an alternative solution to the project team in place of the VENIS® system. The concrete retaining wall was "nailed" in place using a row of soil nails designed to support the retaining wall as well as support the lateral loads imposed on the shoring wall by the adjacent apartment building. The concrete retaining wall then served in place of a temporary shotcrete facing. Once Press II was completed, a new concrete wall was poured against the older retaining wall to cover the soil nail heads and hardware. Figure 11 illustrates the older retaining wall and the new concrete wall.

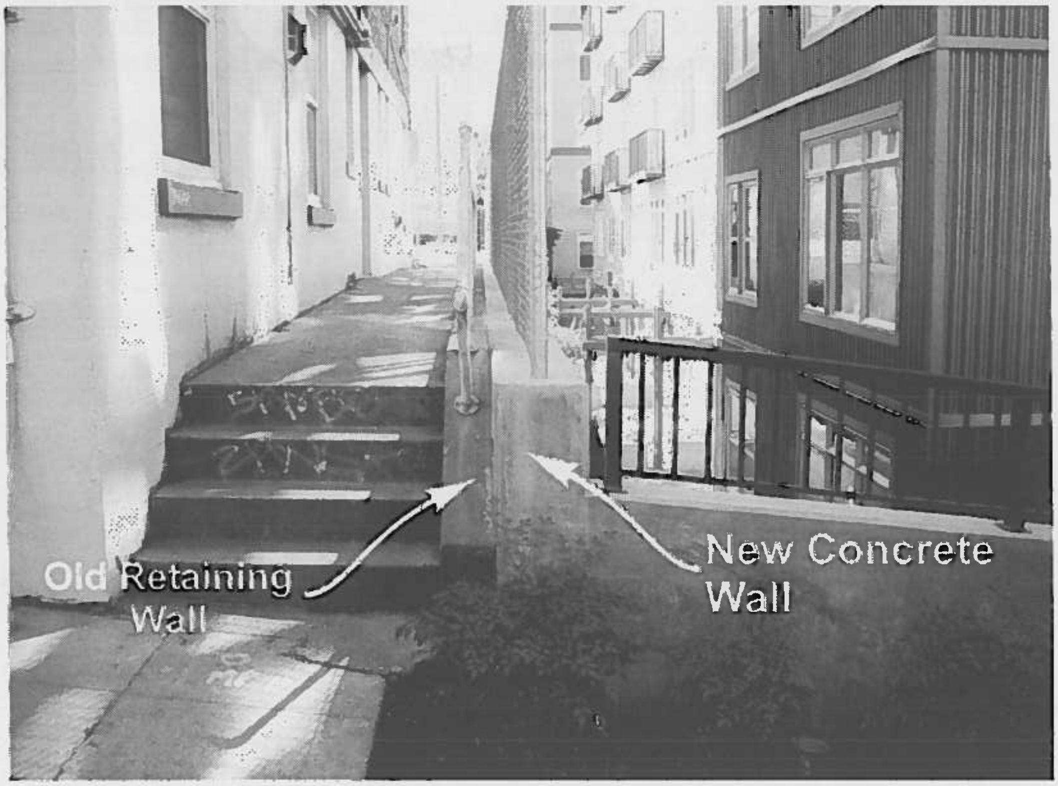

Figure 11: Older Concrete Retaining Wall With The New Concrete Wall Adjacent To The Completed Press II Apartment Building.

Below the level of the concrete retaining wall, a conventional soil nail shoring system was used. This flexible use of soil nail designed allowed the design team to not only take full advantage of the cost-effectiveness of soil nail design, but also saved additional costs on facing materials and labor for the upper 8 feet of wall surface. The only additional work element required was equipment and time to core the concrete retaining wall at each soil nail location.

3. The concrete retaining wall described previously continued south and supported an existing gravel-surfaced parking lot. Approximately 10 vertical feet of soil was retained behind this wall above the ground level elevation of Press II, and a geotechnical boring performed in the parking lot just behind the retaining wall, indicated that the in-situ compaction of the retained fill soils was very poor. A cut slope behind the existing concrete retaining wall was not feasible as the parking lot had to remain.

   The project team reused this portion of the concrete retaining wall in the same manner as the portion between Press II and the existing apartment building. The first two rows of soil nails were installed through the concrete retaining wall, and designed to support the wall as well as the retained fill soils. The concrete retaining wall was used as the temporary facing element for the shoring, in place of a conventional temporary shotcrete facing. As with the length of retaining wall adjacent to the existing apartment building, a new concrete wall was poured against the older retaining wall to cover the soil nail heads and hardware.

As before, the ability of soil nailing to integrate with existing site features is a significant advantage over other shoring systems, particularly in an urban environment where such features are commonly encountered along property lines.

4. Preservation of the Japanese Maple was an integral component of the shoring design for Press II, just as it was for Press I. Since the tree was actually located in the property, the shoring wall had to be "jogged" to the north to provide a vertical wedge of soil sufficient to both support and maintain the tree. This wedge had to be supported with the shoring system and, since it was a concave curve in the wall from north to west, splayed soil nails had to be used for support. These splayed nails had to, again, be located with the assistance of an arborist to preserve the tree's root structure. Because of the sensitive nature of maintaining that wedge of soil, vertical elements were installed along the wall face as mentioned previously. The splayed soil nails and vertical elements used in the vicinity of the tree are illustrated in Figure 12.

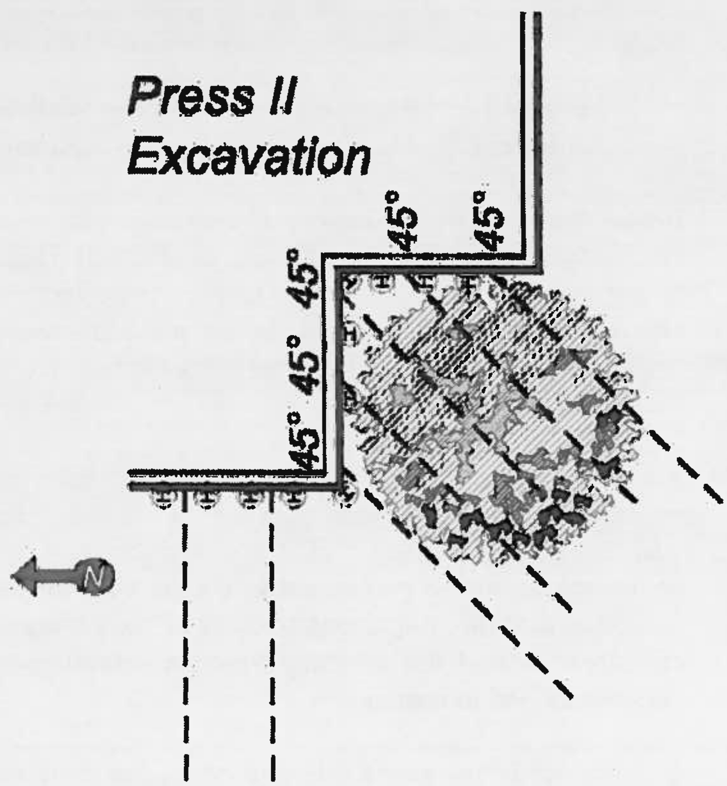

Figure 12: Press II Shoring System In The Vicinity Of The Japanese Maple Tree

Approximately three years have transpired since the first soil nail system was installed and one year since the second was installed. As of the publication of this paper, the tree is alive and healthy (See Figure 13), and continues to provide the natural element to Belmont Avenue that the City of Seattle desired.

Figure 13: The Japanese Maple Tree At The Time of Publication. The Press I Apartment Building Can Be Seen Behind The Tree.

5. As mentioned previously, the south subsurface wall of Press II was located very close to the existing north subsurface wall of Press I. The distance was too small for shoring, but the design team elected to use the Press I subsurface wall as the "face" of the south excavation. This demonstrated another strength of the soil nail method as a temporary earth retention system: it was relatively easy to demolish and remove the soil nails and grout from the Press I system as part of the excavation work for Press II.

FUTURE PRESS APARTMENTS

As of the publication of this paper, preliminary plans for the last two phases of the Press Apartment development (Press III and Press IV) have not been developed. Assuming the apartment buildings are similar to Press I and Press II, including one to two levels of underground parking, the challenges anticipated for design and construction of temporary soil nail shoring systems for Press III and Press IV should be fairly similar to those previously used. Specific challenges to these future shoring systems, based on observation of the current on-site developments include:

1. Right-of-way utilities are anticipated under the street and sidewalks adjoining the north, east, and south sides of the half of the city block not yet developed as part of the Press Apartment development project.

2. Existing buildings will need to be underpinned and laterally supported during construction activities for Press III and Press IV. At some stage, this will include the settlement-sensitive buildings that adjoined Press I.

3. Trees located on the city sidewalks that adjoin the site will need to be maintained during the construction of subsequent soil nail shoring systems.

4. Considerations during subsequent Press Apartment development phases will need to be given towards the demolition and removal of previous temporary shoring systems used during the construction of previous Press Apartment phases.

5. If City of Seattle underground parking requirements change such that additional levels of underground parking are required for future phases of the Press Apartment development project, it will be necessary to underpin and laterally support the Press I and Press II apartment buildings.

CONCLUSIONS

Soil nailing remains a cost-effective, flexible, and efficient means of supporting temporary excavation cut faces in areas where the subgrade soil and groundwater conditions permit its use. In an urban construction environment, where both anticipated and unexpected physical challenges can be encountered, soil nailing demonstrates its worth as an effective shoring system. As temporary excavations get deeper and the area of construction is pushed as close to the property lines as possible, soil nailing may be one of the few shoring methods that will retain effectiveness in the urban environment.

However, there are a few very characteristic challenges to using any shoring method in an urban environment, where the developments that exist beyond the footprint of the excavation have a significant influence and impact on both design and construction. Soil nailing provides project teams with a flexible system for addressing these challenges, including "hybrid" shoring designs, underpinning, and the mixing soil nail shoring systems with supplemental design elements like slope cuts and vertical elements. Project teams should always include the following specific elements in their projects when using soil nailing, in order to maximize the cost-effectiveness and flexibility of this method of shoring:

1. Care and diligence should be applied in gathering as much information on the surrounding utilities, structures, and other site features as can be obtained;

2. It should be expected that the researched information is not always accurate and plan accordingly;

3. Good communication should be maintained between the soil nail designer, the project team, the contractors (particularly the soil nail shoring contractor), and the city engineering and transportation departments in order to address sudden problems quickly and satisfactorily.

Ron Ebelhar, H. C. Nutting | A Terracon Company and Mike Ricke, Anchor Properties

Erskine Commons–Successful Urban Development on a Closed Landfill Site

Abstract:

This paper discusses a case study of the successful development of a closed landfill site in an urban setting. The Fritterling Landfill was a 20-acre construction and demolition debris / foundry sand landfill site that operated from 1969 to 1982 on the south side of South Bend, Indiana. The site development originated as sand and gravel borrow area for construction of a highway interchange adjacent to the property. Landfilling of foundry sand and related materials started shortly thereafter. It was closed in 1982 and an engineered final cover was never provided. The landfill did not have a liner, leachate management collection system or any landfill gas control systems. Anchor Properties (the Developer) purchased 56 acres including the landfill for redevelopment, which included a Wal-Mart Supercenter, Lowe's Home Improvement Store, an additional anchor and seven outlots for various retail uses.

The Developer put together a team of engineers and scientists to evaluate the most efficient approach to developing the property. H. C. Nutting (a Terracon Company) was retained to perform geotechnical/geo-environmental consulting services for the site and to provide recommendations for development of the site to avoid potential adverse effects such as short- and long-term settlement, landfill gas, and impacts on groundwater. From these recommendations and a series of meetings with the civil designers and construction managers, the site improvements were addressed in preparation of construction plans and specifications to implement that work.

Those improvements included, but were not limited to:

- Locate building pads outside waste areas or undercut and replace with select fill in the limited area where encroachment was unavoidable
- Evaluate sources of borrow under shallow areas of landfill
- Dynamic compaction of landfill waste area - locate parking areas and roadways in dynamic-compacted landfill areas
- Install passive landfill gas vents in building walls adjacent to landfill areas, and
- Install slurry cutoff wall to prevent infiltration from detention pond into landfill areas

Site development work required extensive construction quality assurance during all phases. The site has been successfully developed and the infrastructure is performing at or above expectations since 2006.

Erskine Commons–Successful Urban Development on a Closed Landfill Site

Ron Ebelhar, H. C. Nutting | A Terracon Company and Mike Ricke, Anchor Properties

Introduction

The Fritterling Landfill was a 20-acre construction and demolition debris / foundry sand landfill site that operated from 1969 to 1982 on the south side of South Bend, Indiana. The site development originated as sand and gravel borrow area for construction of a highway interchange adjacent to the property. Landfilling of foundry sand and related materials started shortly thereafter. It was closed in 1982 and an engineered final cover was never provided. The landfill did not have a liner, leachate management collection system or any landfill gas c
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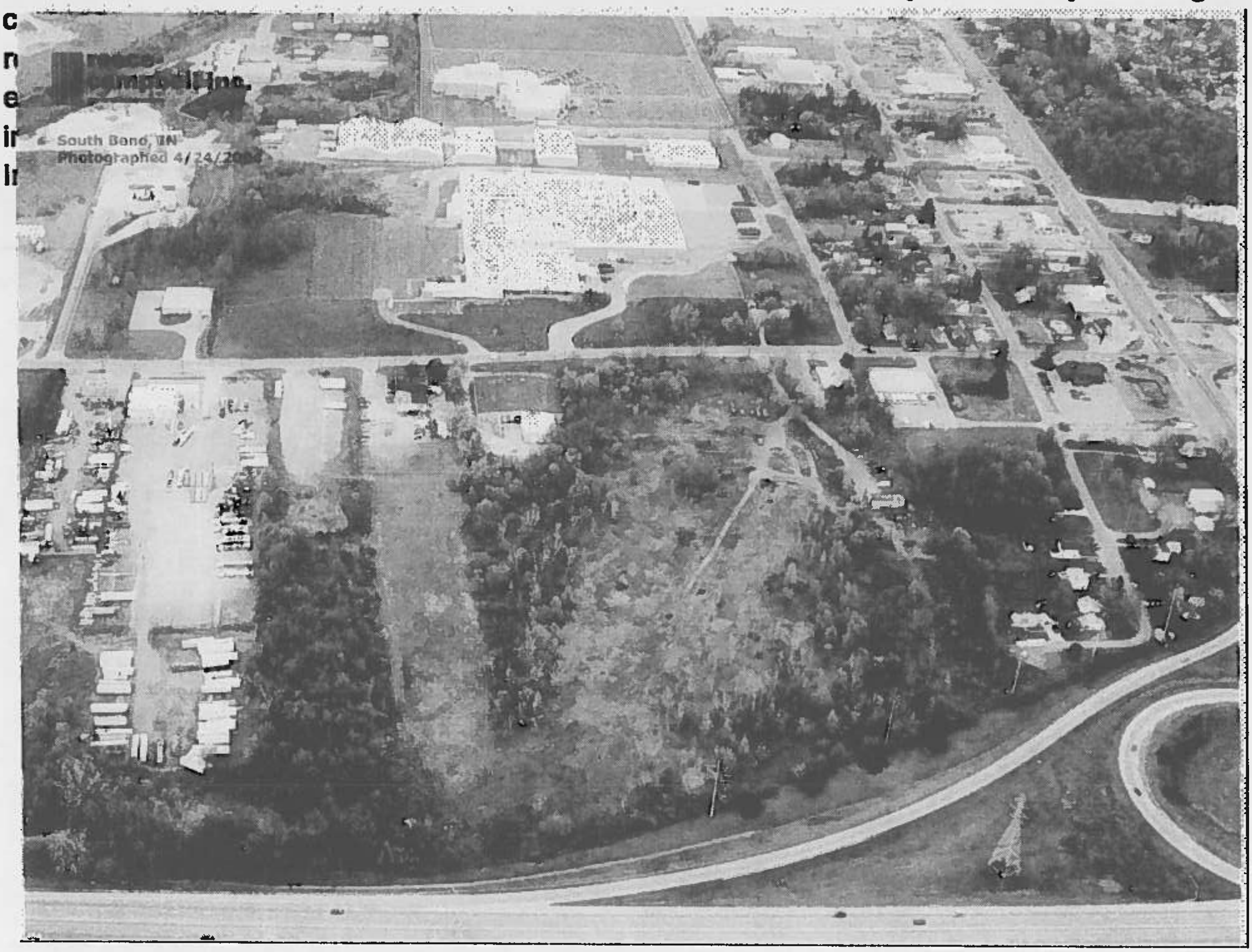

Figure 1. Pre-Development Aerial Photograph, looking north

The Developer put together a team of engineers and scientists to evaluate the most efficient approach to developing the property, recognizing the impact of the landfill and two major utility lines crossing the site. H. C. Nutting (a Terracon Company) was retained to perform geotechnical/geo-environmental consulting services for the site and to provide recommendations for development of the site to avoid potential adverse effects such as short- and long-term settlement, landfill gas, and impacts on groundwater. From these recommendations and a series of meetings with the developer, civil designers and construction managers, the site improvements were addressed in preparation of construction plans and specifications to implement that work.

From the developer's standpoint, this project could not proceed, unless the environmental liability was limited by the Indiana Department of Environmental Management (IDEM) and the project made economic sense. In addition, the retailers had their own requirements that needed to be anticipated in the final schemes. H.C. Nutting assisted the developer and design team analyzing alternative development approaches that satisfied all of those criteria.

The analysis stage of the project was also complicated by the fact that the developer did not have the retailers formally committed to the project. Therefore, the level of analysis and the associated costs were continually evaluated.

Geotechnical and geo-environmental work requirements were fairly extensive due to the presence of the landfill and other non-related environmental issues. Prior to Anchor's involvement, some preliminary geotechnical work had been done to evaluate the subsurface conditions. Due, at least in part, to the presence of the deep waste fills, that development never proceeded. The geotechnical exploration program was developed to use as much of the pre-existing data as possible. Three of the proposed retailers (end users) had fairly comprehensive requirements for geotechnical engineering reports. The work plans on each of those parcels was designed to meet their minimum requirements and the specific needs for the overall site. A portion of the site is shown in Figure 2, which illustrates the distribution of soil borings.

Development Considerations

According to the original site plan, the project included the construction of three large retail stores (Kohl's, Wal-Mart and Lowe's) along the south side of West Ireland Road, west of U.S. Highway 31 and north of U.S. Highway 20 in the southern part of South Bend, Indiana. For the remainder of this paper, each of the three parcels will be denoted as Areas K, W, and L representing Kohl's, Wal-Mart, and Lowe's stores respectively. Kohl's later dropped out of the project, taking space in another location, at the request of the South Bend city government.

The Area K building was planned to be an approximate 88,250-sq. ft. facility with maximum plan dimensions on the order of 400 ft. in a general north-south direction by 250 ft. in the east-west orientation. The proposed building location was in the northwest corner of the overall project site. The finished floor elevation for the store was established at El. 806 ft. An asphalt-paved parking area consisting of about 4.5 acres (550 spaces) was planned along the east side of the building. This parking area measured approximately 440 ft. by 440 ft. in plan dimensions. A new entrance drive was to be constructed on the northwest corner of this parking area.

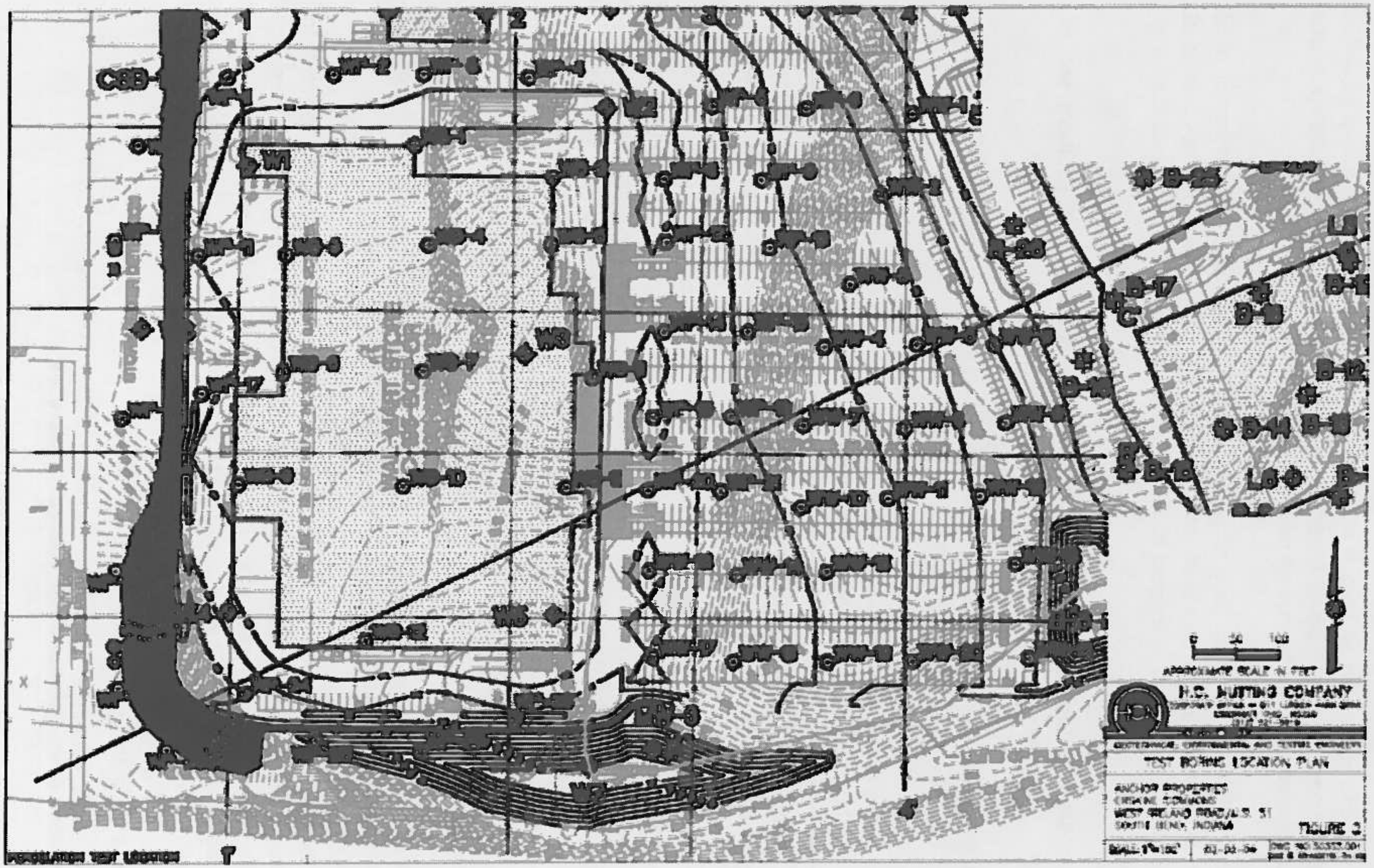

Figure 2. Test Boring Location Plan – Southwest Quadrant of Site

The Area W building covered an area of about 203,000 sq. ft. with a finished floor elevation set at 806 ft. and a parking area of about 7.5 acres (1044 spaces). It is located to the south of Area K, in the southwest corner of the overall project area.

The Area L building covered an area of about 116,000 sq. ft. plus a garden center of about 31,000 sq. ft., a finished floor elevation set at about 819 ft, and a parking area covering about 4.75 acres (619 spaces). It is located to the east of Area W, in the southeast corner of the overall project site.

Stormwater detention areas were planned for the west boundary and southern end of the project area. These were generally the lowest areas of the site topographically.

Other features of the proposed site development include seven (7) out lots ranging from 1 to 1.75 acres located north and east of Area L.

In general, the northern portion of the site (frontage along West Ireland) was previously developed commercial property characterized by nearly level to gradually sloping topography. The old Fritterling Landfill with frontage on West Ireland within the central third of the site had deep fills of construction debris and foundry sand, and extended southward to the U.S. 20 interchange. A gas pipeline ran east-west through the site about 350 ft south of West Ireland Road. In addition an overhead high voltage electric line ran north-south, through the site. Existing grades within the site ranged from Elevation 778 ft near the southwest corner to Elevation 837 ft near the central eastern border. Consequently, cuts on the order of 20 ft. in the east portion and fills up to approximately 25 ft. were required to achieve the final grades within the west portion of the site. Figure 3 shows a cross-section through a portion of the site. This figure illustrates the pre-existing and proposed grades and the soil / fill stratigraphy encountered at the site.

Design Considerations

After over a year of analysis and evaluation of alternative development schemes the developer, H.C. Nutting and the design team created the following design considerations:

- Locate building pads outside waste areas or undercut and replace with select fill in the limited area where encroachment was unavoidable. Due to the relatively central location of the landfill in the overall parcel, it was generally possible to locate building structures around the perimeter of the old landfill. The building pad at Area L (Lowe's) was undercut to remove all waste under the building footprint to limit differential settlement.

- Relocation of the gas pipeline

- Relocation of the overhead high voltage electric lines

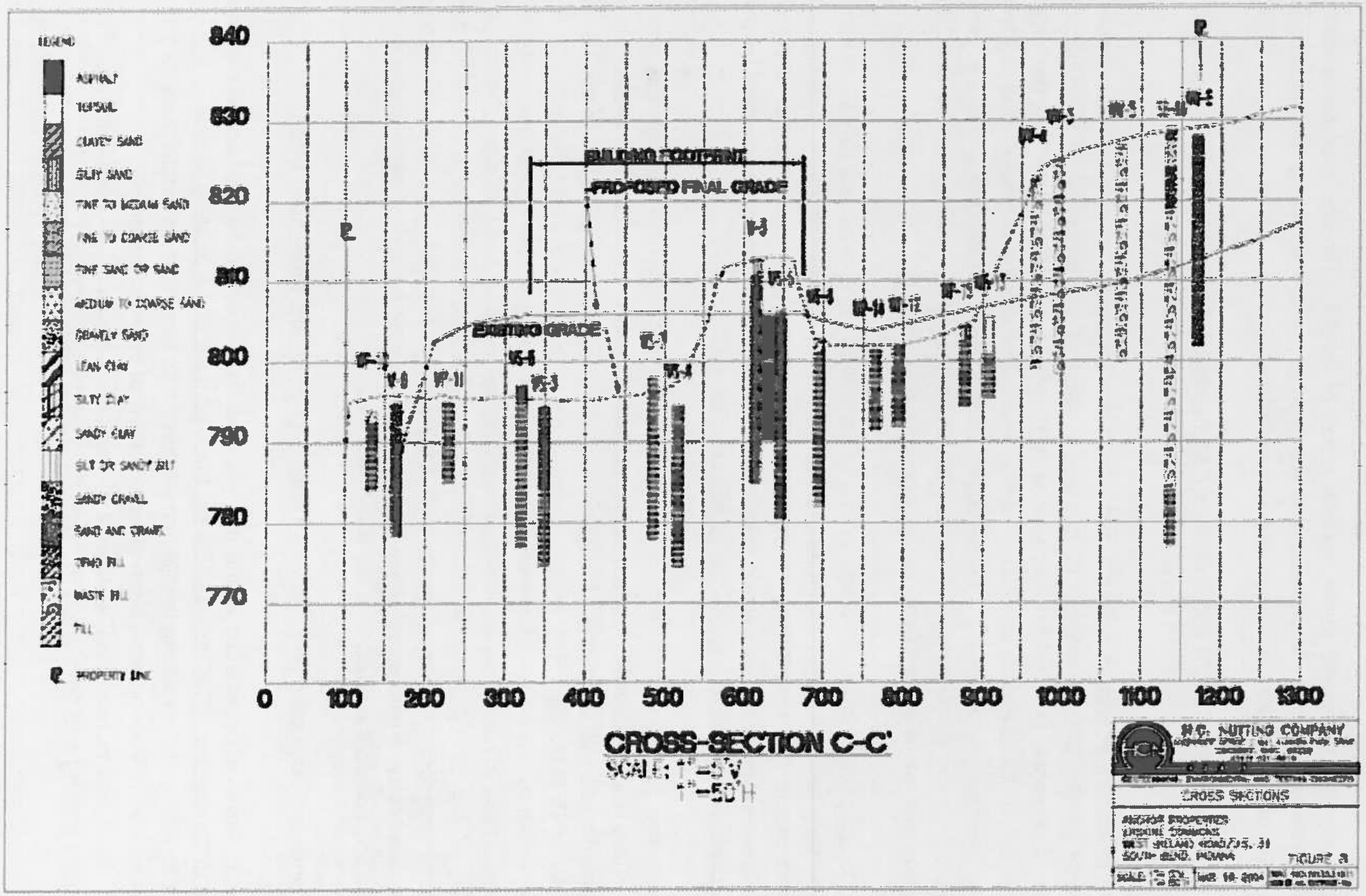

Figure 3. Example Cross-Section of Site

- Evaluate sources of borrow under shallow areas of landfill. On-site sources of clean structural fill were desired. There was also the need to provide additional landfill storage capacity to take materials from areas requiring undercut.

- Estimate quantities of landfill soil and "clean" fill necessary to balance the site earthwork and avoid hauling soils in, or off the site

- On-site shredding of tires disposed in old landfill and transportation offsite. Discussions with local people with knowledge of the site advised of the presence of a significant volume of disposed tires. After discussions with the team, it was found that the most economical / politically correct solution was to bring in a mobile shredder, to shred the existing tires and to dispose of the shredded tires offsite. Other onsite re-uses were considered for the tire shreds, i.e., as leachate collection media but for various reasons this option was not used for this site.

- Dynamic compaction of landfill waste area. The relatively large area and central location of the existing landfill required that it be used as part of the site infrastructure. The highest and best use for the landfill was determined to be parking and access roads. Several methods were discussed but it was quickly decided that dynamic compaction would be the most economic method to improve the landfill surface.

  It was known that dynamic compaction would not likely be effective through the entire depth of the landfill material. It has been found from measurements on other project sites that densification with conventional dynamic compaction equipment will occur to depths on the order of 15 to 20 feet. The ground improvement is not uniform within the entire 20 ft.; the upper zones will show the greatest improvement with gradual reduction with depth. The upper zone of landfill material was generally in the loosest condition since the deeper portions of the landfill material became partially consolidated due to the overburden surcharge pressures. An approximate 15-ft.-thick zone of densified material was planned to act as a mat with enhanced stiffness to make the upper portion of the material more uniform and spread the loads from the pavement areas.

  The Dynamic Compaction program was initially broken up into three main areas:

  Area 1: Areas of the existing landfill that will have a minimum of 10 ft. of landfill material prior to compaction. This included the southern portions of the existing landfill.
  Area 2: Areas of the existing landfill that will have less than 10 ft. of landfill material prior to compaction. This included the northern portions of the existing landfill.
  Area 3: Areas of new landfill material placement in structural fill borrow areas. This included parking and drive areas in Areas K, W, and some of Area L.

Due to various project changes, this was reduced to two areas, essentially combining Areas 1 and 3. The two main areas of work and undercut areas are shown on Figure 4. After trial drops at several locations, the densification program was selected to be a heavier-than-normal ironing pass consisting of 3 drops of the 15-ton tamper from a height of 30 ft. on an 8 ft. center-to-center grid.

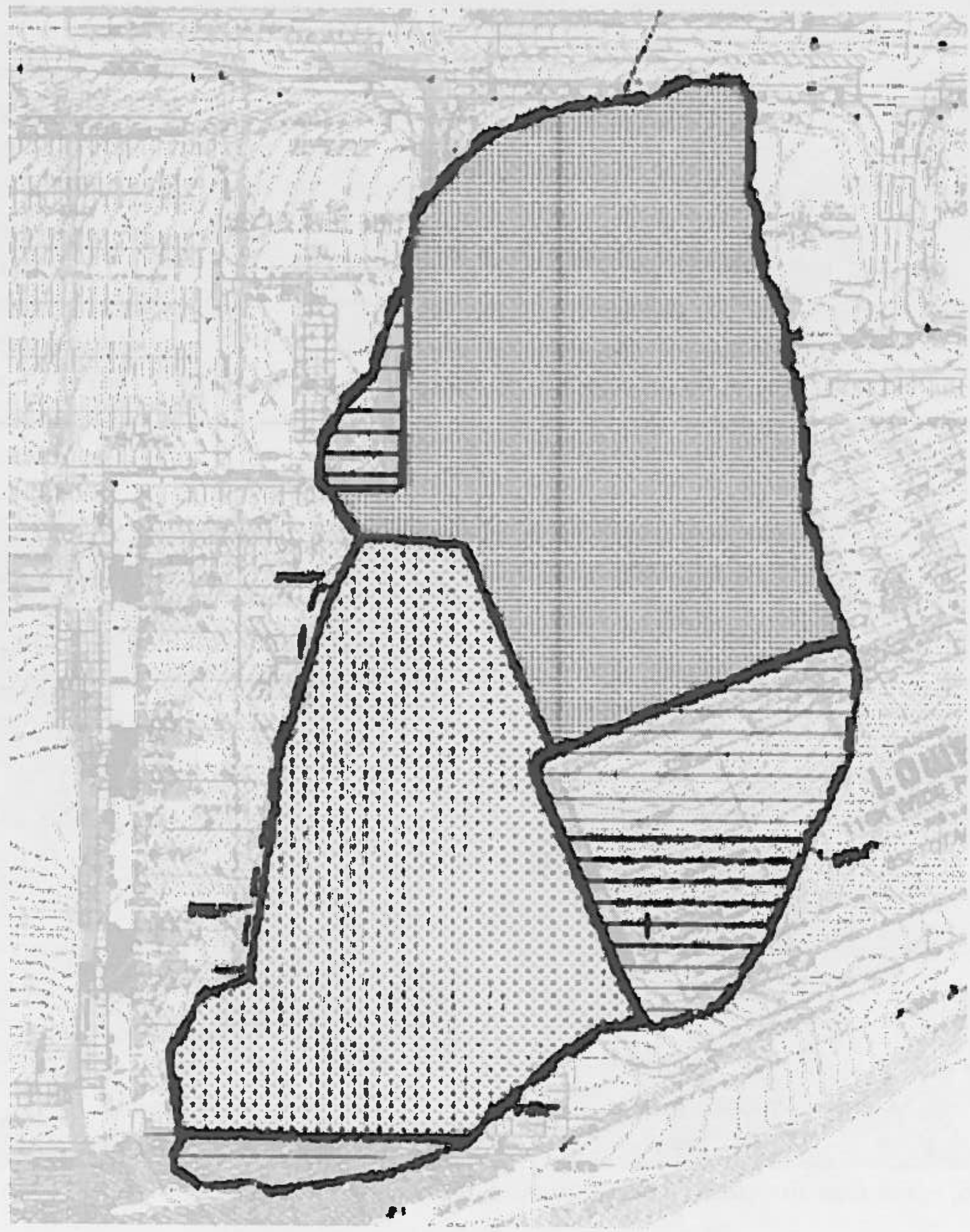

Figure 4. Dynamic Compaction Areas

- Locate parking areas and roadways in dynamic-compacted landfill areas. Due to the uncertainty of fill depths, variety of waste fill types, etc., the team decided that the landfill surface area should be improved to adequately carry traffic and parking loads. In some cases due to scheduling conflicts, waste fill areas under proposed parking lots did not receive dynamic compaction. These areas were further stabilized using a recycled stone layer enclosed in a high-strength woven geotextile (Mirafi HP 570). Traffic control with the dump trucks and the spreader-loader was critical during construction to avoid turning after dumping to avoid shearing and separation at panel intersections.

- Landfill liner / leachate collection system design/installation. The need for structural fill to level the building pad areas of the site required that we consider obtaining soil borrow material from under the shallow areas of the old landfill. This created capacity to place waste material generated by undercuts in other areas of the site, including but not limited to the Lowe's building pad and detention basins. Since this was considered a vertical expansion of the landfill, IDEM required that these excavated areas be lined before waste materials were replaced in these areas. Due to the lack of clay material, a geosynthetic clay liner was used to line the excavated landfill cells. On-site sand was used as the leachate collection drainage layer. A leachate riser and submersible pump system was installed to remove leachate from the cell with discharge to the sanitary sewer system. Figure 5 shows a photograph about mid-way through the project development. The eastern extension of the partially-lined landfill cell is seen on the left-hand side of the photograph.

Figure 5. Site Development Activities About Mid-way through Project – looking south

- Cement stabilization of building pad threatened by extended rain forecast. A tight deadline on building pad delivery and impending rain required that the building pad be stabilized to avoid impacts to its integrity. Due to the granular, cohesionless nature of the subgrade, we elected to use cement stabilization to save the pad. The subgrade was stabilized to a nominal depth of about 12 inches, using a nominal cement mix of about 5 percent by weight mixed from the surface using rotary soil mixers.

- Install passive landfill gas vents in building walls adjacent to landfill areas. Any building pad located within 50 feet of the edge of waste fill material was designed to have a passive venting trench to intercept vapors and reduce the potential for migration into the building itself. Methane bulkheads were also installed at utility entrances into building pads.

- Install slurry cutoff wall to prevent infiltration from detention pond into landfill areas. Excavation of the southern detention pond encountered a strip of landfill that had not been observed in the soil borings. Since the detention ponds were anticipated to both detain and allow infiltration, concerns over the infiltration coming into contact with waste material and creating leachate led to a requirement for a slurry cutoff wall. The wall was designed to separate the waste material from the detention pond infiltrate. A one-pass trenching/slurry installation system was used to install the cutoff wall.

- Field Permeability (percolation) tests to check infiltration rates for detention pond areas

Figure 6. Site Development Activities Substantially Complete, looking south

Figure 6 shows the project at substantial completion with the two anchor stores completed and in operation. The detention ponds are apparent on the south and west ends of the site.

Closing

All work was performed to comply with the Indiana Department of Environmental Management requirements.

The site development work was very complicated. As earthwork proceeded, continual reevaluation of design parameters was necessary, due to actual quantities and types of landfill soils, weather and environmental quality issues. Site development work required extensive construction quality assurance during all phases. The site has been successfully developed and the infrastructure is performing at or above expectations since 2006.

In spite of the initial challenges, site constraints, surprises during the construction phases, etc., the Developer is ready to consider and develop the next landfill site that offers similar return on the investment.

Acknowledgements

The authors would like acknowledge the support of the project team, including, but not limited to, Tim Greive (TGA), Lance Turley (Hull, Inc.), Dick Dierkes and Steve Ripberger (Reece Campbell), Joe Kruger (HCN) and Matt Grever (Anchor Properties), without whose patience and teamwork this project could not have been done.

LANDSLIDE IN AN URBAN ENVRIONMENT

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LANDSLIDE IN AN URBAN ENVIRONMENT

By: Timothy D. Stark and Erik J. Newman

ABSTRACT: The causation of distress in two housing developments located downslope of a large development is discussed. The investigation shows that distress in the housing developments was caused by a large, deep bedrock landslide triggered by upslope fill placement. A large fill was placed to create a visual barrier between the existing housing developments and the large upslope structure and to balance the cut and fill quantities of the upslope development. This case history illustrates some of the ramifications of fill placement on natural slopes surrounded by urban areas such as, overstressing underlying weak bedrock material that may exist below the depth of subsurface investigations that are typically conducted for single family residences, the importance of surface and subsurface information in complicated geologic settings, and the effect of natural and man-made changes to a slope, such as rainfall, surficial grading, home construction, and fill placement, on slope stability. This case history also illustrates the importance of locating the critical cross-section before construction and designing the slope to ensure that this cross-section remains stable, the proper use of back-analyses in landslide investigations, the use of the critical cross-section in back-analyses, and the importance of installing a number of slope inclinometers shortly after distress is reported.

Keywords: Soil Mechanics, Landslides, Clays, Shear strength, Slope stability, Subsurface Investigation.

Introduction

Cut and fill operations are routinely required to facilitate hillside development. Because these operations can affect the stability of the hillside on which they are imposed, the design process should address the potential impact of these operations on the surrounding landscape and developments. This involves considering the impact of hillside development on the structures upslope and downslope of the proposed development because frequently the site investigation only considers the impact of the cut and/or fill on the particular project site.

A factor complicating hillside development is the usually significant cost of disposing of excess cut or excavated material from the project site. Environmental regulations usually make disposal of large amounts of cut material at an offsite location expensive. As a result, there is usually a significant cost incentive to "balance the site", which requires balancing the amount of cut material and the amount of fill material for the hillside development. If the site is "balanced" no fill would need to be imported or exported from the site.

The goal of balancing a site can lead to placement of a large amount of fill at a single location on a natural slope as occurred in this case. The details of the large fill and some surficial grading that occurred at the top and bottom of the slope, respectively, in this case history are presented herein. This case history highlights the need for adequate subsurface investigation and stability analyses to assess the stability of a natural hillside in an urban subjected to a large fill and surficial grading. The case history also illustrates the responsibility of an engineer in foreseeing the magnitude of future upslope development to guide the design of downslope developments. This foreseeability requirement can impact the conservatism that an engineer should adopt for the downslope development.

Landslide Chronology

Between 1988 and 1989, a housing development with about 50 units was completed on an undeveloped hillside near Novato, California and is referred herein as the Knolls. Novato, California is located about 30 miles north of the Golden Gate Bridge in San Francisco. An 11 unit housing development was constructed upslope of the Knolls and is referred to herein as the Vista. Only 7 of the 11 Vista lots were developed at the time of the 1996 landslide. Figure 1 presents an aerial view of these housing developments, the subsequent upslope development referred to as the BC Development, and an outline of the slide mass. Only a portion of the housing units in the Knolls and Vista development are shown in Figure 1.

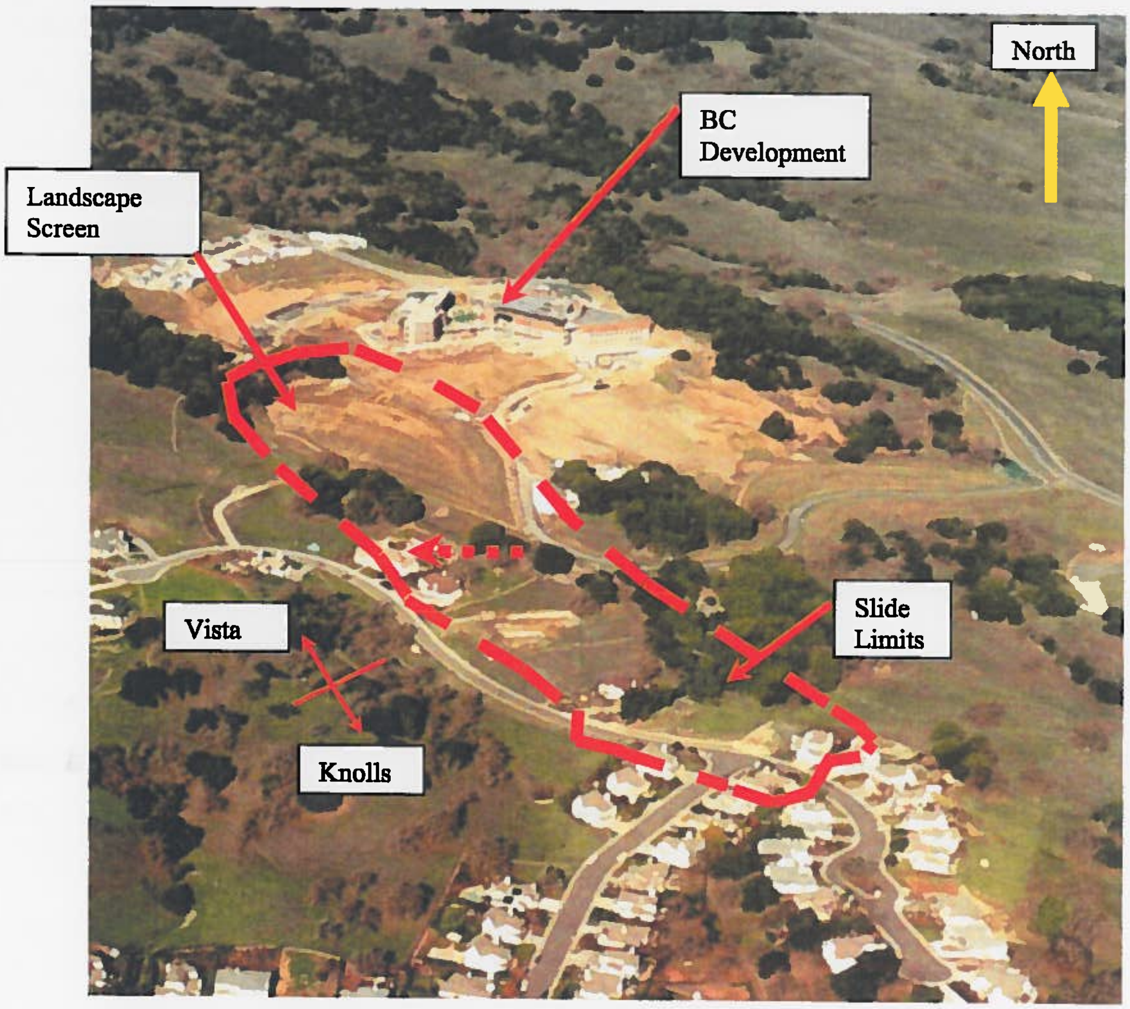

Fig. 1. Aerial view of housing developments, BC Development, and an outline of the slide mass

The Vista and Knolls housing developments did not experience any complaints of distress until September 1996. In September, 1996, the O'Rourke residence (see dotted arrow in Figure 1) experienced significant drywall cracking in the kitchen. In December, 1996, homeowners along the western edge of the slide mass in the Knolls development started experiencing distress in December, 1996. In January, 1997, homeowners along the toe of the slide mass in the Knolls started experiencing distress and damage. This damage chronology is significant because it suggests that slide movement occurred from the top of slope to the bottom of the slope instead of from bottom of the slope to the top. However, the upslope development would claim that the slide was caused by excavation at the toe of the slope and the slide progressed upslope to the BC development.

Shortly after January, 1997, homeowners mobilized a lawsuit against the BC Development. The BC Development subsequently sued the Knolls developer on the theory that the surficial grading performed for the Knolls development removed toe support and allowed a lower landslide to develop which triggered an upper slide that undermined the BC development. This undermining resulted in a large fill created for the BC development undergoing downslope movement.

An important event in the slide chronology is the placement of a large fill by the BC development just above the Vista development in June 1996. The reported purpose of the large fill was to create a visual barrier between the BC development and the Vista development. Another purpose, albeit possibly an indirect purpose, of the large fill was to balance the cut and fill quantities of the BC site. As a result, the large fill is referred as a landscape screen herein. Fill placement for the landscape screen ceased in late December 1996 with the onset of homeowner complaints even though the fill had not reached full height. The final height of the landscape screen is not known and it may have been related to the amount of cut material that had to be disposed. Figure 1 shows the landscape screen at the upslope end of the limits of the landslide. The surface area of the landscape screen is approximately 61,000 square meters and the estimated volume of the landscape screen is 76,600 cubic meters. The estimated volume of the landslide mass is 2.0 million cubic meters.

In summary, there was no significant change in slope geometry after completion of the Knolls housing development and the construction to date in the Vista development until the BC development commenced in late 1995. In particular, there was no significant fill placement related to the BC Development until June 1996 with the start of the landscape screen.

Landslide Factors

In general, a critical combination of the following three factors are required to initiate a landslide: (1) a weak layer underlying the site, (2) subsurface water, and (3) some driving force.

When a critical combination of these three factors coalesces at a site, a landslide can, and probably will, occur.

Weak Layer

The site, on the east side of Mount Burdell, exhibits complex soil and bedrock conditions. The soils overlying the bedrock involve many surficial landslides and thus there are a number of colluvial scarps and colluvial soil deposits in the housing developments and the BC Development site. The colluvial deposits consist of unconsolidated clay, silt, sand, and some gravel derived from weathering of the underlying bedrock materials that have been transported by downslope movement. The colluvial slides have occurred and are occurring above the bedrock and thus are independent of the 1996-1997 slide movement which was observed to occur in the underlying bedrock based on slope inclinometer data.

The predominant bedrock units underlying the near surface soils are the Tertiary Volcanics and the Franciscan Complex. The volcanic rocks overlie the Franciscan Complex at the upper portion of the Vista development shown in Figure 1. The volcanic rocks generally consist of hard andesitic rocks and a weaker agglomerate of ash and block flow rocks. Slope inclinometer data show that the landsliding occurred below the volcanic rocks and thus in the Franciscan Complex. The Jurassic-Cretaceous rocks of the Franciscan Complex include sandstones, claystones, mudstones, shale, conglomerates, and serpentinite. The Franciscan Complex is frequently referred as a mélange, or mixture, because the deposit was formed near the forward edge of a subduction plate boundary (Goodman 1993). The California Coastal Range, of which Mount Burdell is part, was created by an east-dipping subduction zone between the Pacific and North American tectonic plates (Wakabayashi 1999). The intense mixing and deformation of the bedrock materials is explained by the overriding North American plate scraping sediment and rock off the subducting Pacific plate. This results in a jumbled mix of highly sheared and deformed bedrock (Scholl et al. 1980). Over time this highly sheared and deformed rock can accumulate enough volume in a small area to create the California Coastal Range.

The jumbled nature of the Franciscan Complex/mélange presents a difficult challenge for engineers because it is not possible to predict the engineering properties of the rock that would be encountered at a particular site without a substantial amount of subsurface exploration and testing (Goodman 1993). Given the increasing tendency of clients to limit or even reduce subsurface investigation costs, a site underlain by the Franciscan Complex creates an extreme challenge for the design of hillside developments.

In cases where the client is receptive to a substantial amount of subsurface exploration and testing, the design engineer should determine the depth of influence of the cut and fill activities and design a subsurface investigation to sample and test the material that would be impacted by the development. For example, the maximum depth of influence of the landscape screen constructed for the BC Development is estimated to be about 120 meters using Boussinesq stress distribution theory for an inclined embankment loading (Holtz and Kovacs 1981). However, the various geotechnical engineers employed for the BC Development drilled almost 80 boring across the site and none of the borings exceeded a depth of about 15 m within the slide limits shown in Figure 1. The slope inclinometers installed in the BC Development site after homeowner complaints show the depth of sliding to be 24 to 30 m. Thus, none of the borings drilled within the slide limits for the BC development are deep enough to reach the problematic serpentinite (discussed below). As a result, the designers may not have been aware of the weak layer underlying the site although the serpentinite is outcropping at numerous locations across the project site.

In summary, stress distribution analyses should be performed to assess the depth of influence for fill operations and the subsurface investigation should sample and investigate the materials within this zone of influence to determine if a weak layer exists. If a weak layer does exist, stability analyses should be conducted to assess the change in the factor of safety of the slope caused by the fill placement.

One material frequently found in the Franciscan Complex/mélange that usually presents a severe siting and slope stability hazard is serpentinite. The serpentinite at the BC Development site consists of large intact rocks surrounded by a high plasticity clay matrix and thus the rock is referred as a block-in-matrix rock (Goodman and Ahlgren 2000). Frequently, the percentage of the clay matrix is such that the engineering properties of the serpentinite are controlled by the clay matrix instead of the intact rock. The clay matrix is usually fully softened to highly sheared and thus exhibits a shear strength at or below the fully softened strength (Skempton 1970 and 1977). Even the fully softened strength of the clay matrix can be extremely low because it usually consists of highly plastic clay minerals such as montmorillonite (Stark and Eid 1997).

Testing of the serpentinite conducted during this study shows a liquid limit from 83 to 95, a plasticity index from 60 to 68, and a clay-size fraction (% < 0.002 mm) of 55 to 60%. As a result, the clay matrix classifies as a high plasticity clay (CH) according to the Unified Soil Classification system. If a linear failure envelope is passed through the torsional ring shear test results generated according to ASTM D6467, the resulting secant residual friction angle for the serpentinite is only six degrees, which is in agreement with the empirical correlations presented by Stark et al. (2005). The fully softened friction angle, also measured using torsional ring shear tests and a linear failure envelope, corresponds to about 12 degrees, which is in agreement with the empirical correlations presented by Stark et al. (2005). Twelve degrees also is in agreement with field observations of marginally stable serpentinite landslides in the area, such as the landslide at Land's End in the Golden Gate National Recreation Area, which have an average slope of 12 degrees (Goodman 1993).

A number of researchers, e.g., Dickinson 1966; Moiseyev 1970; Blake et al. 1974; Cowan and Mansfield 1970; Phipps 1984, have reported large landslides involving serpentinite. Table 1 shows a number of the long and wide serpentinite slides that have occurred. The length to width ratios of these slides range from 2.3 to 17.5. The length to width ratio of the current slide, 3.8, is also presented in Table 1 and is in agreement with previously reported serpentinite slides even though a large fill is involved.

Table 1: Length and width of serpentinite slides

Length (m)	Width (m)	Length/Width Ratio	Reference
1615	460 – 700	3.5 – 2.3	Dickinson (1966)
1070	60 - 155	17.5 – 7	Dickinson (1966)
610	120 - 215	5 – 2.8	Dickinson (1966)
1525	305 - 610	5 – 2.5	Dickinson (1966)
700	250	2.8	Phipps (1984)
460	120	3.8	Subject slide

A number of researchers, e.g., Berkland 1969; Blake et al. 1974; Rice 1975, have reported a number of large, deep-seated bedrock landslides that underlie the surficial colluvial slide deposits in the Mount Burdell area and there is evidence of serpentinite slides in roadcuts along the eastern edge of the project area. Investigation into possible prior landsliding in serpentinite throughout the San Francisco Bay area, and in particular in the Mount Burdell area, should have revealed the potential for deep-seated sliding with the placement of the large landscape screen.

In summary, engineers designing hillside developments should review local landslide history, assess the depth of influence of the fill/development operations, and investigate potentially weak material through the full depth of influence to ensure a weak layer is not overstressed and does not cause a slope failure.

To investigate the serpentinite in this study, three 0.6 m diameter borings were drilled to view and sample the serpentinite and to supplement three previous 0.6 m diameter borings that had been drilled previously within the slide limits shown in Figure 1. Two of the new borings reached the serpentinite while the third boring could not pass through the hard volcanic material on the BC Development property. Figure 2 is a photograph taken by the first author in one of the 0.6 m diameter borings that reached the serpentinite. The top of the serpentinite is encountered at a depth of 13 m and extends to a depth of 23 m. The depth of shear movement observed in a slope inclinometer within 4 m of the boring is about 16 m, which indicates that shearing or sliding was occurring in the serpentinite. Knowing that shear movement was occurring in the serpentinite, inspection of the serpentinite and the water condition were the main objectives of the down-hole inspection.

The first important observation from Figure 2 is that the serpentinite caved in and thus the clay matrix in the serpentinite could not support the blocky material in the open boring. As a result, frequent splashes from serpentinite entering the water filled bottom of the boring could be heard. The serpentinite caved in such that a 2.0 to 2.5 m of material from the edge of the boring was removed as shown in the diagram to the right of the photograph in Figure 2. Also shown in the photograph in Figure 2 is the diameter of the boring returned to about 0.6 m after the 4 to 5 m thick zone of caving serpentinite. This is in agreement with sliding being observed through this zone of serpentinite in the adjacent slope inclinometer. Because the serpentinite caved in 2.0 to 2.5 m from the edge of the boring, samples of this serpentinite could not be obtained. As a result, the testing described previously was conducted on grab samples from the auger while the two 0.6m diameter borings progressed through this layer and on grab samples obtained from previous 0.6 m diameter borings. Above the location shown in Figure 2, the soil materials were able to support themselves and the boring maintained a diameter of about 0.6 m. This is in agreement with the adjacent inclinometer indicating a well defined movement plane below these stronger materials that overlie the serpentinite.

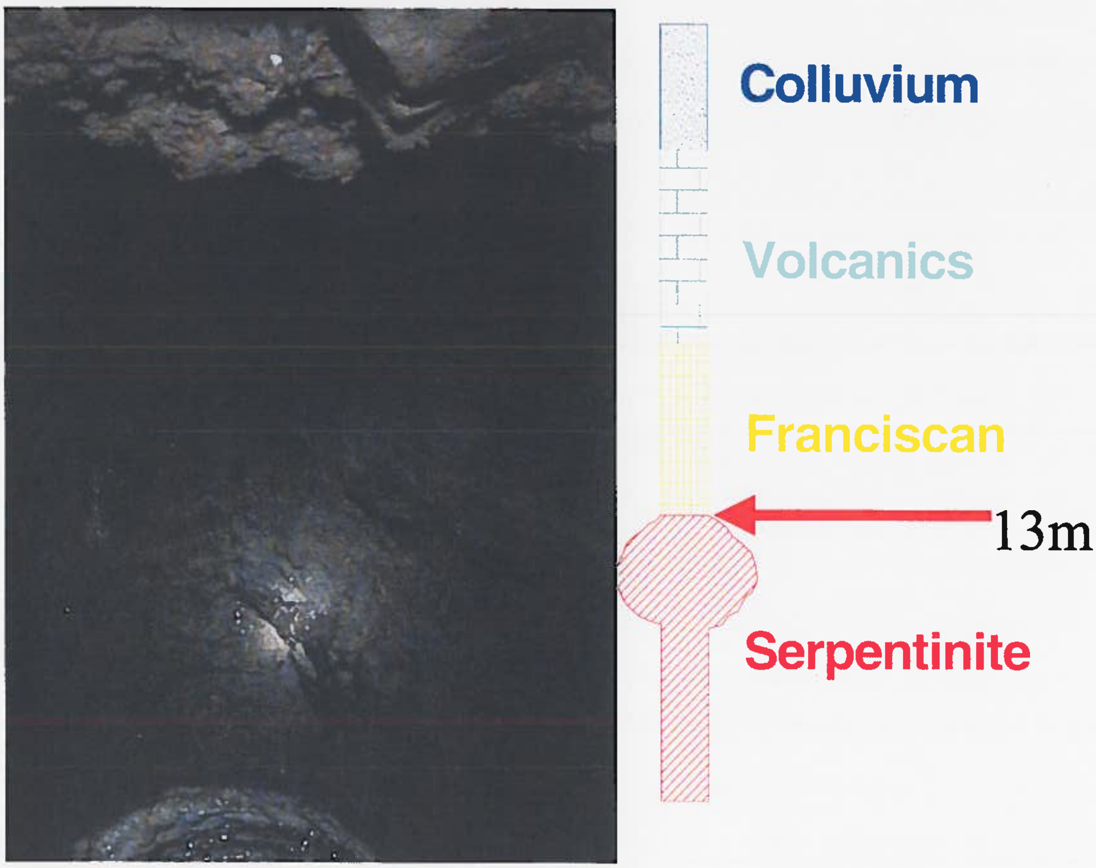

Fig. 2. View of caving serpentinite at a depth of 13 to 15 m in a 0.6 m diameter boring

In summary, the serpentinite is naturally occurring at the site and has been lurking below the project site for millions of years waiting for human development. Thus, the discussion of the three landslide factors turns to the second factor required for slope instability, which is water.

Water

The 0.6 m diameter borings also revealed a groundwater level above the top of the serpentinite. This water had to be pumped out before the boring could be entered and inspected. While suspended in the boring, water could be seen flowing into the boring through the serpentinite layer and filling the bottom of the boring. The three 0.6 m diameter borings drilled for this study were constructed in early April 2003 and thus the rainy season was nearing an end.

In general, the depth to groundwater is related to the amount of precipitation. High levels of precipitation usually result in higher levels of groundwater and vice versa. Thus, it is important to compare rainfall records prior to and during the year that movement is first reported to determine if rainfall is the trigger of the landsliding. To investigate the impact of rainfall on the causation of the landslide, the rainfall records from the nearby Petaluma Fire Station are summarized in Figure 3. Petaluma is the next town north of Novato, California. The yearly rainfall total from July 1 to June 30 of each year in presented in Figure 3. The Knolls housing development, which suffered the most damage, was completed between 1988 and 1989. Between 1989 and 1992-1993 rainy season, the area received below average rainfall. The fifty-three year average rainfall for the Petaluma Fire Station from 1948 to 2001 is 64.0 cm. In the 1992-1993 rainy season, 77 cm of rainfall or 13 cm above average rainfall occurred. The area also experienced above average rainfall in the 1994-1995 (113.2 cm) and 1995-1996 (80.7 cm) rainy seasons without any reports of distress even though the 1994-1995 rainfall exceeded the 53 year average by 49.2 cm. The reports of distress started in late December 1996 during a year of essentially average rainfall (63.8 cm) see Figure 3.

In summary, prior to the reported distress in late December 1996, more rainfall occurred in three prior years and no landslide occurred. The major difference between these years and 1996-1997 when landsliding did occur is the landscape screen had not been constructed. In the 1996-1997 rainy season, less rainfall occurred than in several previous years and a landslide occurred. As a result, it can be concluded that rainfall alone did not trigger this landslide and thus the investigation focused on the third factor, driving force.

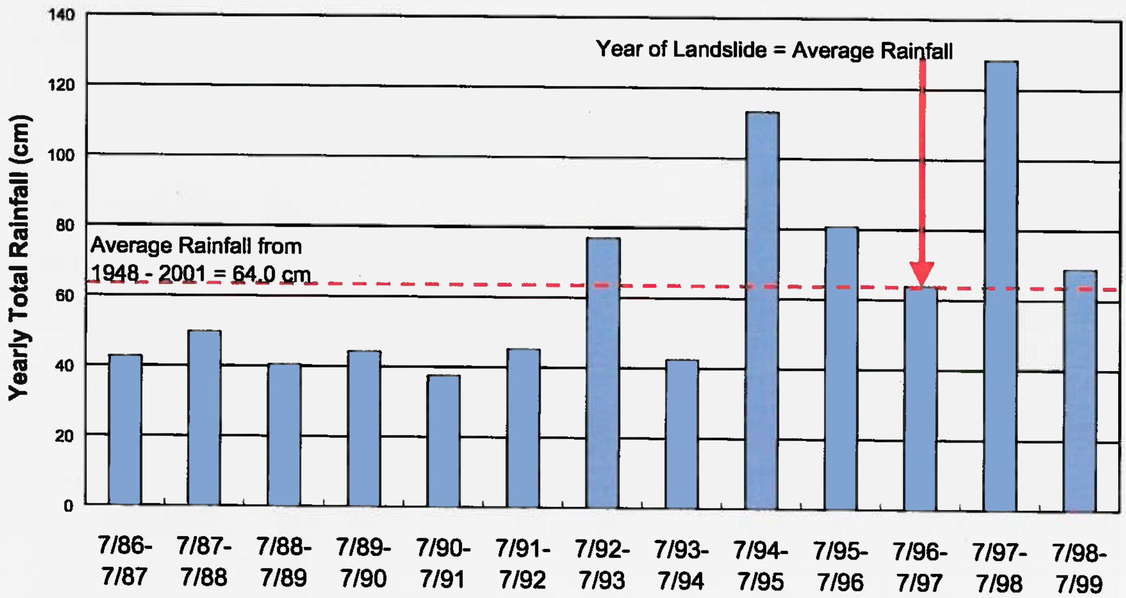

Fig. 3. Annual rainfall data from nearby fire station

Driving Force

The natural slope of the hillside shown in Figure 1 was not sufficient to initiate a deep landslide even with the above average rainfall that occurred in the 1992, 1994, and 1995 rainy seasons because homeowner complaints at the slope toe did not begin until late December 1996. In addition, the slope geometry or slope angle did not change significantly from 1989 until June 1996 when construction of the landscape screen commenced. As a result, this investigation focuses on two other sources of driving force, which are, in chronologic order, surficial grading for the Knolls development and placement of the landscape screen for the BC Development.

The surficial grading removed some material in the vicinity of the landslide toe, reducing the stability of the slope in two ways. The first way is the reduction in buttressing force caused by the removal of soil and rock from the landslide toe. This reduction increases the difference between the driving force imposed by the slope angle and the landscape screen at the top of the slope and the resisting force caused by the soil at the slope toe. Increasing the difference between the driving and resisting forces causes a decrease in the factor of safety.

The second way the stability of the slope is reduced is by the reduction in available shear strength along the failure surface. The available shear strength increases as the normal stress applied to the soil increases. The normal stress is related to the thickness of soil and/or rock above the failure surface and thus is reduced by removal of soil and/or rock from the slope toe.

Figure 4 presents a plan view of the landslide area with the slide limits from Figure 1 superimposed. In the eastern portion of the landslide toe the maximum depth of material removed during the surficial grading is approximately 6.5 m. The area corresponding to a removal of more than 6.1 m of material is indicated by the small ellipsoidal area in the eastern portion of the landslide toe. This maximum depth of excavation is due to a small hill that was situated at that location and had to be removed to create a level building pad for the house that would be constructed on the pad. Figure 4 also shows a larger area that corresponds to a depth of excavation exceeding 3.5 m. This area again is in the eastern portion of the landslide toe which is significant because subsequent stability analyses will show that the critical cross-section is located in the western portion of the landslide toe and outside both shaded areas in Figure 4. If the removal of the small hill had caused the landslide by reducing the buttressing effect and lowering the normal stress, most, if not all, of the landslide toe would have occurred through the point of maximum excavation. However, Figure 4 shows that a large portion of the landslide toe is the western portion where the removal of material is less than 2.5m. The maximum depth of excavation to create building lots in the western portion of the slide is about 2.5 m.

In summary, the surficial grading conducted to facilitate the construction of the Knolls development did not trigger the landslide in 1996 because (1) the landslide occurred 7 to 8 years after the surficial grading and after several years of significantly above average rainfall, (2) the maximum amount of excavation did not occur in the critical portion of the slope and thus did not impact the triggering of the landslide, (3) a large portion of the landslide toe occurs outside of the area of the largest surficial grading and if grading did destabilize the slope the landslide toe would be concentrated at the point of the deepest excavation, and (4) the maximum depth of excavation is 6.5 m and is insignificant compared to a depth of landsliding of 36 m measured in two slope inclinometers because shallow excavations usually do not trigger deep bedrock landslides.

The other change in the driving forces acting on the slope is the placement of the landscape screen. Figure 5 provides a comparison of the landscape screen to the homes in the Vista development. The landscape screen has a height of at least 22 m above the adjacent natural terrain and has a length and width of about 165 and 80 meters, respectively. The volume of the landscape screen when fill placement ceased is approximately 76,600 cubic meters, which corresponds to about 147 million kg of soil assuming a soil unit weight of 18.8 kN/m^3.

Fig. 4. Plan view illustrating the areas of greater than 3.5 m and 6.1 m (ellipsoidal area) of surface grading

In summary, the three landslide factors, weak layer, water, and driving force, coalesced in September and continued through December 1996 with the triggering event appearing to be the placement of the large landscape screen because (1) the weak layer had always been present, (2) the surficial grading had occurred 7 to 8 years before the reporting of distress, and (3) the site experienced years of greater rainfall after the grading than the year distress initiated. The following sections of the paper present the stability analyses used to quantify the impact of the surficial grading and the landscape screen on the stability of the slope.

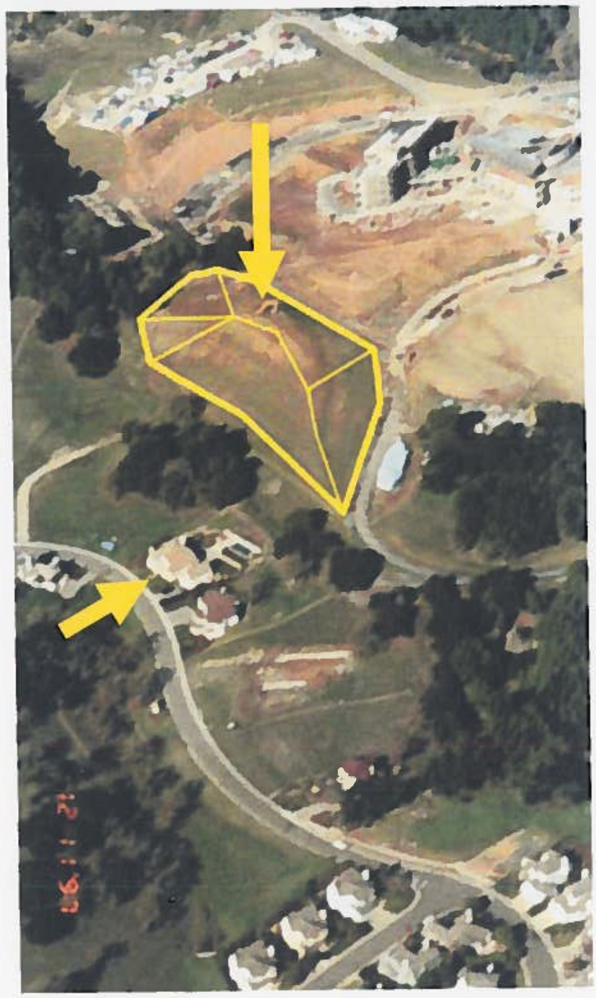

Fig. 5. Aerial illustrating size of landscape screen in relation to a 375 m^2 single family residence (see arrow) and an excavator on the top of the landscape screen (see arrow)

Forensic Investigation

The main steps in the forensic investigation to determine the causation of the 1996 landslide are: (1) develop a number of cross-sections to understand the variability and geometry of the subsurface materials, (2) determine the failure mechanism or failure surface from surface observations and slope inclinometer results, (3) develop material properties for the materials involved and appropriate groundwater levels, (4) perform a back-analysis to locate the critical cross-section, (5) use the back-analysis to estimate the mobilized shear strength of the weak layer and compare it with laboratory test results and field observations to ensure consistency and thus accuracy, and (6) conduct stability analyses to determine the effect of surficial grading and placement of the landscape screen on the stability of the hillside.

Cross-Sections

Knowing that the landslide is underlain by the highly variable Franciscan Complex, six cross-sections were drawn to gain an understanding of the materials present, the variability of the materials, and the presence of unusual and varying subsurface features, such as the buried sandstone ridge under the eastern portion of the slide mass. These six cross-sections are shown in Figure 6 and not only extend the length of the slide mass but also traverse the slide mass to determine material variability and distribution of the weak layer. The cross-sections are labeled TDS1 through TDS6 and stability analyses were performed on all of the cross-sections that extend the length of the slide mass, i.e., are in the direction of sliding. Cross-sections TDS1 and TDS5 (see Figure 6) extend through the western portion of the landslide toe while TDS2 and TDS6 extend through the eastern portion of the landslide toe. The stability analyses reveal that TDS5 and TDS6 are the critical cross-sections, i.e., yield the lowest factors of safety, for the western and eastern portions of the landslide toe, respectively. As a result, the forensic analysis described subsequently provides a comparison of the results obtained using cross-sections TDS5 and TDS6. However, drawing of the six cross-sections provided an invaluable insight to the subsurface conditions underlying the landslide especially cross-sections TDS3 and TDS4 which traverse the slide mass. These cross-sections reveal the presence of a buried sandstone ridge that increases from a depth of about 40m on the western portion of the slide mass to a depth of about only 18m on the eastern portion of the slide mass in the vicinity of TDS4.

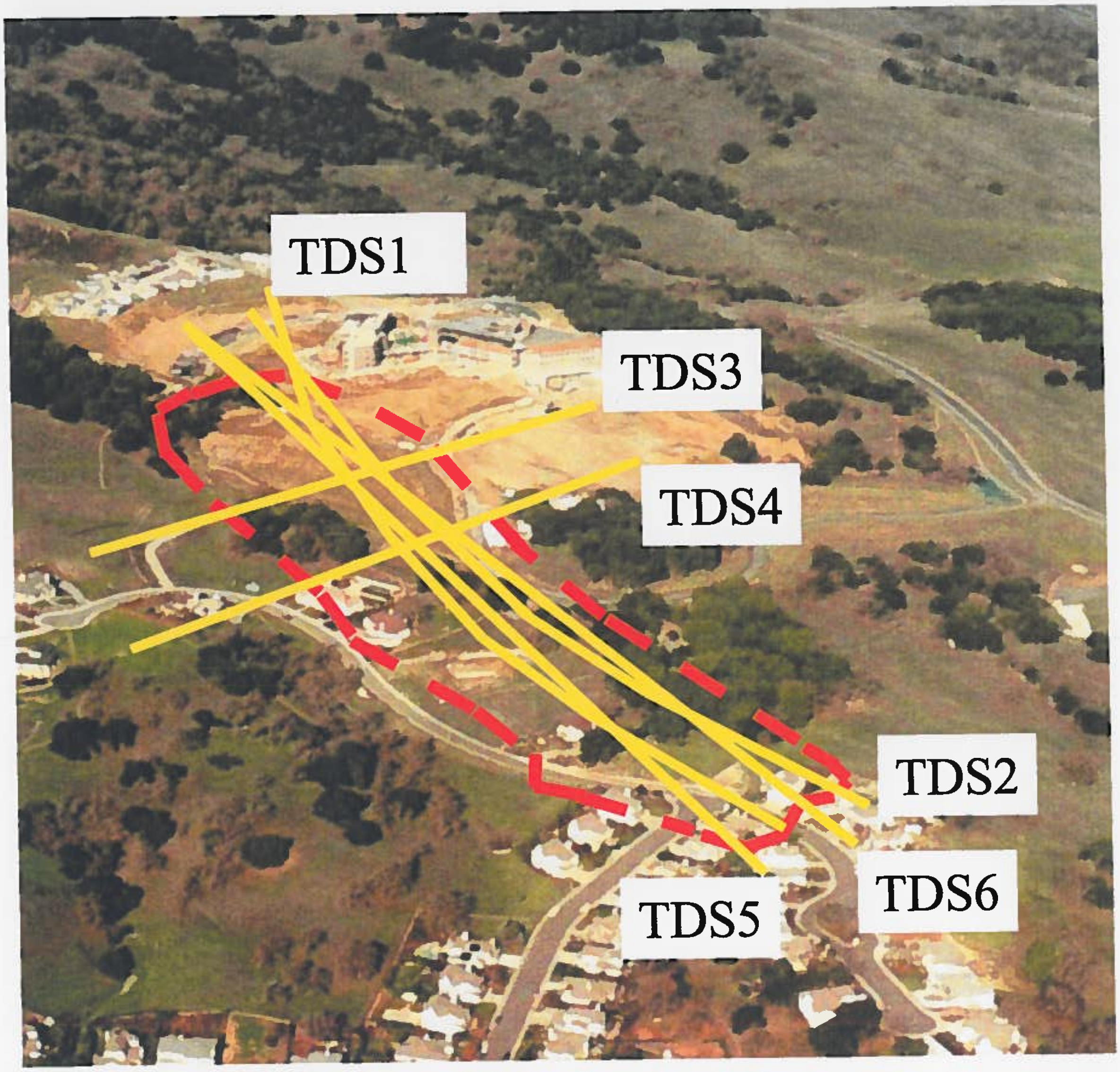

Fig. 6: Aerial view of the landslide illustrating the six cross-sections considered in the forensic study.

Failure Mechanism

The appearance of continuous and substantial tension cracks along the top of the slide limits, i.e., upslope of and around the landscape screen, indicated that the screen area pulled away from the natural materials upslope of this area. This is an indication of a translational failure mechanism instead of a rotational failure mechanism (Cruden and Varnes 1996). In addition, no vertical offset was associated with these tension cracks, which is expected for a translational slide that has only undergone about 20 to 25 cm of deep-seated movement. In a rotational slide, as the slide mass rotates to reduce the driving force, a vertical offset would be associated with the cracking at the top of the slide mass. Vertical offset can be associated with a translational slide if a graben starts to develop. The tension cracks upslope of the landscape screen continued to widen until the landscape screen was completely removed in April 1997 and indicate that the slide mass was simply pulling away from the natural material upslope of the fill area.

Nine of the fifteen slope inclinometers installed after the initial report of distress provide useful information but the other six are either too shallow or outside the slide limits shown in Figure 1 and do not provide direct information on shear movement of the slide. Each of the nine useful inclinometers show only one slide plane at depths ranging from 5 m near the landslide toe to 40 m near the middle of the slide mass. The depth of movement from the inclinometers is plotted on cross-section TDS5 in Figure 7.

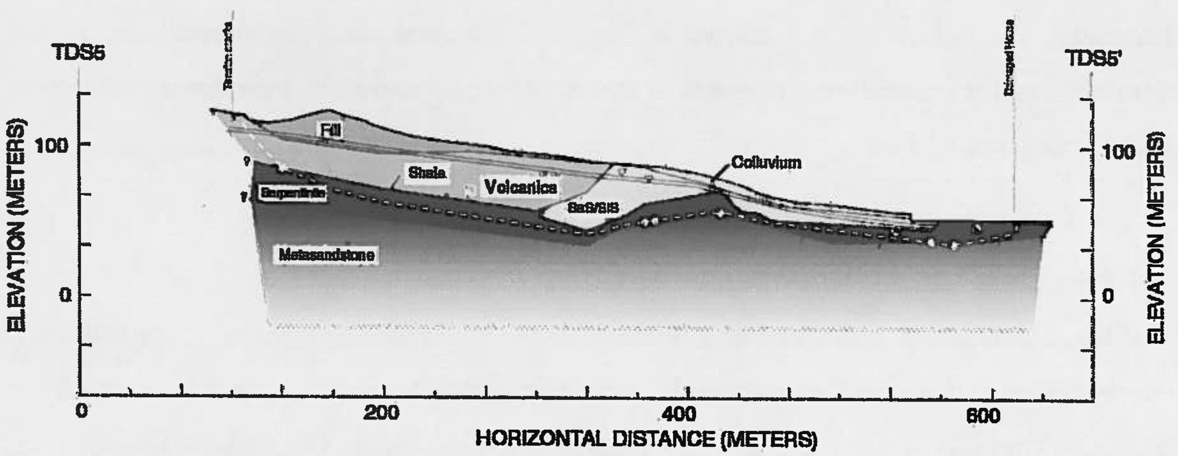

Fig. 7: Cross-section TDS 5 through the western portion of the landslide after surficial grading and placement of the landscape screen

Figure 7 presents the depth of movement or the total depth of the inclinometer in the fifteen inclinometers with various data symbols. The solid circle corresponds to an inclinometer that is within 30 m of cross-section TDS5 and distinct shear displacement is observed. The partially shaded circle corresponds to an inclinometer that that is not within 30 m of TDS5 but shear displacement is present, and a partially shaded square corresponds to an inclinometer that is not within 30 m of TDS5 and the inclinometer is too shallow and thus the failure surface should pass below this inclinometer. The failure surface shown in Figure 7 (see dashed line) was developed by connecting the location of shear movement in the inclinometers, following the various material types, and passing the failure surface through the cracks observed at the top of the landslide and the housing distress observed at the landslide toe.

Based on field observations, slope inclinometer results, and the stratigraphy developed from the results of the subsurface investigation, the failure surface passes through the Tertiary Volcanics upslope of the landscape screen at a steep inclination to the underlying weak and saturated serpentinite (Figure 7). The failure surface continues along the serpentinite layer until the depth of overburden allows it to daylight in the Knolls housing development and destroy four homes at the landslide toe. The landslide also damaged the homes in the Vista development that are at or near the western edge of the slide mass. The other homes did not show substantial landslide damage because they are located inside of the slide boundaries and are essentially "along for the ride".

In summary, the failure surface shown in Figure 7 corresponds to a translational failure mechanism although it is somewhat atypical in that it passes up and over a sandstone ridge near the middle of the cross-section.

Material Properties and Groundwater Levels

One of the uncertainties in the stability analyses is the shear strength of the serpentinite and thus a back-analysis, described subsequently, was conducted to estimate and/or confirm the mobilized shear strength of this material. The engineering properties of the other materials, i.e., unit weight and shear strength, involved in cross-section TDS5 are shown in Table 2.

The groundwater level acting on the failure surface at the time of the initial movement in October to December 1996 is not known. As a result, a range of groundwater level is used in the analysis with the high level corresponding to the rainy season and the low level corresponding to the dry season as shown in Figure 7. These two groundwater levels were developed from water levels observed in small and large diameter borings, monitoring wells installed as part of the remedial measures, and water levels used by other experts.

Table 2. Material properties used in stability analyses

Material Description	Moist Unit Weight (kN/m^3)	SHEAR STRENGTH PARAMETERS: Effective Stress Cohesion Intercept (kPa)	SHEAR STRENGTH PARAMETERS: Effective Stress Friction Angle (degrees)	Source
Volcanics	21.2	0	35	Laboratory testing
Sandstone (SaS) & Siltstone (SlS)	21.0	48	30	Laboratory testing
Metasandstone	21.2	145	30	Laboratory testing
Shale	20.4	0	12.5	Testing and Stark et al. (2005)
Colluvium	19.7	0	25	Testing and Stark et al. (2005)
Serpentinite	19.7	0	?	Back-analysis & Stark et al. (2005)

Back-Analysis of the Landslide

One of the main uncertainties in the stability analysis is the mobilized shear strength of the serpentinite because it is difficult to obtain a representative sample of the block-in-matrix rock and prepare it for testing in a laboratory shear device. This is due to the large rocks in the serpentinite and the variation in the quantity and consistency of the clay matrix across the site. Thus, it is difficult to ascertain whether or not the clay matrix controls the engineering properties of the serpentinite. To overcome this dilemma a two-dimensional limit-equilibrium back-analysis of the landslide was conducted to investigate the shear behavior and overall shear strength of the serpentinite. The back-analysis and the stability analyses discussed subsequently utilize Spencer's (1967) two-dimensional, limit equilibrium method because this method satisfies all conditions of static equilibrium. The slope stability program XSTABL Version 5 (Sharma 1996) was used for all of the analyses. A three-dimensional stability analysis was not conducted because of the large length of the slide which resulted in a length to width ratio of 3.8 (Stark and Arellano, 2000).

In each back-analysis, the friction angle of the serpentinite was varied until a factor of safety of 0.99, i.e., failure, was obtained. In each analysis the effective cohesion of the serpentinite was assumed to be zero because of the fully softened and/or sheared nature of the serpentinite (Stark et al. 2005). The engineering properties of the other materials used in the back analysis are presented in Table 2.

In a back-analysis, the failure surface that has undergone sufficient movement to result in the mobilization of an overall factor of safety of just less than unity, e.g., 0.99, should be analyzed. If sufficient movement has not occurred along the failure surface, the factor of safety is greater than unity and thus the back analysis would under estimate the shear strength of the weak layer because the factor of safety is greater than unity but the exact value is not known. In other words, the factor of safety could 1.1 to 1.5 and the slide mass would not be exhibiting movement. The magnitude of the under estimation is not known because the relationship between factor of safety and back-calculated friction angle of a material is not linear. As a result, a search for the failure surface in a back-analysis should not be performed but it is acceptable to vary the failure surface *between* known points, e.g., cracks at the top of the slide mass, shearing observed in inclinometers, and distress at the toe of the slide mass, to ensure the minimum friction angle is back-calculated. It is inappropriate to disregard the surface and subsurface features and search for a failure surface that may yield a friction angle that is less than the friction angle back-calculated for the observed failure surface. This is inappropriate because this is a new failure surface that has not undergone failure and thus the friction angle *should* be higher than the friction angle back-calculated for the observed failure surface. If the friction angle is greater than the back-calculated friction angle for the observed failure surface there is a flaw in the analysis because movement did not occur along the new failure surface.

In summary, a forensic investigation differs from a design investigation because the failure surface is known whereas in design the failure is not known and the engineer searches for the weakest or least stable portion of the hillside to ensure that it is safe. If the weakest portion of the hillside exhibits a suitable factor of safety in design, it is presumed that the remainder of the hillside would be stable. This design process is different from a forensic analysis and a search for the critical failure surface should not be conducted. Most importantly, the forensic analysis should utilize the failure mechanism (translational v. rotational) determined from the surface and subsurface observations and not conduct a back-analysis with a failure mechanism that is not present in the field. For example, a back-analysis for the present case should not use a search with circular failure surfaces to back-calculate the shear strength of the serpentinite because the failure mechanism is translational based on the location of the failure surface identified at a number of locations. However, it is prudent to vary the failure surface between these known locations to ensure that the minimum back-calculated friction angle is obtained.

The results of the back-analysis of cross-sections TDS1, TDS2, TDS5, and TDS6 are presented in Table 3. The cross-section that yields the highest back-calculated friction angle is the critical cross-section, i.e., the weakest or least stable part of the hillside, because the highest is required to achieve a factor of safety of unity. Therefore, in the western portion of the slide mass, i.e., TDS1 and TDS5, TDS5 is critical because it yields a higher back-calculated friction angles for both the high water (rainy season) and low water (dry season) cases than TDS1. In the eastern portion of the slide mass, TDS2 and TDS6 yield similar back-calculated friction angles ranging from 7.3 to 7.9 degrees for the low water (dry season) and high water (rainy season) cases.

Table 3. Back-calculated friction angles for serpentinite in cross-sections TDS1, TDS2, TDS5, and TDS6

	Back-calculated friction angle	
Cross-section	High water	Low water
TDS1	9.6	8.9
TDS2	7.9	7.6
TDS5	9.9	9.5
TDS6	7.7	7.3

A comparison of the back-calculated friction angles for cross-sections TDS5 and TDS6 reveals a significantly greater back-calculated friction angle for TDS5. Thus, the critical cross-section for the entire slide mass is TDS5 which is located through the western portion of the landslide toe. This is in agreement with distress being first reported in homes in the western portion of the Vista development and the western portion of the landslide toe. Therefore, it is concluded that shear movement started along or near cross-section TDS5 and induced movement along TDS6 to create the slide limits shown in Figure 1.

After conducting a back-analysis it is important to compare the back-calculated friction angles with the results of laboratory shear tests and empirical correlations to ensure the back-analysis yielded reasonable values of friction angle. The effective stress cohesion is assumed to be zero because of the fully softened nature of the serpentinite (Stark et al. 2005). The fully softened and residual failure envelopes estimated from torsional ring shear tests conducted on samples of serpentinite obtained from a 0.6 m diameter boring on the BC Development property are shown in Figure 8 and compared to the range of friction angle or failure envelopes for the high and low water conditions in cross-sections TDS5 and TDS6. The back-calculated failure envelopes for TDS5, i.e., friction angles of 9.9 to 9.5 degrees, are slightly below the fully softened failure envelope measured in the torsional ring shear tests. The back-calculated failure envelopes for TDS6, i.e., friction angles of 7.7 to 7.3 degrees, are well below the measured fully softened failure envelope and are in better agreement with the measured residual strength failure envelope.

The serpentinite has undergone shearing over geologic time but there is no evidence of prior landsliding along the failure surface shown in Figure 7 and thus a residual strength condition is probably not applicable to the observed failure surface for the causation of the landslide. However, a residual condition may be applicable for the design of remedial measures. In addition, if a residual strength condition had been mobilized along the failure surface shown in Figure 7 a small increase in the driving force or reduction in resistance, e.g., surficial grading, would have caused a reactivation of movement. The landscape screen is a large increase in driving force and the movements were increasing when the removal of screen commenced indicating the onset of post-peak behavior.

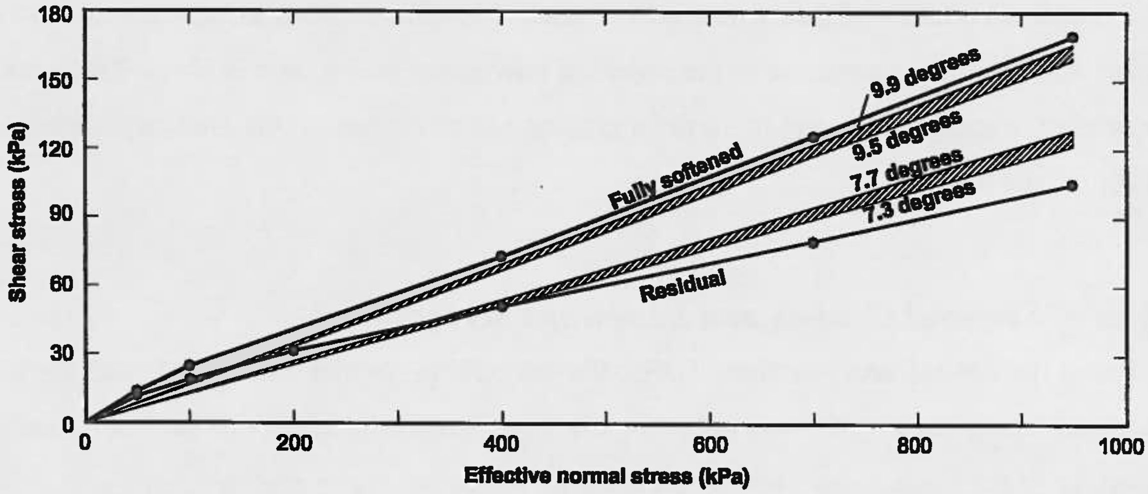

Fig. 8. Serpentinite failure envelopes derived from torsional ring shear tests and back-analyses

Analysis of a number of case histories by Stark and Eid (1997) and supplemented by Mesri and Shahien (2003) shows that it is reasonable to back-calculate friction angles below the full softened failure envelope for sites that have not undergone previous sliding. A strength below the fully softened value can be caused by progressive failure, which is discussed subsequently. Thus, the back-calculated friction angles for TDS5 and TDS6 falling between the fully softened and residual failure envelopes is in agreement with prior landslide observations and suggests consistency between field observations and the back-analysis. If the back-calculated friction angle plots above the fully softened failure envelope or below the residual failure envelope, then it is inconsistent with field observations and laboratory test results, which suggests a flaw or inconsistency in the back-analysis.

The back-calculated friction angles should also be compared with empirical correlations to ensure the back-analysis yields reasonable values of friction angle. Using a liquid limit of 83 to 95, a clay-size fraction (% < 0.002 mm) of 55 to 60%, and the fully softened and residual friction angle correlations presented by Stark et al. (2005), the back-calculated friction angles for TDS5 are in agreement with these empirical correlations that have been verified using field case histories.

In summary, the back-calculated friction angles for TDS5 are in agreement with laboratory test results, empirical correlations, and field observations of distress occurring first in the western portion of the landslide and sufficient deformation occurring to destroy the homes, which confirms a factor of safety near unity. Thus, it is reasonable to assume that the mobilized friction angle of the serpentinite at the on-set of movement in this case is about 9.9 degrees and can be used to study the impact of surficial grading and placement of the landscape screen on the stability of the hillside.

Effect of Surficial Grading and Landscape Screen

Using the critical cross-section, TDS5, the material properties in Table 2, and the a back-calculated friction angle of 9.9 degrees for the serpentinite, the impact of surficial grading and placement of the landscape screen on the stability of the hillside is presented in Table 4. Cross-section TDS5 was modified to reflect the four conditions shown in Table 4 because the cross-section shown in Figure 7 represents the slope geometry after surficial grading and after placement of the landscape screen, i.e., the circumstances under which the landslide was triggered. The cross-section was modified using the topography before surficial grading and before placement of the landscape screen that is available from the grading plans for the Knolls development and the BC Development, respectively.

Table 4 shows that the slide mass in Figure 1 exhibits a factor of safety between 1.10 and 1.15 before any surficial grading or fill placement occurred. (This slide mass is probably different than the slide mass used to illustrate a factor of safety of 1.5 or greater to obtain a building permit for the various projects.) After the surficial grading occurred in the vicinity of the western portion of the landslide toe, the factor of safety was still between 1.10 and 1.14 indicating a stable condition and in agreement with no homeowner complaints even though three years of heavy rainfall occurred before placement of the landscape screen. After surficial grading and placement of the landscape screen, the factor of safety decreased to between 0.99 and 1.03 indicating the slope was unstable regardless of the water level. This is in agreement with the observation of tension cracks occurring in the road that intersects the slide limits near the structures in the BC Development in October 1996 (see Figure 1). These tension cracks are located in a limited area and reappeared in the same location after the road was repaved. This is significant because only 2.5 cm of the annual rainfall had fallen at the Petaluma Fire Station at the time the road cracking was observed in October 1996. In December 1996 and January 1997, 27.4 and 22.0 cm, respectively, of the 63.8 cm of rainfall that occurred during the 1996 and 1997 rainy season occurred. This is in agreement with homeowner complaints in the Knolls development starting in late December.

In summary, this analysis and field observations confirm that movement started at the landscape screen area as early as September to October 1996 and progressed down slope until a continuous failure surface was created from just upslope of the landscape screen to the Knolls development below. When the entire failure surface had been created or mobilized, the lower portion of the slide mass started to impact the Knolls development until the landscape screen was removed in April 1998. The start of movement was caused by the coalescence of the presence of a weak layer, water in the serpentinite layer, and enough fill placement for the landscape screen, i.e., driving force, to reduce the factor of safety to near unity even for the low water case. This is reinforced by the last analysis that shows the factor of safety is unchanged if the landscape screen is in place and no surficial grading has occurred for the Knolls development.

Table 4. Effect of surficial grading and landscape screen on the factor of safety for TDS5 using a serpentinite back-calculated friction angle of 9.9 degrees

Condition	High water	Low water
Before surficial grading	1.10	1.15
After surficial grading	1.10	1.14
After surficial grading and landscape screen	0.99	1.03
Landscape screen and no surficial grading	0.99	1.03

Progressive Failure

Another possible mechanism alluded to earlier in the paper for the triggering of a landslide and considered during the investigation is progressive failure because the landslide occurred 7 to 8 years after the surficial grading and the delay in the slide may have been triggered by the time required for progressive failure to develop from the slope toe to the top of the slope (Mesri and Shahien 2003). Progressive failure occurs in slopes in which the driving force exceeds the mobilized strength of the weak layer, i.e., the slope angle exceeds the friction angle of the weak layer, at the excavation. If this occurs, soil at the location of the excavation or surficial grading becomes overstressed. If the local overstressing is large enough that the soil near the surficial grading yields, the applied shear stresses are transferred to the soil element just upslope of this overstressing. If the existing shear stresses and the transferred shear stresses are great enough to cause the upslope soil to yield, the overstressing would be transferred upslope again. This process can continue until enough soil is overstressed that a slope failure occurs. If the shear strength of the weak layer soil increases sufficiently upslope, initial progressive failure can be arrested.

In this case the back-calculated friction angle for the critical cross-section is 9.9 degrees and the slope angle estimated from survey measurements along the ground surface along cross-section TDS5 is 5.9 degrees. Thus, the mobilized friction angle along the observed failure surface is greater than the slope angle. It is concluded that the soil element at the slope toe did not yield and thus a progressive failure mechanism did not commence at this site. This is in agreement with no Knolls development homeowner reporting distress until late December 1996. If the initial soil element at the slope toe had yielded and then subsequent soil elements had yielded, some of the houses and associated sidewalks and pavements in the Knolls development would have exhibited movement but none was reported.

Legal Aspects of Landslides in an Urban Environment

In some respects a landslide in an urban environment is easier to investigate than a landslide in a remote area. It may be easier to investigate because there is more hardscape present which makes it easier to delineate the limits and shape of the slide mass. However, a landslide in an urban environment usually results in substantially higher damages and thus litigation. For example in the case described herein, the total damages awarded before legal fees and costs is about $15 million. In this case a four month jury trial was conducted and the jury voted 9 to 3 against the upslope development which resulted in the upslope development paying all of the damages and legal costs.

In a landslide case, a number of causes of action can be filed by plaintiffs, i.e., the damaged parties, to recover their loses. The causes of action range from negligence, public or private nuisance, breach of contract if there is privity of contract between the parties, and trespass. The easiest cause of action is trespass because the plaintiffs only have to show that the upslope property moved outside of its property boundaries and onto the downslope properties. This can be proven via property surveys that show the upslope property "physically invaded" the downslope properties. Thus, if a landslide occurs in an urban environment it is usually easy for plaintiffs to prove trespass which can create liability for many, including engineers, developers, homeowners, and contractors.

Negligence is difficult to prove because it requires a plaintiff to prove the following four elements against the defendant: (1) the defendant owed the plaintiff a duty of care, (2) the defendant breached the duty of care by not following local industry standards, (3) the breach of care caused the landslide, and (4) the amount of damage incurred by the landslide. Negligence is difficult for a plaintiff to prove because it requires expert testimony on the local industry standard of care. Thus, plaintiffs usually pursue a breach of contract claim before a negligence action but if there is no privity of contract between the damaged parties, e.g., downslope homeowners in this case, and the defendant, upslope development in this case, a breach of contract claim in not possible. As a result, another cause of action can be sought such as nuisance.

Nuisance can be plead against private or public entities. Nuisance is also difficult to prove because it requires a plaintiff to prove the following eight elements against the defendant: (1) plaintiff owns the property, (2) plaintiff's use and enjoyment of the property is affected in some way, (3) plaintiff suffered unreasonable interference with his/her use of the property, (4) defendant acted intentionally or negligently, (5) interference was caused by defendant's use of the land, (6) interference caused substantial harm to the property, and (7) the amount of damage incurred by the landslide.

In summary, a trespass or breach of contract action is usually pursued against developers, homeowners for various activities such as property irrigation or excavation, engineers, contractors, and building departments for a landslide in an urban environment.

Foreseeability of Future Development

In addition to determining whether natural events or construction activities would start a new landslide or reactivate an old landslide, the design engineer for a downslope housing development should be concerned about the level of foreseeability that is required for their design. In general, the size of structures usually decreases as development progresses up natural hillsides. Thus, is it foreseeable that a much larger and heavier development could occur above the single-family housing developments that the engineer is designing? If so, the housing development may have to be designed with large slope stabilization techniques to ensure stability during and after construction of the upslope development.

In this case the downslope design engineer did not know that the BC Development would occur and thus did not design stabilization techniques to resist the large landscape screen. It is recommended that design engineers clearly state their assumptions in the stability analyses in regards to future upslope development so engineers for future upslope development can identify the prior assumptions and perform stability analyses to assess the impact, if any, of a future upslope development on the existing downslope development. In this case, some of the design professionals involved in the Knolls development were also involved in the BC Development which made for some interesting foreseeability questions.

Conclusion

Landslide observations, data, and analyses used to investigate the cause of distress in two housing developments downslope of a hillside development near Novato, California are presented. The only mechanism that explains ALL of the surface and sub-surface movements observed in or near the two housing developments is a deep-seated translational failure surface that follows a layer of weak serpentinite and exits beneath the downslope housing development (see Figure 7). Movement along this translational failure surface was activated when fill placement for a large visual barrier was sufficient to reduce the factor of safety to near unity in September 1996 and the movement progressed from upslope to downslope until a continuous failure surface was created.

This case history illustrates some of the ramifications of constructing a large fill on a natural hillside upslope of housing developments and the importance of determining the relationship between a weak layer, groundwater, and static and seismic driving forces imposed on the slope. Most importantly, this case history illustrates the importance of understanding the depth of influence of the proposed development via a stress distribution analysis and conducting a subsurface investigation that extends through the depth of influence of the development. If this subsurface investigation is not conducted, it would be impossible to thoroughly assess the slope behavior and its effect on surrounding development because the presence or absence of potentially problematic layers would not be known.

In a back-analysis, the failure surface that has undergone sufficient movement to result in the mobilization of an overall factor of safety of just less than unity, e.g., 0.99, should be analyzed. If sufficient movement has not occurred along the failure surface, the factor of safety is greater than unity and thus the back analysis would under estimate the shear strength of the weak layer because the factor of safety is greater than unity and the exact value is not known. The only time that the factor of safety is known is when movement occurs and thus the factor of safety along the observed failure surface is at or near unity. It is inappropriate to disregard surface and subsurface features that indicate a translational failure mechanism and search for a circular failure surface that may yield a friction angle that is less than the friction angle back-calculated for the observed failure surface. This is inappropriate because this new failure surface has not undergone failure and thus the friction angle should be higher than the friction angle back-calculated for the observed failure surface. If it is greater than the back-calculated friction angle there is probably a flaw in the analysis with the new failure surface. However, it is suitable to vary the failure surface between known points, e.g., cracks at the top of the slide mass, shear movement observed in inclinometers, and distress at the toe of the slide mass, to ensure the minimum friction angle is back-calculated for the observed failure surface.

Acknowledgments

The contents and views in this paper are the authors and do not necessarily reflect those of any of the developments, homeowners, consultants, or anyone else involved in this project.

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LANDSLIDE IN AN URBAN ENVRIONMENT

By: Timothy D. Stark and Erik J. Newman

Figure Captions:

Fig. 1. Aerial view of housing developments, BC Development, and an outline of the slide mass

Fig. 2. View of caving serpentinite at a depth of 13 to 15 m in a 0.6 m diameter boring

Fig. 3. Annual rainfall data from nearby fire station

Fig. 4. Plan view illustrating the areas of greater than 3.5 m and 6.1 m (ellipsoidal area) of surface grading

Fig. 5. Aerial illustrating size of landscape screen in relation to a 375 m^2 single family residence (see arrow) and an excavator on the top of the landscape screen (see arrow)

Fig. 6. Aerial view of the landslide illustrating the six cross-sections considered in the forensic study

Fig. 7. Cross-section TDS 5 through the western portion of the landslide after surficial grading and placement of the landscape screen

Fig. 8. Serpentinite failure envelopes derived from torsional ring shear tests and back-analyses

Table Captions:

Table 1. Length and width of serpentinite slides

Table 2. Material properties used in stability analyses

Table 3. Back-calculated friction angles for serpentinite in cross-sections TDS1, TDS2, TDS5, and TDS6

Table 4. Effect of surficial grading and landscape screen on the factor of safety for TDS5 using a serpentinite back-calculated friction angle of 9.9 degrees

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Unstable Slopes in the Cuyahoga River Valley: Their effects on Urban Development, the Geologic History, and Planning for Future Geotechnical Investigations

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Many constructed facilities (public works and private development) within the heavily industrialized lower Cuyahoga River valley in the greater Cleveland, Ohio area, are built on or near the valley wall side slopes that are continually subject to creep movement. Subsurface investigations, geotechnical monitoring, and geological evaluations have confirmed the presence of multiple presheared planes within the Cuyahoga River valley. This paper presents examples of unstable slopes and their adverse affects on public and private works within the valley. Highlighted in these examples will be the results of over 10 years of inclinometer and piezometer monitoring performed by BBCM on the west bank of the existing I-90 Central Viaduct Bridge; and an evaluation of a deep seated slope movement on a 150 foot high valley wall which is located below W25th Street in Ohio City. This slope is owned by at least four public agencies and has been moving since the development of the City of Cleveland.

This paper will also discuss the Quaternary geologic history and its influence on the observed behavior of lacustrine clay slopes within the Cuyahoga River valley. Large fluctuations in the elevation of glacial Lake Erie caused deep, rapid incision of the Cuyahoga River into the overburden soil deposits. Steep bluffs, prone to creep and landslides, developed adjacent to the Cuyahoga River. The presheared planes, along which shear displacement occurred, developed within the lacustrine clay and were subsequently buried as the river aggraded fluvial deposits as it rose to its current elevation. These ancient presheared planes are likely responsible for areas of large unstable slopes that continue to cause damage to constructed facilities throughout the Cuyahoga River valley. These presheared planes compare quite well to the levels of movement encountered in long-term inclinometer testing of various sites.

These examples demonstrate the necessity for geotechnical engineers to link the geologic history and observed slope behavior prior to developing a scope of work for geotechnical investigations.

Introduction

The heavily industrialized Cuyahoga River valley in greater metro Cleveland, Ohio, is prone to landslides and creeping hillsides. As a result, the unstable slopes within the river valley often negatively affect the performance of the constructed facilities. The purpose of this paper is to summarize the Quaternary geologic history of the Cuyahoga River valley as it relates to the observed geotechnical behavior of the valley walls. Two examples are presented of unstable slopes, located near downtown Cleveland, Ohio.

Quaternary Geologic History

In the vicinity of downtown Cleveland, the present Cuyahoga River runs along the western bench of a buried ancient river valley. The uppermost stratum of bedrock in the Cuyahoga River valley, in the vicinity of downtown Cleveland, is Devonian aged Ohio shale. The now buried axis of the ancient Cuyahoga River valley is at an elevation near sea level (Peck 1954 and Gardner 1972). Ohio shale and the underlying deeper bedrock strata, contain organic matter that produces natural gas, which percolates upwards through the fractured shale becoming trapped in pockets within the lower portion of the overlying sediments.

Near the end of the Late Wisconsinan glacial period, a large proglacial lake, typically bound by the Defiance and Wabash moraines, was present in the Lake Erie basin. The Saginaw, Huron and Erie glacial ice lobes advanced and retreated numerous times, which closed or opened various drainage outlets, causing the lake elevation to fluctuate over time. Calkin and Feenstra (1985) summarized each of the proglacial lake phases (lake phases distinguished using different names) and correlated lake elevation with drainage outlet and ice margin location. Figure 1 presents a history plot of proglacial lake elevation in the Lake Erie basin.

The maximum lake elevation during the Lake Maumee phases was approximately 800 feet, which was sufficiently stagnant to permit fine-grained soil particles to settle. The lacustrine silt and clay was deposited throughout the lower Cuyahoga River valley (Gardner 1972 and Szabo *et al.* 2003) during the Lake Maumee phases.

Calkin and Feenstra (1985) postulate that a rapid lake lowering event occurred during Lake Ypsilanti. The ice margin during Lake Ypsilanti may have retreated enough to permit lake drainage through New York's Mohawk River valley and possibly through the Niagara escarpment. They found evidence to suggest that during the Lake Ypsilanti phase, the lake elevation may have dropped to Elevation 400 feet, approximately 170 feet lower than the elevation of current Lake Erie.

Subsequent to Lake Ypsilanti, glacial ice readvance blocked off the New York outlets. As a result, the lake elevation rose, which again submerged much of the Cuyahoga River valley and initiated the Lake Whittlesey phases. During the Lake Whittlesey phases, the Cuyahoga River deposited deltaic sand and silt within the Cuyahoga River valley (Toten 1985 and Peck 1954) to approximate Elevation 760, an elevation approximately 90 feet above the current high ground in the downtown Cleveland vicinity. The gradual drop in lake elevation after the Lake Whittlesey phases caused wave action redistribution and river incising, which removed much of the deltaic material. Within this timeframe, Calkin and Feenstra (1985) postulate that the Mohawk River valley drainage outlet was opened once again (Lake Wayne) creating a rapid lake elevation drop (approximately 60 feet below the current Lake Erie elevation).

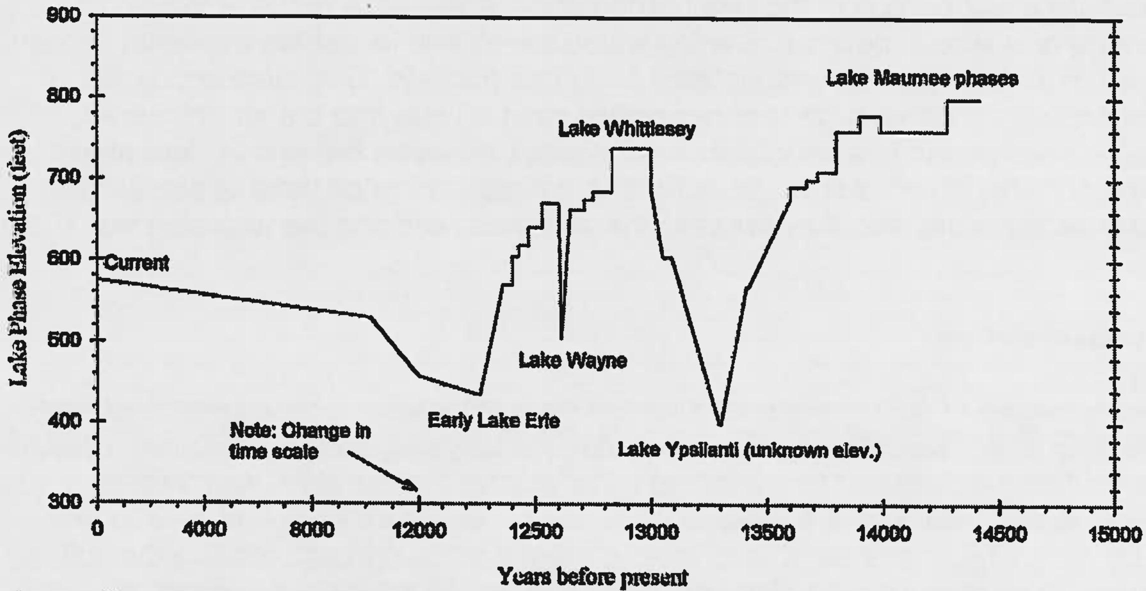

Figure 1. History of proglacial lake elevation within the Lake Erie basin

Finally, the northward retreat of glacier ice reopened the Niagara escarpment for good, which initiated the Early Lake Erie phase and caused the lake elevation to drop to Elevation 430 (approximately 170 feet below the current Lake Erie elevation). The Early Lake Erie phase also corresponds to when geologists believe that glacial Lake Algonquin, located in the Huron basin, drained into the Ontario basin through the Kirkfield outlet in Ontario instead of draining into the Erie basin (Karrow *et al.* 1975).

The dramatic drop in lake elevation associated with Early Lake Erie, Lake Wayne, and potentially Lake Ypsilanti caused the Cuyahoga River to incise rapidly through the Wisconsinan deposits and into the Illinoian deposits (Szabo *et al.* 1985), creating steep bluffs adjacent to the Cuyahoga River. Szabo *et al.* (1985) identified the bottom of the scoured river valley at Elevation 445, approximately 130 feet below the present river elevation. Isostatic rebound of the Niagara escarpment caused a gradual rise in the Early Lake Erie elevation to the current Lake Erie elevation (Calkin and Feenstra 1985). As the lake elevation rose, the Cuyahoga River aggraded fluvial material (Bradley, 2002 and Szabo *et al.* 1985).

Influence of Geology on Engineering Behavior

The geologic history of the Cuyahoga River valley has significantly influenced the geotechnical characteristics of the valley sediments, which has contributed to the unstable valley slope existing in the Cuyahoga River valley today. The key geotechnical characteristics that have been influenced by geologic history are summarized below.

1. Excess Pore Pressure

Devonian aged Ohio shale, and deeper bedrock strata, contain natural gas that percolates upwards through the fractured shale becoming trapped in pockets throughout the lower portion of the overburden sediments. As a result, elevated or artesian pore pressure conditions develop within the till and lacustrine deposits. Recent subsurface explorations have encountered such gas pockets. One such encounter produced pressure high enough to shoot drilling mud 60 feet into the air. However, more typically water/gas has been observed to erupt between five and 20 feet above the ground surface. Excess pore pressure that develops from pockets of pressurized natural gas reduces the effective stress in the soil, thus reducing the available soil shear strength.

2. Overconsolidation

Valley wide erosion of deltaic material after the Lake Whittlesey phase overconsolidated the underlying soil. The compression induced in the silty clay by the overburden deltaic deposits led to an increase in strength and initial stiffness of the clay. Compression also reoriented the clay particles to a generally horizontal alignment such that face to face particle contact is more prevalent than face to edge particle contact, making the soil more fissile. Overconsolidation also creates or exacerbates the development of fissures. However, as pointed out by Terzaghi *et al.* (1996), the influence of fissures on available shear strength is minimized for glacial clays of low plasticity ($I_p < 20\%$), like the deposits encountered in the Cuyahoga River valley.

3. Buried slip planes at residual strength conditions

The elevation of at least two (Lake Wayne and Early Lake Erie) and possibly three (Lake Ypsilanti) lake phases lowered rapidly as the receding glacier exposed large drainage outlets. As the Cuyahoga River incised rapidly into the sediments, steep bluffs were created, triggering large landslides or time-dependent shear deformation. The ancient landslides created planes along which shear displacement occurred that are at drained residual strength conditions (Terzaghi *et al.* 1996). Evidence of these landslides have been found and could be expected from the existing ground surface down to approximate Elevation 400, corresponding to the elevation of the lowest lake phase.

Mesri and Shahien (2003) also explain the development of residual shear strength conditions for stiff clays prior to a first-time slope failure. Processes active during deposition, consolidation, erosion, and preshearing can facilitate discontinuities that can reduce the strength to or near residual strength conditions after only a small shear displacement.

4. Change in soil stress state

Even where a landslide or deviatoric creep did not develop, the process of river incision would have changed the soil's stress state. The process of river incision, especially in the bluffs adjacent to the outer bend of a tight river curve, would have likely reduced the horizontal effective normal stresses within the influence of the created slopes. This process would have also induced shear stresses in the affected areas. At least a portion of any shear stresses that were induced by river incision likely remain today as part of the soil stress state. As a result of these additional in-situ shear stresses, there is additional uncertainty in slope stability analyses.

5. Creep

Time-dependent shear deformation (creep) requires that a sufficient amount of energy, termed activation energy, be obtained to overcome an energy barrier (Mitchell 1964). As the ratio of the shear inter-particle contact force to the normal inter-particle contact force increases, the required energy barrier decreases.

As the Cuyahoga River incised into the sediments creating steep bluffs, the soil's stress state changed. In particular, shear stresses were induced into the soil mass. Both the normal and shear stress components of the applied boundary stress can be expected to generate both normal and shear components of contact force. However, for practical purposes, it is reasonable to assume that as a greater shear stress is induced in the adjacent river bluffs, the likelihood of creep increases.

Examples of Unstable Slopes in the Cuyahoga River Valley

The geologic processes presented in the previous sections contribute to the unstable slopes present in the Cuyahoga River valley today. Two examples of unstable creeping slopes, located near downtown Cleveland, are summarized. The approximate locations of the West 25th Street slide and the I-90 slide are illustrated in a map (map taken from www.google.com) shown in Figure 2. Both of these two slides are occurring in the hillside located on the outside of tight river bends where toe erosion caused by the river currents is the greatest.

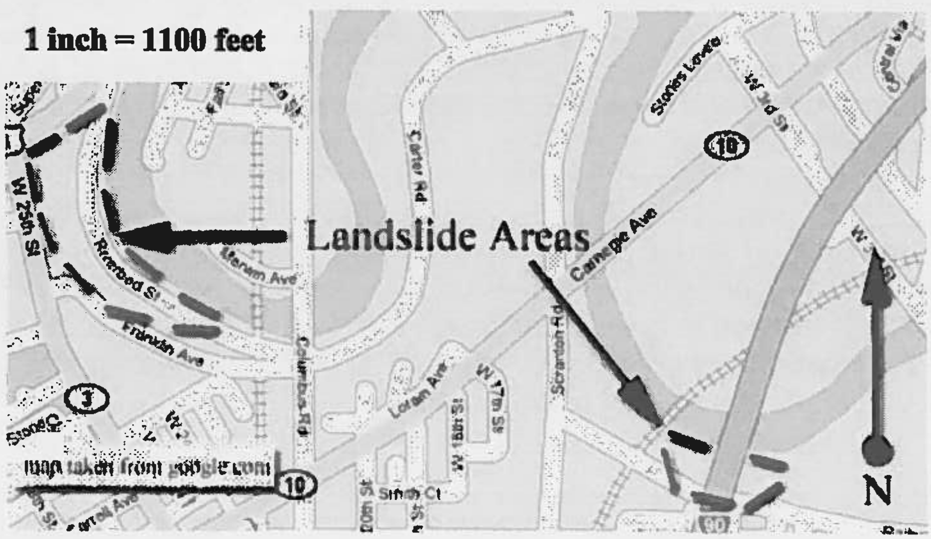

Figure 2. Approximate locations of West 25th Street, and I-90 slides in Cleveland, Ohio

1. West 25^{th} Street and Riverbed Street

There are multiple slides occurring along the west bank of the Cuyahoga River in the vicinity of West 25^{th} Street. First, there are a series of smaller landslides occurring along the river bank, creating scarps in Riverbed Street. There is also a much larger landslide with an exposed scarp on the upper terrace of the slope running parallel to the river, ranging from about 150 feet to 230 feet east of West 25th Street and extending continuously approximately 3000 feet. In general, the soil stratigraphy can be described in descending order as follows: non engineered fill, fluvial sand, fluvial and/or lacustrine silt, lacustrine silty clay, and silty clay till. All the natural soil layers were deposited during the Wisconsinan glacial period, with the exception of the till, which was deposited during the Illinoian glacial period. Figure 3 illustrates a cross section along the hillside showing the idealized soil stratigraphy for that section. There are three boxes superimposed on Figure 3 that denote the approximate locations of observed inclinometer movement. The slip planes associated with a smaller slide that has a scarp along Riverbed Street are shown in red, while the slip planes associated with the large slide that has a scarp running parallel to West 25^{th} Street are shown in blue.

The scarp for the large slip surface developed in the 1960's when fill was placed east of West 25^{th} Street to provide the area necessary for parking lots. The relative vertical displacement of the portion of the scarp illustrated in Figure 3 is approximately four feet. The slip surface dives down to a nearly horizontal slip plane, located at approximate Elevation 555. Figure 1 indicates that the glacial lake level was at approximate Elevation 500 during the Lake Wayne phase.

Evidence suggests that the lower portion of the slip plane for the slides along Riverbed Street are at an elevation similar to the elevation of the horizontal portion of the large slip plane. However, the scarp along Riverbed Street is separating and dropping much more rapidly than the scarp adjacent to West 25^{th} Street. The Riverbed Street scarp redeveloped to the condition illustrated in Figure 3 approximately 6 months after Riverbed Street was repaved.

Since the horizontal portion of slip plane for both slides are near the same elevation, supporting the Riverbed Street slide using a standard river bulkhead wall becomes unfeasible without also arresting movement of the West 25^{th} Street slide. If the West 25^{th} Street slide is allowed to continually creep, any new river bulkhead designed to support the Riverbed Street slide would either go along for the ride with the large slide mass or fail due to excessive load or deflection.

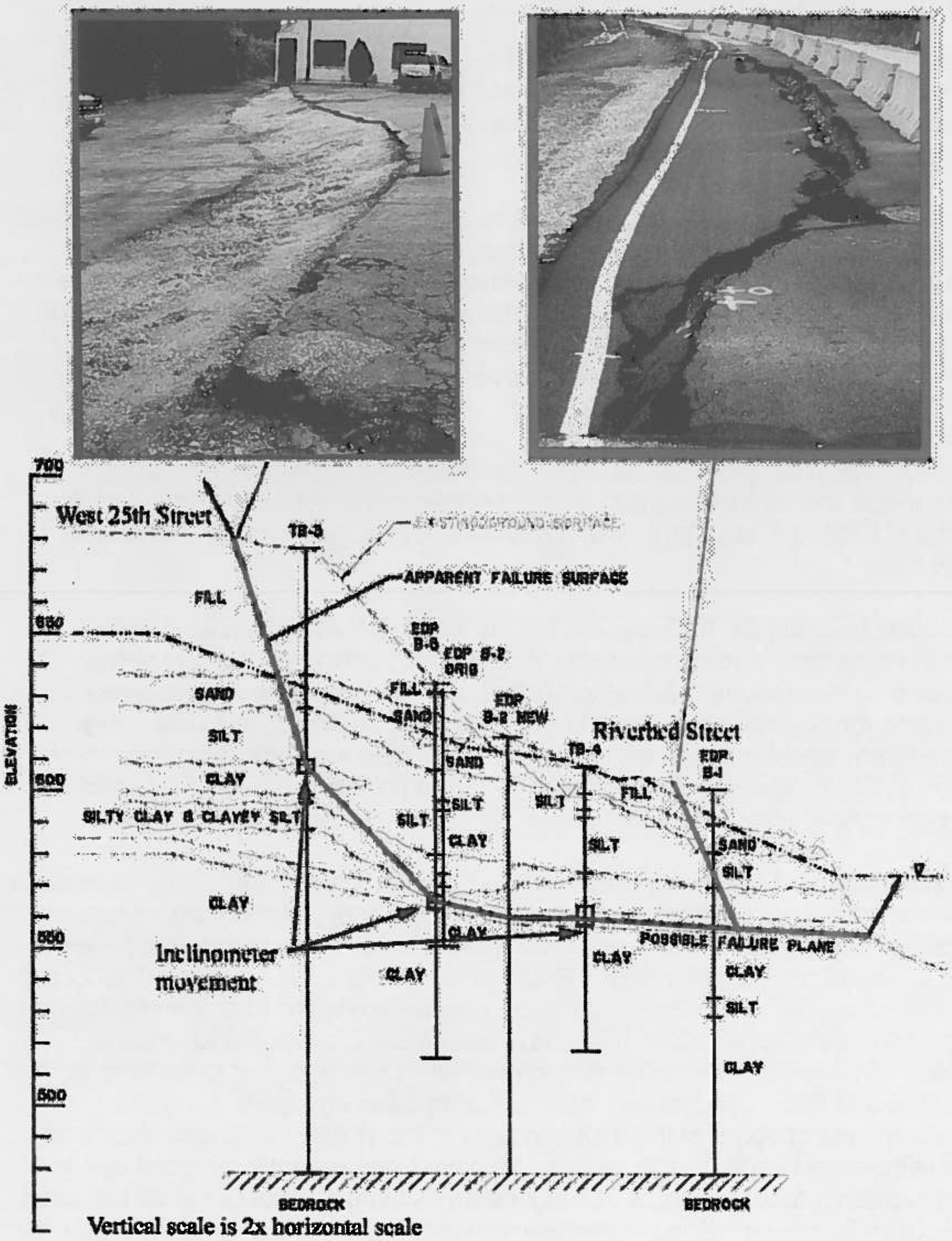

Figure 3. Idealized cross section of slope adjacent to West 25th St., near downtown Cleveland (slope is down to the left in the pictures instead of down to the right in the cross section).

2. I-90 over the Cuyahoga River

The I-90 Central Viaduct Bridge crossing over the Cuyahoga River was completed in 1959. The total length of the bridge is 5,080 feet. The trusses are supported on a series of piers, each of which are supported on friction piles. Two sets of piers are founded on the west bank of the Cuyahoga River.

By the late 1980's, sufficient soil creep induced superstructure movement had occurred to precipitate the need for remediation. A structural stabilization structure was constructed to reduce the movement of pier I, which is illustrated in Figure 4. Figure 4 also presents a plan view and idealized cross section of the west bank slope beneath the I-90 bridge. The stabilization structure, consisting of a drilled shaft wall located between the Cuyahoga River and pier 1, is tied-back with steel H-piles to an anchor block located upslope of pier 1. The drilled shafts are socketed into bedrock approximately 140 feet below ground surface. The anchor block is supported on a series of H-piles driven to refusal in bedrock. A series of rock anchors extend downward from the anchor block, approximately 45° from the horizontal, into bedrock upslope of the stabilization structure. Construction of the stabilization structure was completed in 1999.

The soil stratigraphy is similar to that noted for the West 25th Street slide. Artesian gas/water pressures were encountered nine times during drilling operations at this location. At one such encounter, 8-feet to 10-feet of water/gas erupted constantly above the casing for 10 minutes during the first core run. During core barrel removal, water escaped from top of the core barrel, which at the time was approximately 15 feet above ground surface. Water/gas/debris also escaped horizontally from the top of the core barrel connection, spraying approximately 40-feet horizontally from the drill rig.

The creep rates observed in the slope beneath the I-90 bridge are sufficiently slow as to require approximately 2 to 3 years before the amount of observed deflection exceeds the deflection error threshold of the inclinometer. As a result, long monitoring periods are required to obtain meaningful trends. Creep is occurring on at least two distinct slip planes, the lowest portion of the lower slip plane is at approximate Elevation 490 while the lowest portion of the upper slip plane is at approximate Elevation 560. Figure 1 indicates that the glacial lake elevation during the Early Lake Erie and Lake Wayne phases was 435 and 500, respectively. Also superimposed on Figure 4 are the approximate locations of some of the inclinometers in the vicinity. Of those shown on Figure 4, inclinometers B-105, B-108, and B-110 have been recently replaced due to excessive movement. Inclinometer B-204 is located within the limits of the stabilization structure. Four inclinometer tracings are superimposed on the cross section: B-203, B-103 (B-103 was replaced by B-303 due to excessive movement), B-204, and B-105. The inclinometer tracing clearly indicate at least two slip planes in the vicinity of pier 1. Inclinometer B-204, which is located within the stabilization structure, shows significantly less movement than the other inclinometers.

Figure 5 presents four inclinometer deflection history plots for inclinometers B-105, B-110, and B-204. Inclinometer B-204 was installed within the limits of the stabilization structure after its completion in 1999. Construction of the stabilization structure, which included temporary toe unloading, increased the creep rate in the vicinity of pier 1, as is clearly indicated in the B-105 (93 ft to 97 ft) inclinometer tracing for the deep slip plane. Mitchell (1993) describes this onset of the dramatic increase in creep rate as the primary creep phase, which is characterized by a continuously decreasing creep rate. Data indicates that the creep rate began to level off to a nearly constant value around 2004. The soil is likely transitioning from primary to secondary creep phase, which is typically characterized by a nearly constant creep rate.

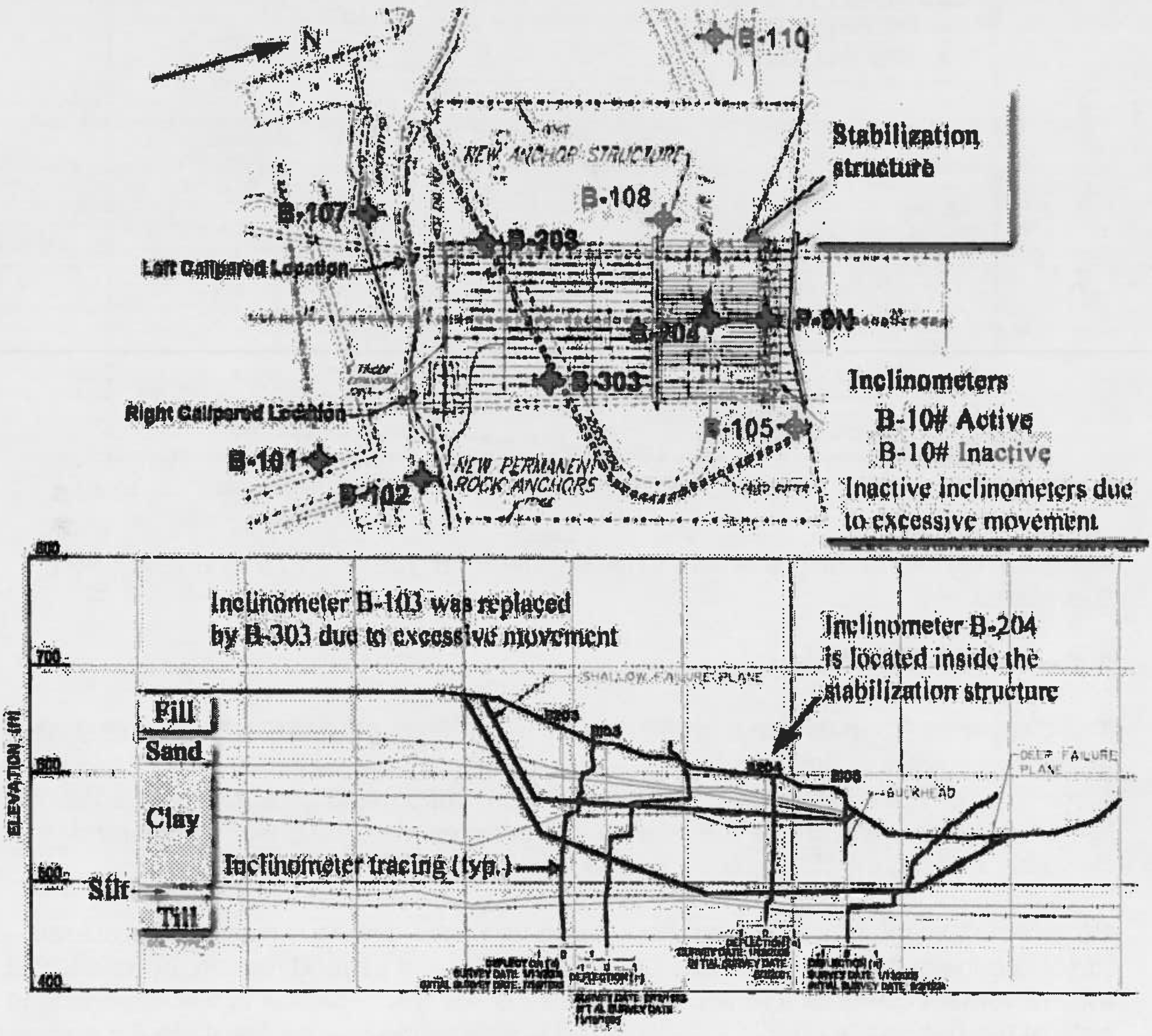

Figure 4. Plan view and cross section of slope beneath I-90, near downtown Cleveland

However, the change in creep rate associated with the movements observed in inclinometer B-110 was significantly less than the change associated with inclinometer B-105. In general, the construction of the stabilization structure had very little impact on the creep rate associated with inclinometer B-110 due to its location relative to the stabilization structure. As shown in Figure 4, inclinometer B-110 is well outside the vicinity of the pier 1 stabilization structure. Creep still continues at nearly the same rate within the footprint of the stabilization structure along the deep slip plane, as is noted in inclinometer B-204 (117 ft to 119 ft), as outside of the stabilization structure. However, within the limits of the stabilization structure, creep along the shallow slip plane has essentially halted.

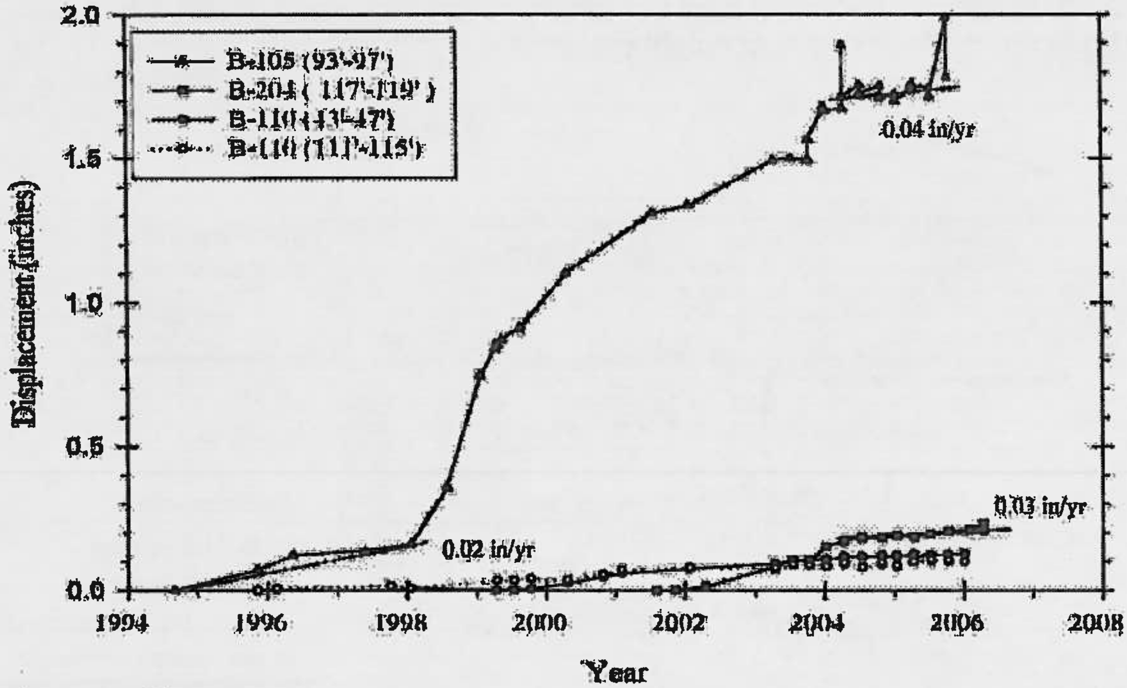

Figure 5. Measured displacement in inclinometers B-105, B-110 and B-204 (I-90 in Cleveland)

3. Columbus Road Bridge

The Columbus Road Bridge is a lift bridge (Figure 6) constructed in 1939, and is located immediately north of the Riverbed Street Landslide, as shown in Figure 2. The bridge is currently undergoing inspection for complete renovation, and preliminary information has revealed that the bridge has experienced approximately 3.5 inches of lateral movement based on old benchmarks, and that the bridge is "binding".

Other than this information, there have been no known previous reports of problems associated with bridge or roadway movement. It is not beyond reason, however, that this movement could be a result of deep seated soil creep, similar to that experienced at both of the previously mentioned slopes. It is interesting to note that if the 3.5 inches of movement were spread evenly over the 69 years of bridge life, the calculated rate of movement would be 0.05 inches per year, which closely matches the results of 13 years of inclinometer readings for the West Bank of the I-90 Bridge.

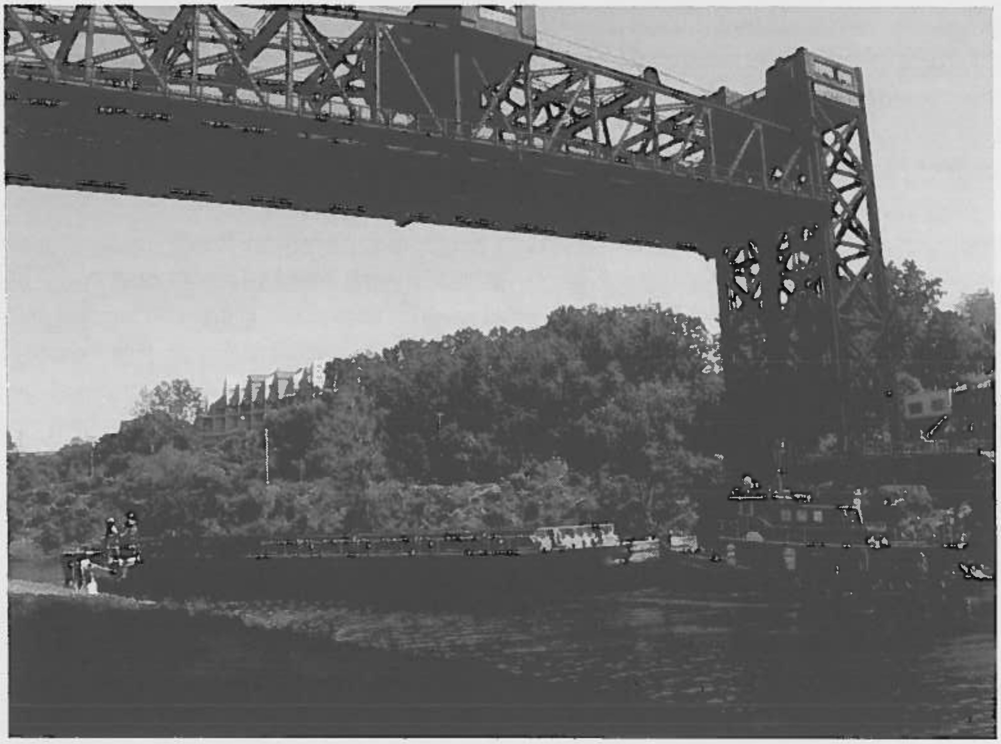

Figure 6. - Columbus Road Lift Bridge

Conclusions

Due to geologic history, the slopes along the banks of the Cuyahoga River are unstable from the mouth of the river in Cleveland, Ohio to the headwaters located 40 miles south in Akron, Ohio. The challenges to civil works (buildings, roadways, and bridges) are most prevalent in the downtown Cleveland region.

Geotechnical investigations for new civil works located on or near the slopes of this valley need to consider the "hazards" not typically encountered or addressed in conventional geotechnical investigations. These conditions make it difficult, if not impossible, to detect and predict future structure performance using information obtained from a standard subsurface investigation.

Topics of special consideration for geotechnical investigations in this area include:

1. Accurate pore pressure information may be difficult to obtain due to the presence of isolated and random trapped natural gas pockets. It has been our experience that up to half of the deep borings encountered gas pockets during drilling.

2. The creep rate associated with some of the hillsides within the river valley is sufficiently slow as to require a few years of monitoring before the measured

deflection values are greater than the typical error associated with reading the inclinometer. Thus, it is difficult to obtain accurate information about the depth and rates of inclinometer movements, as well as define the geometry of the creeping slide mass without a long-term monitoring program. Most owners and their construction schedules do not allow sufficient monitoring time necessary to detect reliable trends.

3. The strength of the soil must be tested in such a way as to model the shearing conditions applicable to the field situation. Conventional triaxial and direct shear testing of these soils would largely ***over-predict*** the available soil strength along the predefined failure surfaces. Testing should focus on quantifying the residual shear strength of the material. In most situations, the prudent engineer should analyze the slope as if multiple pre-existing shear planes are present regardless of whether or not they were encountered in a subsurface investigation due to the high likelihood of their presence and the difficulty of verifying their presence using typical subsurface investigation techniques.

4. It would be advisable, from a loss prevention standpoint, for practicing geotechnical engineers to make this information available to both their clients and the project owners for projects which are located along the Cuyahoga River Valley. It may be difficult for owners to accept the fact that geologic occurrences in this river valley have caused on-going deep-seated creep movements which may adversely affect their proposed developments in the future. However, it is better for owners to understand and to accept the risks associated with their proposed development ***before*** design and construction, than to realize it many years after from damage due to ground movements. It is also better for the geotechnical consultant and the design team.

Acknowledgments: The authors would like to thank the Ohio Department of Transportation, the owner of the I-90 Riverbed Street projects, as well as the lead members of the design teams which include Richland Engineering, Limited and Michael Baker Jr., Inc.

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Innovative Construction Techniques used at North Shore Connector Project

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Abstract:

The North Shore Connector (NSC) project will extend the Allegheny County Port Authority's Light Rail Transit System from downtown Pittsburgh below the Allegheny River to the North Shore via twin bore tunnels. Nicholson Construction was subcontracted to perform jet grouting ground improvement and support of excavation using diaphragm wall and soil mixing techniques.

Jet grouting was specified to provide break-out and break-in block sections at the tunnel boring machine (TBM) launching and receiving pits. Jet grout treatment was also used adjacent to buildings with shallow foundations and below retaining walls along the tunnel alignment. A new method for measuring jet grout column diameter based on electrical resistivity was employed on the project.

New equipment was introduced in the U.S for constructing soil mix temporary support of excavation using a soil-mix technique termed Cutter Soil Mixing (CSM) due to the use of two vertical cutting wheels that have evolved from hydrofraise technology. The system allows for continuous water tight soil mix panel construction including steel beams.

This paper highlights jet grouting and various support of excavation systems (e.g. slurry walls, cement bentonite walls and CSM walls), including new innovative techniques (e.g. Cyljet and CSM) used under the constraints of an urban environment.

1 Introduction

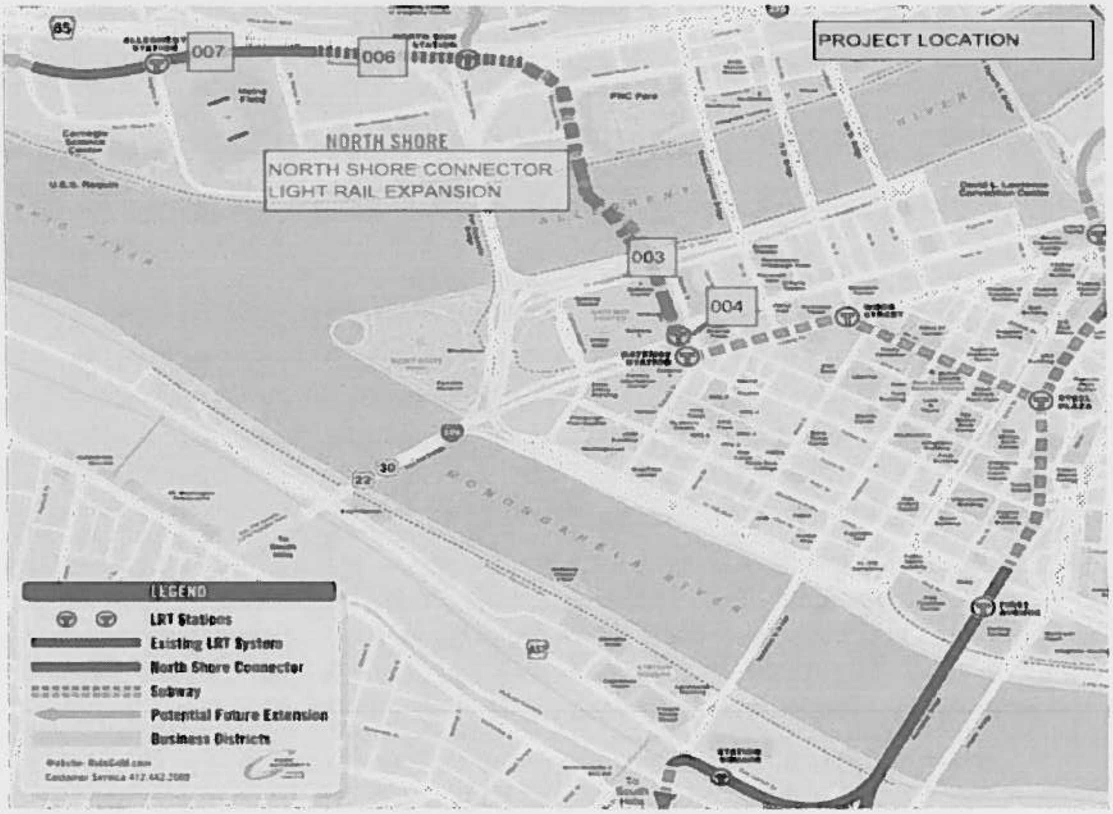
1.1 Project

The North Shore Connector (NSC) project in Pittsburgh, PA, will extend the Port Authority of Allegheny County's Light Rail Transit System approximately 1.2 miles from the Gateway Subway Station, underneath Stanwix Street and Allegheny River, and to the north shore of the river using twin bore tunnels (Figure 1). Construction is extended further on the north shore using the cut and cover method. Nicholson Construction was subcontracted by North Shore Constructors, an Obayashi/Trumbull Joint Venture (General Contractor), to perform ground improvement and construct support of excavation.

1.2 Scope of Work

Nicholson's main scope of work was as follows:

- Jet Grouting (JG): Column Confirmation Test Program (CCTP), 14 columns to select jet grout parameters. Total 32,700 cubic yard including CCTP.
- Cutter Soil Mixing (CSM): 110,000 SF (31.5-inch thick).
- Diaphragm Wall (DW): 37,000 SF (31.5-inch thick).
- Cement Bentonite Wall (CB): 22,000 SF (31.5-inch thick).



Nicholson's work at North Shore

Nicholson's work at South Shore

Figure 1. Nicholson's work at North Shore Connector

2 Geology of NSC Project

The North shore connector project is located in the floodplains at the confluence of the Allegheny, Monongahela and Ohio Rivers. The floodplain consists of unconsolidated deposits that overlie rock. Two separate types of valley soils are recognized; an upper alluvium deposit and lower fluvioglacial deposit.

The alluvium deposit is 0 to 15-ft. thick primarily consisting of silty clay or clayey sand having low strength, high compressibility, and moderate to low permeability. Typical grain size distribution of the alluvium deposit is 20% Gravel, 15% Sand and 65% Fines.

The fluvioglacial deposit is approximately 25 to 50-ft. thick overlying the rock. It consists of coarse grained glacial deposits typically having high frictional strength, low compressibility and high permeability. The fluvioglacial deposit was found to be thicker and coarser on the north side with a typical grain size distribution of 57% Gravel, 38% Sand, and 5% Fines. On the south side the deposit was found to be 40% Gravel, 50% Sand, and 10% Fines. The majority of the tunnel, both cut and cover and tunnel boring, is constructed through the fluvioglacial deposit, though the tunnel extends into the underlying bedrock below the river.

In general, the ground water table exists under phreatic condition. The alluvial and especially the fluvioglacial deposits provide a relatively permeable aquifer.

Geologic maps of the project area indicate that the top of rock at the tunnel alignment rises from elevation 660-ft. from the receiving pit at the south end to 680-ft. at the launching pit at the north end (existing ground level varies from elevation 723 to 729).

The upper layer of rock is shale with inter bedded siltstone seams to siltstone with shale foliage. A thin layer of approximately 1-2 ft. thick limestone underlies the shale. Below the limestone is a layer of calcareous clay stone. The lowermost rock unit sampled was found to be fine grained sandstone. The uniaxial compressive strength of rock and permeability of soil/rock is shown in the following Tables 1 and 2.

Table 1. Strength of Rock based on GBR

Rock Type	Uniaxial Comp. Strength (psi)	Remarks
Shale	5,400 to 11,800	Mostly
Limestone	2,400 to 11,300	Shale and
Claystone	200 to 4,300	Limestone
Siltstone	3,400 to 11,000	encountered
Sandstone	3,400 to 12,000	at NSC.

Table 2. Permeability based on GBR

Location	TBM Receiv-ing Pit	Stanwix St.	TBM Launch Pit	North Side Station
Deposit	Co-efficient of Permeability (cm/sec)			
Fill	5×10^{-5} 1×10^{-6}	5×10^{-5} 1×10^{-6}	7.1×10^{-3} 1×10^{-6}	7.1×10^{-5} 1×10^{-6}
Alluvial	1×10^{-3} 1×10^{-6}	1×10^{-3} 1×10^{-6}	2.7×10^{-4} 1.8×10^{-7}	2.7×10^{-4} 1.8×10^{-7}
Fluvio-glacial	1×10^{-2} 1×10^{-4}	1×10^{-2} 1×10^{-4}	1.4×10^{-3} 1×10^{-4}	1.4×10^{-3} 1×10^{-4}
Rock	1×10^{-4} 1×10^{-5}	1×10^{-4} 1×10^{-5}	2×10^{-4} 1×10^{-4}	2×10^{-4} 1×10^{-4}

3 Jet Grouting

Jet grouting (JG) was specified per contract to improve the ground at the launching and receiving pit of the Tunnel Boring Machine (TBM). The contract requirement was to create a jet grout block mass encompassing both tunnels (each 22-ft. diameter) and extending 6-ft. around the tunnel. Requirements were 100% coverage and 28 day minimum unconfined compressive strength (UCS) of 150-psi.

Additionally, ground treatment was specified for the east tunnel at Stanwix St. to minimize ground movement and avoid settlement of shallow foundation structures. Requirements were 75% coverage and 28 day minimum strength of 150-psi. Jet grouting was also considered below retaining walls to replace pile foundations located along the tunnel alignment.

3.1 Construction Method

Based on available data and past experience, double fluid JG system using air and grout was considered for the NSC project.

In the double fluid system the cement grout is injected at high pressure and is aided by compressed air, which helps to produce bigger column diameters.

Before beginning production, the Column Confirmatory Test Program (CCTP) was performed to select jet grout parameters and grout mixes. The test program was performed directly adjacent to the south shore of the Allegheny River.

Initial jet grout parameters were selected based on soil type, soil consistency, bulk density, atterberg limits, grain size distribution, water content and permeability of in-situ soil. 5 to 8-ft. diameter grout columns were tested during CCTP. On the basis of trial columns, the graph of jet grouting energy versus column diameter was drawn (Figure 2) to finalize the jetting parameters.

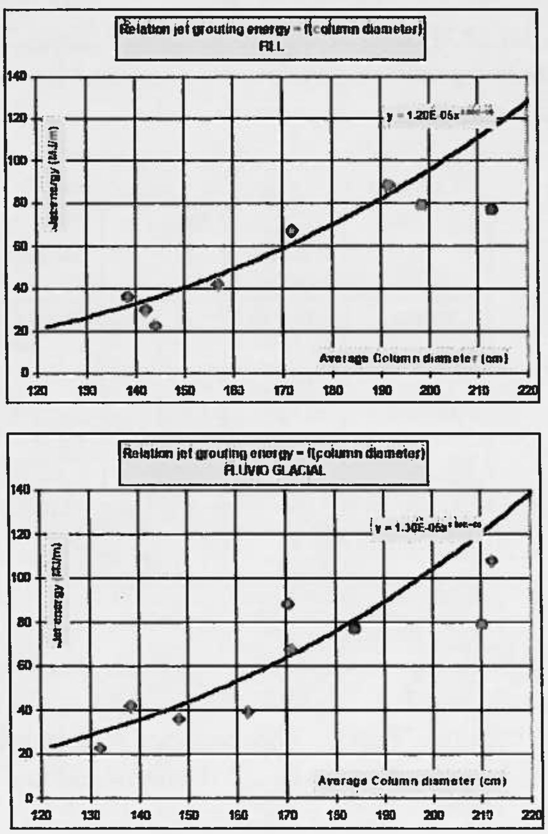


Figure 2. Energy vs. diameter of JG column based on field test

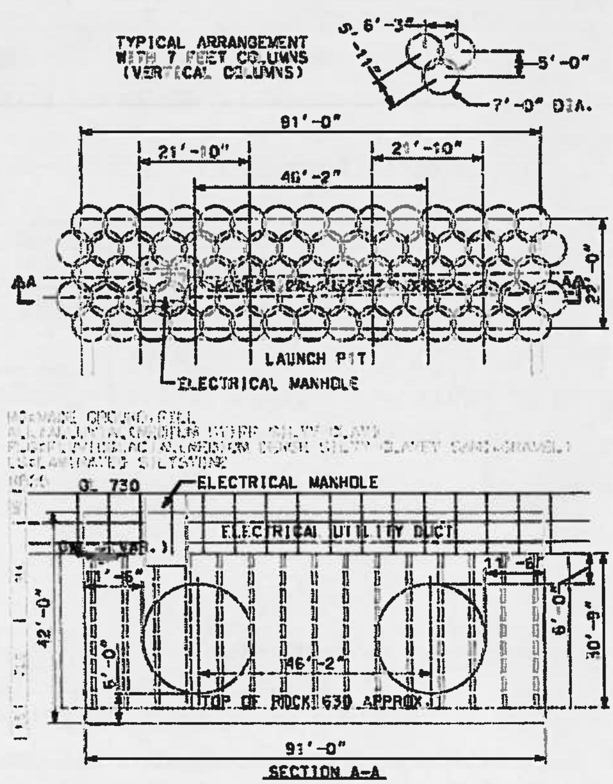


Figure 3. Layout of JG Columns at Launch Pit

7-ft. diameter JG columns were used for production (Layout of JG Columns at the Launch Pit is shown in Figure 3). The JG spoils were directly deposited to the barges and truck mounted dumpsters during CCTP and production respectively (Figure 4 and 5). The following were considered to check the effectiveness of the jet grouting method.

- Density and Unconfined Compressive Strength (collecting spoil and core samples).

- Diameter and continuity of jet grouted columns (inspecting core samples at the half radius and periphery of the jet grouted column).
- Physical inspection of the jet grouted hole (as the holes were deep and below water table open excavation was not conceived; cyljet method was proposed).

Figure 4. Spoil Management for CCTP

Figure 5. Spoil Management at Launch Pit

3.2 Selection of Grout

The grout mix for jetting was cement, bentonite, and water. Bulk ordinary portland cement (Type-II) with high yield bentonite was used to reduce bleed.

3.3 Quality Control

An innovative technique known as Cyljet, developed by Europenne de Geophysique (E.D.G) of France was used to confirm the diameter of jetted columns.

The specific gravity of grout was measured at the plant and jet grout parameters were monitored using a Lutz system. In addition, spoil samples were collected to measure specific gravity and strength testing. Cored samples from the JG columns were tested for strength requirement (results shown in Figure 15).

3.4 Cyljet Method

Jet grouted column diameter was predicted using the cyljet.

The cyljet method consists of recording and analyzing the potential differences generated by an induced electrical current around a borehole. A multi conductor cable is inserted in the borehole connects regularly-spaced current injecting electrodes (A) and receiver electrodes (M) to a computer controlled selector (See Figure 6).

In this method the potential difference is measured with the induced electric current around a borehole in cylinder 5 to 10-m in diameter. Depending on ground resistance the responses are varied. The measurements show the geo-electrical pseudo-section showing resistance of equi-potential lines. A calibration borehole is drilled and measured to know the results of the original ground. Subsequently when a measurement is performed on a freshly grouted column, the result shows the contrast in resistance between the original ground and of the soilcrete forming the column.

These resulting signals are processed by a computer and column diameters are simulated (typical result from NSC project is attached in Figure 7). The diameter of the columns is obtained by comparing various simulations with the actual field readings.

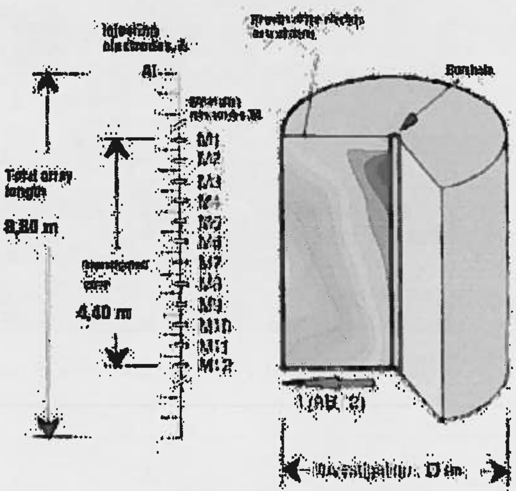


Figure 6. Cyljet measurement

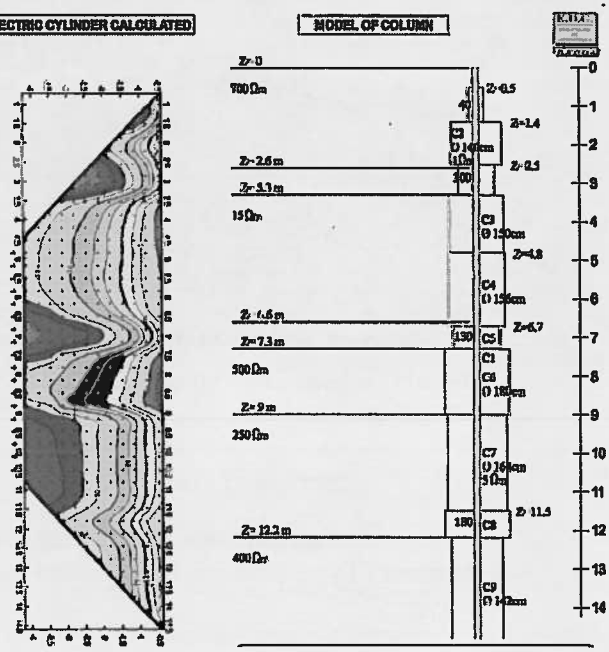


Figure 7. Cyljet Test result from NSC

3.5 Observations

The use of jet grouting was found to be an effective ground improvement system for the congested urban environment. The JG had several advantages over other ground improvement technologies:

- The ability to create large treated cylindrical elements using small drill holes.
- Small drill holes can be used to avoid utilities.
- Ability to treat specific zones.
- Different energies could be used in different soil layers.
- Flexibility of drill equipment to drill angled holes.
- Ability to control and transfer spoil directly for better spoil management under space constraints of urban construction.
- Cyljet proved to be an effective method for determining the size of the jet grouted columns.

3.5 Challenges

The presence of coarser than anticipated soils precluded the ability to obtain continuous core samples at low compressive strengths.

4 Support of Excavation

The temporary support of excavation (SOE) was specified in the contract using cement deep soil mix (CDSM) at the cut-and-cover section and also at the launching/receiving pits for the Tunnel Boring Machine (TBM). Nicholson proposed a CSM (Cutter Soil Mixing) wall. The choice of this working method was based on the geotechnical, environmental and economical reasons. Low spoil gives a particular advantage when working in restricted zones, saving the cost of expensive waste disposal. Under low head room areas, the temporary SOE was constructed using a cement-bentonite slurry.

The permanent support system for the new station shell at the north shore was constructed using diaphragm wall as contract specified.

4.1 CSM

The CSM wall is primarily deep soil mixing constructed by mixing cementitious binder with the natural soil in situ developed by Soletanache Bachy. The CSM tool is based on Hydrofraise technology used for construction of slurry walls.

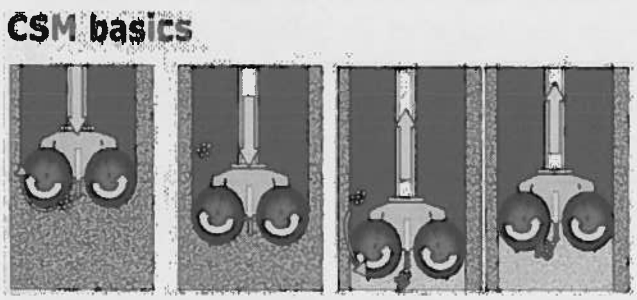


Figure 8. Basics of CSM

Figure 9. Cutting/Mixing Drums of CSM

4.1.1 Construction Method

The CSM process consists of primarily two phases. Penetration of the tool with outward rotation of the drums whilst injecting a "drilling in" bentonite slurry in between the drums, followed by inward rotation of the drums and withdrawal with continued injection employing a cementitious binder suspension (Basics of CSM method is shown in Figure 8).

The cutting and mixing drums are mounted on compact hydraulic motors. The drums are designed to combine high penetration rates and excellent soil/cement mixing. The cutting/mixing wheels are mounted on a Kelly bar supported by a hydraulic track rig (Figure 9 and 10). The Kelly bar allows accurate positioning and verticality of the soil mixed structure. The CSM machine is equipped with an onboard real time monitoring system.

The CSM walls (800 mm thick) consisted of overlapping primary and secondary panels. The panels were overlapped to account for a deviation equivalent to 1% of the panel depth (ranging from 3-ft. to 6-ft.). The design of SOE for the CSM wall at NSC was based on toe embedment in rock.

Figure 10. CSM Rig at North Shore

Figure 11. Grout Plant at North Shore

A compact automated grout mixer was used for preparation of grout for CSM wall system (Figure 11). A standard bentonite mixer and slurry tanks were used for preparing bentonite slurry. The soldier piles were installed after completion of a panel. Most of the soldier piles were embedded in the bedrock for structural purposes.

4.1.2 Selection of Grout

- The specified unconfined compressive strength of the soilcrete was minimum 70-psi and average about 150-psi. Nicholson designed the grout mix considering 190-kg of cement per cubic meter of treated soil.

4.1.3 Quality Control

Real time monitoring system (Figure 12) measured the following parameters:

- Verticality by a two axis (X, Y) inclinometer
- Depth
- Fluid flow (slurry injection, cementitious binder)
- Motor hydraulic pressure
- Hydrostatic pressure outside the tool
- Wheel rotation speed

The bentonite slurry and grout were tested during the construction for checking following parameters:

- Specific gravity of cementitious binder.
- Marsh Funnel Viscosity of bentonite slurry.
- Marsh Funnel Viscosity of cementitious binder.
- UCS samples taken from fresh in-situ soil cement mix.
- Confirmation core samples as specified by the contract.

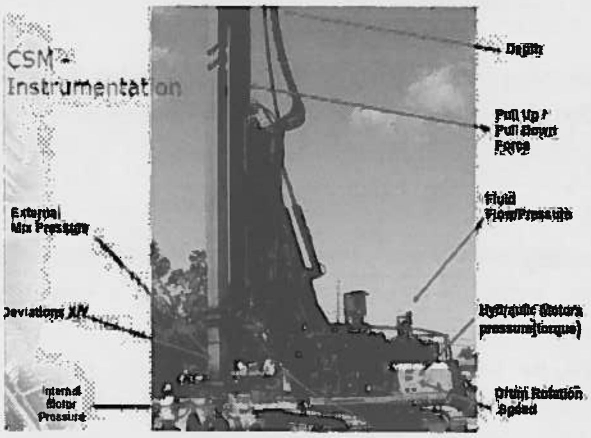


Figure 12. Instrumentation for CSM

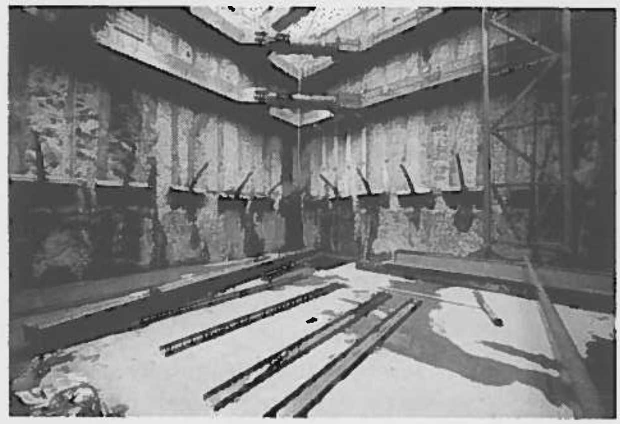


Figure 13. CSM wall at the Launching Pit

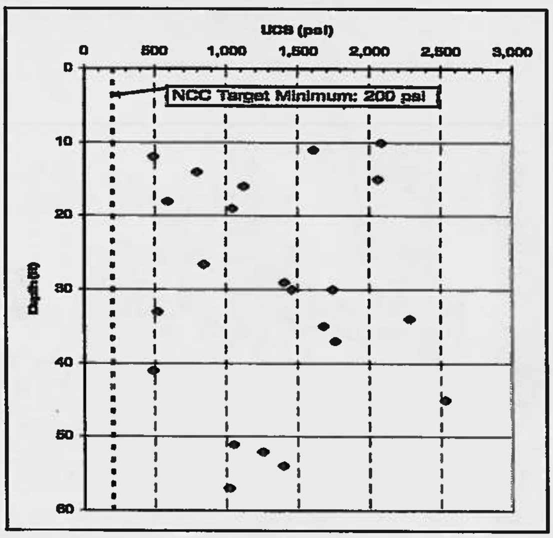


Figure 14. Cored UCS results of CSM at Launch Pit

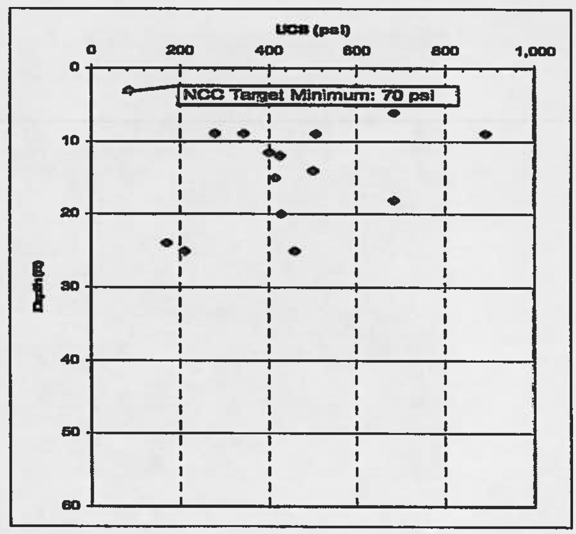


Figure 15. Cored UCS results of JG at Launch Pit

4.1.4 Observations

The CSM system was found to be an effective water cut-off and temporary SOE system for urban construction. As the prevailing soil was utilized as construction material, spoil removal was minimal. Also, spoil can be disposed off economically as solid waste. Vibration-free and low noise of plant/equipment was helpful for deployment in inner city areas.

Mixing was found more effective in the fluvioglacial deposit. In general the CSM method was found to treat soil uniformly (Exposed face of the CSM wall at Launch Pit is shown in Figure 13). The soilcrete created using CSM system reached a minimum UCS 500-psi (Figure 14). Verticality was maintained accurately using real time data management system.

4.2 Cement Bentonite Wall

Cement bentonite (CB) walls were proposed for temporary SOE under low head room areas (under highways I-279 & SR-65) along the alignment of CDSM/CSM wall using cement bentonite mixing method.

4.2.1 Construction Method

The CB walls consist of overlapping primary and secondary panels excavated using conventional slurry wall technology. The trenches were excavated using a low head room grab with bentonite slurry as stabilizing agent (Figure 16).

Once the panels were excavated to full depth, steel beams were placed and the stabilizing agent was substituted with the designed cement-bentonite grout mix. Soil disposal system is shown in Figure 17.

The bentonite slurry consisted of a uniform mixture of bentonite and water. The designed cement bentonite mix was supplied at the bottom of the excavated panel through multiple tremie pipes utilizing grout pumps.

Figure 16. CB wall below I-279

Figure 17. Soil disposal system for CB wall

4.2.2 Selection of Grout

- The grout mix for the CB walls was based on laboratory test results and past work experiences. The specified unconfined strength of the soilcrete was minimum 70-psi and average 150-psi.

4.2.3 Quality Control

In order to guide the grab during initial excavation, and to ensure the position and verticality of the CB wall, a guide wall was constructed prior to commencement of the CB walls. During excavation measurement of the position of the suspension cable of the grab with respect to the reference points to the guide wall was measured. The field personnel tested the

the viscosity, specific gravity and sand content of the bentonite slurry during and after completion of panel excavation.

Samples were also collected from the fresh grout mix for UCS test. The confirmatory test program using core samples was implemented.

4.2.4 Observations

CB walls were found to be an effective hydraulic cut-off and SOE for variable and permeable ground conditions in urban construction. Under restricted head room areas this could be constructed using low head room plant and equipment.

4.3 Diaphragm Wall

A concrete diaphragm wall was constructed as a support of excavation during temporary stages and also to form the structural walls for the new North Shore station. Diaphragm wall construction sequence (typical) is shown in the Figure 18.

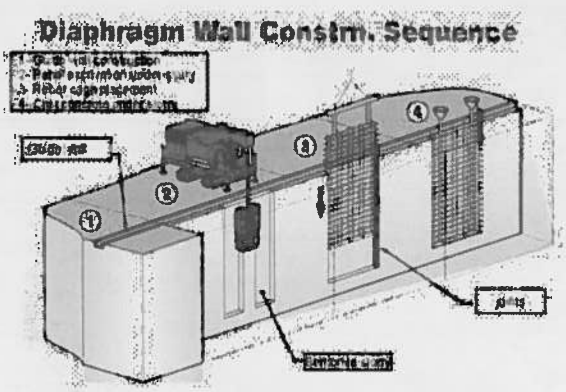


Figure 18. Construction Sequence of Diaphragm wall

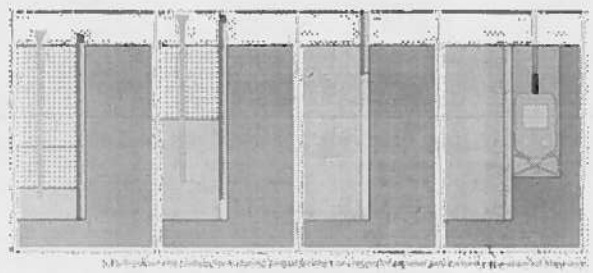


Figure 19. Construction sequence with CWS endstop

4.3.1 Construction Method

Excavation was carried out using mechanical grabs. The primary panels are constructed using two removable endstop (Figure 19). The follow up panels were constructed progressively away from the completed primary panels using one endstop. Finally, the secondary panels (located in between two completed panels) were constructed.

Once the excavation was completed up to the designated depth, the bottom of the trench was cleaned by a clamshell bucket (Figure 20). Then the contaminated bentonite slurry was pumped out and desanded. The fresh bentonite slurry was recycled until the bentonite slurry complied with the specification.

Later the reinforcement cages were lowered and the concreting was carried out using steel tremie tubes. Ready mix 4000-psi high slump concrete was used for diaphragm walls. The bottom of the tremie pipes was kept approximately five feet into the concrete to avoid contamination with bentonite.

4.3.2 Quality Control

In order to guide the grab during initial excavation, and to ensure the position and verticality of the diaphragm wall, a guide wall was constructed prior to commencement. During excavation measurement of the position of the suspension cable of the grab with respect to the reference points to the guide wall were measured. In addition to above, verticality can also be monitored using real time data management system (typical data management system shown in Figure 21).

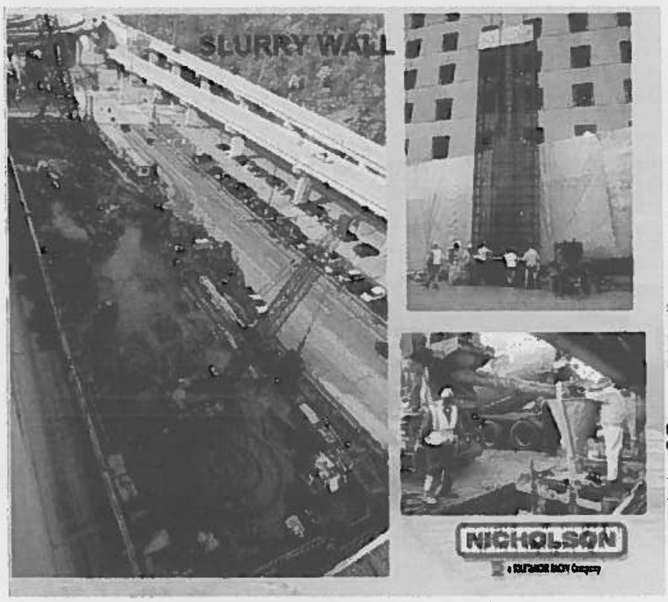


Figure 20. Diaphragm wall construction

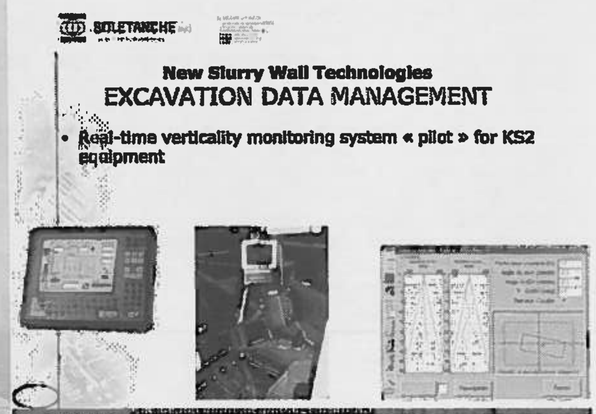


Figure 21. Real time data management system for Diaphragm wall

Field personnel tested the properties of the bentonite slurry during the excavation and concreting phase using mud balance, marsh funnel, filter press, pH meter etc.

The concrete level in the panel was measured regularly. A concrete curve giving the volume of actual concrete volume of concrete poured versus theoretical volume was measured to check unusual concrete overruns.

4.3.3 Observations

The diaphragm wall can be constructed in hard, variable and highly permeable ground condition using bentonite as a stabilizing agent. The finished wall was practically impervious. The minimum noise and vibration during construction proved the effectiveness of the diaphragm wall system as permanent SOE for congested urban environment.

5. Conclusions

Double fluid jet grouting method was found to be an effective ground improvement method in variable ground conditions at North Shore Connector. In the absence of suitable method for finding the efficacy of jet grouted soil below water table, the Cyljet method was effectively used for determining the diameter of the jetted columns.

CSM seemed to be the most attractive solution as a temporary support of excavation system for urban construction in soft ground conditions. The reduced handling of spoil, less noise and vibration during operation makes it particularly attractive in urban construction. With the advancement of plant and equipment it is likely to extend its reach further including ground improvement.

Diaphragm wall was found an effective method for permanent support of excavation system in urban construction. Innovative design and improved plant and equipment has practically eliminated the physical constraints for working under low head room and/or restricted areas.

6. Acknowledgments

The authors like to thank Chris Hynes, Fred Tarquinio and Rick Deschamps of Nicholson Construction Company for providing information and valuable suggestions for preparation of this paper.

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REAL TIME RESPONSE TO THE LANDSLIDE AT THE LEXINGTON APARTMENTS

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Abstract: In late February, 2003, a steep hillside behind three apartment buildings near Nashville, Tennessee displayed signs of instability. Evidence of movement was noted as small bulges emerged in various places along the slope. In the following days, additional movement was observed and the residents from 70 units were evacuated and relocated.

A geotechnical review confirmed that the physical symptoms were indicative of a landslide. Closer inspection revealed tension cracks developing several hundred feet up the slope and extending laterally approximately 500 feet resulting in an active slide area of about 6 acres with less than 20 feet of clearance between the moving slope and remaining unoccupied apartments.

Data derived from slope inclinometers recorded movements up to 35 feet deep and surveys confirmed that the scarp occurred at least 450 feet up the slope crossing well over the property line. Due to property line constraints, the repair was restricted to the lower half of the active slide area.

As spring rains intensified and the slide movement accelerated, the design-build retention system was converted to a build-design system as the construction moved to a fast-track schedule. An anchored soldier beam and lagging wall was constructed to create the first line of defense.

The permanent repair scheme included an initial installation of the anchored soldier beam and lagging wall augmented by two rows of reaction blocks and anchors located upslope of the wall. As the movement of the slope slowed in response to the initial repairs, some surface grading was performed and surface ditches were constructed to divert surface water from the lower landslide mass to further enhance slide stability.

This paper will cover the emergency action plan, an overview of the design considerations, challenging aspects of the construction and the unique and complicated relationships developed between multiple owners, property managers, designers, peer reviewers, adjacent property owners and the contractor.

INTRODUCTION

In 1996, a large parcel of land in Bellevue, Tennessee was investigated for the development of a multi-unit apartment complex. The overall property encompassed approximately 80 acres of moderately to very hilly terrain. The topographic relief in the general vicinity of the development is on the order of 320 feet. The original geotechnical investigation was performed to provide recommendations for foundation design for the structures and included discussions pertaining to retaining walls. During site grading in the fall of 1996, two or three shallow slides occurred behind two of the proposed buildings at the base of the steeper portion of the site. These slides were reportedly repaired by excavation of the loose slide material and replacement with shot rock that was capped with topsoil. After the repairs, building construction resumed and the development was completed in 1998.

In December 2002, the development was sold to an investment firm and a separate property management firm was retained. Following a somewhat wetter than average winter, significant surface bulging and seepage from the slope became apparent. With only fifteen to twenty feet separating the apartments from the toe of the slope, the management staff called for the evacuation of three of the buildings, a total of 70 occupied units. Representatives of the new ownership and management teams convened at the site to interview geotechnical firms and initiated a study. As the study was being completed and the options reviewed, record rainfalls occurred at the site and the slide began a more rapid advancement toward the buildings. On May 11, 2003, the toe of the slide came in contact with one of the buildings. The slope movement cut off all site drainage behind the buildings and many of the ground floor apartments were flooded. A local grading contractor had been on standby and immediately mobilized to begin excavating slide material away from the structures.

Within a few days, the geotechnical contractor was retained to provide a design/build solution, mobilized to the site and began to install soldier piles near the base of the slope as a temporary barrier to protect the structures as the team members brainstormed to determine the composition of the final repair scheme. The detailed design process began a few days following the construction process. As the repair proceeded, the design was constantly altered to adjust to the changing field conditions within the limits of the property.

As the movement of the slide slowed following installation of the soldier beams, other components of the repair could be installed. These included the temporary shot rock buttress, permanent tiebacks for the soldier beam wall, subsurface drainage components at the wall and within the slope, ground anchors and surface drains. Finally, after four months on-site, the remediation was completed and the site cleaned up. The slopes were seeded and strawed, the pavement was repaired in the work area and along the driveways, fences were installed and post-construction monitoring systems were implemented.

GEOLOGY

The topographic setting of the Bellevue area of Nashville, Tennessee is represented by undulating hills that occasionally represent significant changes in elevation that are a result of ancient weathering of the geologic profile. Often, the erosion process exposes several formations of different character and composition. In this case, the site is underlain by limestone of the Liepers and Catheys Formations. However, the bad actors are the Fort Payne and Sequatchie Formations that are present at the higher elevations adjacent to the site. Those formations typically weather significantly and produce variable thicknesses of colluvium along the lower portions of slopes. The composition of these gravity-deposited soils generated from the weathering of the upper formations is different than the makeup of the residual soils overlying the Liepers and Catheys Formation. In this region, the colluvium is usually somewhat permeable, contains wide variations in grain size in the soil fraction and often contains numerous rock chips and chert fragments. As a result of their method of deposition, colluvial soils in this setting usually exhibit preferential bedding planes and, therefore, are often pseudo stable. The residual soil above the Liepers and Catheys is usually slightly silty to silty stiff clay with occasional chert fragments. It is not uncommon for a thin zone above bedrock to be water bearing and much softer.

While the weathering of the Sequatchie and Fort Payne Formations produces colluvial soils, its presence is not ubiquitous along the lower slopes. Occasionally, terraces along typically steeper hillsides are a hint of colluvial deposits. However, the soils maps produced by the US

Department of Agriculture are often a very reliable indicator of colluvial soils. In this case, the Dellrose soils contain many of the characteristics of colluvium produced by the geographically higher parent bedrocks. As can be seen in Figure 1, the landslide occurred well-within the boundaries of the mapped Dellrose soils.

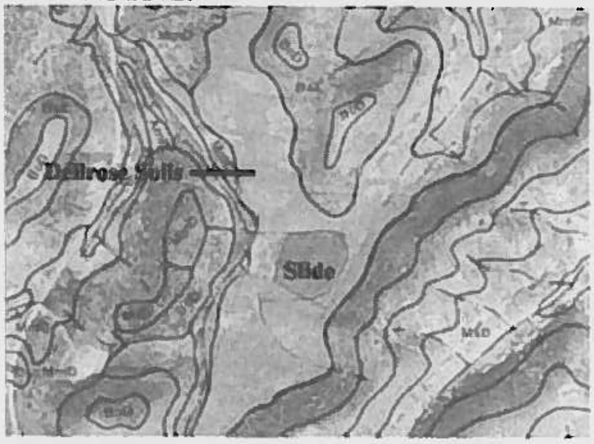

Figure 1. Geologic setting

Other early warning signs and historical evidence of previous slides were noted along the hillside. An older roadbed that transected the hillside and bounded on each end by very deep drainage swales was very likely a graben, or wedge shaped plunge into the subsurface at the upper end of a significant, historical slide. The character of the upper end of the current slide was nearly identical in its early stages to the nature of that feature. As the slope began to seep water and initiate small bulges, a 2 to 3 foot scarp became evident approximately 450 feet up the hillside.

Figure 2. Early warning signs, small bulges in the slope.

TRIGGERS

The landslide had a number of factors that contributed to its failure. As previously discussed, the thick deposit of colluvium had a significant impact on the ultimate instability of the slope. The natural, historic slides increased the risk of future sliding due to relic features embedded within the profile. Since the colluvial deposit at the site appears to have been naturally pseudo stable, the steepening of the toe of the slope increased the probability of failure. The bottom 150 feet of the slope was excavated from a 4H:1V slope to a 2H:1V slope to facilitate construction of the apartments. The lower portion of the slope was also denuded of trees.

As frequently is the case, water played a very important role in the loss of stability. The winter precipitation was above normal and the local colluvium absorbs water rather quickly but doesn't discharge it as readily. This causes an increase in weight of the sliding mass and a decrease in shear strength, decreasing the factor of safety of the slope. In addition, the downward sloping strata created by the method of deposition contribute to the easier development of slide planes. Moreover, the previous repair work actually exacerbated the problem rather than helping it. These zones of shot rock buried in the slope created permeable pockets that readily filled with subsurface water, added to the weight and allowed the infiltration to penetrate deeper into the subsurface soils. Also, a diversion ditch constructed at the top of the cleared area was so flat that it actually retained water rather than discharging it. On or around the week of May 11, 2003, over 10 inches of rain was recorded in the area. The slide had already begun moving, but the large volume of available water and the considerable inflow into the open tension cracks rapidly accelerated the movement of the slide.

ANALYSIS

The initial geotechnical investigation of the slope consisted of conventional borings and test pits within the slide area. A copy of the original geotechnical report was also reviewed for pertinent information. Topographic and geologic maps were studied and a set of aerial photographs were taken to study the area in detail. A survey crew was mobilized to record changes in the slope surface and to map the location and extent of visible tension cracks. Inclinometers were installed into bedrock in most of the borings and readings were taken almost immediately. Feedback as to the depth of slide planes was almost instantaneous. Unfortunately, the life span of the slope inclinometer casings was very short with most of the casings deflected beyond use within one week (Figure 3).

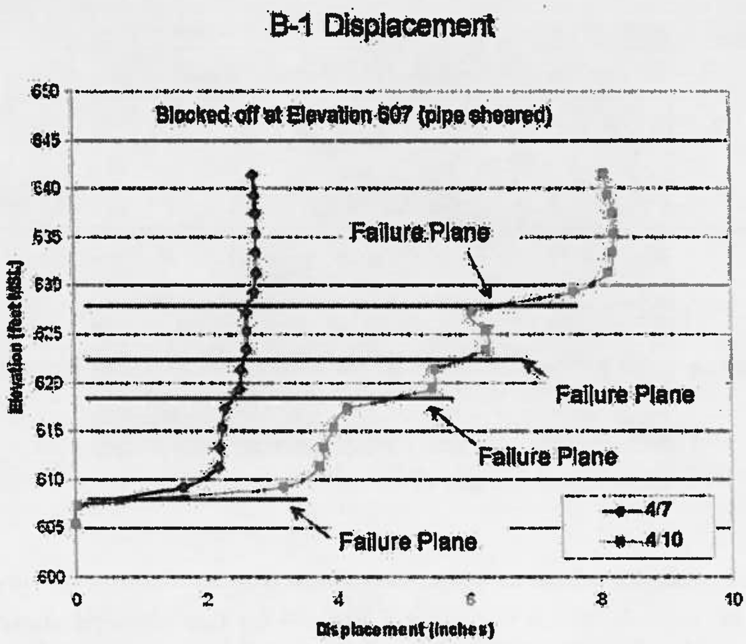

Figure 3. Typical slope indicator results

Nonetheless, the zones of movement identified in the inclinometers enabled an estimate of soil parameters by back-calculation using the profiles developed with the recently acquired data.

Subsequent to the development of probable failure surfaces, a series of options was developed:

- Do Nothing (demolish the buildings and nominally stabilize the slope)
- Move Buildings (nominally stabilize the slope)
- Excavate Slide Material/Replace with Engineered Fill
- Excavate to Construct a Shot rock Buttress
- Gravity Retaining Walls
- Deep Vertical Drains
- Soldier Beam and Lagging Wall(s)
- Reticulated Micropile Wall(s)
- Soil Nails
- Ground Anchors

Once the apparent magnitude of the slide was established and the likely degree of repair required, the ownership team performed its set of financial analyses to accompany the developing repair development. Cost/benefit calculations were performed to compare repair costs to the value of the units and length of time to recover the outlay of capital.

The evaluation of options was somewhat challenging due to the size and complexity of the team. The property was owned by an investment fund that was managed in New York City and had construction oversight out of its Chicago office. They retained peer review services from a geotechnical firm in Chicago. The fund's principal stakeholder was located in the Middle East. The property management company was located in Atlanta with oversight from Baltimore and construction review from West Virginia. The contractor was located in Atlanta and their geotechnical designer was in Pittsburgh. The geotechnical firm providing data and options was located in Nashville. Additionally, the slide crossed a property line onto a parcel owned by a local investment group. Needless to say, communications were at times complicated and sometimes slow.

PROPOSED SOLUTION

The initial plan was to stabilize the entire slope with ground anchors. The number and location of anchors was being developed as the slide began its accelerated movement. Consequently, the construction of a temporary soldier beam and lagging wall was initiated to slow the progress of the slide and protect the buildings until the ground anchors could be installed. Figure 4 illustrates the advancing slide materials during wall construction.

Figure 4. Soldier Beam and Lagging Wall

However, as the magnitude of the slide grew, it became necessary to make the wall a permanent feature and include tiebacks for added stability, as shown in Figure 5. The resulting final design is illustrated on Figure 6.

Figure 5. Permanent Wall

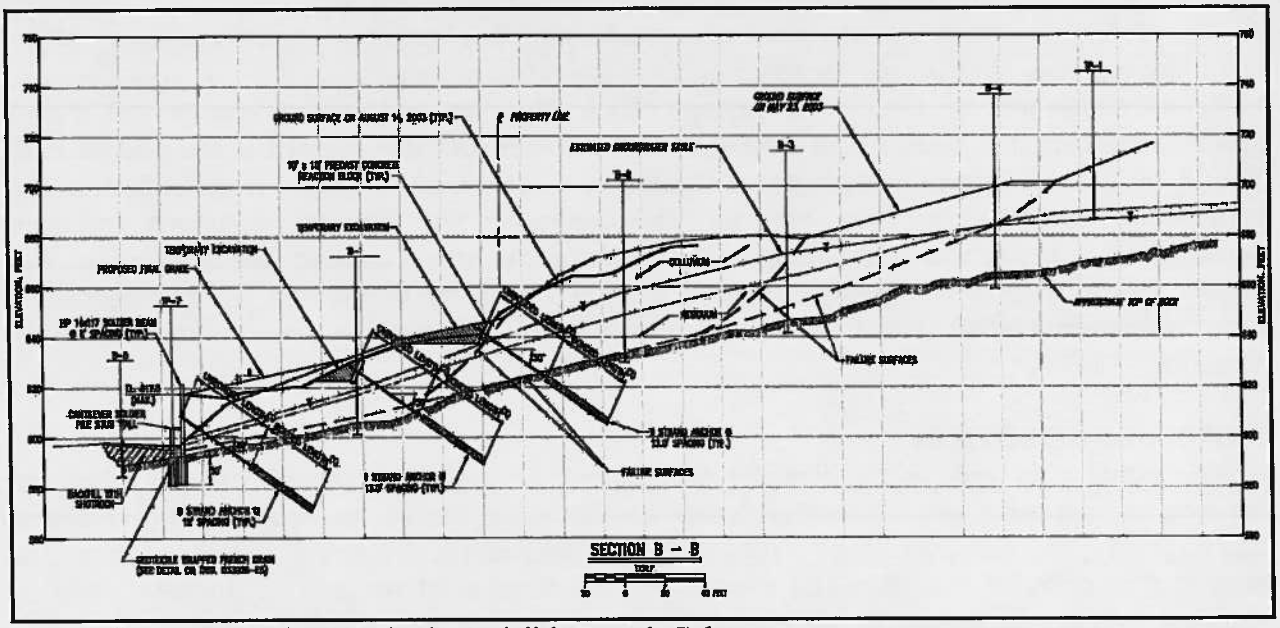

Figure 6. Cross-Section of Final Landslide Repair Scheme

A temporary shot rock berm was placed in front of the soldier beam and lagging wall to create a buttress for short term support and also provide a working platform for the installation of tiebacks. When the adjacent landowners decided not to participate in the repair, the solution had to be modified to be constructed entirely on the apartment complex property. Two additional rows of ground anchors above the wall were analyzed and deemed satisfactory provided that a subsurface easement was obtained and the top row of anchors was allowed to extend onto the adjacent property. A small cantilever soldier pile stub wall was constructed in front of the anchored soldier pile wall to provide additional support at locations where the soldier beams were distressed by slope movements prior to installation of the anchors. Other ancillary features to the anchors and wall included several rows of subsurface drains and several tiers of surface ditches to divert water around the repaired area. In addition, the surface drainage in front the wall was considerably improved.

EMERGENCY RESPONSE

During the evaluation phase while the ownership considered their options, the landslide continued to move in response to frequent rainfall events. The magnitude of movement appeared to be in nearly direct correlation to the magnitude of precipitation. For about two weeks, the slide moved between a few inches and a foot per day. During this period the apartment staff removed appliances and other valuable objects from the ground floor units, and in the case of one building, from all three floors. As the slide advanced toward the buildings, minimal excavation was performed to maintain surface water drainage behind the buildings. Following some increased rain in the first part of May, movement increased; causing an active slide of about 6 acres, and the grading contractor mobilized his crew on Sunday morning, May 11 (Mother's Day). Several pieces of tracked excavation equipment and a small fleet of dump trucks were delivered to the site to begin several days of 24-hours per day excavation to save the buildings. Rock was hauled in to create stable haul roads as mud and excavated slide materials were hauled out to create access to work. During this time, the slide movement increased to about 25 feet per day. The contractor had his eye on the slide and everyone else had theirs on the weather forecast.

As shown in Figures 7 & 8, the movement of the slope blocked surface drainage behind the units and significant flooding occurred inside several of the ground floor apartment units.

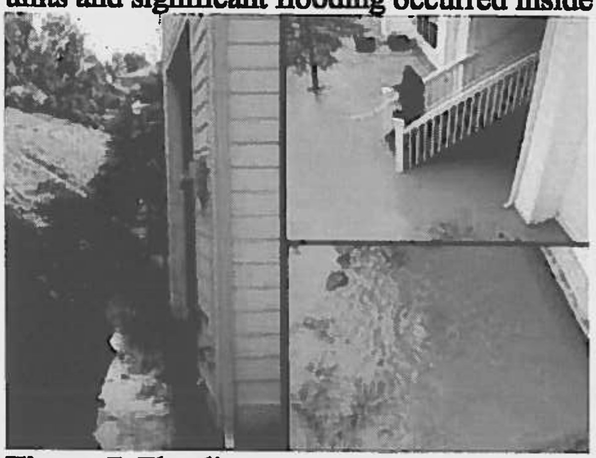

Figure 7. Flooding

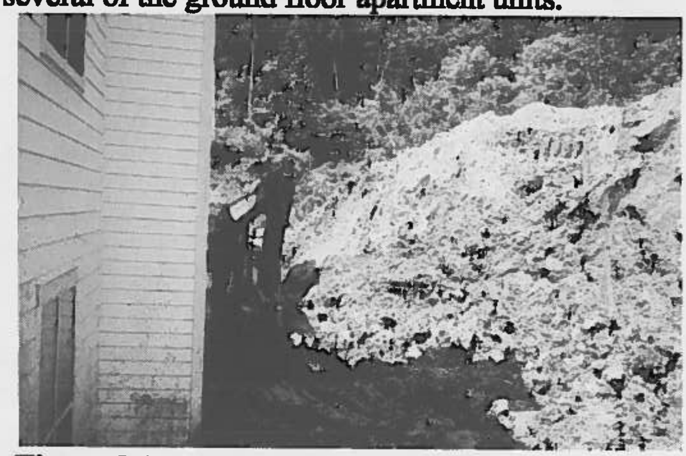

Figure 8. Emergency excavation of slide material

Pumps were installed to control the accumulating surface water. Also during this time, the utility companies were notified to disconnect service behind the buildings. The gas company was resistant to do the disconnect believing that their plastic lines would be flexible enough to tolerate the movement. Following the insistence of the ownership team, the service was isolated from the area just before the "flexible" gas line was sheared.

The specialty contractor was able to quickly procure HP 14 X 117 soldier beams and had deliveries arriving on site within a few days. Heavy equipment capable of drilling 24-inch diameter holes through soil and into the shallow bedrock arrived by the end of the week and the soldier beam installation began. The moving soil had to be excavated back from the soldier pile line several feet to allow installation of the piles before it rapidly advanced back past the construction line. Likewise, the soil required over excavation to allow installation of the wood lagging. No backfilling was required since the soil quickly moved into place behind the lagging!

DESIGN

The design process required consideration of the progressive failure (Figure 9) that was occurring in the slope.

Figure 9. Surface expression of progressive failure

Analysis was additionally complicated by the changing geometry of the slope as it moved downslope. The interim profile presented in Figure 10 illustrates some of the subsidence in progress in the higher elevations with corresponding bulging occurring in the lower slope.

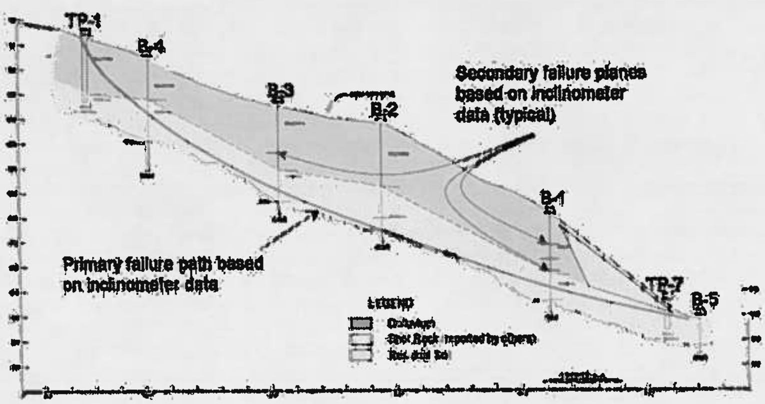

Figure 10. One of the many profiles of the slope geometry developed as the slide progressed

The most effective parameters used for analysis were derived from back calculation using the location of the slide plane from the inclinometers and the slide surfaces exposed in the construction, such as the failure surface seen in Figure 11.

Figure 11. Exposed Slide Plane

The final parameters used for the design of the repair components are presented in Table 1. Water levels were extrapolated from the exploratory data.

Table 1. Design Parameters

Stratum	Angle of Internal Friction, Φ	Cohesion; psf	Moist Unit Weight, γ_m	Saturated Unit Weight, γ_{sat}
Colluvium	27 degrees	0	115 psf	130 psf
Slickensided Residual Soil	15.7 degrees (back calculated)	0	120 psf	130 psf
Limestone Bedrock	45 degrees	5000	170 psf	170 psf

As the initial slope model seemed to approximate the conditions observed in the field, loading conditions were assessed for the design of the soldier beams, reaction blocks and the ground anchors. The slope stabilization system was designed to increase the forces resisting sliding by 30 per cent so as to increase the factor of safety of the slope from approximately 1.0 to 1.3. The required allowable horizontal resistance to be provided by the anchored reaction blocks and anchored soldier pile walls to achieve this goal was calculated to be 72 kips per linear foot. The anchors and soldier piles were designed to bond in the underlying limestone bedrock. Anchor length varied from about 70 feet to over 85 feet. Anchor design loads were on the order of 316 kips each.

As the design considered the several different loading conditions, features to control the subsurface water were designed and implemented. Several rows of subsurface drains were installed parallel to the slide. The subsurface drain intended for the base of the soldier beam and lagging wall was constructed in front of the wall because of the dangers to personnel and equipment due to the advancing slide.

CONSTRUCTION

Construction in front of and on a moving slide (occasionally rapidly moving) presented several challenges. Close coordination between the wall construction and the excavation was critical to construct each segment of wall in front of the slide. Similarly, coordinated excavation and lagging placement was required to successfully create the wall, especially in light of the narrow work area between the slide and the apartment buildings. During this time, the magnitude of lateral movement of the slide was vivid illustrated when one of the inclinometer casings was excavated from between the soldier beams. The casing had originally been installed over 100 feet upslope of the anchored soldier pile wall.

The presence of the existing apartment buildings presented a unique opportunity to observe and modify the design and construction "on the fly". An available internet connection allowed a webcam with pan and zoom capability to be installed high on one of the buildings which provided observation and communication of conditions to anyone on the design team or a member of the ownership and property management team. In several instances, a person at the slide site could be in contact by cell phone to the designer and project manager to review a condition that could be observed remotely using the webcam. Even though the connection was severed several times due to either the slide movement or equipment contact, it was quickly repaired. Time lapse video also provided interesting data for observation and analysis.

The parking lot provided a convenient staging area for the equipment and materials. An assembly line for construction of pre-cast anchor blocks (Figure 12) and lagging panels supported the field activities behind the buildings.

Figure 12. Finished Anchor Blocks

Some of the vacant apartments (safely located in nearby structures) were used as construction offices and for housing of the construction personnel.

The emergency excavation of slide material and the importation of shot rock took its toll on the roads and parking lot in the complex. Over 2000 loads of earth were hauled off site and nearly 500 loads of shot rock were imported. The heavy axle loads were much higher than the design loads for the infrastructure. Consequently, additional cost was incurred to repave a significant portion of the complex as construction wound down.

NON-TECHNICAL COMPLICATIONS

Design and construction is often complicated by factors that are non-technical. So, in addition to determining design parameters in a frequently changing environment, the final construction had to consider a number of non-technical factors. The landslide was massive; it incorporated approximately 6 acres, half of which occurred upslope on an adjacent landowner's property. After several meetings, the adjacent landowner, whose property was adversely affected by the slide, chose not to participate in any repairs on his side of the property line even though he wanted to maintain the potential for upslope development. This meant that the apartments had to be protected by a repair that was constructed only on the lower half of the slope and that the design assumptions could change if and when the upslope property is developed. It also required that a subsurface easement be obtained to allow placement of some anchors as high as possible within the slide.

As with many urban construction sites, the landslide remediation team had to deal with complaints about noise, dust, lights, etc. A fence was constructed around the construction zone to minimize intrusion by residents, and fall protection was required above the soldier beam and lagging wall to keep children from exploring around the wall.

Since the property was acquired as an investment, the potential for sale of the property was a possibility. In order to plan for the future sale, comprehensive records were maintained to document the decision trail and the design. That documentation proved to be very effective when used recently for a smooth sale of the property.

POST-REPAIR MONITORING

In order to monitor the performance of the hillside subsequent to the remediation, another series of inclinometers were installed in the slope. These devices have been read with declining frequency and have demonstrated satisfactory post-construction performance. A less sophisticated early warning system was also installed to allow the groundskeeper or other non-technical staff to report basic levels of performance. Simply, several rows of steel fence posts were very accurately installed in the surface of the slope in straight alignment. A visual sighting along the rows of posts can indicate possible surface movement if the alignment changes. Based on recent observations, slope movements at the site have ceased.

CONCLUSIONS AND SUMMARY

The rapid movement of the landslide often required immediate communication and decisions amongst the property ownership, the designers and the construction personnel. The slope stabilization scheme was able to be constructed in the timely manner required to save the apartment buildings located near the toe of the slope. Installation of the web cam facilitated instant feedback on the status of the slope and visualization of construction hurdles by the remote team members to enable quick modifications to the design to accommodate the changing conditions. The repair scheme was designed to use materials that were available for immediate delivery at a reasonable cost and increase the slope's resistance to sliding by 30 per cent. The slope repair scheme has been successful in stabilizing the slope and has allowed residents to occupy the affected apartment buildings.

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Which in-situ test should I use?—A designer's guide

Abstract: In-situ tests can greatly increase the volume of geomaterial investigated at a foundation site, with savings in both cost and speed when compared to sampling and lab testing. Historically, they have been developed to evaluate specific parameters for geotechnical design. Some tests directly measure the response to a particular type of load, such as a plate load test or a pile load test. These tests verify design assumptions, and possibly determine soil or rock properties by inversion. The two most common in-situ tests, the Standard Penetration Test and the Cone Penetrometer Test, primarily identify soil type and stratigraphy, along with a relative measure of strength. Interpretation of these two tests may also utilize indirect correlations with specific soil properties, but typically with high statistical variability (partly due to inherent testing variability, partly due to ignoring the soil's stress history test, and partly due to crude empiricism). Other tests, such as the Iowa Borehole Shear Test, the Dilatometer Test, and the Pressuremeter Test, attempt to directly measure in-situ the soil properties that might be otherwise determined from laboratory tests of "undisturbed" (more accurately termed "intact") samples. Stress-path variations, disturbance effects due to insertion of the test device, and alternative test procedures may affect the results of these tests. The research literature contains numerous correlations between in-situ test results and various geotechnical parameters. To use these correlations with reliability, the engineer must understand their basis and potential for error, and then choose the in-situ test(s) that provide the most reliable correlation(s) for the desired soil properties and design parameters. In general, this requires a test that closely models the intended design use or directly measures the soil properties required for design. This paper examines the commonly available in-situ tests and provides testing recommendations for specific geotechnical design applications.

Introduction

In-situ tests generally investigate a much greater volume of soil more quickly than possible for sampling and laboratory tests, and therefore they have the potential to realize both cost savings and increased statistical reliability for foundation design. Many well-written technical papers and manuals have previously discussed and compared the various in-situ tests, e.g. Schmertmann (1975) and Kulhawy and Mayne (1990). This paper presents an overview of the available in-situ tests and points out some important details often overlooked by practicing engineers. After discussing different geotechnical foundation design needs, it provides recommendations to help the engineer choose the most appropriate in-situ test to satisfy the design requirements of specific types of foundations.

Available In-Situ Tests

The following sections briefly discuss the basic details of the available in-situ tests, and some important, yet sometimes unrecognized, details. Additional information can be found in technical papers shown in the references and the Standard Test Methods available from ASTM International.

Standard Penetration Test (SPT), ASTM D 1586, D 4633, and D 6066: While the standard penetration test is probably the most common in-situ test performed in North and South America, the term "standard" is misleading. Although the test is relatively simple to perform, only skilled drillers routinely achieve meaningful results. In 1902, C.R. Gow designed a 1-inch diameter heavy-wall sampler to be driven with a 110 pound weight. In 1927, L. Hart and G.A Fletcher developed the standard 2-inch-diameter "split-spoon" sampler (Figure 1). Later, Fletcher and H. A. Mohr standardized the test using a 140-pound hammer with a 30-inch drop to measure the blow count for three consecutive 6-inch increments of penetration, reporting the total blow count for final 12 inches as the N_{SPT} value. Terzaghi and Peck (1948) published early geotechnical design correlations, which popularized the SPT and encouraged its acceptance as a "standard".

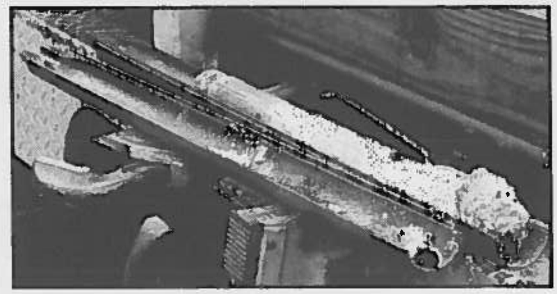

Figure 1: Split spoon SPT sampler

The three styles of SPT hammer in common use (see Figure 2) deliver energy to the drill rods that varies from about 35 % to 95% of the theoretically available driving energy of 4200 in-lbs. This variation, plus the use of non-standardized drilling techniques, led Schmertmann (1978) to investigate their effect on the value of N_{SPT}, which he found to exceed a factor of two. In addition, Schmertmann (1979) also found that N_{SPT} varied approximately inversely in proportion to the hammer energy delivered to the drill rods. With the advent of modern computers, energy measurement devices allow technicians to easily measure the actual driving energy entering the rods as described in ASTM D4633. The engineer can then correct the

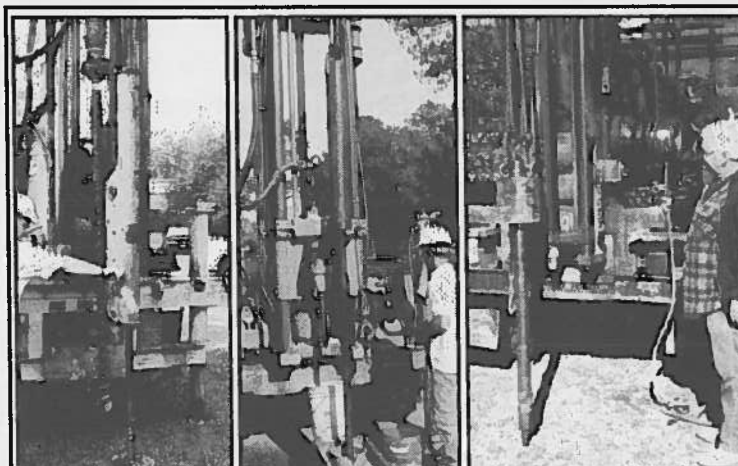

Figure 2: a) Automatic Hammer ~95% eff.,
b) Safety Hammer ~60% eff.,
c) Donut Hammer ~35% eff.
(photo from GeoServices Corp.)

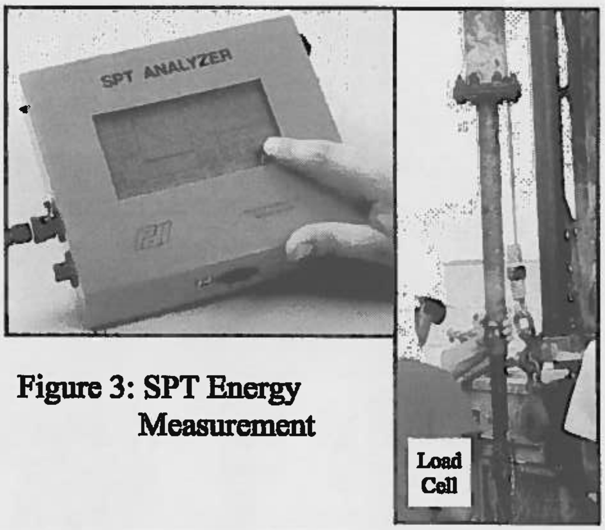

Figure 3: SPT Energy Measurement

measured value of N_{SPT} to N_{60}, the equivalent blow count at 60% of the theoretical hammer energy (thought to represent the average energy in the correlation database). Skempton (1986) presented a method to compute N_{60} values from raw N_{SPT} data, which is incorporated in ASTM D 6066.

Unfortunately, N_{60} values rarely appear on boring logs. The barrel on the old samplers had the same inner diameter as the shoe. Today, an alternative sampler barrel in common use has a larger inside diameter to accommodate liners with an inner diameter the same as the shoe. However, liners are rarely used - Skempton suggests multiplying the N-value by 1.2 for this correction. Automatic trip hammers, now in widespread use, may deliver almost 95% of the theoretical energy if well-maintained. For these hammers, a correction of 1.58 may be needed to get N_{60}. Without making the N_{60} correction, designs tend to be overly conservative and costly. Even with the best techniques, predicting how the soil responds to static structural loading based on the results of a dynamic test can be highly inaccurate.

Dilatometer Test (DMT), ASTM D 6635: In 1975, Dr. Silvano Marchetti invented the Flat Dilatometer, consisting of sharpened blade with a circular membrane located on one side, to investigate H-pile behavior for lateral loads. He performed tests at ten well-documented research sites and developed empirical correlations with classical soil properties. In 1980, he published a classic paper presenting those correlations; most of which are routinely used today. In 1981, Marchetti traveled to the United States on sabbatical and worked with Drs. John Schmertmann and David Crapps. While they were initially skeptical of Dr. Marchetti's invention, they were convinced by the impressive speed and accuracy of the results.

Figure 4: DMT Blade

Figure 4 shows a photograph of the stainless steel Dilatometer blade under a direct push rig. The blade, 15 mm thick and 96 mm wide in cross-section, is pushed into the soil at a constant rate of 2 cm/sec, preferably using a load cell to measure the penetration thrust as shown in Figure 5. Generally the operator stops penetration at 20 cm depth intervals, records the thrust at the test depth using a load cell, and then inflates the membrane.

The surrounding soil usually collapses the 60-mm-diameter stainless steel membrane flush against the blade during the penetration. (In very weak soils, a vacuum must be applied prior to pushing.) Electrical conductivity between the center of the membrane and the underlying body of the blade completes a circuit that activates a buzzer and a light on the dilatometer control unit. To run the test, the operator slowly inflates the membrane with nitrogen gas supplied from the control unit. When the membrane center moves away from the blade, the electrical continuity is lost and the light and buzzer go off. At that instant the operator reads the gas pressure at the control unit and records the membrane lift-off pressure as the "A-pressure" on the data sheet. The operator then continues to inflate the membrane. When the membrane has inflated an additional 1.1 mm at its center, an electrical switch inside the blade reestablishes the electrical circuit and reactivates the buzzer and light, prompting the operator to record the corresponding gas pressure as the "B-pressure". When below the water table, the operator can slowly deflate the membrane, and record the water pressure that pushes the membrane back in contact with the blade as the "C-pressure". Nearly all of the correlations are based on the thrust, "A-pressure" and "B-pressure". The "C-pressure" can be used to determine the groundwater table in clean sands and to determine the undrained shear strengths of soft clay (Lutenegger, 2006).

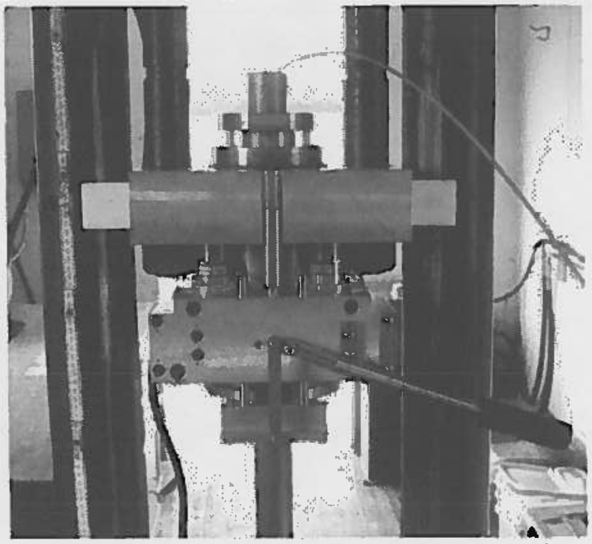

Figure 5: Push Clamp using Four Load Cells to Measure Thrust

The dilatometer blade has a cross-sectional area of about 14 cm^2 and can be pushed with a direct push rig into soil with an N_{60}-value of about 45 blows per foot or with a heavy drill rig into soil with an N_{60}-value of about 35 blows per foot. Tests can be successfully performed in all penetrable soils, including clay, silt, and sand. If the soil contains a significant amount of gravel, there may be point contacts against the membrane instead of a continuous medium, causing inaccurate results. Furthermore, the gravel will often tear a hole in the membrane.

DMT results have been correlated with the parameters that geotechnical engineers need the most -- soil shear strength and deformation properties. The computer program for the dilatometer data reduction evaluates and outputs the following soil properties and parameters:

- Tangent vertical constrained modulus [M],
- Undrained shear strength for clays [c_u],
- Drained friction angle for sands [ϕ'],
- Total unit weight of soil [γ_t],
- Coefficient of lateral earth pressure at rest [K_o],

- Preconsolidation pressure [p_c], and
- Overconsolidation ratio [OCR].

Cone Penetrometer Test (CPT), ASTM D 3441 and D 5778: The mechanical cone penetrometer probe, invented in The Netherlands in 1932 by P. Barentsen, measures the quasi-static thrust required to push a solid, conical tip having a 60 degree apex angle and a cross-sectional area of 10 cm^2 into the foundation soil. The operator advances the cone using a nested, dual-rod system, the outer rods providing strength to penetrate the cone in a collapsed configuration, and the inner rods allowing him or her to advance only the cone tip at each test depth (generally at 20-cm intervals) while measuring the hydraulic thrust pressure at the top of the rods. In 1953, Begemann modified the probe to include a friction sleeve just behind the tip. For the friction cone test, the inner rods initially advance only the tip for a short distance, and then engage both the tip and a friction sleeve together. The center of the friction sleeve is located 20 cm above the tip, and the value of unit soil adhesion acting on it is computed by subtracting the tip-only thrust force (from the previous test depth) and dividing by the sleeve area of 150 cm^2. The engineer then divides the unit tip bearing from the previous test depth by the unit adhesion to determine the friction ratio (both readings then apply to the same depth), and uses an empirical chart to identify the type of soil. Depth plots of unit bearing and friction ratio also provide a relative profile of the site stratigraphy.

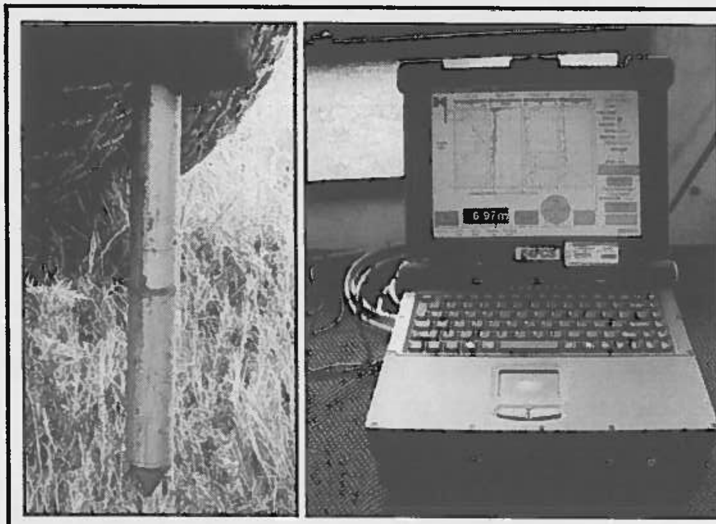

Figure 6: CPT and Data Acquisition Computer

The improvement of electronics and computers in the 1980s led to the development of stainless steel electrical cone penetrometer probes that obtain and record more reliable test measurements and eliminate the dual-rod system (Figure 6). Strain gauges are used to measure the tip and friction values and a pressure transducer measures the pore water pressures generated during penetration. With the electric cone, data are collected at penetration increments of 0.5 cm to 5 cm depending on the computer acquisition system, such as the one shown in Figure 6. Engineers prefer the electronic cone's accuracy and productivity, relegating the mechanical cone to profiles containing strong materials that might damage the more expensive electrical cone. The cone penetrometer can be pushed with a direct push rig into soil with an N_{60}-value of about 50 blows per foot or with a heavy drill rig into soil with an N_{60}-value of about 40 blows per foot.

Engineers have obtained reasonable accuracy in correlations between the CPT unit bearing and soil strength parameters, such as friction angle and undrained cohesion (see

Lunne, et al., 1997). More indirect correlations with at rest coefficient of lateral earth pressure, modulus, and overconsolidation ratio are much less reliable due to the significant effects of stress history and the in-situ state of stress. The addition of pore pressure measurements, generally made just behind the tip, to the electrical cone (CPTU) improves stratigraphy profiling and various indirect correlations. By collecting data at close depth intervals, thin layers are detected. Two correlation charts are used to identify the soil type: Friction Ratio (R_f) vs. Corrected Cone Bearing (q_T) and Pore Pressure Ratio (B_q) vs. Corrected Cone Bearing (q_T). Generally, the pore pressure ratio correlation chart is more sensitive to thinner layers, while the friction ratio chart is better for cohesionless soils. When there is a discrepancy in soil type between the two charts, either pore pressure dissipation tests or sampling can be used to identify the correct soil type.

Pressuremeter Test (PMT), ASTM D 4719: Louis Menard began his work with the pressuremeter test in 1954 while still a college student, studying first under Professor Kerisel in France, and later under Professor Ralph Peck at the University of Illinois. Menard improved and advanced a foundation test concept begun by Kogler in 1933, and then returned to France in 1957 where he started a company to build and use the PMT. He compiled a large data base of load tests and companion pressuremeter tests to refine his empirical design formulas and persuade other engineers to use the PMT. To show his confidence and encourage acceptance of the test, Menard guaranteed foundation designs based on the PMT with $10,000,000 of professional liability insurance from Lloyds of London (Hartmann, 2008).

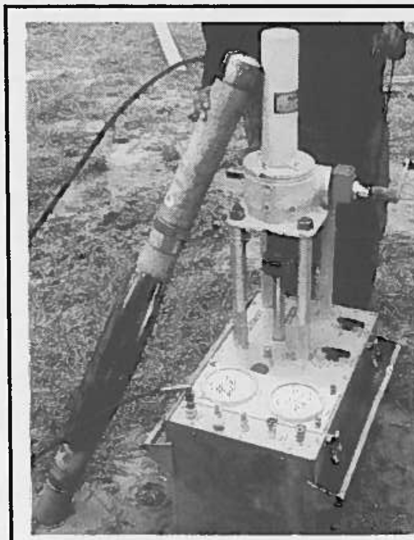

Figure 7: Texam PMT

The PMT is typically performed by inserting a cylindrical probe into an open borehole, supporting it at the test depth, and then inflating a flexible membrane in the lateral direction to a radial strain of as much as 40% depending on the probe design. The PMT operator may expand the pressuremeter probe in equal pressure increments (stress controlled test) or in equal volume increments (strain controlled test), typically stopping the test when initial volume of the probe has doubled or when reaching the maximum allowable pressure. About 40 data points are obtained from a strain controlled test versus and about 10 data points from a stress controlled test, thus a better defined curve can be obtained from strain controlled tests. Creep tests can be performed near the yield point of the test to evaluate time effects of the modulus. Ideally the PMT provides an axisymmetric, plane strain test (the horizontal plane), typically drained in sands and silts, and undrained in cohesive soils. Early PMT probes employed guard cells at their top and bottom to force the measurement cell located between them to expand only in the lateral direction. Briaud (1992) showed that the error in test results did not exceed 5% for single-cell probes (Texam in Figure 7) with a length at least six times its diameter.

Researchers have also used self-boring and push-in probes with some success in specific types of soils. Probes may also be designed with very stiff membranes for testing at high pressures and lower strain in soft rock.

The PMT results include the at-rest horizontal earth pressure, the pressuremeter elastic modulus, the reload modulus, and the pressuremeter limit pressure (plastic failure), but generally require an empirical approach for foundation design or for correlation with classic geotechnical parameters such as the shear strength or Young's modulus. While the PMT stress path can be modeled theoretically, the effects of stress history and anisotropy, testing in the direction of the minor principal stress (usually) in a material with behavior controlled by confining stress, and the disturbance of stress release and softening at the borehole wall (or stress increase for push-in probes), usually lead to an empirical approach. Good test results begin with a high quality borehole having minimal disturbance to its side walls, typically requiring mud wash rotary techniques. Maintaining the drilling mud level at or near the top of the borehole minimizes the horizontal stress release from drilling. During drilling, the operator should carefully monitor the rotation rate, advance rate, and mud flow rate to obtain a high quality borehole.

Modern data acquisition systems speed field testing and computer programs relieve the drudgery of data analysis, but the PMT remains one of the most labor-intensive and time-consuming in-situ tests. Pressuremeter tests are particularly valuable in dense sands, hard clays and weathered rock, if the DMT and CPT cannot penetrate those formations. Pressuremeter tests can also be used in remote sites that only skid rigs can access.

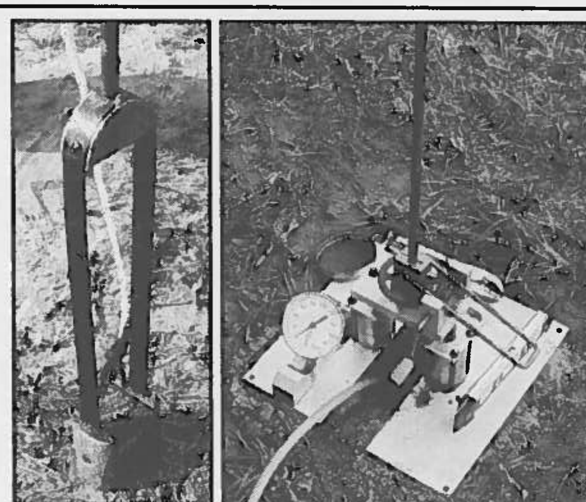

Figure 8: BST shear head (left) and shear dynamometer (right)

Iowa Borehole Shear Test (BST): While the shear strength of soils can be critical for the design of earth slopes, the calculation of earth pressure against retaining walls, and the determination of foundation bearing capacity, it can be a time consuming and expensive to measure with laboratory shear tests. The BST (Figure 8), developed by Dr. R.L. Handy at Iowa State University, provides a convenient method to accurately measure the drained shear strength of soils in-situ. Tests typically require between 30 and 60 minutes to perform, and the results are immediately available. It is similar to a laboratory direct shear test with the sides of the borehole being sheared.

To perform the BST, the operator inserts the shear head into borehole into a 3-inch diameter borehole to the chosen test depth. A normal stress is then applied to push apart two serrated stainless steel plates (total area 10 in^2), pressing them laterally against the sidewalls of a borehole. After allowing the soil to consolidate at the applied normal stress, usually between 5 minutes for cohesionless soil and about 10 to 20 minutes for cohesive soil, the operator pulls the shear head upward to measure the shear strength of the soil in contact with the plates. This shear test is typically repeated four to five times at progressively higher normal stresses to prepare a plot of normal stress versus shear strength. In sands, silts, and stiff clays, the BST provides a drained test, while results for softer cohesive soils may be partially drained. An available pore pressure sensor located in the shear head can provide an indication of drainage. Because the same soil is tested, the data can usually be fitted linearly with a coefficient of correlation of 0.99 or better.

For soils with an N_{60} value of 15 or more blows per foot, the smaller set of plates (total area 1.6 in^2) should be used to ensure that the plates are fully embedded into the soil. Because the pressure gauges are calibrated to measure the stress of the larger (standard size) plates, for the smaller plates the recorded pressures must be multiplied by 6.25 to account for the differences in the plate areas.

An oversize borehole can adversely affect the accuracy of the test results, as can loosening or softening of the borehole sidewalls. A borehole prepared with a76-mm (3 inch) diameter Shelby tube usually tends to minimize disturbance. Hand augers are also a good choice for more remote locations. Boreholes prepared using mud-rotary drilling methods will reduce the shear strength until the normal stress causes the shear heads to penetrate through any mud-caking.

Research is being performed to evaluate the residual shear strength in over-consolidated clays. After measuring the peak shear strength value, the BST plates are collapsed and lowered back to the starting depth for the data point. A normal stress equal to about 90% of the peak normal stress is then reapplied to the clay and the plates are pulled upward to the ending depth of the peak value. The resulting shear stress is recorded. This procedure is repeated until the shear stress becomes a constant value. An example set of residual borehole shear test data is shown as Figure 9.

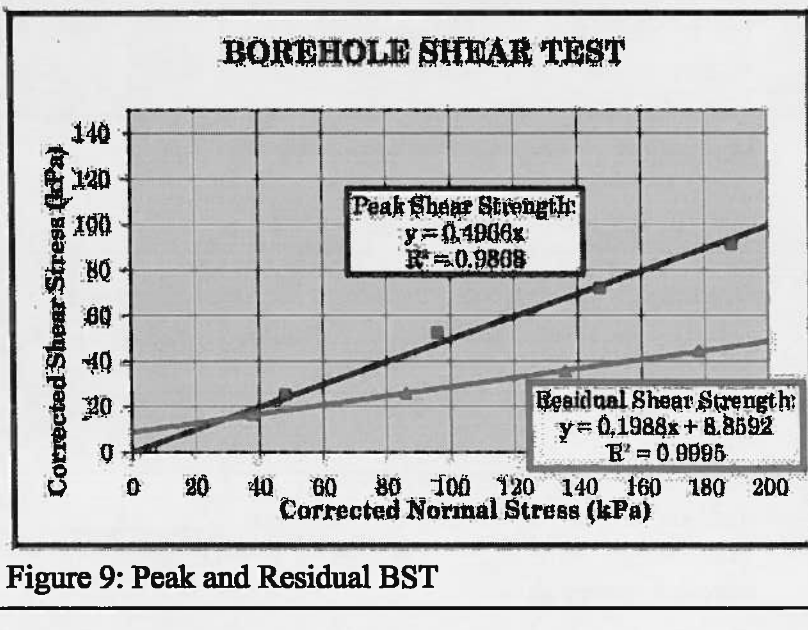

Figure 9: Peak and Residual BST

Dr. Handy also has developed a rock borehole shear test (RBST) device to measure the shear strength properties of rock. The device is quite robust and can apply a normal stress of 80 MPa and a shear stress of 50 MPa. The device is placed inside of a cored borehole, and the test is conducted using hydraulic pressure to apply the normal stress and to pull the plates for the shear stress. A shale or siltstone is likely to be smeared during the test and, after each data point, the plates are rotated axially by 45^0 for the next normal stress, obtaining a maximum of four data sets. With granite, the rock is likely to chip during each shearing. The rock shear device will probably need to be removed from the borehole and the rock chips cleaned from the device. The device should then be lowered to about 5 mm above the previous shear depth for the next test data set.

Ko Step Blade (KSB): While engineers can estimate the vertical stress of soil relatively well, they cannot estimate the horizontal stress. The coefficient of horizontal stress, K, ranges from 0.2 to 6 times the vertical stress (Schmertmann, 1985). When a vertical force is applied to the soil, it is resisted by the soil in three dimensions, two of which are horizontal, emphasizing the importance of the horizontal stress.

Unfortunately, horizontal stresses are difficult to measure. When we drill a hole, we remove them. When we push a device into the soil, we tend to increase them in looser soils and may decrease them in denser soils. Soil sampling causes too much disturbance for the engineer to measure horizontal stresses with laboratory tests.

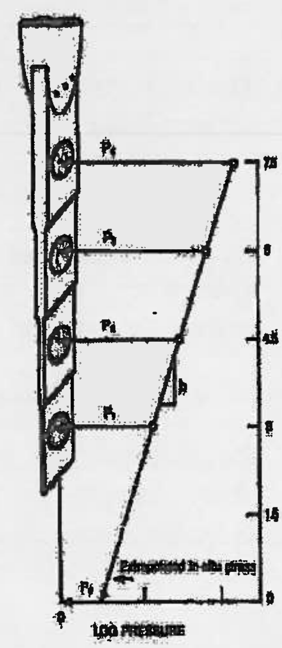

Figure 10: Ko step blade (Handy, 2004)

The K_o step blade was invented to measure this difficult to obtain soil parameter. The blade contains four steps going from thin to thick from its bottom to top (Figure 10). At each step there is a circular membrane that is exerted outward, measuring the soil's horizontal stress. It is recognized that even the thinnest step causes disturbance to the horizontal stresses when it is pushed into the soil. At the desired test depth, the engineer measures the horizontal stress of the soil for each blade step. By plotting the blade thickness versus the log horizontal stress, engineer can extrapolate the horizontal stress at a zero blade thickness. The documented accuracy of this method is $\pm$10% (Handy, 2008). (Note that the maximum 7.5-mm-thickness of the K_o step blade is half that of the 15-mm-thick DMT blade.)

Vane Shear Test (VST), ASTM D 2573: This test accurately determines the undrained shear strength of purely cohesive soils by rotating a small vane having four blades (Figure 11) around its vertical axis to fail a cylinder of soil in torsional shear. The friction acting on the rods must be subtracted from the total torque applied at the top of the rod string, but nearly all test equipment is designed to make this subtraction. Vane size can be varied to allow testing a range of soil strength using the same torque head.

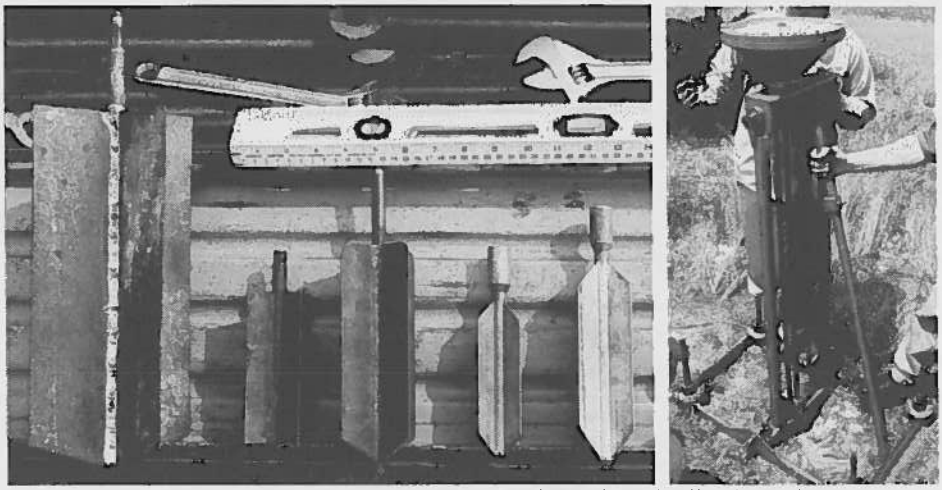

Figure 11: Various size VST shear heads (left) and manual penetration rig with torque head (right)

The undrained shear strength of clay, s_u, can be obtained directly from the maximum torque (T_{max}) by the simple equation:

$$s_u = 2T_{max} / (\pi D^2 H) \quad \text{(ignoring end effects)} \ldots\ldots (1)$$

By 1972 Bjerrum had realized that, when used in stability analyses, the vane s_u did not always give a factor of safety of 1.0 when failures had occurred. He recommended correcting the vane undrained shear strength using the following equation:

$$s_{u(field)} = s_{u\,(vane)} \times \mu, \quad \text{where } \mu = 1.7 - 0.54 \log PI\% \ldots\ldots (2)$$

By continuing to turn the vane blades five to ten revolutions, the residual undrained shear strength and the resulting sensitivity of the soil can also be readily determined. Note that sand, silt, or fibrous (roots or peat) inclusions disrupt the cylindrical failure surface around the vane, leading to erroneous results.

Falling Head Permeability Test (FHPT) or BAT outflow: In 1984, Torstensson invented a probe with a discrete filter (Figure 12) for performing outflow tests and inflow tests that also served to collect groundwater samples. Wilson and Campanella (1997) showed that the filter can clog with inflow tests, which can lead to inaccurate permeability measurements, particularly in more permeable soils. They also replaced Tortenson's hyperdermic needles with 0.375 inch diameter quick connects so that the more permeable soils could be tested. The test is similar to the laboratory falling head permeability test and uses Boyle's-Mariotte's law as the basis for the computations.

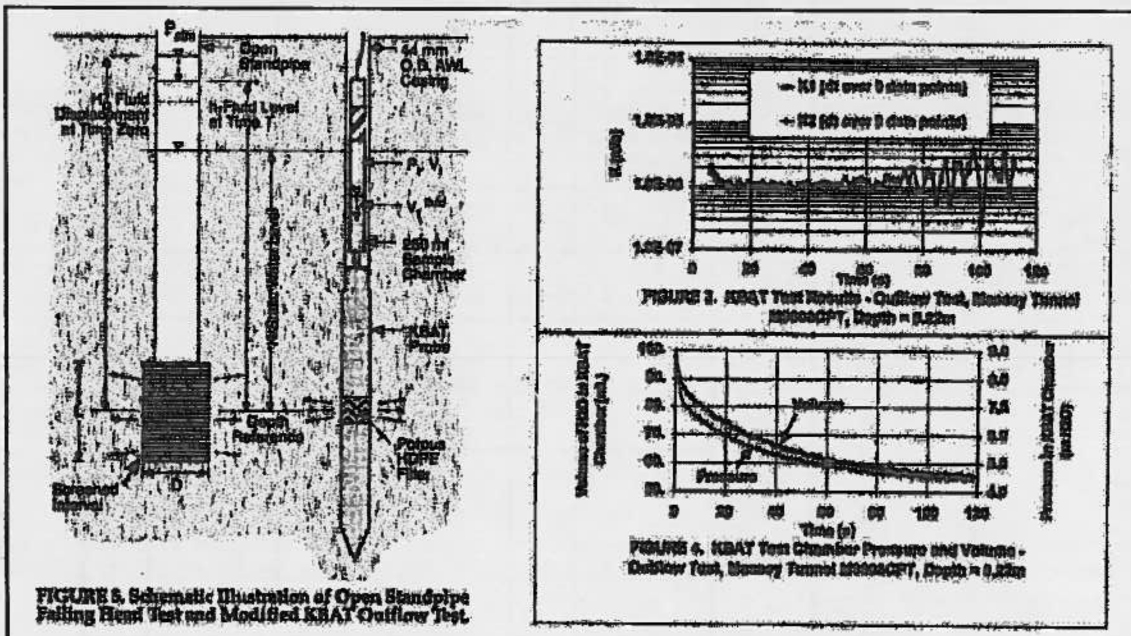

Figure 12: Falling head outflow test probe and typical results (Wilson, 1997)

Design Guide for Geotechnical Engineering

In following sections, the most appropriate in-situ test(s) is recommended for specific design applications. Table 1 summarizes these recommendations.

Table 1: Summary of Geotechnical Engineering Design Guide of Appropriate In-Situ Tests

Geotechnical Design Application	Soil Type	Suggested In-Situ Test Ranking (1 => most appropriate; 3 => least appropriate & N/A => not applicable)							
		SPT	DMT	CPT	PMT	BST	KSB	VST	FHPT
Shallow Foundations -settlement	cohesive	N/A	1	2[1]-3	1	N/A	N/A	N/A	N/A
	cohesionless	3	1	2[1]-3	1	N/A	N/A	N/A	N/A
-time rate of settlement	cohesive	N/A	2	2	N/A	N/A	N/A	N/A	1
-bearing capacity	cohesive	3	1	1	2	1	N/A	1	N/A
	cohesionless	3	1	1	2	1	N/A	N/A	N/A
Slope Stability	cohesive (total stress)	3	1	3	N/A	N/A	N/A	1	N/A
	cohesive (effective stress)	N/A	N/A	N/A	N/A	1	N/A	N/A	N/A
	cohesionless	3	1	2	N/A	1	N/A	N/A	N/A
Ground Improvement	cohesive	3	1	1[1]-2	1-2	N/A	1-2	N/A	N/A
	cohesionless	2[1]-3	1	1[1]-2	1-2	1-2	1-2	N/A	N/A
Deep Foundations -axial capacity	cohesive	1	2	1	1	2	N/A	2	N/A
	cohesionless	1	2	1	1	2	N/A	N/A	N/A
-lateral capacity	cohesive	3	1	N/A	1	N/A	N/A	3	N/A
	cohesionless	3	1	N/A	1	N/A	N/A	N/A	N/A

Note #1: Requires site specific correlations

Shallow Foundations

The engineer should always prove that a shallow foundation will not adequately support the load before recommending a deep foundation or ground improvement. Shallow foundations should be designed for sufficient bearing capacity and tolerable settlement. Bearing capacity depends on the soil's shear strength, while settlement depends on its deformation modulus. The settlement criterion generally controls design provided that the footings are wide enough.

Settlement: Dilatometer and pressuremeter tests are static deformation tests and reliably measure the soil's static deformation modulus. Both tests can provide an initial tangent modulus representing a strain level in the elastic range of loading (about 0.5 to 1%), similar to the working load that most structures impose on soil. Menard developed empirical formulas for the PMT, based on numerous case studies, to compute settlement from the pressuremeter initial modulus. Settlement analysis with the DMT follows a more traditional approach, applying an elastic estimate of the expected stress increase profile to the DMT profile of the vertical modulus.

Schmertmann (1986) presented a method to compute settlement based on DMT results. He demonstrated the accuracy of the method in 16 case histories. Hayes (1986) had additional case studies again validating the method. In Schmertmann's "ordinary" method settlement is simply calculated using the following equation:

$$S = (\Delta\sigma)\,(\Delta H)/\,M \quad \text{...} \quad (3)$$

Where S is the settlement; $\Delta\sigma$ is the increase in vertical stress; ΔH is the layer thickness and M is the constrained deformation modulus measured with the DMT. Because the modulus value is in the denominator of this equation, one cannot simply average the modulus value. Rather, the engineer should use the modulus from each test depth to represent a thin layer (thickness = test depth interval) at that depth. Schmertmann (1986) also presented a "special" method that attempts to account for lightly overconsolidated soils in which the modulus may decrease when the applied load exceeds the preconsolidation pressure. Generally, these two methods agree within 10%.

Penetration tests, such as the quasi-static CPT and the dynamic SPT, strain the soil to failure and therefore provide strength parameters that represent failure. The ratio of stiffness to strength increases significantly as overconsolidation increases (past stress history). As a result, modulus correlations with strength extrapolated from plastic (failure) behavior to elastic behavior necessarily include significant scatter and are usually chosen very conservatively. Site specific correlations with more accurate lab or in-situ tests can prove useful to reduce this conservatism.

Plate and conical test loads are methods to test the soil response to a directly-applied foundation stress. Plate load tests, usually a square plate 1 ft on a side, may need to be performed at several depths if the stress bulb from the plate is much smaller than the footing stress bulb. The conical test load (CTL) places the base of a cone of gravel or fill material directly on the surface of the test location, resulting in a full-scale stress increase beneath the center of the conical load. It is a convenient proof test and should be used more frequently (Schmertmann, 1993).

The time rate of settlement: In cohesive soils, excess pore water pressure is developed when the CPT or DMT probe is pushed into them. When the penetration stops, those pressures decrease. As the excess pore water pressure decreases, the engineer can measure the pressure and elapsed time. Like laboratory consolidation tests, the time for 50% dissipation to occur is computed and this value is needed to compute the coefficient of consolidation and coefficient of permeability in the horizontal direction, c_h and k_h. However, the method includes many correlation coefficients, making the accuracy of the method about one order of magnitude. A better method is the field falling head permeability test or KBAT outflow test, which provides a direct measurement of permeability using Boyle's law.

Bearing Capacity: The bearing capacity for the foundation can be evaluated using classic formulas, which have form similar to Meyerhof equation (Das, 1998):

$$q_u = (q_c + q_q + q_\gamma) = cN_c\lambda_{cs}\lambda_{cd}\lambda_{ci} + qN_q\lambda_{qs}\lambda_{qd}\lambda_{qi} + \tfrac{1}{2}\gamma BN_\gamma\lambda_{\gamma s}\lambda_{\gamma d}\lambda_{\gamma I} \quad \ldots\ldots\ldots\ldots\ldots (4)$$

where:

q_u	= ultimate bearing capacity
c	= cohesion
q	= stress at depth of foundation = γD_f
γ	= average unit weight of soil under footing (effective unit weight if submerged)
B	= width (or diameter) of foundation
$\lambda_{cs}, \lambda_{qs}, \lambda_{\gamma s}$	= shape factors, based on footing plan dimensions
$\lambda_{cd}, \lambda_{qd}, \lambda_{\gamma d}$	= depth factors, based on width and embedment
$\lambda_{ci}, \lambda_{qi}, \lambda_{\gamma i}$	= load inclination factors, based on inclination
N_c, N_q, N_γ	= bearing capacity factors, based on friction angle

The engineer must evaluate the soils' shear strength to calculate the bearing capacity. The BST can accurately measure the drained shear strength properties. The DMT provides the friction angle in cohesionless soils by back calculation based on the thrust measured during penetration and the normal stress and side shear acting on the DMT. In cohesive soils, the DMT provides well-documented correlation with the undrained shear strength. For the CPT, the friction angle is fairly well correlated with tip resistance based on tests performed in large triaxial chambers. The undrained shear strength of cohesive soils is also commonly correlated with the CPT tip resistance using a factor that varies between 10 and 20, depending on the geology and sensitivity of the clays. Shear strength correlations with SPT N-values tend to be conservative and crude.

The bearing capacity can also be predicted using empirical correlations with the net limit pressure from pressuremeter tests using the following formula:

$$q_{ult} = (k)(p^*_{Le}) + q_o \quad \ldots\ldots\ldots\ldots\ldots (5)$$

Where q_{ult} is the ultimate bearing capacity,
k is a pressuremeter bearing capacity factor,
p^*_{Le} is the equivalent net limit pressure near the foundation level, and
q_o is the total stress overburden pressure at the foundation level.

Note that for soils exhibiting strong anisotropic behavior, the orientation of the failure plane developed during an in-situ test may prove important for predicting the shear strength along the failure plane for the footing. The DMT, BST, PMT, and VST force a failure in the vertical plane and are sensitive to lateral stress variations, which can be beneficial to the bearing capacity analysis. The CPT and SPT cause failure to occur due vertical loading and may provide a better model of the actual load behavior.

Slope Stability: Slope stability analyses generally address two limit states, total stress (undrained) and effective stress (drained). Sands are generally permeable enough to be considered as drained with the exception of earthquake or other dynamic loading conditions. For clays, the engineer should analyze the slope using both drained and undrained shear strength properties. Overconsolidated clays tend to have high undrained shear strengths and are more critical when using drained shear strengths, while normally consolidated clays tend to have lower undrained shear strengths and are more critical when using undrained shear strengths. Overconsolidated clays often have residual shear strengths that are significantly lower than peak shear strengths. Residual strengths should be used in the analyses when there are preexisting failure surfaces or slickensides that are oriented in the direction of the critical failure surface.

The BST is the only in-situ test that measures the drained shear strength of cohesive soils. Some preliminary testing has been performed to measure the residual shear strengths by repeatedly shearing the soil. The BST can also accurately measure the drained angle of internal friction for cohesionless soils, provided that there are no particles larger than 1 cm in diameter.

The DMT and CPT can also provide reasonable measurements of the friction angle for cohesionless soils, while SPT data provides very conservative estimates. The VST and DMT provide good estimates for the undrained shear strength of cohesive soil; the CPT is correlated with undrained shear strength, which depends on correlation coefficients that typically range from 10 to 20; and the SPT again tends to provide conservative estimates. As noted above for bearing capacity, orientation of the failure plane may again prove important for strongly anisotropic soils.

Ground Improvement: Often soils are improved so that the structure can be safely supported on shallow foundations by previously inadequate soils. Loose granular soils are densified, usually by a dynamic method. Soft cohesive soils are usually preloaded, often using wick drains to shorten the consolidation time. The end result is that soils' deformation moduli and shear strengths are increased. Often soils are tested before and after the improvement effort to evaluate its effectiveness. The SPT variability and relative insensitivity to ground improvement changes make it a relatively poor choice for this type of testing, and lab testing of field samples cannot provide the quantity of data required to verify improvement of the overall mass of material.

In cohesionless soils, ground improvement techniques often both increase lateral stresses and compact the soil. These changes lead to both a greater friction angle and increased stiffness as any excess pore pressures rapidly dissipate. They also may encourage an "ageing" process that further increases the shear strength and stiffness. The amount of improvement that occurs depends on the dynamic effort and the distance away from the dynamic source. The improved soils will be fairly heterogeneous in both the vertical and horizontal directions. A large number of tests are needed to confirm that the soils have been adequately improved at all desired locations.

In-situ tests with high shear strain and disturbance effects measure ground improvement poorly because they destroy the improvement during the test. Because the DMT, and possibly the PMT, accurately measure both the soil's deformation modulus and the at rest lateral pressure with minimal ground disturbance, they provide the best choice to determine whether sufficient ground improvement has been performed (see Figure 12). By performing a few DMT and CPT soundings close to each other, a site specific correlation can sometimes be developed to more reliably compute the deformation modulus from the CPT tip resistance (Schmertmann, et al., 1986). Then, because a CPT sounding requires only about half the time needed for a DMT sounding, the CPT can provide the bulk of the verification tests, saving time and reducing testing costs.

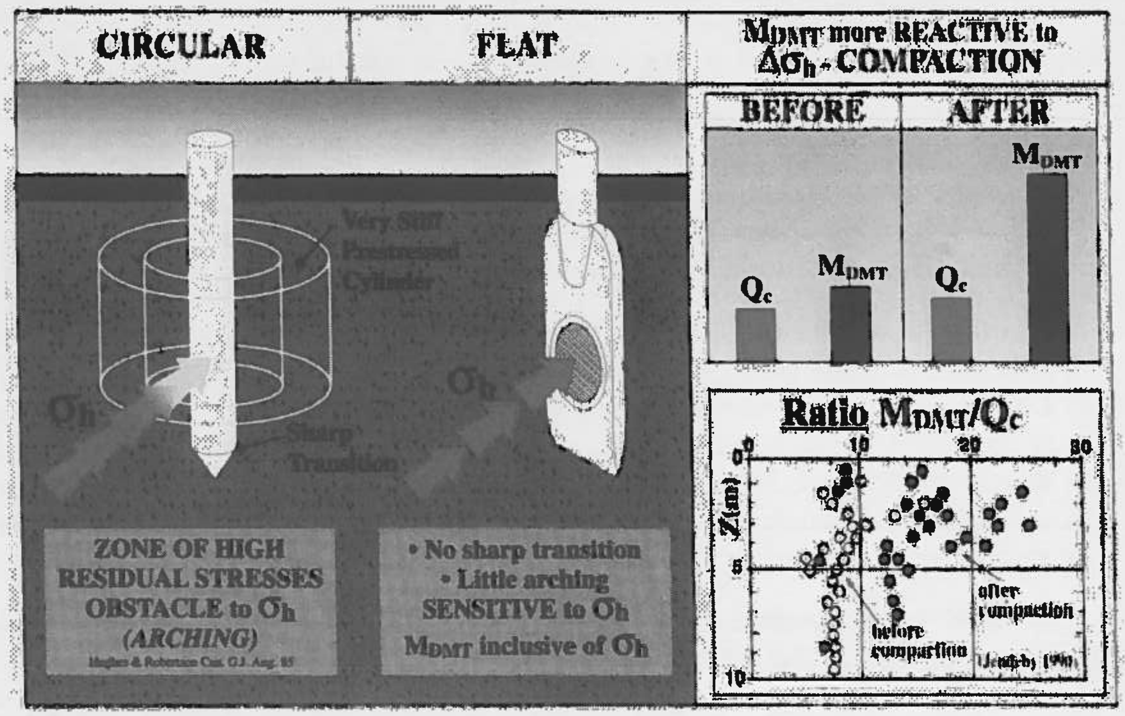

Figure 13: Better measurements of ground improvement using DMT instead of CPT (Marchetti, 1998)

When the ground improvement uses cement or chemical grouting, there may be cemented layers that cannot be penetrated using direct push methods of DMT or CPT. Pressuremeter tests should be done to measure the deformation modulus and serve as the calibration test. A site-specific correlation between the PMT and the SPT could then increase testing productivity. The minimum acceptable N_{60}-value should be chosen based on the comparison with the acceptable PMT modulus. Penetration may also not be possible if the improved materials contain rock fragments or concrete/urban debris. In this case a slotted casing pressuremeter is required.

In cohesive soils, ground improvement is often monitored by measuring the decrease of in-situ pore pressures as the soil consolidates under the applied pre-load. However, this will only confirm the completion of the consolidation process. In-situ tests are then required to confirm the improvement of strength and stiffness both of which the DMT and PMT can verify. Alternatively, the CPT, BST, and VST can verify strength improvement. The CPT tip resistance can again be calibrated with the DMT modulus, and then the bulk of the testing can be performed with the quicker, cheaper CPT. PMT results could also verify improvement, but with greater cost due to additional time of testing and analysis effort.

Deep Foundations

Axial Capacity: Both the SPT and the CPT provide good models for determining the vertical capacity of a deep foundation, with the SPT generally better for driven piles. While numerous analytical methods have been developed to determine vertical capacity, the methods that directly use N_{60} or the CPT tip resistance are more accurate than classic methods that use shear strength parameters determined from empirical correlations. Because both tests provide a depth profile of test results (more data points with the CPT), the engineer can also prepare depth plots of total pile capacity, side resistance, and end bearing. Furthermore from those plots, the engineer can make a contour map of the required tip depth for the entire site. (see Failmezger & Bullock, 2004). In stronger soils, the SPT provides more reliable test results. The CPT may reach refusal in strong thin layers that will not stop either the SPT or a pile. The SPT also provides the best tool to determine the drivability of a pile, and is the most likely test to recognize potential capacity reduction due to dynamic penetration in lightly cemented soils and sensitive clays. Correlations with both SPT and CPT usually include a database of comparisons with static load testing.

If the CPT or SPT cannot penetrate the foundation materials (soil or rock), then the PMT can be performed to calculate vertical capacity. Numerous pressuremeter tests have been performed in conjunction with pile load tests and correlation coefficients have been refined for the PMT-based analytical methods.

Historically, engineers have grossly underestimated the vertical capacity of rock sockets, primarily because they have not been able to accurately measure the rock's shear strength or run a load test on the rock to failure. The rock borehole shear test is a new method to determine the rock's shear strength. Classic shear strength capacity equations can be used to predict the vertical capacity. Osterberg load tests should be used to measure the rock socket failure capacity and to refine correlations with the RBST. While this is an area of research, site specific correlations can be used now.

Negative Skin Friction: When the soil surrounding a pile moves more than the design pile settlement, then negative skin friction or "downdrag" occurs. The axial capacity of the pile will not decrease, but undesirable foundation settlement may occur as the capacity is "remobilized". This often happens when fill is placed on a site that contains soft compressible soils. The engineer must determine the neutral point, where the negative skin friction ends and the positive skin friction begins. As above, the pile's side resistance can be estimated from SPT or CPT testing. However, the engineer must also accurately compute settlement to quantify identify the zone over which the soil settles more than the pile. The best way to do this is with DMT data. By cumulatively calculating settlement from the bottom of the sounding to the ground surface, the depth where the design settlement occurs can be determined. Above this depth is negative skin friction and below it is positive skin friction. Only a small amount of movement (<0.25 inches) is required to fully mobilize friction, whether positive or negative.

Lateral Capacity: Correlations have been developed to estimate the deformation behavior of laterally loaded piles from strength parameters, but tests that actually measure both strength and stiffness will provide superior design parameters. Because both the DMT and PMT test the soil horizontally, they are the best methods to evaluate lateral capacity. The engineer can determine accurate P-y curves from those methods and use them with numerical computer programs such as LPILE and COM624. The Dilatometer is the best choice if it can be pushed, because a continuous P-y profile can be easily established. The pressuremeter is needed for harder soils and for rock.

Conclusions

Though usually testing less than 0.01% of the overall mass of soil and rock supporting a foundation, in-situ tests generally investigate a much greater volume of soil more quickly than possible for sampling and laboratory tests. Thus they provide both cost savings and increased statistical reliability for foundation design. The additional foundation cost from poor geotechnical design greatly exceeds the additional cost of these tests to obtain better engineering design. Therefore, performing appropriate in-situ tests to support more reliable design should prove economical on every significant project.

The type of information required for a particular design application should drive the choice of an in-situ test. The in-situ test chosen should compare favorably with the application, including stress path, test type (static vs. dynamic), orientation (lateral vs. vertical), level of stress (elastic vs. plastic), and the controlling design parameters (strength, stiffness, stress).

For shallow foundation design, the DMT, CPT, and PMT provide bearing capacity parameters in all penetrable soils, but only the DMT for penetrable soils and the PMT for harder/stronger soils provide reasonable settlement estimates. (The CTL provides a good full-scale proof test of settlement.)

For slope stability design, the BST should be used to quantify the drained shear strength parameters; the DMT to determine the drained friction angle of cohesionless soils and undrained shear strength of the cohesive soils; and the VST for the undrained shear strength of the cohesive soils.

For ground improvement verification the DMT provides the best sensitivity, but the CPT tests the mass of foundation material more efficiently if it can be correlated with the dilatometer results at the site.

For deep foundation axial capacity, the SPT or CPT should be used if they can penetrate to the depth desired. The PMT should be used in harder soils. For lateral capacity of deep foundations, the DMT should be used where it can be pushed and the PMT should be used in the harder soils.

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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	SHALE 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 PRESSURE 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 INTRUMENTATION, 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.

PAST OHIO RIVER VALLEY SOILS SEMINARS (CONT.)

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 INFRASTUCTURE RECONSTRUCTION, October 15, 1993, Fort Mitchell, KY.
ORVSS XXV	RECENT ADVANCES IN DEEP FOUNDATIONS, October 21, 1994, Lexington, KY.
ORVSS XXVI	SITE INVESTIGATIONS: GEOTECHNICAL AND ENVIRONMENTAL, October 20, 1995, Clarksville, IN.
ORVSS XXVII	FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING, October 11, 1996, Cincinnati, OH.
ORVSS XXVIII	UNCONVENTIONAL FILLS: DESIGN, CONSTRUCTION AND PERFORMANCE, October 10, 1997, Lexington, KY.
ORVSS XXIX	PROBLEMATIC GEOTECHNCIAL MATERIALS, October 16, 1998, Louisville, KY.
ORVSS XXX	VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION, October 1, 1999, Cincinnati, OH.
ORVSS XXXI	INSTRUMENTATION, September 15, 2000, Lexington, KY
ORVSS XXXII	REGIONAL SEISMICITY AND GROUND VIBRATIONS, October 24, 2001, Louisville, KY.
ORVSS XXXIII	GROUND STABILIZATION AND MODIFICATION, October 18, 2002, Covington, KY
ORVSS XXXIV	APPLICATIONS OF EARTH RETAINING SYSTEMS AND GEOSYNTHETIC MATERIALS, September 19, 2003, Lexington, KY
ORVSS XXXV	ROCK ENGINEERING AND TUNNELING, October 20, 2004, Louisville, KY
ORVSS XXXVI	GEOTECHNICAL INNOVATIONS IN TRANSPORTATION ENGINEERING, October 14, 2005, Covington, KY
ORVSS XXXVII	INNOVATIONS IN EXPLORATION OF SUBSURFACE VOIDS, October 27, 2006, Lexington, KY
ORVSS XXXVIII	CIVIL INFRASTRUCTURE AND THE ROLE OF GEOTECHNICAL ENGINEERING, November 14, 2007, Louisville, KY
ORVSS XXXIX	*URBAN CONSTRUCTION, October 17, 2008, Covington, KY*