



OHIO RIVER VALLEY SOILS SEMINAR XXXVI OCTOBER 14, 2005

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Ohio River Valley Soils Seminar October 14, 2005

Role of Performance Monitoring for Construction of Transportation Facilities in Urban Areas

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ABSTRACT

Construction in an urban environment adds challenges to the already difficult task of constructing on and in soil and rock. In addition to working in tight spaces with limited access, dealing with numerous utilities, obtaining permissions and rights-of-way are the impacts of the work on neighboring structures and people. These impacts are increasingly affecting the cost and schedule for infrastructure projects. This paper addresses the role of performance monitoring to help minimize the impacts of the new construction on existing structures and people.

INTRODUCTION

Construction of transportation facilities must deal with many unknowns and limited data. This is especially true for those projects in urban areas that involved construction on or in soil and rock. We are working in materials with properties that can change instantly and significantly from one point to the next. These changes may result from the actions of nature in laying down the earth, from prior activities of man on the site, or from actions of the contractor as he works with the site. Further complications may come from uncertainties in the loads that the new facility must withstand during construction and operation. These various uncertainties combine to produce substantial uncertainty in how the completed facility will perform throughout its life.

Compounding the importance of these uncertain conditions are the potential large consequences of unexpected performance by the facility. Unexpected performance may adversely impact the project, neighboring structures and utilities, and people. Unexpected performance may delay the project, increase its cost, and lead to lengthy and expensive litigations.

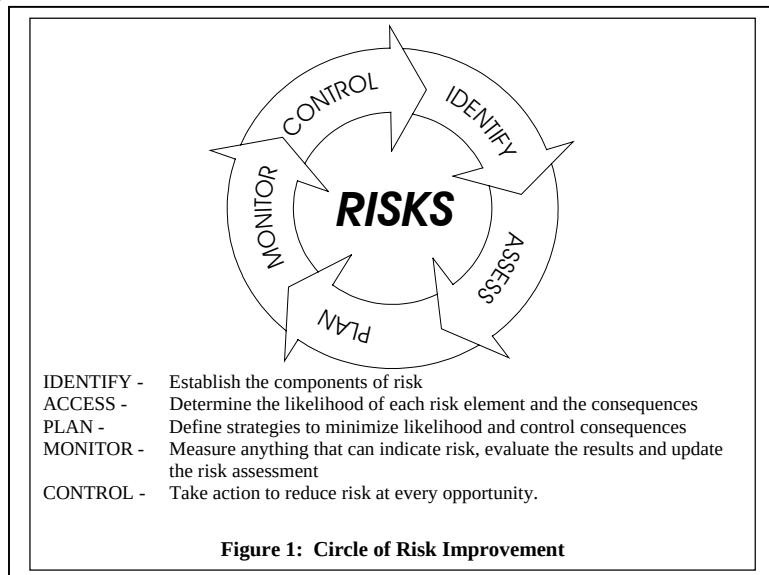
Urban work amplifies these issues because there are more structures within the potential influence zone, urban structures tend to be more significant, there are more people to be impacted, the population tends to be less tolerant, and more unknowns exist due to previous activities at the site. Additionally, one may be working in and around existing structures that must stay in operation and joining new construction to existing facilities and completed sections of the work.

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RISK AND MONITORING

Uncertainties and large consequences produce risk. Owners and contractors don't like risk. They are increasingly employing ways to manage and reduce risk to control budget and completion time. Figure 1 illustrates the process of risk management. Many of today's so-called risk management programs for infrastructure projects identify and assess risks, then seek to lay them off on someone else, usually the Contractor or the insurer. This is risk allocation and not risk management. In the long run, the Owner pays a higher price through higher insurance premiums and more costly construction. True risk management adds steps to plan strategies that minimize likelihood and control consequences, measure anything that can indicate risk, and take action to reduce risk at every opportunity. As illustrated in Figure 1, monitoring is an essential part of any true risk management program. For heavy civil construction, performance monitoring has a central role in risk management.

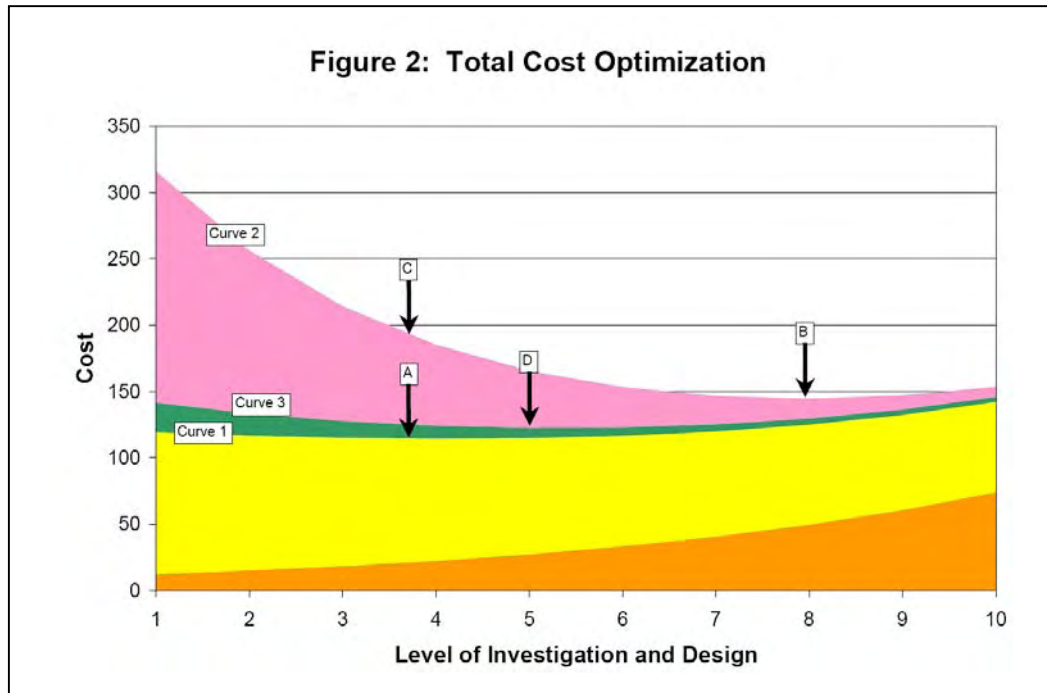
The traditional philosophy of most engineers has been to deal with unknowns and uncertainties by designing with caution and conservatism. The actual risks are arbitrarily masked by a factor of safety. Their aim is to get the facility big enough and strong enough that all the uncertainties don't matter. However this tact leads to higher costs for the owner without his knowledge of what those costs are or what they are buying.



An alternate approach is to do more investigations and studies in the pre-construction phase to reduce uncertainty and conservatism. At some point, the cost of additional investigations and studies becomes high relative to the reduction in uncertainty obtained with the additional work and some uncertainty remains. Figure 2 illustrates these points in a conceptual diagram. The horizontal axis depicts level of effort expended in investigations and design. As this level increases, the conservatism in the design should decrease leading to lower construction costs. Curve 1 indicates the total of investigation and construction costs. Point A indicates the optimal level of investigation that produces the minimum of investigation and construction costs.

The costs under Curve 1 give no consideration to risk costs. Risk costs result from unexpected events and conditions not anticipated in the design that increase the cost of construction, produce delays and/or cause damages to people and property. These costs are higher for low levels of investigation and design because the probability of failure is higher due to the large uncertainty in the information used for design. These costs decrease with increasing level of investigation and design, largely because the uncertainty in predicted performance decreases due to the additional investigation. When we add the possible risk costs to the total cost picture, we get Curve 2. The optimal level of investigation and design increases to Point B. The optimal total cost has increased because we have increased the level of investigation and we have included risk costs; however the optimal total cost is significantly less than at the level of investigation for Point A, if we include risk costs, e.g. Point C.

The really interesting observation is what happens when we consider the effect on total cost of performance monitoring. There is some evidence to indicate that an “effective” monitoring program can reduce risk by an order of magnitude (Lambe, Silva and Marr, 1981). This reduction comes from reduced uncertainty in predicted performance and reduced consequences. Curve 3 shows total costs when the costs of a monitoring program are added to investigation, construction and reduced risk costs. The optimal level of investigation shifts to Point D. The optimal level of investigation is somewhat higher than for the case where we ignore risk costs (Point A), but much less than the case where we include risk costs but do not monitor (Point C). The total costs with “effective” monitoring are significantly less than those without monitoring (Point B). Point D represents true optimization of the design-construction process by employing an appropriate level of investigation and design to remove costly conservatism and using “effective” monitoring to reduce uncertainty about performance and better control the consequences of unacceptable performance.



ROLE OF PERFORMANCE MONITORING

Monitoring provides us with quantitative information on actual performance. We compare the measured performance with the predicted or expected performance. Differences indicate the effects of uncertainties in our design. We need to evaluate those differences to determine what they indicate for future performance. If the anticipated future performance is unacceptable, we look for changes, modifications, and remediation that can be made to alter the future performance.

During design we have data that represents some indication of the true state of nature. We use our knowledge and judgment to combine these data with models to predict ultimate performance. If the predicted ultimate performance is unacceptable the Engineer alters the design. Traditional

design treated predictions as discrete values but in fact every prediction has uncertainty. Measured performance is nature's indication of the true condition. Measured performance reduces the range of uncertainty caused by all the unknowns present during design.

Traditional approaches attributed unexpected performance to an act of god; this defense has become increasingly useless as lawyers and experts seek relief for those who are allegedly damaged. More recently the blame has shifted to acts of the contractor or acts of the design professional. By measuring performance and taking action, the goal is to reduce unexpected performance and take the blame game out of the project equation.

Consider a situation involving a deep excavation in the center of a city. We know that lateral movements of the excavation support system of more than 1 inch will cause architectural damage to adjacent buildings and disruptions to adjacent utilities. We also know that lateral movements greater than 2 inches is an indication of impending failure of the mat foundation for an adjacent high-rise building. Figure 3 illustrates how this information interacts. For monitoring purposes we might consider the 1-inch value as a Threshold Value. If measured lateral deformations approach this value we will be concerned with the adverse impacts to adjacent facilities. The 2-inch value might be considered a Limiting Value. If measured lateral deformations approach this value, we will seriously consider stopping construction until corrective action could be taken. We use an accepted method to predict maximum deformation of the wall and determine it to be $\frac{1}{2}$ inch. It appears that the proposed design will work. But can we be sure? There is uncertainty about the material parameters we chose to represent the soil, the wall and the retention system. There is uncertainty about the groundwater conditions during construction and there is considerable uncertainty over how the contractor's means and methods will affect the actual deformation of the wall.

If we were able to quantify these uncertainties, we could use probabilistic methods to obtain a probability distribution of the predicted deformation. This might look like the cumulative distribution on lateral movement shown in Figure 3. Let's assume that the mean value of the probabilistic prediction equals the deterministic value of $\frac{1}{2}$ inch. The example shows that there is a 50% chance that the actual deformation will exceed $\frac{1}{2}$ inch; there is a 20% chance that it will exceed the 1 inch Threshold Value; and there is a 4% chance that it will exceed the 2 inch Limiting Value. Seen from this perspective the design looks too risky. With a closer look into the prediction we estimate that 30% of our uncertainty comes from uncertainties in the material properties and loads, 20% comes from the predictive method we used and 50% comes from uncertainty about the contractor's means and methods. We could (1) ignore the risk elements all together and face grim reality if it appears, (2) redesign to reduce predicted lateral displacement but at higher cost, (3) perform additional investigations to reduce uncertainty about material parameters, (4) use a more reliable method to predict lateral movements, and/or (5) incorporate an excavation support system that can be modified to control deformations and monitor performance during construction to kept the actual movements below the Threshold Value. This last approach has been wonderfully laid down by Peck in his "Observational Method." (Peck, 1969)

"EFFECTIVE" MONITORING

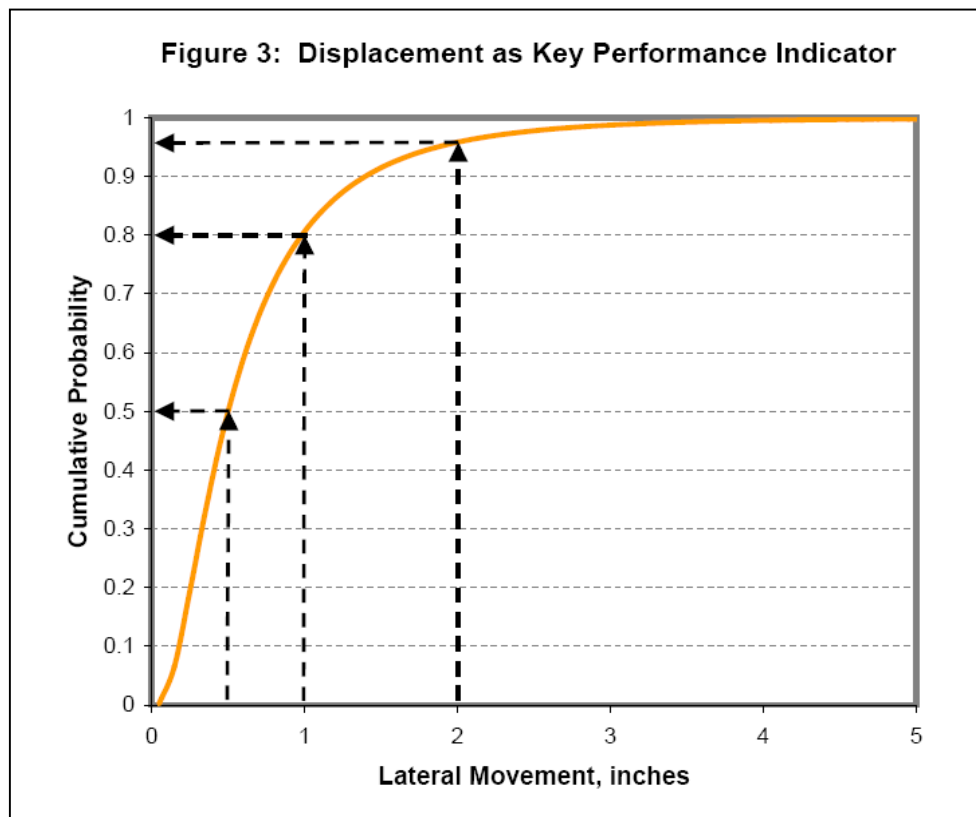
The observant reader will have noticed that I have placed the adjective "effective" in quotes when used in front of monitoring. This is to emphasize the obvious but often ignored fact that the benefits of performance monitoring result only when the work is performed in an effective manner. Table 1 lists the components of an effective performance monitoring program. Each of these components is considered below:

Table 1: Components of an Effective Performance Monitoring Program

• Measure one or more Key Performance Indicators
• Action Levels and responses must be established up front.
• Data must be reliable
• Measurements must be taken with sufficient frequency to capture the unexpected performance as earliest possible stage.
• Measurements must be evaluated in a timely manner
• Preplanned action must be taken when Action Levels are reached.

Measure one or more Key Performance Indicators

A Key Performance Indicator is something that gives us a quantification of current and future true performance. Typical key performance indicators for structures are deformation, differential movement, rotation, strain, force and pressure. There are literally thousands of different sensors to measure these parameters. In our current technological economy, the capability and reliability of sensors are increasing all the time while size and cost are decreasing.



Generally, the most useful Key Performance Indicator for infrastructure construction is some aspect of deformation. Unexpected deformations are the consequence of most of the unexpected behavior we must deal with. Undesirable deformations may be static (inertia not significant) or dynamic (inertia affects performance). As discussed earlier, unexpected deformations result from uncertainties in our predictive models and the input data as well as variables introduced by the construction processes. Static deformations progress from minor acceptable values to complete

collapse. It is precisely this continuous aspect of deformation that makes it a useful Key Performance Indicator. Measured deformation can be a reliable predictor of future performance. Table 2 summarizes the effects of deformations as a progression in increasingly severe consequences. Clearly risk increases as the level of deformation progresses from one state to the next. Measurements of deformation which establish the magnitude and rate of change allows us to predict the future with increasing reliability as we progress from the early stages of design through construction. The better we can anticipate the future and reduce unexpected performance, the better we can manage risk. The goal of all performance monitoring programs should be to keep actual performance from progressing to any level above that we have accepted and prepared for.

Table 2: Performance Levels for Deformation

Level	Effects on Facilities	Effects on People
I	As designed, as expected, acceptable consequence	None
II	Architectural damage, minor inconveniences	Nuisance
III	Loss of function of doors, elevators, sensitive equipment	Annoying
IV	Loss of tolerances that produce interferences in construction	Disruptive to normal activity
V	Loss of function of the facility	Causing tissue damage
VI	Collapse	Causing death

Some measurements help us anticipate and predict future deformations. Some examples are:

- Measure excess pore water pressures in the ground that will dissipate over time and cause movement.
- Measure drawdown of groundwater that may cause movements over time.
- Measure corrosion rate or volume change to detect deterioration of materials from chemical causes.
- Measure rate of weathering, erosion, or clogging to detect deterioration of materials from physical causes.
- Measure rate of wear or fatigue to detect deterioration of materials from mechanical causes.
- Measure change in forces, stresses or strains to detect unexpected loading
- Measure construction processes to infer likely effects on material properties and hence future performance.

There may be Key Performance Indicators other than deformation. For projects in urban areas, noise and discharges of gas, fluids and solids can be important elements affecting the progress of the work; they can be Key Performance Indicators. In soft ground tunneling projects, ground performance can be a direct function of how the tunneling machine is operated; consequently we may monitor machine parameters like thrust and slurry pressure as Key Performance Indicators.

Data must be reliable.

A performance monitoring program works only if the project staff believe the data it provides. Strong pressures to ignore the measurements develop if there is any indication that the data might not be reliable. Once the integrity of the measurements comes into question, it is very difficult to regain trust in a monitoring system.

A reliable monitoring program comes from good design and systematic execution. Table 3 summarizes the key steps of a systematic program for a performance monitoring system. Dunnicliff (1988, 1993) provides much more detail on the steps of a systematic instrumentation program. He uses the analogy of each step being a link in a chain. The chain is only as strong as the weakest link. Likewise a monitoring system is only as reliable as each step in Table 3. Each of the twelve steps must receive careful attention to all details if the overall system is to provide high reliability.

Table 3: Systematic Program for Reliable Performance Monitoring System

1. Identify what is to be measured.
2. Determine measurement level, range and precision.
3. Determine monitoring frequency.
4. Design appropriate monitoring system.
5. Provide means to check measurements, validate readings and give redundancy for key measurement points.
6. Plan installation, calibration, maintenance and data management.
7. Prepare budget that includes costs for data collection and evaluation.
8. Prepare specifications for instrumentation that establishes minimum acceptable quality and reliability of equipment.
9. Procure, test, install and verify instruments.
10. Calibrate and maintain instruments.
11. Collect, process and evaluate data.
12. Check and explain all unexpected readings.

Measurements must be taken with sufficient frequency to capture the unexpected performance at earliest possible stage.

I'm often asked for a summary table of recommended reading intervals for constructed facilities. For example FERC (1991) gives some recommendations for earth dams. One approach used on infrastructure projects is to take one measurement a month until construction occurs within 200 ft of the sensor, then one reading a week until construction occurs within 50 ft of the sensor, then daily while construction occurs within 50 ft of the sensor. However, these recommendations or any others I could provide will surely be misused. Frequency of measurement is closely tied to the rate of change of the performance indicator one is measuring. The time for significant change may be as short as minutes for static loads and seconds for dynamic loads. For example many of the performance problems we encounter in underground urban construction result from deformations caused by excavations. Excavations produce an unloading. In an unloading, soil or rock rebounds nearly elastically with relatively small strains until it almost reaches a state of failure; then large plastic strains can develop in a few minutes to few hours. A measurement system must obtain readings more frequently than the rate at which significant changes occur for the change to be detectable and acted upon. Thus a performance monitoring system for an excavation must measure movements several times every few minutes to few hours to detect these movements and provide an adequate warning. This is a very tough point to get across to people who have had years of experience observing excavations that showed no visible signs of distress; yet were unknowingly close to collapse and disaster.

Sensor readings change with changes in environmental conditions. Infrequent readings cannot reveal these environmental effects. They show up as scatter in the data and reduce the precision of the data for use as a Key Performance Indicator. We increasingly take measurements several times a day and include measurements on temperature sensors for two reasons. Most

sensors show some response to changes in temperature. Temperature typically changes over the course of a day. Sensors experiencing a change in temperature will show a change in reading proportional to the temperature. By observing the sensor reading changing in proportion to the change in temperature, we are confident that the sensor is working properly. We can also use the data to correct the readings to remove the effects of temperature on the measurements if desired. A similar approach can be taken along coastal areas where groundwater levels and structural forces fluctuate with the tide. These procedures greatly improve our confidence in the measurement system.

As the pace of construction work increases, performance monitoring programs must obtain readings at much closer intervals than traditionally used for them to be effective. I think a strong case can be made on risky projects for instruments to be read several times a day to increase the reliability of the measurement system and to make the changes in the trend of the data detectable at an earlier time.

Measurements must be evaluated in a timely manner.

A measurement that is not evaluated soon after it is obtained is useful only to the lawyers and experts doing cleanup work. Either it shows no significant change and therefore is of little interest to anyone; or it shows a significant change but no one knows about it until the damage is done. Ideally every measurement would be evaluated moments after it is obtained and the appropriate action initiated shortly thereafter. Unfortunately file cabinets and computer disks are littered with reams of carefully recorded data that no one with sufficient knowledge paid attention to. This state results from misunderstood goals of the monitoring program, inadequate funding for data evaluation, or ignorance in the management team. We are working on ways to program computers to help with this task to reduce the time between reading and evaluation and reduce the cost. In one approach we make the computer compare the latest reading to the recent history of readings. If the latest reading significantly departs from the historical behavior, then the computer sends an electronic notice get a responsible person involved in the evaluation. If the latest reading is consistent with the historical behavior, then it is only recorded in a database. This approach greatly reduces the information that a person must deal with and the time required for evaluation; yet, the data get immediate attention when required.

Preplanned action must be taken when Action Levels are reached.

For a performance monitoring program to be an effective risk management tool, preplanned actions must be taken to alter performance and/or consequences when the measurements approach Action Levels. Action Levels must be set in advance so there is contractual agreement among all parties on conditions and responsibilities. Preventative and remedial measures must have been laid out in advance so that materials are available, chain of command and responsibility are defined, and preplanned effective actions can be readily implemented. If one waits until the measurements reach a level that causes concern before establishing Action Levels and appropriate responses, all effort will go to arguing over whether there is a problem and who is responsible, rather than dealing with the situation in a timely fashion.

MONITORING TECHNOLOGIES

There are literally thousands of different sensors we can use to monitor performance of infrastructure. Technological advances are adding new types of sensors and additional

capabilities to existing types at an unparalleled rate. Some examples of recent and current developments follow.

In-place Inclinometers and Tilt Beams

Inclinometers measure tilt relative to the constant pull of gravity. Inclinometers are widely used to measure horizontal movements of structural elements and the underground. The traditional approach has been to install a casing and use an inclinometer to measure the deviation of the casing away from vertical at fixed points along the casing. This requires a person to pull the inclinometer through the casing for each reading set and takes time. Due to costs, reading intervals have been limited to once per week or less, except in special circumstances. The reduced cost of tilt sensors now permits us to position several inclinometers within the casing and leave them in place for the duration of the project. These sensors are connected to a data logger equipped with remote communications to give us a continuous access to the sensors. We can now measure horizontal movement of an excavation support system every hour. This reading frequency is very helpful in situations where the rate of advance of excavation is tens of feet per work shift.

The same tilt sensors can be mounted onto a small beam-like element that is fastened to a structure at its two ends. A change in the tilt reading indicates that one end has moved relative to the other end. One can join a number of these tilt beams together with one end fastened to a fixed point and use the tilt measurement on each beam to calculate the movement of each joint. Tilt sensors and tilt beams are increasingly used on existing structures to detect movements caused by new construction because they are so precise. A good tilt sensor can reliably detect a change as low as 1 arc second which corresponds to a change in tilt of 1 in 200,000 or 0.000005. At these levels, we easily see the response of the structure to temperature changes and wind loads.

Automated Total Stations and Global Positioning Systems

Manufacturers of these specialized devices have made great improvements in their accuracy, resolution and capability. The ability to obtain rapid readings from a remote location makes them useful for real-time performance monitoring.

An automated total station is similar to a total station used by surveyors but it has motors with encoders that rotate and tilt the instrument by precise amounts. An automated total station can find a target and measure distance, azimuth and tilt between the instrument and the target. In performance monitoring, we are generally most interested in differences in movement in the work vicinity and less interested in absolute positions. Automated total stations give much better resolution for differential movement than for absolute position. Today's equipment can measure differences in movement in the x, y and z directions to a working accuracy of ± 2 mm. Even better accuracy is possible with advanced processing of the data. The total station is relatively expensive but it can be used to monitor a large number of targets located within a 500 ft radius of the instrument, provided they are in direct line-of-sight with the instrument. We are using automated total stations to remotely monitor the movements of building faces while excavation occurs in the street, existing subway lines and stations while they are undermined for new facilities, and excavation support systems for cut-and-cover tunneling operations.

GPS systems also offer means to monitor x, y and z deformations as long as the target is visible from at least five satellites. This is a severe restriction for much of the monitoring required on infrastructure projects. GPS gives an absolute position of a target to within about 5 mm. Better accuracy might be achieved in some circumstances with advanced processing of the data. One

interesting use is to monitor the movement of tall buildings subjected to wind loads and earthquake loads using a GPS station positioned on top of the building. The measurements are used to compare the performance of the building with that intended in the design.

Seismographs

A frequent problem occurring in urban construction is complaints by neighbors about vibrations and noise. Unchecked, these complaints can become serious obstacles to the progress of work. Generally these complaints are founded in people's perceptions that the level of vibration or noise is potentially harmful to themselves or their property. In fact people can be sensitive to vibration levels one thousandth that at which physical damage might begin.

A proactive approach on any infrastructure project is to use seismographs to monitor vibrations and noise at key receptor areas and do so in real-time. These records provide a factual basis to deal with people's complaints quickly. If the records show that Alarm Levels are exceeded then quick action can be taken by the contractor to stop the offending activity. On the other hand the measurements can be shown to those making complaints to convince them that what they are experiencing is not sufficient to cause harm to themselves or their property and to seek their indulgence until the offending activity can be completed.

Crack monitoring

A big source of complaints and litigation from construction in urban areas results from alleged cracks to existing buildings from the construction activity. Much urban construction occurs in areas with buildings close by, many of them historic. Many of these buildings have cracks before any construction starts. Any change in these cracks during and after the construction activity may be blamed on the work. Crack growth can lead to expensive litigation. During the construction of the John Hancock tower in Boston, cracks developed in the masonry of nearby Trinity Church. The Church made a claim of \$40 million dollars for damage and lost future value. The final award was \$11.6 million dollars.

In fact, cracks develop and grow in building elements for a number of reasons unrelated to the new construction, including thermal changes, foundation settlements, wind loads and material degradation. We increasingly mount electronic displacement transducers across existing cracks to monitor the change in crack width with temperature and time. By correlating the measurements with actual construction activities, we are able to identify the likely cause of the crack growth. If attributable to construction activity, we look to alter the effect of that activity to minimize future crack growth. Cracks to neighboring facilities were traditionally seen as a nuisance byproduct of construction; but today's litigious climate forces us to be more proactive in controlling the offsite effects of our construction activities.

Monitoring Equipment Processes

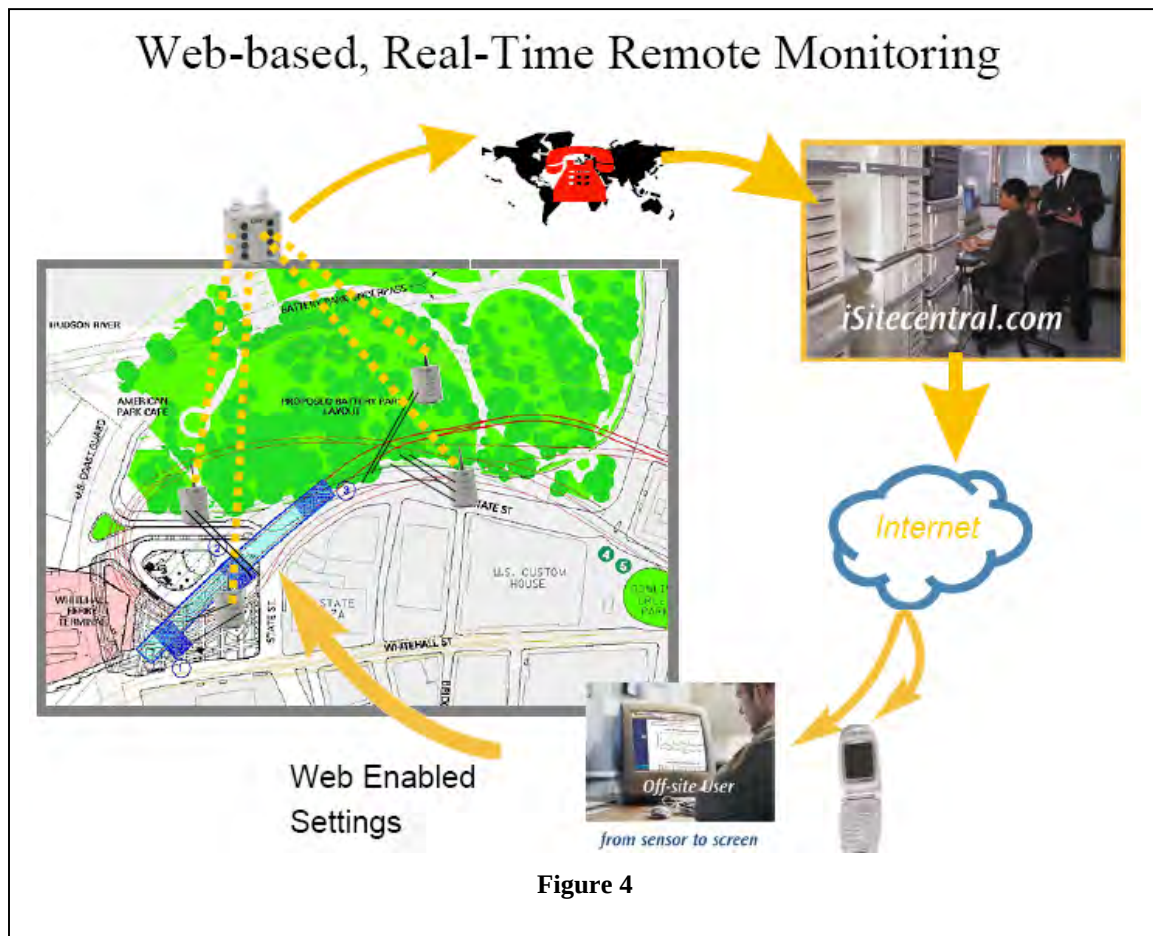
Some elements of poor performance result from poor construction procedures. Attention is being directed to using the performance of the construction equipment as an indicator of future performance problems for the structure. One common example is to monitor the strain and acceleration in a pile during every stroke of the driving operation in real-time. The data are used to estimate the capacity of the pile and to ensure that the pile is not overstressed by driving forces. In the future these data may be used to optimize the operating characteristics of the driving hammer and achieve better driving efficiency.

In Europe and Japan, engineers are requiring contractors to install continuous monitoring equipment on their equipment for deep soil mixing, jet grouting, compaction and other soil improvement methods. These records show power usage, earth resistance and deformation response, volume takes, production rates and other variables that may correlate with future performance. The aim is to use measurements of the equipment performance to help control the quality of the constructed element and obtain the desired future performance.

Small sensors are being used to monitor temperature and resistivity in concrete as it sets. We may soon be able to isolate zones with defective concrete early enough that they can be removed before they become a permanent part of the work and lead to future performance problems. New sensors and real-time monitoring may also allow faster construction rates (Marr, 2005) to address the current desire to reduce the time required to get a new facility into operation.

Real-time Monitoring Systems

A big change in performance monitoring is occurring due to the same technological advances that support the Internet. That change is the ability to show sensor readings in real-time on any device that connects to the Internet. Sensors are connected to dataloggers that are linked to the public data network. The data link may be by hard line, cell network or satellite. Figure 4 illustrates one such system that we operate. This system uses a cluster of servers to maintain electronic contact with data loggers at sites all over the world. Our servers connect to the Internet. The datalogger at a site can constantly determine whether the reading on a sensor is exceeding a Limiting Value.



When that occurs the datalogger contacts the iSiteCentral servers and passes along the current readings on all sensors. The iSiteCentral system verifies the reading by instructing the data logger to read the sensor again. After verification, the iSiteCentral system then proceeds through a prearranged set of instructions that might include sending a recorded message to some people, sending emails to others, or even sending an alarm alert back to the site. At any point in time and from any location, a user can log onto the site and see a status report on the condition of every sensor on the site. She or he may also examine graphs showing the complete history of data for the sensor or a group of sensors to determine whether the situation requires immediate action.

Internet-based systems like iSiteCentral will radically change the way we use performance monitoring on future infrastructure work. As these systems become more reliable and their costs decrease expect to see more measurement points, more monitoring in real-time and faster evaluation of data. These changes will help make performance monitoring a key part of every effective risk management program.

Monitoring loads.

One of the big unknowns in designing a constructed facility and evaluating its performance lies in the actual loads experienced by the facility. We design for loads dictated by codes and standards or for conditions assumed for the design. We design for selected wind speeds, wave heights, earthquake magnitudes and flow rates mostly using semi-empirical relationships between these conditions and the loads they produce on a structure. Some of these relationships use upper bound data to produce conservative estimates of loads. Others make heavy use of data collected at a small scale and extrapolated to the scales we must work with.

As we put more real-time monitoring systems into place, we are going to learn a lot more about the actual loads that develop in our structures and how our structures respond to those loads. I believe that future performance monitoring systems will help us considerably improve our knowledge of the uncertainties in the load side of the design equation.

BENEFITS OF PERFORMANCE MONITORING

From my perspective, the role of performance monitoring in infrastructure is to save owners money. These savings result from the benefits that an effective performance monitoring system can provide. These benefits include avoiding surprise behavior, reducing the likelihood of undesirable performance and providing early warnings of unexpected performance so that remedial actions can be taken to reduce the undesirable consequences. These benefits reduce the potential for delays to the project from unexpected performance. They reduce the possibilities that construction will adversely affect neighboring people and facilities. They also reduce the opportunities for claims arising from unexpected performance.

On projects that involve uncertainties about the existing conditions, new construction methods or materials, low margins of safety, high consequences of adverse performance, or tight restrictions, performance monitoring can provide benefits that may be several times the cost of the monitoring program. As an example the Central Artery/Tunnel project nearing completion in Boston required some of the most daring undertakings in underground construction ever attempted. The design engineers recognized that they faced enormous risks from adverse performance and designed a robust performance monitoring program for the entire project. The monitoring program cost about \$60 million dollars or 0.4% of the total project cost. Engineers working on the project experienced numerous instances where the monitoring program showed problems and

deficiencies in time for corrective action to be taken. Estimates have been made which show that the performance monitoring program for the project helped avoid as much as \$500 million dollars in costs from damages and delays that could have resulted were no monitoring systems in place.

LOOKING AHEAD

Conservative designs based on limited information add significant costs to repairing and constructing infrastructure. Delays and claims resulting from unexpected performance add to these costs. I see conditions favorable for performance monitoring to become a more integral part of the project management process. When more people understand that data from real-time performance monitoring systems can alert them to unexpected performance and allow them to take evasive action early, saving money and time in the process, we will see performance monitoring joining schedule and cost control as parts of the construction manager's resource kit.

The futurists tell us that we are entering a wired world where everything will be monitored and reported anytime, anywhere. One manifestation of this view in our world is something called "structural health monitoring." This involves placing sensors on and within a structure to constantly monitor the pulse of the structure. The idea is that deterioration or malfunction of some part of the structure will alter the pulse in a way that we can identify and correct the problem before failure occurs. The ideal system will tell us the remaining useful life in the structure so that the owner can plan repairs, renovations and replacements. Several bridges are already being wired with sensors to monitor their structural health. We are working with some geotextile materials that have fiber optic strain gages embedded into them as part of the manufacturing process. The instrumented material will be installed just like the virgin material. Data will tell us the level and distribution of strain along the geotextile element over the life of the facility. We see applications for this material to monitor subsidence of roads and railroads constructed over karst features and mined areas where future sudden subsidence may occur.

As discussed above, performance monitoring must be an important part of any effective risk management strategy for a constructed facility. As more owners develop their risk management strategies, I expect to see performance monitoring as a key component of the risk management program. We might even go so far to consider performance monitoring as risk monitoring; that is a real-time quantitative measure of whatever elements of risk that can be measured.

The increasingly important role of performance monitoring to managing risk on a project should make us consider the best delivery method for performance monitoring. There is a strong tendency on infrastructure projects to make performance monitoring a part of the contractor's work. In general this is akin to requiring the contractor to do the quality assurance. Most general contractors are not motivated to make performance monitoring systems work. They generally see instruments as things that get in their way and they think that measured performance only brings bad news for them.

I believe that performance monitoring should become the responsibility of the construction management team. An effective performance monitoring system provides them with solid facts about the engineer's design, the contractor's work and the effects of site conditions.

CONCLUSIONS

Performance monitoring should be a part of any infrastructure project that involves significant uncertainty or large consequences from unexpected adverse performance. Results from a performance monitoring program can help avoid undesirable performance and reduce consequences of unexpected performance.

Performance monitoring is an essential component of effective risk management. As shown in Figure 1, risk management involves a circle of five steps that should be applied throughout the project. Monitoring is one of these five steps.

Performance monitoring must be done in an effective manner. Table 1 lists six elements of an effective performance monitoring program. All elements are equally important to obtaining measured performance that people will believe and act on.

Performance monitoring best belongs to those responsible for risk management on the project. This is generally with the Owner or its representative and not with the contractor. Hopefully, this paper helps engineers and owners understand the value of performance monitoring as an integral part of an overall risk management strategy.

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***Drilled Shafts for Bridge Foundation Stability Improvement
Ohio 833 Bridge over the Ohio River
Pomeroy, Meigs County, Ohio***

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Abstract

The Ohio Department of Transportation (ODOT), the West Virginia Department of Transportation (WVDOT) and the Federal Highway Administration (FHWA) are working together to construct a replacement structure for the existing U.S. Route 33 Bridge over the Ohio River between Pomeroy, Ohio and Mason, West Virginia. The proposed structure is a cable-stayed structure with an abutment-to-abutment length of 1,910 feet. Relocation of Main Street on the Ohio side of the river required a new roadway embankment ranging up to 30 feet in height. The initial geotechnical exploration discovered the presence of existing fill materials and soft alluvial deposits overlying shale bedrock along the proposed Main Street alignment. As a result, a controlled rate of embankment construction was specified. Installation of slope inclinometers and piezometers was recommended to monitor for possible slope movements and increases in pore water pressures.

Embankment construction began in October of 2003. Slope inclinometer data obtained in November of 2003 indicated the presence of deep slope movement. Subsequent explorations and monitoring confirmed the presence of what appeared to be a remnant landslide. The detected failure plane was generally at or below the interface between residual soil and soft shale (mudstone), at depths up to 65 feet. Although the proposed bridge foundations were not within the area of slope movement, it was decided that proactive steps should be taken to protect the new foundations in the event that the movement expanded. Working with URS, the bridge designer, and the ODOT Office of Geotechnical Engineering, a foundation protection scheme was developed consisting of the construction of nine, eight-foot diameter heavily-reinforced drilled shafts on the Ohio river bank, just uphill of the tower pier. The shafts were instrumented with slope inclinometer casing, strain gauges and tiltmeters. The design methodology will be presented along with results of the instrumentation monitoring.

Section 1 – Introduction

1.1. Project Background

The Ohio Department of Transportation (ODOT), the West Virginia Department of Transportation (WVDOT) and the Federal Highway Administration (FHWA) are working together to design and construct a replacement structure for the existing U.S. Route 33 Bridge over the Ohio River between Pomeroy, Ohio and Mason, West Virginia. Associated ramps, approaches and access roads, including the relocation of a portion of Main Street in Pomeroy, comprise the roadway improvements for this project. The new bridge and approach roadway will be renamed Ohio 833. The lead agency for design and construction of the project is ODOT. Figure 1 shows the project location and river

crossing on a portion of the 7.5-minute USGS topographic map of the Pomeroy Quadrangle.

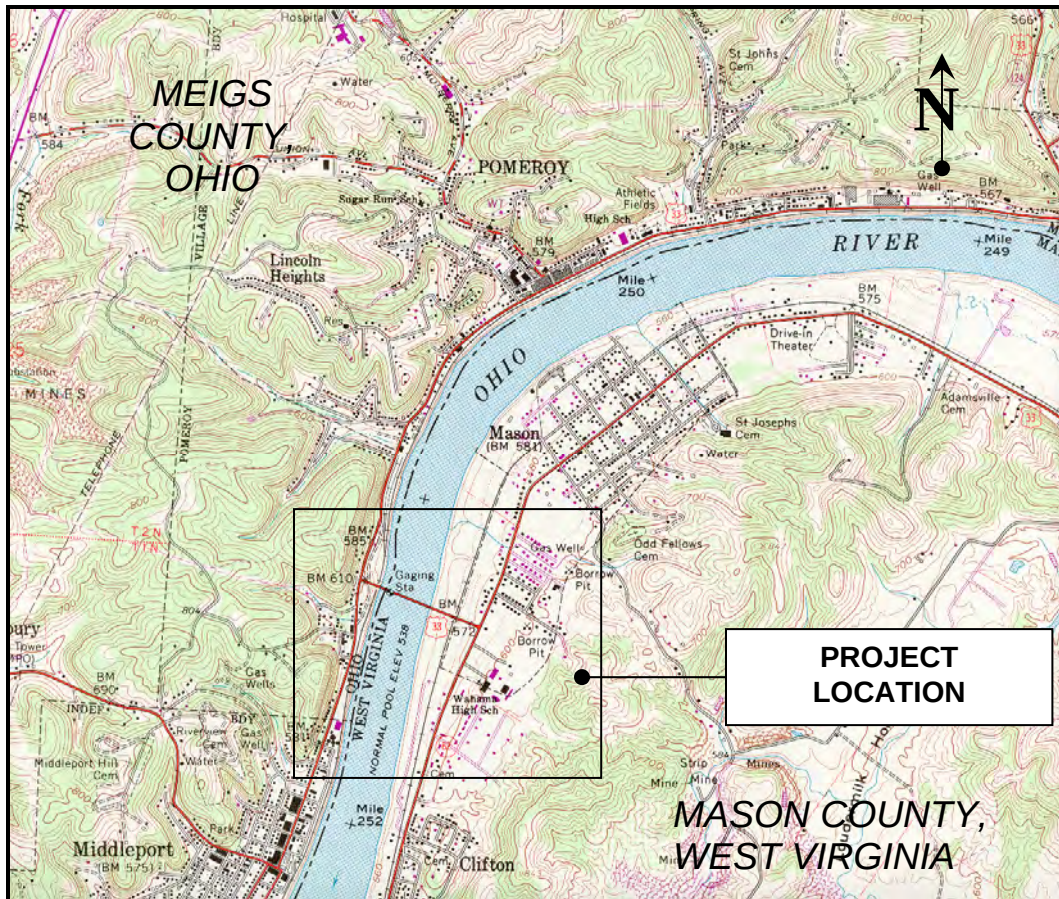


**Figure 1. Project Location and Vicinity
(Portion of USGS 7.5-Minute Topographic Map – Pomeroy Quadrangle)**

Fuller, Mossbarger, Scott and May Engineers, Inc. (FMSM) performed initial geotechnical explorations for the roadway and bridge in support of URS Corporation's (URS) design responsibilities. The geotechnical explorations were completed in various phases from June of 2000 through May of 2002. The explorations were performed in accordance with ODOT's Specifications for Subsurface Investigations dated November 1, 1995.

1.2. Initial Slope Stability Analyses

Several borings were performed during the initial roadway geotechnical exploration within the area of the proposed Main Street Relocation on the Ohio side of the river. Those borings indicated that the foundation soils within that area of the site consisted of existing fill materials, alluvial silt and clay deposits, and residual clays weathered from the underlying shale (mudstone) bedrock. The soils in the area ranged in thicknesses up to about 60 feet.

Slope stability analyses of the proposed Main Street Relocation embankment configuration were completed during the initial geotechnical explorations. The embankment in this area had a maximum proposed height of about 30 feet. Stability analyses were performed using embankment geometry at Stations 138+50, 141+50 and 145+00, and returned low factors of safety for short-term loading conditions (undrained soil conditions). As a result, an embankment construction rate of one foot per week was recommended to reduce the potential for problems associated with short-term loading conditions. Slope inclinometers and piezometers were also recommended to monitor for possible slope movements and increases in pore water pressures, respectively. Two piezometers and two inclinometers were installed in September of 2003 just prior to the beginning of embankment construction in this area.

One piezometer each was installed within the alluvial silt and clay materials at a depth of about 20 feet at Main Street Stations 142+00 and 146+00, on centerline. The two slope inclinometers were located at Stations 142+29 and 146+00 at offsets of 75 feet and 89 feet right of the Proposed Main Street centerline, respectively, and were socketed into bedrock.

1.3. General Site Topography

Topographic relief along the proposed alignment varies from approximately Elevation 600 feet at the existing location of Main Street to approximately Elevation 500 feet at the bottom of the Ohio River. The ground surface generally slopes downward from Main Street towards the Ohio River. The flow of the Ohio River is in an approximate south-southwest direction at the location of the proposed roadway improvements and bridge replacement. A sandstone rock bluff is present on the Ohio side of the river, west of existing Main Street. The topography on the West Virginia side of the river, within the project limits, is relatively flat in comparison to the topography on the Ohio side of the river.

1.4. Geology

The soil survey for the Ohio side of the project (Soil Survey of Meigs County, Ohio, United States Department of Agriculture, 2000) indicates that the site is underlain by Cidermill silt loam (CkB), dumps and mines (Dp), the Upshur-Gilpin complex (UgE) and Vandalia silt loam (VaD2). CkB soils are described as deep, well drained, gently sloping soil on terraces along the Ohio River. These terraces are formed in silty and loamy alluvium underlain by sand and gravel. Dp deposits are located at the bottoms of hillsides near coal mines and coal-processing facilities and consist of piles of coal fragments, roof shales, underclay and rock fragments. The UgE soils are characterized as deep and moderately deep, steep and very steep, well drained soils on side slopes, lesser sloping benches or smooth slopes. VaD2 soils are typically deep, well drained soils on colluvial foot slopes. UgE and VaD2 soils are formed in residuum of red shale and lesser amounts of siltstone and sandstone.

Available geologic mapping (Reconnaissance Bedrock Geology of the Pomeroy, Ohio Quadrangle, Ohio Department of Natural Resources, 1995) shows that the soil materials described above are underlain by bedrock of the Conemaugh Group representing the Pennsylvanian geologic period. The Conemaugh Group typically consists of alternating layers of shale, siltstone, sandstone, mudstone and lesser amounts of limestone and coal. The color of the various bedrock units varies from

shades of gray to green, red, brown and black. These bedrock units range from being nonbedded to massively bedded. The total thickness of the group ranges from approximately 350 feet to about 490 feet. The Conemaugh Group is known for its rapid horizontal and vertical changes in rock type. Other characteristic features include multicolored mudstones, rare coal beds and thin to thick layers of marine limestone in the lower two-thirds of the group.

1.5. Subsurface Soil Conditions

As previously noted, FMSM performed the initial geotechnical explorations in various phases from June of 2000 to May of 2002. The following paragraphs summarize the subsurface conditions observed at the site as interpreted from the borings performed during those explorations. The discussion focuses on conditions observed near the Ohio Tower Pier location and those encountered on the Ohio side of the river.

The borings drilled from floating plant operations within the limits of the Ohio River encountered water depths of approximately 28 feet near the proposed West Virginia Tower Pier location and approximately 23 feet near the proposed position of the Ohio Tower Pier. Below the water, near the Ohio Tower Pier, silty clay and/or sandy silt was observed with a thickness of approximately 20 feet. Interbedded shale and siltstone bedrock, approximately 20 feet thick, was noted at a depth of approximately 45 feet below the surface of the water. A layer of mudstone that was approximately 25 feet thick was encountered below the interbedded shale and siltstone. Hard siltstone was observed below the mudstone.

Soils observed in the borings drilled on the Ohio side of the river generally consisted of three predominant types: fill material, alluvial deposits and residual soils. Fill material was observed in thicknesses up to approximately 20 feet as the upper-most material in a majority of the borings. The fill material primarily consisted of coal remnants and construction debris. This material was further described as brown and black, moist and loose to dense. The fill typically classified as gravel with sand (A-1-b) or gravel with sand and silt (A-2-4). Most samples of this material were determined to be non-plastic.

Generally, silty alluvial deposits were observed underlying the fill material, and were observed in thicknesses ranging up to approximately 30 feet. According to the ODOT classification method, samples of this material, for the most part, classified as silty clay (A-6b), silt and clay (A-6a) and sandy silt (A-4a) and, to a lesser degree, as clay (A-7-6). The alluvial deposits were described as being various shades of brown to gray in color, moist to wet in natural moisture content and very soft to stiff in consistency.

Residual soils and/or highly weathered mudstone were typically encountered underlying the river terrace deposits in thicknesses ranging up to approximately 30 feet. These soils generally classified as clay (A-7-6), silty clay (A-6b), silt and clay (A-6a) or as stone fragments with sand, silt and clay (A-2-7). These soils were described as reddish brown to gray and tan, moist and soft to very stiff. Sandstone and coal remnants were often observed within this horizon.

Beneath the soils and weathered bedrock materials described above, a less weathered mudstone was encountered. This bedrock zone was generally described as slickensided with blocky bedding. Samples of this bedrock type exhibited Rock Quality Designations (RQD's) ranging from 0 to 50 percent.

Underlying the mudstone was interbedded shale and siltstone bedrock. This material was described as laminated to thin bedded and soft to hard. RQD's generally ranged from 30 to 50 percent within samples of this bedrock.

Figure 2 shows a generalized cross-section of the site on the Ohio side of the river based upon the results of the borings and laboratory testing performed during the initial geotechnical explorations.

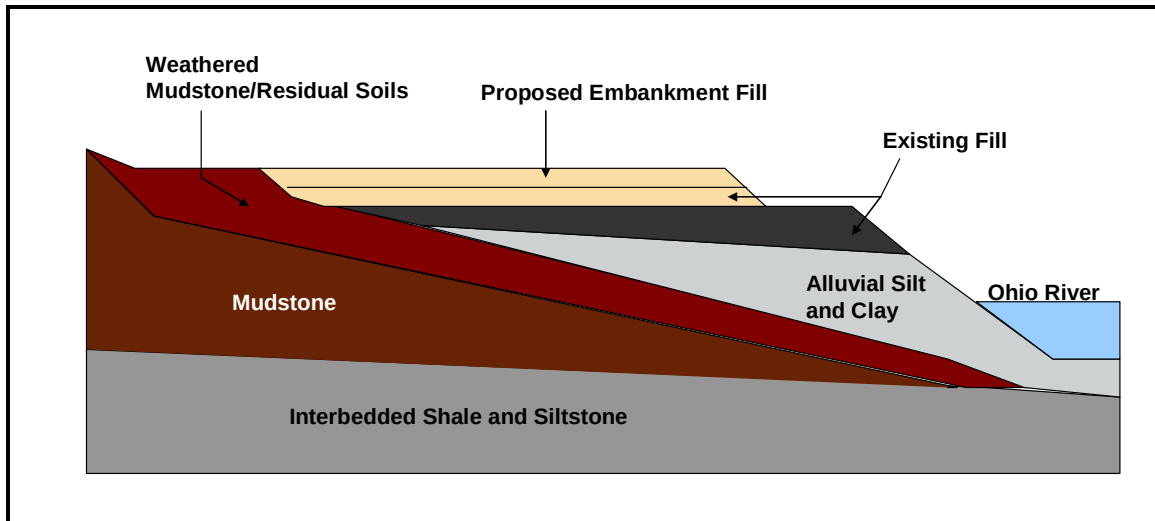


Figure 2. Generalized Cross-Section of Site (Ohio Side of River)

1.6. Detection of Slope Movement

Embankment construction began in October 2003. Slope inclinometer data collected in November 2003 indicated the presence of deep slope movement. Movement was generally detected near the interface between residual soil and the weathered clay shale bedrock, at depths up to approximately 65 feet below the ground surface. The two initial slope inclinometers closed off within a few weeks due to movement. Additional inclinometers were installed in an attempt to define the limits and rate of movement. There was not any evidence of movement at the surface.

Review of the various instrumentation, boring logs and engineering analyses suggested that the movement resulted from reactivation of an ancient landslide in the weak Conemaugh clay shale. Triggering factors probably included embankment construction and fluctuations of the Ohio River.

Section 2 – Review of Instrumentation Data

2.1. General

FMSM prepared and updated instrumentation plans during several different intervals beginning prior to construction and continuing during construction. The instrumentation

initially consisted of slope inclinometers and piezometers. Instrument locations were selected to help estimate the plan limits of slope movement and the depth(s) at which the movement occurred. The data collected from the instrumentation was used by FMSM to perform additional analyses and to assist in developing remedial alternates.

Mahan/National was responsible for instrumentation installation and data collection. Data collected from the instrumentation was periodically forwarded to FMSM for review. A plan view showing the locations and some general data for all instrumentation installed to date is shown as Figure 3.

2.2. Piezometer Results and Interpretations

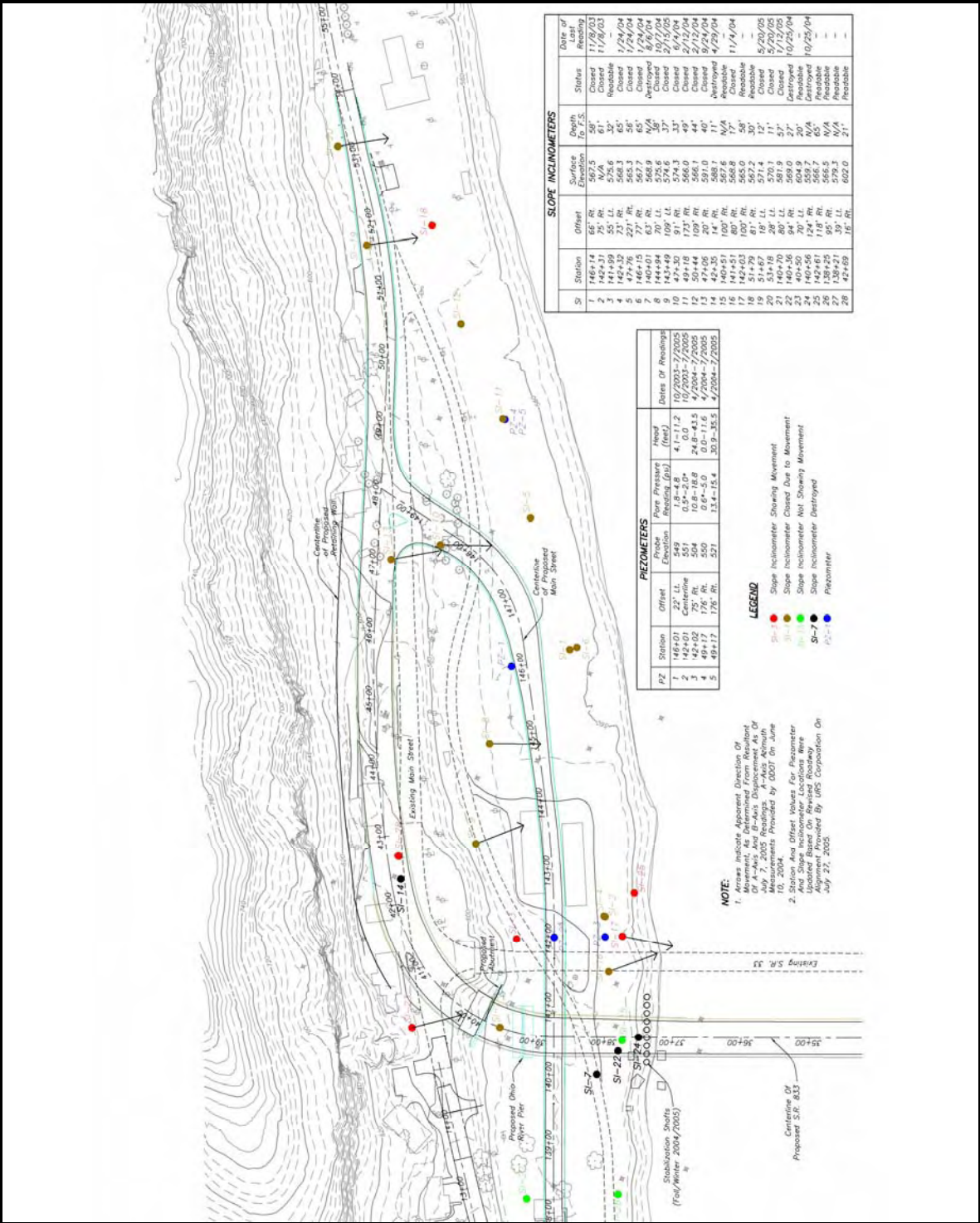
A summary of piezometers 1 and 2 from October 20, 2003 through July 7, 2005 is shown in Table 1. A summary of piezometers 3, 4 and 5 from April 3, 2004 through July 7, 2005 is shown in Table 2. FMSM created summary plots to compare the piezometer readings to Racine Lock and Dam lower pool daily river elevations. As expected, the piezometer plots indicate a rise in pore pressure measurements corresponding with high water events around September 20, 2004, January 9, 2005 and April 1 through April 4, 2005.

**Table 1. Summary of Piezometer Readings from
October 20, 2003 through July 7, 2005**

PZ	Station	Offset	Probe Elevation (Feet)	Normal Pool River Elevation (Feet)	Range of Readings (psi)	Head (Feet)	Piezometric Surface Elevation (Feet)
1	146+01	22' Lt.	549	538	1.8-5.5	4.1-12.6	553-562
2	142+01	CL	551	538	2.3*-1.4*	0.0-3.3	<551-554

**Table 2. Summary of Piezometer Readings from
April 3, 2004 through July 7, 2005**

PZ	Station	Offset	Probe Elevation (Feet)	Normal Pool River Elevation (Feet)	Range of Readings (psi)	Head (Feet)	Piezometric Surface Elevation (Feet)
3	142+02	75' Rt.	504	538	10.8-20.2	24.8-46.6	529-551
4	49+17	176' Rt.	550	538	0.6*-5.8	0.0-13.5	<550-563
5	49+17	176' Rt.	521	538	12.9-16.1	29.8-37.1	551-558



Pomeroy-Mason Bridge over the Ohio River Meigs County, Ohio



Note: Not to Scale

Figure 3. Slope Inclinometer and Piezometer Locations and Arrows Showing Direction of Movement

Negative pore pressure readings are indicated with asterisks in Table 1 and Table 2. It appears several of the instruments have experienced unexplained reading anomalies since installation. Negative pore pressure readings in PZ-2 and PZ-4 possibly indicate the instruments were installed above the normal phreatic surface. A sharp jump in pore pressure readings was indicated in PZ-3 approximately 19 days after installation. One explanation for the sharp jump in readings is the porous stone on the piezometer tip may not have been fully saturated prior to installation of the instrument. Additionally, the results of PZ-3 do not compare well with respect to results from the other piezometers. Piezometer PZ-3 shows an average normal pool phreatic surface elevation of 534 feet. By comparing the probe elevation of PZ-3 to the boring log for inclinometer 17 (which is approximately 25 feet downslope), it appears the piezometer was installed below top of rock within a clay-shale horizon. The slower response may be due to the instrument being installed below top of rock. Despite the perceived reading anomalies, each of the instruments exhibited expected behavior during recorded high water events.

Piezometer readings for PZ-1 and PZ-5 show an average normal pool phreatic surface elevation ranging between 551 and 553 feet. Piezometers PZ-2 and PZ-4 showed negative pressure readings or zero head conditions during normal pool. The normal phreatic surface utilized in FMSM's global stability analyses was 555 feet near proposed Main Street Station 140+50, 10 feet right of centerline, and then sloping down towards the normal pool river elevation of 538 feet.

The 100-year flood elevation modeled in the global stability analyses was 576 feet. River elevations obtained from the Racine Lock and Dam indicate the 100-year flood elevation was reached on September 20, 2004. On January 9, 2005, the Ohio River reached elevation 575 feet. At these times, piezometer readings for PZ-1 and PZ-4 show phreatic surface elevations ranging between 562 and 564 feet. The 100-year flood rapid drawdown analysis phreatic surface utilized was 565 feet near proposed Main Street Station 140+50, 10 feet right of centerline, and then sloping up towards the flood river elevation of 576 feet.

The results for PZ-1 are shown in Figure 4.

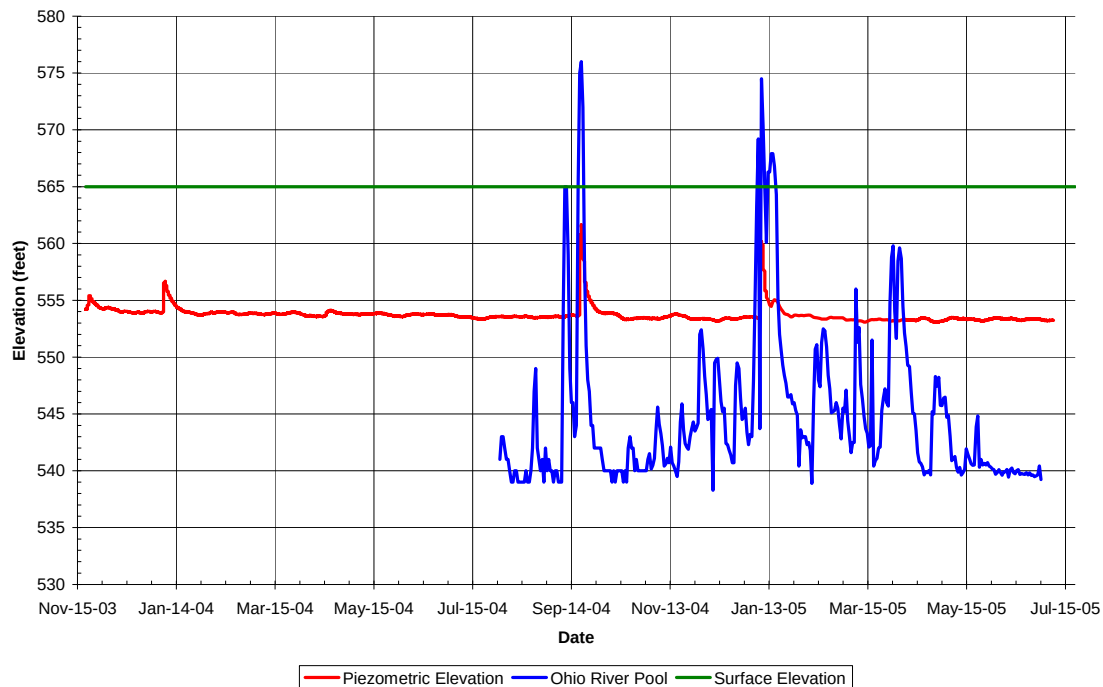


Figure 4. PZ-1 Piezometric Elevation vs. Ohio River Pool

2.3. Slope Inclinator Results and Interpretations

Twenty-eight (28) slope inclinometers have been installed during different intervals at the project site. Slope Inclinator Nos. 1 and 2 were the inclinometers installed before embankment construction began in October of 2003. The second group of inclinometers were installed in November of 2003 and consisted of Nos. 3 through 8. Slope Inclinator Nos. 9 through 14 were installed from January through February of 2004, and Nos. 15 through 20 were installed in March of 2004. Slope Inclinator Nos. 20 through 28 were installed in June of 2004.

Since installation, four slope inclinometers (Nos. 7, 14, 22 and 24) have been destroyed by on-site construction activities. An additional fifteen (15) slope inclinometers have been closed due to excessive movement. The remaining nine inclinometers are being monitored on a bi-weekly basis.

FMSM prepared graphs showing rate of movement over time plotted against measured river elevations for Slope Inclinator Nos. 17, 23 and 25. The graph for SI-17 is shown in Figure 5. Increased rates of movement coincided with elevated phreatic surface levels indicated from the piezometers.

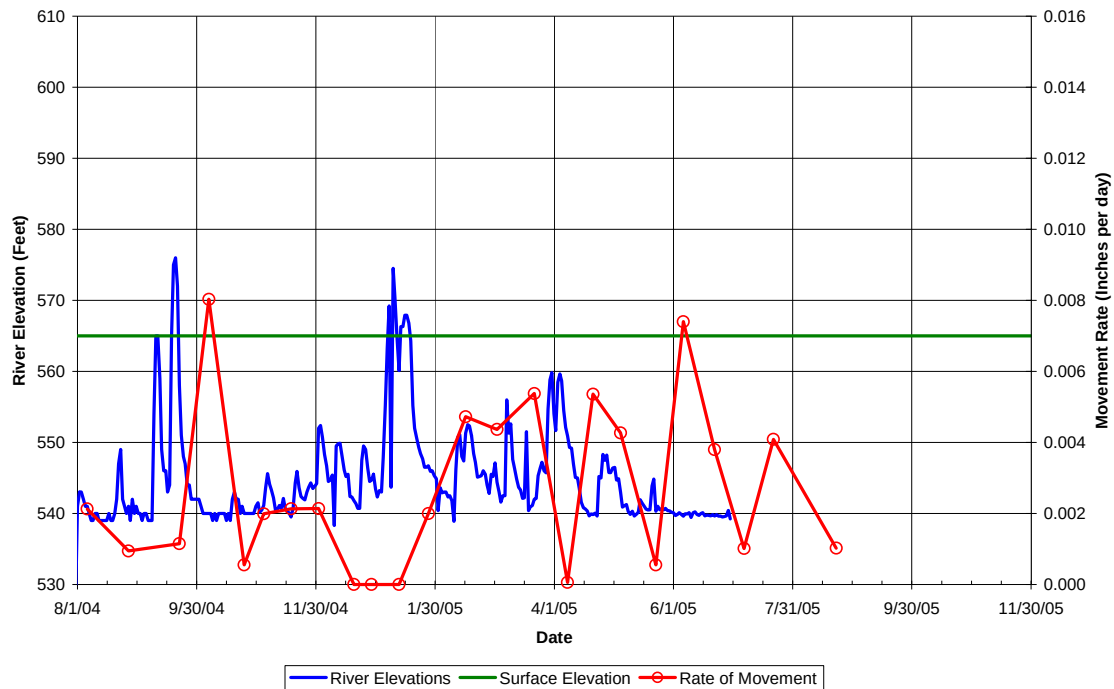


Figure 5. Slope Inclinator No. 17 Rate of Movement vs. Time (With Respect to Ohio River Elevation)

Based on the readings obtained to date, movement of the slope appears to be sensitive to elevated river levels. The movement rate for SI-17, 23 and 25 ranged from 0.0 to 0.008 inches per day. Generally, the rate of movement increased following high water events such as September 20, 2004. The rate of movement was also elevated from February to April 2005, coinciding with a relatively wet spring. Slope Inclinator Nos. 17 and 25 show an unexplained elevated rate of movement during the first few weeks of June 2005.

Comparing the information obtained from slope inclinometers within approximately 160 feet of the normal pool river bank leads to the following observations. The elevation of the movement surface steadily falls from elevation 537 near Station 51+79 to around elevation 505 near Station 47+00. From there, the surface remains relatively constant to Station 142+00. Based on the readings obtained from SI-16, the surface rises approximately 45 feet in elevation over a horizontal length of 52 feet. Heading west from this line, the movement surface appears to follow the rockline and is assumed to daylight near the exposed rock highwall. The movement surface was observed near the top of rock in each inclinometer except SI-3 where it was detected within the overburden (silty clay and clayey silt). It is believed that the existing bridge abutment and Ohio River pier, located approximately 20 feet southwest of SI-3, are providing some resistance to movement in this area. The maximum depth of the movement surface is approximately 65 feet below the ground surface and was measured between approximately Station 142+30 through 146+00.

Inclinometer installations indicate the movement extends as far south as the existing and proposed bridges. Slope inclinometer 23, which is approximately 70 feet west from the

proposed bridge abutment, continues to indicate movement. In this area the limit of movement appears to move due east toward the river. The northernmost inclinometer, SI-20, has been closed due to excessive movement; therefore, the northern limit of movement has not been identified.

Typically, inclinometer probes employ accelerometers to measure tilt. One accelerometer measures tilt in the plane of the inclinometer wheels or A-axis. A second accelerometer measures tilt in the plane that is perpendicular to the wheels or B-axis. A resultant direction of movement can be determined from the A and B-axis readings that are referenced to the grooves in the inclinometer casing. To determine the actual field direction of displacement, several of the inclinometer casings were surveyed by ODOT on June 10, 2004. The resulting direction arrows are shown in Figure 3.

The movement arrows can be grouped into two sections. The first section extends from Station 44+00 through the northern end of the project, where the direction of movement appears to be uniform and perpendicular to the ground surface contours toward the Ohio River. The second section extends from the existing and proposed bridges to Station 44+00. This area is not as uniform, with direction arrows pointing more easterly with a general direction toward the Ohio River. This inconsistency in direction may be caused by the influence of the existing bridge abutment and Ohio River pier locations.

Section 3 – Engineering Analyses and Results

3.1. General

Evaluation of the slide activity began with conducting reviews of existing topographic mapping, geologic and soils information and Ohio River elevation data. The next step was to review and interpret the data collected from on-site inclinometers and piezometers as discussed previously. The information generated during these reviews was then used to perform the following engineering analyses:

- A back-analysis of the slope stability for the embankment configuration at Main Street Station 145+00.
- A feasibility study to review possible remedial alternates to stabilize the slide between approximate Main Street Stations 141+50 and 54+50.
- Evaluation of slope stability and possible remedial alternates for the ahead-station approach embankment configuration in the area of the new bridge.

FMSM performed a back-analysis of slope stability using the embankment geometry at Main Street Relocation Station 145+00. At the time slope movement was first detected in inclinometers SI-1 and SI-2, approximately eight (8) feet of new roadway embankment fill had been placed. This fill height was used to model the cross-section geometry at this Station. The soil profile used in the analysis was approximated based on data available from boring logs generated during FMSM's initial geotechnical exploration for the roadway (October 31, 2001 report) and from boring logs developed during installation of slope inclinometers SI-6 and SI-8.

The slope inclinometer data was used to estimate the location of the failure surface at Main Street Station 145+00. Up-slope from SI-8 and down-slope from SI-6, the failure surface was estimated from results of other inclinometers, information from other borings and stability analyses that attempted to find the most critical condition.

The approximation of the piezometric surface elevation throughout the cross section at Station 145+00 was based upon Ohio River elevation data, available piezometric data, and on information obtained from a review of available boring logs.

A piezometric surface elevation of 556 feet was used at an offset of 70 feet right of centerline. A river surface elevation of 542 feet was used based upon the data collected from the Racine Lock and Dam lower pool gauge data. This elevation represents the average elevation of the river at the approximate time slope movement was first detected. This elevation is about four (4) feet above a normal pool elevation of 538 feet.

The shear strength parameters of the soil materials modeled within the subsurface profile at Station 145+00 were based upon the data collected during FMSM's initial geotechnical exploration of the roadway. The parameters were presented in FMSM's roadway report dated October 31, 2001 and are summarized in Table 3 for clarity.

Table 3. Shear Strength Parameters Used in Stability Back-Analysis

Soil	Unit Weight (pcf)	Effective Cohesion	Effective Angle of Internal Friction
New Embankment Fill	125	200	20.0
Existing Fill	130	0	28.0
Alluvial Silts and Clays	125	140	32.5
Residual Clay	125	350	23.4

The back-analyses of slope stability were performed using the REAME (Rotational Equilibrium Analysis of Multilayered Embankments, two-dimensional, version 2K) computer program. As previously described, the specific results of a back-analysis are the operable shear strength parameters of the materials along/near the failure surface. A cohesion value of zero (0) was assigned to the materials along the failure surface. The angle of internal friction for those materials near the failure surface was then adjusted until a safety factor of 1.0 was obtained for the stability analysis. The resulting angle of internal friction for the materials along the failure surface was 11.9 degrees.

Section 4 – Feasibility Study of Stabilization Alternatives

4.1. General

Upon detection of the initial slope movements north of the new bridge location, ODOT directed FMSM and URS to evaluate possible landslide stabilization alternates that could accommodate continued construction of the proposed roadway and bridge foundation elements on the Ohio side of the river. FMSM and URS performed a feasibility study to evaluate preliminary alternates and to estimate the potential costs of each alternate.

The alternates were developed with the intent of stabilizing the entire area in which movement was indicated by the inclinometer data. This data indicated that the limits of the slide ranged from about Main Street Station 141+50 to Station 54+50.

4.2. Drilled Shafts – Cantilevered or Tied Back

This remediation alternate consisted of installing reinforced concrete drilled shafts near the toe of the proposed embankment. With this option, the shafts would be socketed some depth into rock to interrupt the failure surface, and the tops of the shafts would be extended to the ground surface. The conceptual basis for this option is that soil arching effects between the drilled shafts, in conjunction with the resistance developed by socketing the shafts into rock below the failure plane, would arrest the slide. Depending on the magnitude of the loading, tie-back anchors may be required. Advantages of this option are that the shafts intercept the failure surface, improve stability in the alluvial silt/clay and the contractor was equipped to install drilled shafts. The disadvantage is that the high moment and shear forces induced on the shafts require large diameter shafts, possibly with tiebacks, which would be quite expensive.

4.3. Truncated (Shortened) Drilled Shafts

A concept of using shortened or “truncated” drilled shafts was explored as a way to install a configuration of shafts that could interrupt the failure surface and function as a “shear key”. With this concept, the bottom of the reinforced concrete section of the shaft would extend some distance below the estimated failure surface. The top of the reinforced section would extend above the estimated failure surface but would be terminated well below the ground surface. The shaft excavation above the reinforced section would be back-filled with a flowable fill, or some other material, such that a structural connection between this material and the reinforced concrete section would not exist. This would result in reduced loads and moments on the shafts such that practical shaft diameters and steel reinforcement could be achieved. Evaluation of potential additional failure surfaces above the existing failure surface or above the tops of the shafts would need to be completed with this concept. An advantage of this alternative is that less moment is induced than with the full-length shafts, while still intercepting the failure surface, which results in a less expensive remediation than full-height shafts. The disadvantages are that there would still be short term stability concerns in the alluvial silt and clay and there is some uncertainty about magnitude and distribution of load for this case.

4.4. Tied-Back Precast Blocks

FMSM also contacted a geotechnical specialty contractor and asked them to provide a conceptual remedial alternate to stabilize the slide. The remedial option presented consisted of a system of ten (10)-foot by ten (10)-foot reinforced concrete mats spaced 12.5 feet on center, with one tie-back anchor per mat. The anchor would extend into competent bedrock. It was estimated that two (2) rows of mats would be necessary to stabilize the slope. The estimated cost for this method was lower than other options; however, FMSM again had concerns about the reliability of the tie-back anchors within a moving soil mass, as well as the lack of detailed performance data for this type of system.

4.5. Preferred Options and Related Decisions

A series of meetings were held in August 2004 to discuss alternatives for the project. These meetings included representatives from ODOT, WVDOT, FHWA, FMSM and

URS. As a result of these meetings, several key decisions were made regarding how to deal with the detected movement. These decisions included:

- Structural measures would be employed to protect the foundations for the new bridge, specifically the Ohio River tower foundation, the touch-down pier foundation and the Ohio abutment. The measures would consist of a single-row of full height drilled shafts constructed near the river bank, just uphill of the Ohio River tower.
- No structural measures would be taken to protect the roadway portion of the Ohio approach at this time. The roadway alignment would be modified such that no additional embankment would be required.
- The foundation systems for the Ohio touch-down pier and the Ohio abutment would be changed from driven H-piles to drilled shafts in order to provide more resistance to lateral loads.
- An instrumentation program would be implemented in order to monitor the existing bridge and the proposed foundations for movement.

The final design for the protective measures consisted of nine, eight-foot diameter drilled shafts, spaced twelve feet on center, located 140 feet right of the proposed Main Street centerline. The shafts would extend to the ground surface and be socketed 30 feet into bedrock.

The methodology used to estimate the sizes, spacing and location of the drilled shafts is explained in a paper by Robert Liang, PhD, PE of the University of Akron (in cooperation with ODOT and the FHWA) entitled "Drilled Shaft Foundations for Noise Barrier Walls and Slope Stabilization". The methodology detailed in Dr. Liang's paper was also used to estimate the loadings on the drilled shafts. Generally, the methodology explained in the paper uses a limit equilibrium (method of slices) analysis to compute the load per unit length of landslide earth pressures. This load is then converted to a load per drilled shaft, which is dependent upon the shaft diameter, center-to-center spacing and soil arching properties. In Dr. Liang's paper, this conversion to load per drilled shaft was estimated by performing numerical analyses for typical soil strength parameters, drilled shaft diameters and center-to-center shaft spacing.

The computer program LPILE PLUS was used to model the shaft/soil/bedrock interaction in order to estimate the required rock socket lengths (below the estimated failure surface) and to estimate the shear and moment distributions for the shafts and shaft deflections at the estimated failure surfaces. The maximum moments, maximum shears and deflections of the shafts were estimated to facilitate reinforcing steel design by URS.

Using the methodology described above a factored resultant force of 2,098 kips was predicted for each shaft. The maximum factored shear, maximum factored moment and anticipated deflections for each shaft were as follows:

Maximum factored shear	= 3,679 kips
Maximum factored moment	= 37,942 ft-kips
Deflection (top of shaft)	= 5.4 inches
Deflection (top of bedrock)	= 0.6 inches

Section 5 – Instrumentation Program

5.1. General

The instrumentation program consisted of three components:

- Instrumentation of the existing bridge,
- Instrumentation of selected foundation elements of the Ohio touch-down pier, and
- Instrumentation of the stabilization shafts.

For the existing bridge, ten uniaxial tiltmeters (Applied Geomechanics 6350) were installed at five locations, those being the top and bottom of the Ohio land pier and water pier, and at the top of the Ohio abutment. At each of the five monitoring points, two uniaxial tiltmeters with built-in thermistors were attached to the vertical concrete surface using epoxy bound stainless steel anchors and mounting brackets. In addition, four other thermistors were installed on the bridge to monitor temperature. Survey targets were also mounted on the existing abutment and piers for monitoring by conventional optical techniques. Cabling from the tiltmeters and thermistors was routed to a datalogger and multiplexer located near the Ohio abutment.

As noted previously, the foundation system for the Ohio touch-down pier was revised to consist of seven, six-foot diameter drilled shafts. Instrumentation was installed in two of the drilled shafts, one on the uphill row and one on the downhill row. Slope inclinometer casing was installed in the two shafts, along with nine levels of sister bar strain gauges. At two of the elevations, four individual vibrating wire strain gauges (Geokon 4911) were mounted at locations representing the axes perpendicular and parallel to the surrounding slope. At the remaining seven elevations, one gauge was mounted vertically to the reinforcing cage on the tension (upslope) side of the shaft and one was mounted on the compression (downslope) side. Cabling from the instruments was routed to a datalogger and multiplexer mounted on the pier stem.

Three of the stabilization shafts were instrumented as well, those being the two shafts on the end and the center shaft. The instrumentation package was similar to that installed in the touch-down pier shafts, with the addition of a submersible biaxial tiltmeter (Applied Geomechanics 802) mounted near the tops of the shafts.

5.2. Planned Automation and In-Place Inclinometers

Following the development of the stabilization shaft instrumentation plan, a cost-benefit analysis was developed for completely automating the data collection and storage process in lieu of periodic manual data collection methods performed onsite. It was assumed the monitoring period would be long term extending for the duration of the construction period and beyond. Since the planned group of instruments included datalogger systems at two of the three substructures that are currently monitored, the option to add datalogging capabilities at the third substructure provided the most benefit at the least long-term cost, based on the results of the analysis.

Only the Pier 7 instruments consisting of strain gauges and inclinometers did not initially utilize a datalogger collection component. Furthermore, all of the instruments utilize either vibrating-wire (VW) or electrolytic level sensors; therefore, the CR10x datalogger

is compatible. Programming and configuration of the dataloggers was provided by the tiltmeter supplier Applied Geomechanics Inc., Santa Cruz, CA.

The automated data acquisition system (ADAS) will require the addition of in-place inclinometers at Pier 7 Shafts Nos. 52 and 54 and the Stabilization Shafts Nos. 1, 5 and 9. A datalogger, multiplexers and radio transmitter components will be added to the Pier 7 readout station. The existing bridge datalogger system was selected to serve as the base station. The base station is equipped with an analog phone modem and radio frequency transmitter.

Each of the remaining two instrument clusters will communicate via radio telemetry equipment to the base station. Currently, the existing bridge instrumentation is operating under the ADAS. Readout stations will be assembled for Pier 7 and the stabilization shaft instruments. The remaining ADAS will be implemented in a phased approach by taking advantage of regularly scheduled data collection trips to install the ADAS components as configuration and testing of the equipment are completed. The plan is to implement radio frequency transmitters at the existing bridge readout and base stations. Next, two readout stations will be assembled and protected in free-standing NEMA 4 enclosures mounted to concrete pads. NEMA 4 enclosures are constructed for either indoor or outdoor use to provide a degree of protection against falling dirt, rain, sleet, snow, windblown dust, splashing water, and hose-directed water. The components of each readout station will consist of multiplexers, datalogger, solar panel, regulator, battery and radio frequency transmitter components. The last equipment to be added to the ADAS will be in-place inclinometers.

Five of the eleven inclinometers installed will be instrumented with an in-place inclinometer (IPI) system. The selected inclinometers are located at the Pier 7 Drilled Shafts Nos. 52 and 54 and the Stabilization Shafts Nos. 1, 5 and 9. Manual slope inclinometer readings will continue until the ADAS is brought completely online. To establish a baseline and initial profile of each casing, a minimum of two manual reading sets were recorded for all eleven inclinometers in the project database. In the event of an apparent rotation, the IPI string may be temporarily removed and manual readings will be performed to confirm the event.

In-place inclinometers (IPIs) are essentially tiltmeters used to precisely measure the angular rotation at their installation point. The tilt measurements of several instruments can be used to obtain a displacement profile. Each IPI is designed with guide fins to precisely fit into the primary axis grooves in the inclinometer casing. Each instrumented casing will receive a string of five IPIs hanging in tension on a fixed length, evenly spaced, cable or solid rod system from the top of the casing.

The automated IPI system differs from manual inclinometer data collection method in two ways. First, the automated system monitors five locations within the casing. The manual inclinometer process records rotation over two feet within the casing. Second, manual reading sets are scheduled to be taken by field technicians once every two weeks. The automated system will be configured to record rotation once per hour at each sensor. Stored data will maintain within the datalogger's flash memory until downloaded. The memory capacity of the system will allow for up to four weeks of data collection before the memory reaches capacity and begins overwriting the oldest stored data. Data will be collected, reviewed and analyzed for anomalies twice per week.

Section 6 – Instrumentation Results

6.1. Existing Bridge Tiltmeters

The tiltmeters are very sensitive, capable of measuring tilt accurately to less than 0.001 degrees. Consequently, the instruments are able to detect not only movement due to slope movement, but also the normal small-scale movements of the structure to which they are attached. The most significant of these types of movements is related to the daily changes in ambient temperature. Because these tilt movements are not associated with slope movement, for the purpose of this project they are considered “noise” that should be filtered out of the data if possible.

Initially, a linear regression was computed for each tiltmeter reading as a function of its own temperature sensor reading. Based on the recorded temperature changes, this allowed for the calculation of movement due solely to temperature changes on the structure. This temperature-induced movement was then subtracted from the total movement in order to compute the estimated movement due to other causes. While this process removed a significant amount of the daily fluctuation in the readings, there was still some amount of “noise” in the data. Eventually, it was decided that the uncertainties associated with this approach did not justify its continued use and the application of linear regression analysis was discontinued.

Figure 6 shows the variation of temperature readings from the tiltmeters as well as from independent thermistors that were mounted on the bridge. Figure 7 presents tilt measurements in the “Y-direction”. Several of the instruments indicate a very small (less than 0.05 degrees) downslope tilt. It is possible this apparent movement is due to seasonal variations and will continue throughout the summer before reversing. At present, it is not possible to conclude that slope movements are affecting the existing bridge.

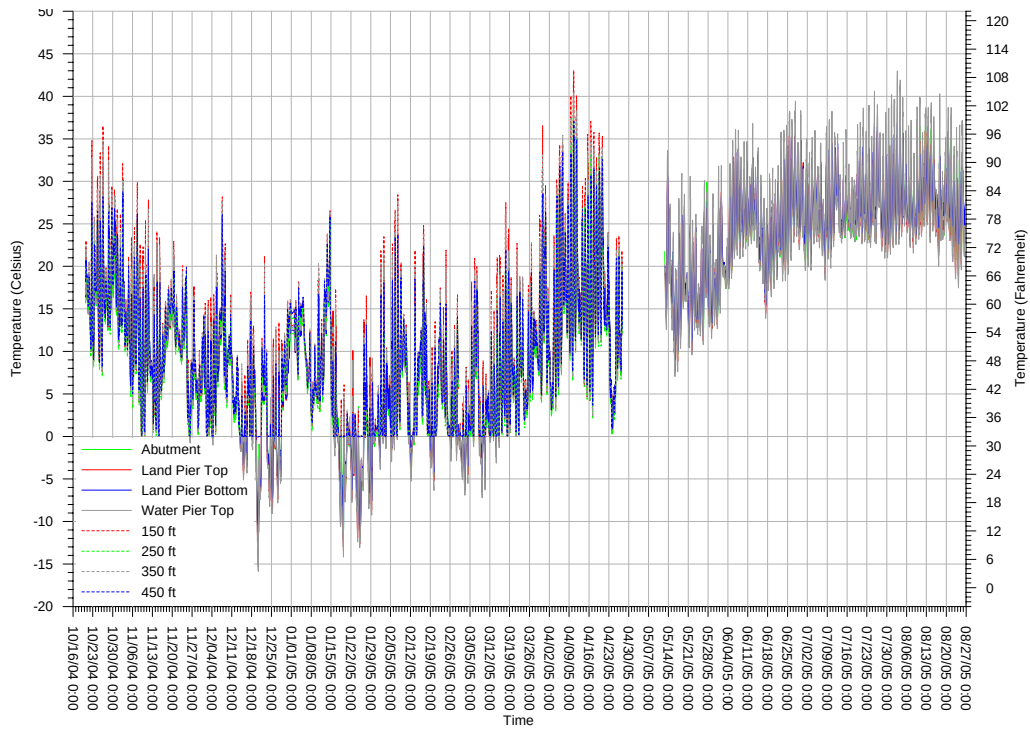


Figure 6. Temperature Readings –Tiltmeters and Independent Thermistors

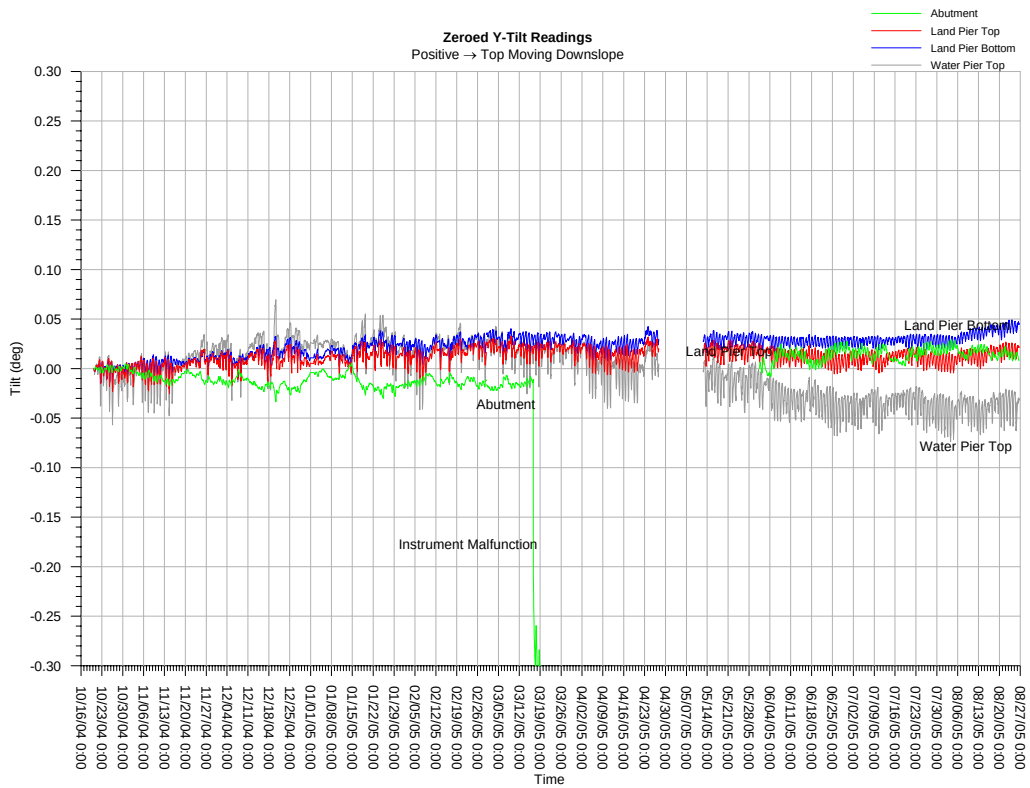


Figure 7. Tilt Measurements in the “Y-direction”

6.2. Touch-Down Pier Drilled Shafts

The most recent observations from both shafts indicate small movements downslope and in the upstream direction. Further observation will be needed over time to see if these apparent trends are the result of construction operations, random variations, or actual shaft displacement caused by slope movement. Figure 8 shows the cumulative displacement at the top of Drilled Shaft 52.

Figure 9 shows the moment diagram for Drilled Shaft 52 computed from the interpreted strain gage readings as a function of time. There was an increase in moment from May to August of 2005. It is unknown if this movement is due to slope movement or to the construction of a temporary steel falsework directly over the shaft, removal of formwork from the pier, and/or pile-driving and soil placement at the site. Construction activities have complicated both the collection and the interpretation of data.

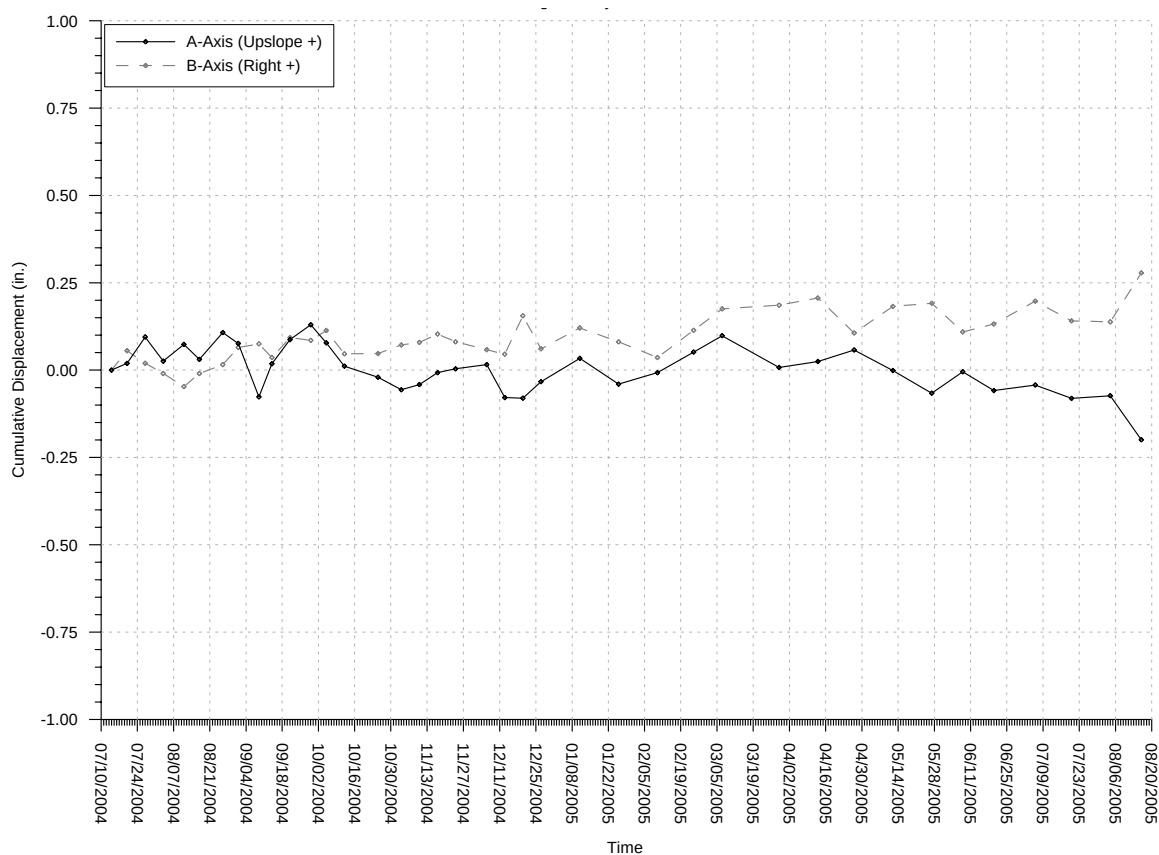


Figure 8. Cumulative Displacement at Pier 7 Drilled Shaft 52

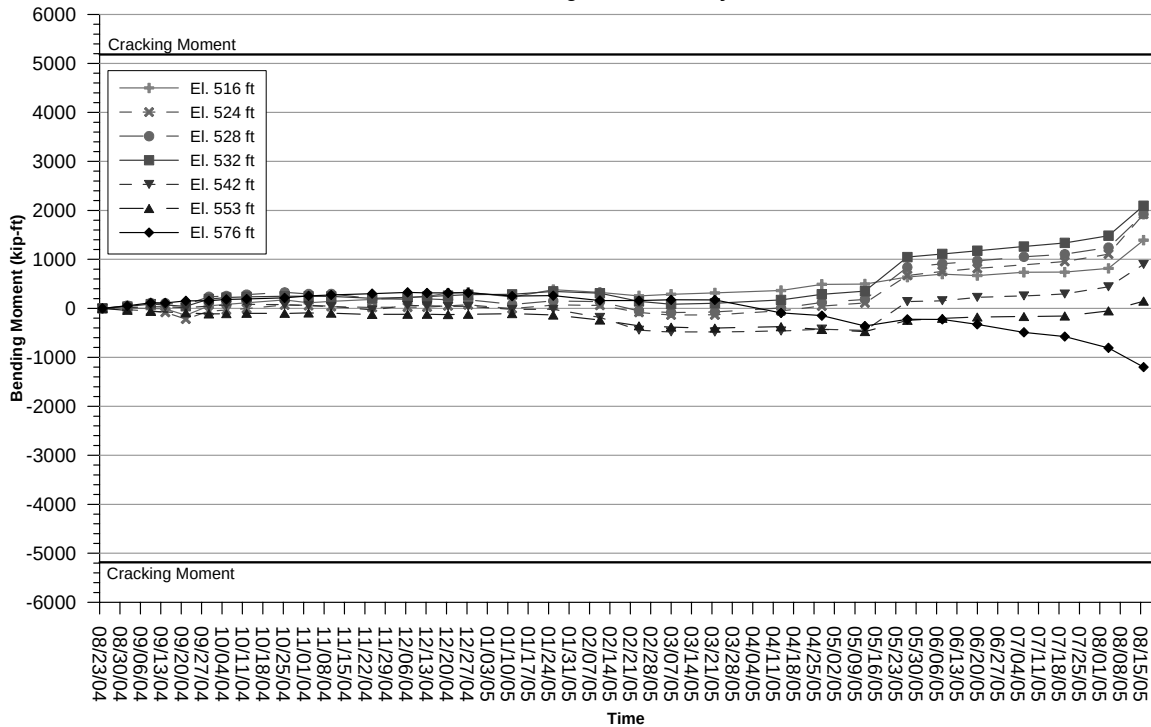


Figure 9. Moment Diagram – Pier 7 Drilled Shaft 52

6.3. Stabilization Shafts

The stabilization shafts were constructed from late November 2004 to late January 2005. Figure 10 shows the work pad being prepared for shaft installation and Figure 11 shows one of the reinforcing steel cages being prepared. The plastic box shown near the top of the reinforcing cage was used to protect the instrument leads. The Contractor prepared a temporary drilling pad and bench to install the shafts, resulting in the tops of the shafts being 10 to 12 feet below the ground surface. It was initially thought that the temporary fill would be removed immediately after completion of shaft installation; however, the fill was not removed until May 2005. Slope inclinometer and stain gauge readings were taken at two week-intervals beginning in early June 2005. The initial data is being used to establish baseline conditions.



Figure 10. Work Pad Being Prepared for Stabilization Shaft Installation



Figure 11. Reinforcing Steel Cage Being Prepared

Section 7 – Future Steps

Completion of the automated data acquisition system will facilitate frequent monitoring of the touch-down pier drilled shafts, stabilization shafts and the existing bridge. The existing bridge is scheduled for demolition in early 2007 following completion of the new bridge; however, the drilled shafts can be monitored for many years using the remote data collection system in a cost effective manner. The collected data will be analyzed periodically, and the results can be used to evaluate and refine the design methodology.

References

Liang, Robert Y. (2002), Drilled Shaft Foundations for Noise Barrier Walls and Slope Stabilization.

Drilled Shaft Design and Construction - Case Histories in New Jersey

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ABSTRACT: Drilled shafts designed by Gannett Fleming for two projects in New Jersey have recently been constructed. The drilled shafts included embedment in the Triassic age Brunswick Formation, a soft red shale with some interbedded sandstone formations and material properties difficult to estimate. The Route 1/130/171 Interchange included thirty-seven (37) drilled shafts, 1.2 meters (4 feet) in diameter, designed using AASHTO's Standard Specifications for Highway Bridges, 16th Edition. The Route 18 Extension, Section 2A included eight (8) drilled shafts, 1.8 meters (6 feet) in diameter, designed using FHWA's Intermediate Geomaterial Method (IGM). Three load tests using Osterberg Cells to verify axial and lateral capacity were performed for these projects. A comparison between the different approaches taken to determine the axial and lateral capacity in the Brunswick Formation and how these estimates compared to the load test results was then performed. The present AASHTO method suggests using equivalent soil mass for estimating tip the resistance when rocks are defined by very poor quality (RQD<25%), which results in minimum tip resistance. In contrast, FHWA's IGM method allows using intermediate material when the rock is also of very poor quality which results in higher capacity. The comparison showed that for the axial capacity estimation, the AASHTO method provided less capacity compared to the IGM method. In addition, the estimated capacity using the IGM method matched better with the load test results. The comparison also revealed that the load test results matched well with the lateral capacity estimation modeled with L-Pile using cohesive properties rather than cohesionless properties.

INTRODUCTION

The Route 1/130/171 Interchange project included the design and construction of interchange improvements for Route U.S. 1 Section 6T, Route U.S. 130 Section 16E, and N. J. Route 171 Section 1B in the Township of North Brunswick, Middlesex County, New Jersey. The project eliminated the signalized intersection of Route 1/130/171 with grade separated structures and provided direct connections for the major traffic movements – specifically Route 1 southbound to Route 130 southbound and Route 130 northbound to Route 1 northbound. The project also consisted of widening Route 1 to furnish a continuous three lanes in each direction and to furnish full shoulders and auxiliary lanes.

Route 130 northbound and southbound extended over Route U.S. 1 by way of two bridge structures. The design included the connector road (Connector A) from Route US 1 southbound to Route U.S. 130 southbound. The connector crossed over Route U.S. 1 and Route U.S. 130 northbound. Connector B diverged from Route US 130 northbound, underpasses Frontage Road, and accessed Route U.S. 1 northbound. The Frontage Road intersection with Route U.S. 130 is a modified single point urban intersection. The connectors A and B passed beneath the intersection.

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In order to facilitate the proposed interchange, six ramps, four bridge structures, eleven retaining walls, one culvert, thirteen sign support structures, four tower lighting structures, and one access road were designed and constructed. A project location map with completed structures is presented as Figure 1.



Figure 1: Project Location Map with Completed Structures for Rt. 1/130/171

The Route 18 Extension, Section 2A project is located in Piscataway Township, Middlesex County, New Jersey. The primary purpose of this project was to improve the proposed Route 18 corridor along Metlars Lane by eliminating the existing “bottleneck” condition. This improvement provided the missing link in the State Highway System, a continuous four-lane arterial-highway extending through Piscataway Township, providing traffic with improved access to Interstate 287, Rutgers University, and employment centers of Piscataway Township.

The project involved construction of a four-lane, 2.7 kilometer (1.7 mile) limited access roadway from the John A. Lynch, Sr. Memorial Bridge (over the Raritan River), south of River Road, along the current Metlars Lane corridor (County Route 609), and then connected to existing Hoes Lane (Route 18, Section 3A). Metlars Lane was an existing two-lane collector roadway, which connected the existing four-lane highway at the John A. Lynch Bridge to Hoes Lane. Hoes Lane was an existing four-lane median divided land service arterial which provides access to Northern Piscataway Township’s employment centers and Interstate 287. The new Route 18 four-lane alignment eliminated the existing two-lane “bottleneck” on Metlars Lane and provide access to River Road (County Route 622), Busch Campus of Rutgers University, Metlars Lane, Davidson Road, Avenue E (Livingston Campus of Rutgers University) via grade separated interchanges.

The proposed construction included six bridge structures, twelve retaining walls, one box and six arch culverts, and eleven sign structures. A project map is included in Figure 2.

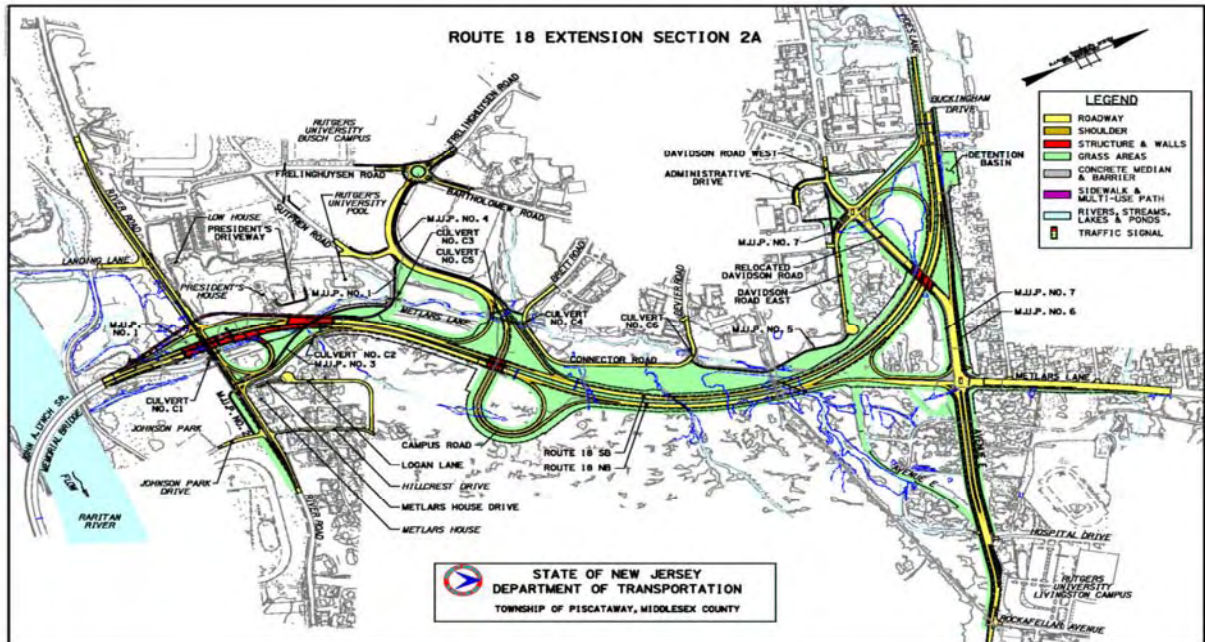


Figure 2: Project Map for Rt. 18 Extension, Section 2A

SITE GEOLOGY

Physiography

According to the New Jersey Geological Survey (1994), the projects are situated in the Piedmont physiographic province, which is characterized by undulating to gently rolling hills, separated by flat valleys. Differences in elevation are comparatively slight. The soils consist of silt and clayey silt overlying Triassic age shale bedrock (shown as Brunswick shale on the Geologic Map of New Jersey) with minor occurrences of interbedded sandstone.

Bedrock

The projects are underlain by Triassic age Brunswick shale, estimated to be 1800 to 2400 meters thick, which is composed primarily of soft red shale with some interbedded sandstone (Rogers, 1955). The shale is friable and generally severely weathered and highly fractured. Generally the bedrock is characterized by thin to thick beds of soft shale, with occasional beds of fine-grained sandstone, all dipping gently to the northwest. The shale crumbles readily into small fragments. Near diabase intrusions, the shale has been altered somewhat and is slightly harder. The Brunswick shale has a dull-red color (Holman and Jumikis, 1953).

Site Soils

According to the *Engineering Soil Survey of New Jersey, Report No. 10*, the predominant soil type within the vicinity of the project is designated as SH-4. The SH-4 soil is primarily silt and clayey silt and is described in the literature as fine-grained or A-4 in accordance with the AASHTO classification system. Although some of the soils may have large shale fragments,

upon weathering or when subjected to mechanical abrasion, the fragments readily breakdown into silt particles, and an initial visual classification of A-2 readily degrades to A-4. The depth to bedrock is usually 1 to 2 meters (Holman and Jumikis, 1953).

SUBSURFACE CONDITIONS AT DRILLED SHAFT LOCATIONS

Rt. 1/130/171

A typical subsurface profile for pier locations where drilled shafts were used is presented in Figure 3. Two borings, B-6 and B-7 were used to evaluate the subsurface conditions at this location. The overburden soil predominantly consists of 1.5 to 1.9 meters of residual clayey silt or silt and sand underlain by 7.6 to 10.2 meters of decomposed shale. The SPT N-value of the clayey silt and sand layer was 15 blows per 0.3 meter. The decomposed shale is very dense and consists of medium to fine sand and clayey silt with rock fragments. The SPT N-values of the decomposed shale was generally over 100 with values as low as 23 blows per 0.3 meter encountered.

Shale bedrock was encountered from 9.2 to 12.0 meters below existing grade. The bedrock varies from soft to hard and is slightly to moderately weathered with closely to medium spaced fractures. The core recoveries varied from 80 to 100 percent with Rock Quality Designations (RQD) varying from 7 to 95 percent.

The 24-hour groundwater readings indicate that the groundwater elevations varied from 26.0 to 28.8 meters at the time of the subsurface investigation. Three long-term observation wells were installed in the vicinity of bridges. The recorded long-term groundwater levels from the observation wells fluctuated from Elevation 26.1 to 28.6 from October 1997 to January 1999.

Rt. 18 Extension, Section 2A

A typical subsurface profile for piers where drilled shafts were used is presented in Figure 4. In general the overburden soil along the bridge sites consists of silty clay/clayey silt overlying decomposed shale. The decomposed shale is underlain by shale bedrock.

The thickness of the decomposed shale layer varied between 0.15 and 5.7 meters. The decomposed shale layer was generally in a medium dense to very dense state. The SPT N-values are ranged between 16 over 100 blows per 0.3 meter.

Shale bedrock was encountered at elevations ranging from 14.5 to 3.5 meters. Bedrock is described as soft to moderately hard shale, severely to slightly weathered, with moderately close to extremely close joint spacing. Individual core recoveries ranged from 60 to 100 percent, averaging 76 percent. Rock Quality Designation (RQD) ranged from 0 to 48 percent, averaging 22 percent.

Twenty-four-hour groundwater readings indicate that groundwater varies between elevations 2.5 and 14.5 meters at the time of the investigation. Two long-term observation wells were installed in the vicinity of bridges. Groundwater readings recorded in observation wells between September of 1996 and June of 2001 indicate that the groundwater level elevation ranged from 8.90 meters to 4.44 meters.

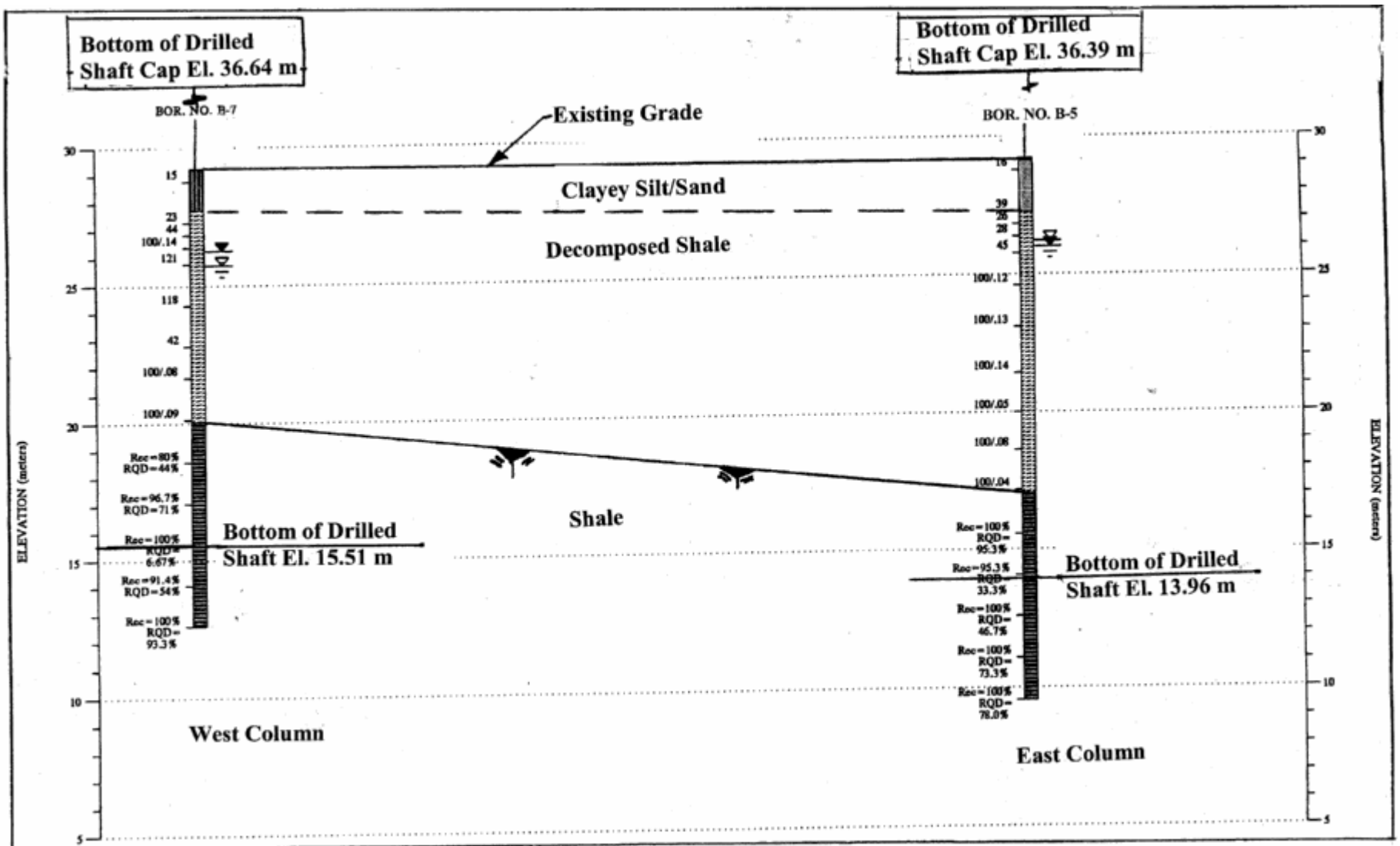


Figure 3: Typical Subsurface Profile at Pier Locations for Rt. 1/130/171

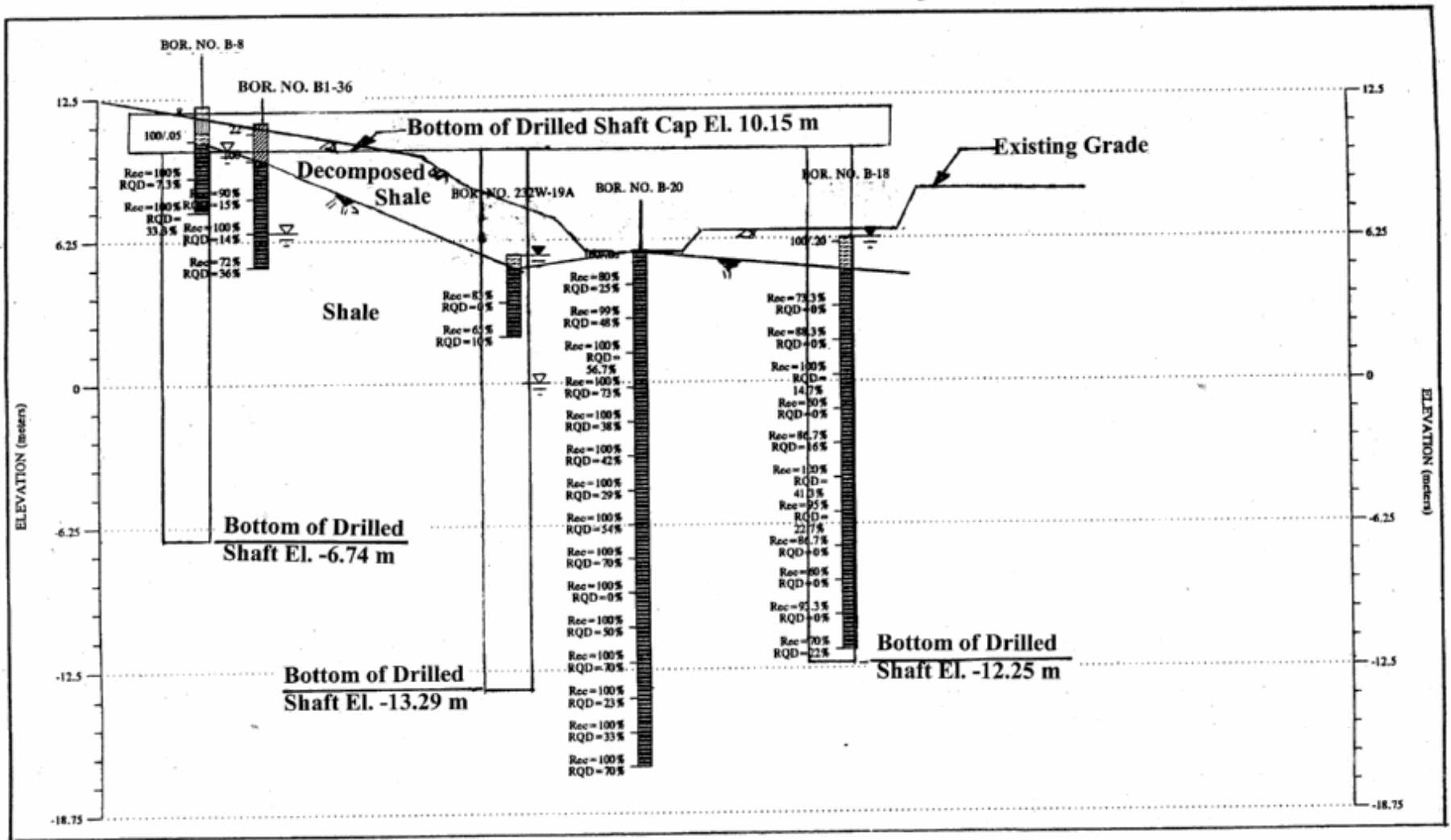


Figure 4: Typical Subsurface Profile at Pier Locations for Rt. 18 Extension, Section 2A

LABORATORY TESTING

Three undisturbed samples of the decomposed shale were obtained using a Denison sampler within the Rt. 1/130/171 project limit. Consolidated Undrained Triaxial (CIU) tests, grain size distribution, Atterberg limits, and unit weight determinations were performed on these samples by URS Corporation Inc. The decomposed shale is classified as SW-SC in accordance with the USCS classification system and denoted as A-2-7 in accordance with the AASHTO classification system. The unit weight of the decomposed shale varies from 18.7 kN/m^3 to 21.3 kN/m^3 (119.0 to 135.5 pcf). Based on the CIU tests, the friction angle of the decomposed shale varies from 39° to 53° .

Point load tests were performed at Gannett Fleming's in-house laboratory in accordance with ASTM D 5731. A total of 25 rock core samples were selected from the Rt. 1/130/171 project site. The axial unconfined compressive strength of intact shale ranges from 7.3 to 25.1 MPa (1057 to 3638 psi) and averages 15.5 MPa (2,244 psi). The diametral unconfined compressive strength ranges from 2.0 to 29.8 MPa (from 284 to 4,315 psi) and averages 15.9 MPa (2,309 psi). A total of 36 point-load tests on selected rock samples from Rt. 18 Extension, Section 2A were also performed. The 36 point load tests on the rock core specimens indicate that the axial unconfined compressive strengths range from 36.4 to 112.2 MPa with an average of 60.3 MPa and diametral unconfined compressive strengths vary from 23.0 to 55.6 MPa with an average of 41.5 MPa.

AXIAL CAPACITY DESIGN METHODS AND FIELD LOAD TESTING

AASHTO Method:

Axial capacity of the drilled shafts for the Rt. 1/130/171 project was estimated in accordance with AASHTO (1996), "Standard Specifications for Highway Bridges". Thirty-seven drilled shafts were designed and constructed with a 1.2 meters (4 feet) diameter shaft through the overburden soil and a 1.07-meter (3.5 feet) diameter shaft through the bedrock. The design length of the drilled shafts varied from 13.7 to 18.3 meters (from 45 to 60 feet). The variation in the design length attributed to mainly two factors. One factor was the magnitude of the actual axial load, which varied from 1,890 to 3,113 kN (from 425 to 700 kips). Another factor that impacted the design length was the variation of RQD. Based on the AASHTO Table 4.4.8.1.2A equivalent soil mass was used for estimating the side and tip resistance when RQD is less than 25 percent. Side resistance was estimated using AASHTO Equation 4.6.5.3.1-1 and tip resistance was ignored (since the settlement is less than 100 mm (4 inch)) when RQD is equal to or greater than 25 percent. In determining the allowable axial capacity at the top of each column, a group efficiency factor of 0.62 was used based on the center-to-center spacing of the drilled shafts. Since load tests were performed, in accordance with AASHTO (1996), Section 4.6.5.4, a factor of safety of 2 against bearing capacity failure was used. Settlement of the drilled shafts was estimated using the design axial loads. The estimated settlement for the shafts was calculated to be less than 2 mm (0.1 inch). A summary of design length, diameter, socket length and estimated allowable axial capacity of drilled shafts is presented in Table 1.

TABLE 1: AASHTO METHOD (RT. 1/130/171 PROJECT)

Design Length m (ft)	Diameter m (ft)	Socket Length in Shale (RQD>25%) m (ft)	Axial Capacity kN (Kips)
13.7-18.3 (45-60)	1.2 (4)	0-4.9 0-15)	1,935-3,140 (435-706)

IGM Method:

Axial capacity of drilled shafts for Rt. 18 Extension, Section 2A project was estimated based on IGM method as outlined in FHWA Publication No. FHWA-IF-99-025. Eight (8) drilled shafts were designed and constructed with a 1.8-meter (6 feet) diameter shaft through both the overburden soil and decomposed shale (IGM). Decomposed shale with SPT blow counts greater than 50 blows per 0.3 meter were considered as cohesionless IGM. Side resistance was estimated using Equation 11.26 and tip resistance was estimated using Equation 11.11. The design length of the drilled shafts varied from 13.7 to 18.3 meters (from 45 to 60 feet). The design length in IGM varied from 13.7 to 18.3 (from 45 to 60 feet). The design length in IGM varied due to the variation in provided axial load, which are from 8,540 to 14,235 kN (from 1,920 to 3,200 kips). A factor of safety of 2.5 against bearing capacity failure was used. Settlement of the drilled shafts was estimated using the design axial loads and the estimated settlement was negligible. A summary of design length, diameter, length in IGM and estimated allowable axial capacity of drilled shafts is presented in Table 1.

TABLE2: IGM METHOD (RT. 18 EXTENSION, SECTION 2A PROJECT)

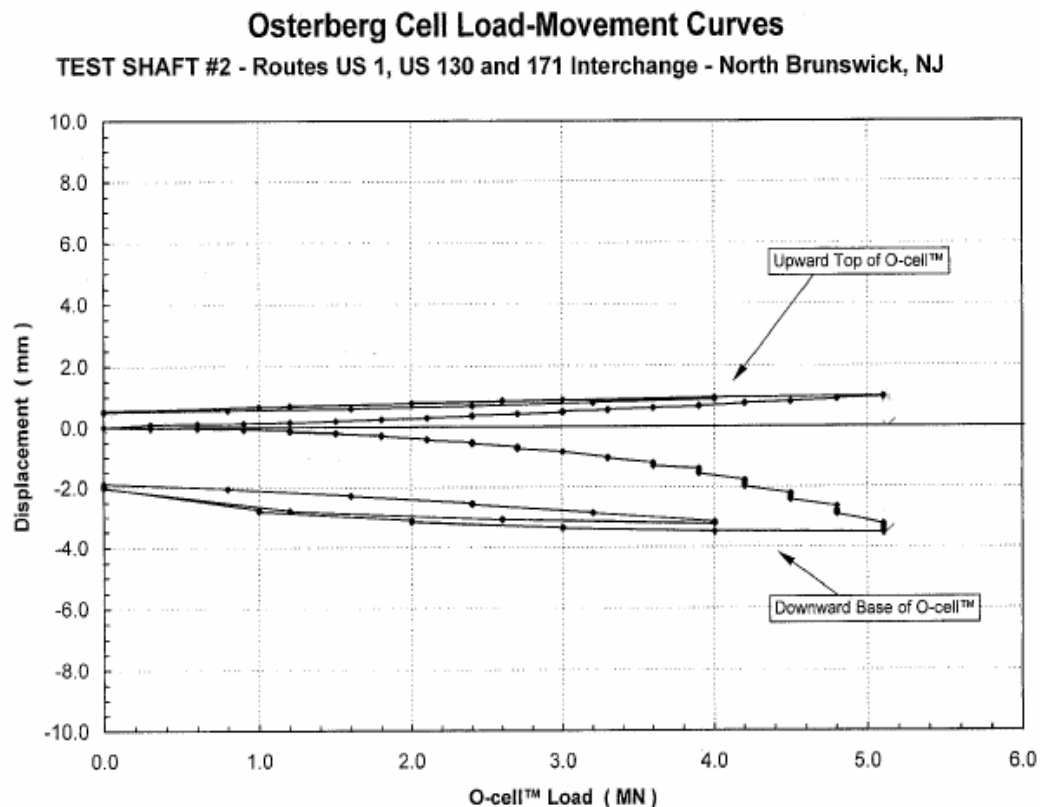
Design Length m (ft)	Diameter m (ft)	Length in IGM (RQD<25%) m (ft)	Axial Capacity kN (Kips)
13.7-18.3 (45-60)	1.8 (6)	13.7-18.3 (45-60)	8,559-14,364 (1,924-3,229)

Axial Load Testing

Two (2) full scale axial load testing was performed in Rt. 1/130/171 project and the load tests are designated as LT-1 and LT-2.

The load test LT-1 was performed on a 1.2-meter (4 feet) diameter uniform and 17.9-meter (58.6 feet) long demonstration shaft. A 330 mm (13 inch) diameter bi-directional O-cell was used to pressurize the drilled shaft. The maximum upward applied net load to the upper side shear was 5.6 MN (1,259 kips). At this loading, the upward movement of the O-cell top was 0.31 mm (0.011 inch). The applied net load to the end bearing was 4.67 MN (1,049 kips). At this loading the average downward movement of the lower O-cell base was 2.03 mm (0.08 inch). The unit end bearing at the base of the shaft was calculated to be 3.99 MPa (83.4 ksf) at the above noted displacement.

The load test LT-2 was performed on a 15.7-meter (51.5 feet) long production shaft. The test shaft consisted of a 1.2-meter (4 feet) diameter and 10.1-meter long shaft through the overburden soil and a 1.07-meter (3.5 feet) diameter and 5.6-meter long shaft through the bedrock. A 330 mm (13 inch) diameter bi-directional O-cell was used to pressurize the drilled shaft. The maximum upward applied net load to the upper side shear was 4.8 MN (1,080 kips). At this loading, the upward movement of the O-cell top was 1.0 mm (0.04 inch). The applied net load to the end bearing was 3.36 MN (755 kips). At this loading the average downward movement of the lower O-cell base was 3.5 mm (0.14 inch). The unit end bearing at the base of the shaft was calculated to be 3.8 MPa (78.5 ksf) at the above noted displacement. The load test results for LT-1 are included in Figure 5.



LOADTEST, Inc. Project No. LT-8835-2

Figure 5: Axial Load Test Result for Rt. 1/130/171 Project

One (1) full-scale axial load test was performed in Rt. 18 Extension, Section 2A project and load test is designated as LT-3. The load test LT-3 was performed on a 1.8-meter (6 feet) diameter and 19.3-meter (63.2 feet) long demonstration shaft. A 670 mm (26 inch) diameter bi-directional O-cell was used to pressurize the drilled shaft. The maximum upward applied net load to the upper side shear was 13.8 MN (3,110 kips). At this loading, the upward movement of the O-cell top was 0.23 mm (0.009 inch). The applied net load to the end bearing was 12.3 MN (2,760 kips). At this loading the average downward movement of the lower O-cell base was 75.3 mm (2.96 inch). The unit end bearing at the base of the shaft was calculated to be 4.67 MPa (97.5 ksf) at the above noted displacement. The load test results for LT-3 are included in Figure 6.

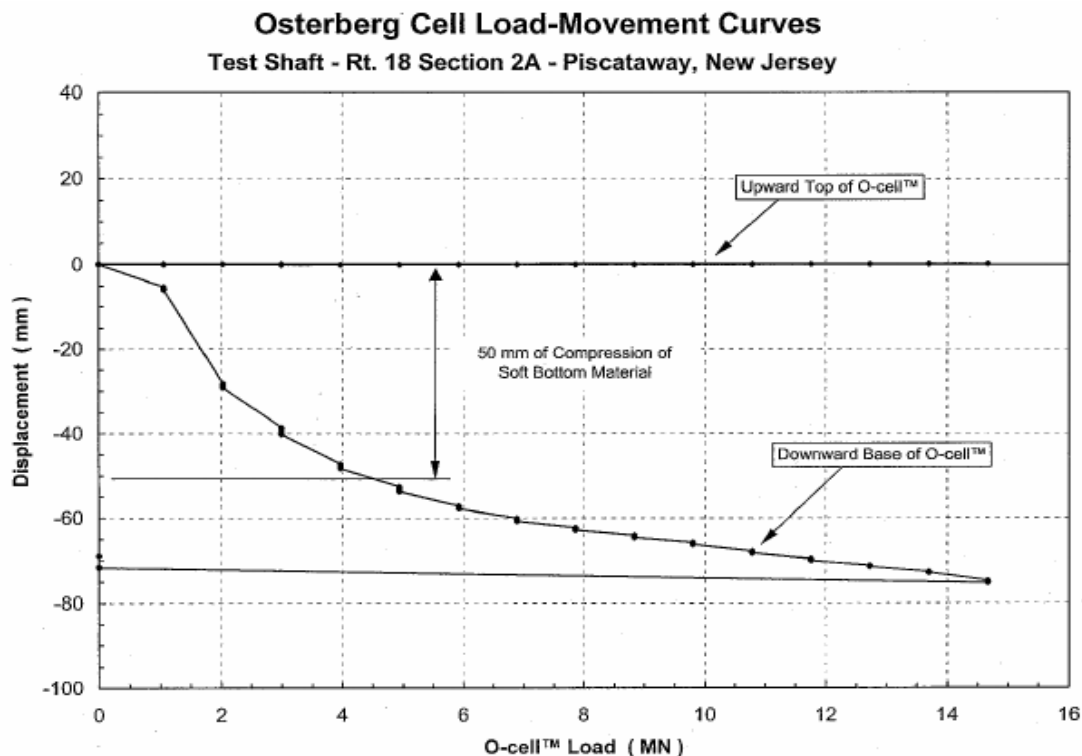


Figure 6: Axial Load Test Result for Rt. 18 Extension, Section 2A

A summary of the three axial load test (LT-1, LT-2, and LT-3) results and another test performed by others at the Menlo Park Mall, NJ, in Brunswick Formation reported in FHWA-SA-94-035 "The Osterberg CELL for Load Testing Drilled Shafts and Driven Piles" February, 1995 is included in Table 3.

TABLE 3: COMPARISON OF ESTIMATED AXIAL CAPACITY AND LOAD TEST RESULTS

	Estimated Capacity			Load Test Results			
	AASHTO Method		IGM Method	LT-1	LT-2	LT-3	Menlo Park Mall**
	LT-1	LT-2	LT-3				
Unit Side Resistance (ksf)	0.9-D.Shale 10.8-Shale	1.3	6.2	1.8 $\Delta = 0.013''$	1.9 $\Delta = 0.04''$	3.0 $\Delta = 0.009''$	7.4 $\Delta = 0.5''$
Unit End Bearing (ksf)	N/A	90.0	54.7	83.4 $\Delta = 0.08''$	78.5 $\Delta = 0.14''$	97.5 $\Delta = 3.0'' *$	68.0 $\Delta = 0.6''$
Where, Δ is deformation for the corresponding unit side resistance and unit end bearing * The displacement started almost as soon as the load was applied and gradually increased to 3.0 inch. Since the hole kept open for several days, the material at the bottom of the open hole might have expanded upward or the bottom of the hole was not cleaned properly resulting in the observed settlement under test load. ** Under Failed Condition							

From Table 3 the following conclusion are made:

1. The deformations due to the side friction are insignificant in LT-1, LT-2, and LT-3 compared to the Menlo Park Mall test result. In the Menlo Park Mall test failure

condition reached at 0.5 inch deformation. Therefore, it can be concluded that side frictions did not reach to a failure condition for Load Tests LT-1, LT-2, and LT-3.

2. The unit side resistance estimated by AASHTO method varied from 0.9 to 1.3 ksf. These values are very less than the unit side resistance of 7.4 ksf which is obtained in the Menlo Park Mall test.
3. Unit side resistance estimated by IGM method was 6.2 ksf which is relatively closer to the unit side resistance of 7.4 ksf obtained in the Menlo Park Mall test.
4. Estimated unit end bearing using AASHTO method was 90 ksf and by IGM method was 54.7 ksf, and the unit end bearing obtained in the Menlo Park Mall test result was 68 ksf. Therefore, the estimated unit end bearing using IGM method is closer to Menlo Park Mall test result.

LATERAL CAPACITY DESIGN METHODS AND FIELD LOAD TESTING

The computer program LPILE Plus 3.0 was utilized to analyze the lateral behavior of the drilled shafts. The drilled shafts were modeled using a fixed head condition in the transverse direction and a free head condition in the longitudinal direction. Reduction of the k-values due to group effects was considered in the transverse direction based on AASHTO Section 4.6.5.6.1.4. The lateral capacity and corresponding deflection at the drilled shaft heads are summarized in Table 4 for the maximum anticipated lateral load at that pier or column.

TABLE 4: ESTIMATED LATERAL CAPACITY USING L-PILE

Project Name	Pile Size		Lateral Capacity and Corresponding Deflection kN (mm)
	Diameter m (ft)	Length m (ft)	
Rt. 1/130/171	1.2 (4)	17.8 (55)	31-209 (1.5-48)
Rt. 18 Extension Section 2A	1.8 (6)	18.6 (61)	62-387 (0.5-18.2)
Note: Modulus of horizontal reaction, $k = 40$ pci for Sand $k = 225$ pci for Decomposed Shale (RQD <25%) $k = 1600$ pci for Shale Bedrock (RQD >25%) Shear strength, $c = 0$ psi and Frictional angle, $\phi = 36$ for Decomposed Shale Shear strength, $c = 350$ psi and Friction angle, $\phi = 0$, and Strain corresponding to one-half the maximum principal stress difference, $e_{50} = 0.002$ for Shale Bedrock			

Lateral Load Testing

Two (2) full scale lateral load testing was performed, one in Rt. 1/130/171 project and the other one in Rt. 18 Extension, Section 2A Project.

For Rt. 1/130/171 Project lateral load test was performed on a production drilled shaft. The length and diameter of the tested drilled shaft were 1.2 meters (4 feet) and 17.8 meters (55 feet), respectively. The applied lateral load on the drilled shaft was 408 kN and the corresponding deflection was 0.0 mm.

For Rt. 18 Extension, Section Project the lateral load test was performed on a demonstration drilled shaft. The length and diameter of the tested shaft were 1.8 meters (6 feet) and 18.6 meters (61 feet), respectively. A 8.9 MN (2,000 kips) hydraulic jack was used to apply the lateral load. A 4.4 MN (1,000 kips) vibrating load cell was used to monitor the applied load. The maximum lateral load applied to the test shaft was 3.31 MN (743 kips). At this loading, the measured deflection at the point of loading was 6.09 mm (0.24 inch). After the lateral load testing was performed the drilled shaft was reanalyzed by LPILE Plus 3.0 computer program using cohesive properties of decomposed shale (IGM). Length versus deflection curves for the two methods and load test results are shown in Figure 7 for the comparison.

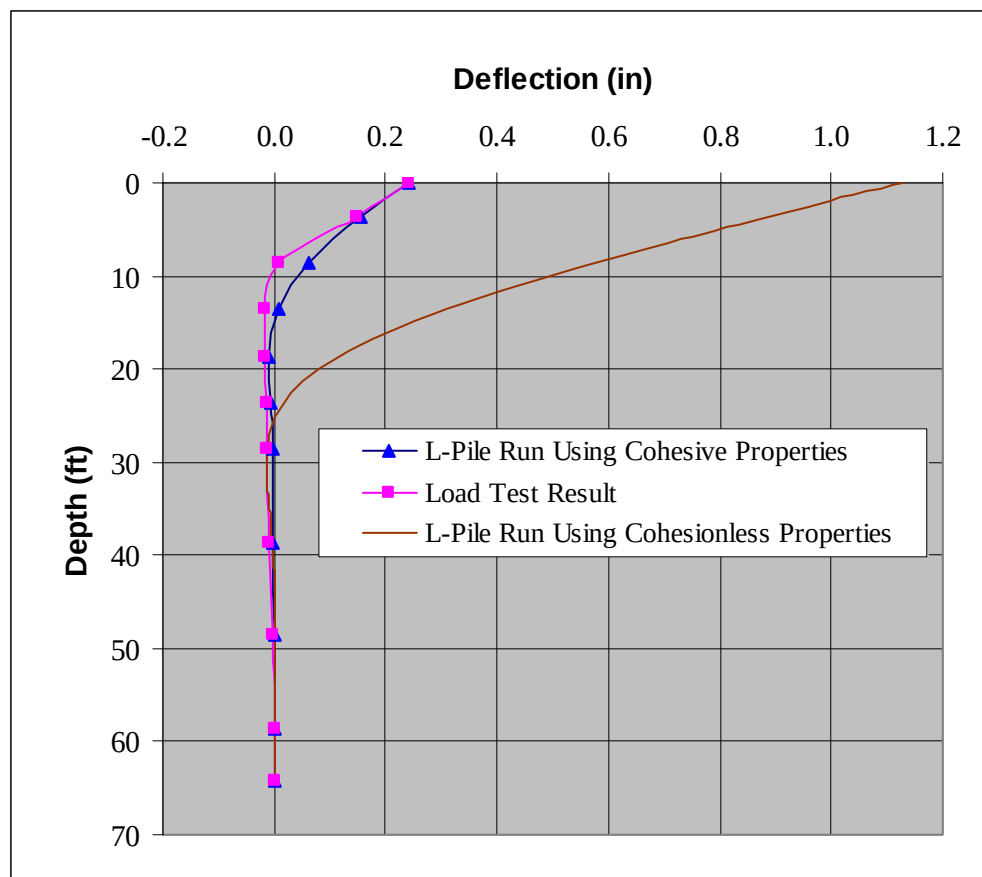


Figure 7: Comparison of Estimated Lateral Deflections and Load Test Results

From Figure 7 the following conclusion are made:

1. The estimated drilled shaft deflection curve by using cohesive properties ($c = 200$ psi, $c_{50} = 0.003$, and $k = 2,000$ pci) matched very well with the field measured deflection curve.

2. The estimated drilled shaft head deflection by using cohesionless properties (as included in Table 4) is 350 percent higher than the field measured deflection.
3. All three curves indicated that the point of fixity is approximately at 40 feet deep.
4. It is more appropriate to model decomposed shale as Weak Rock with undrained shear strength (c), modulus of horizontal reaction (k) and strain corresponding to one-half the maximum principal stress difference (e_{50}) values instead of Sand with friction angle (ϕ) and k value.

CONCLUSIONS AND RECOMMENDATIONS

- IGM method is better method for estimating axial capacity of drilled shaft in Brunswick Formation
- Use of cohesive properties rather than cohesionless properties of Brunswick Formation will provide better result for lateral capacity estimation
- Easy to construct drilled shaft in Brunswick formation
- Can auger decomposed shale/shale bedrock without using any core barrel
- Can construct drilled shaft without reducing socket diameter therefore, can get higher capacity
- Temporary casing is required for the soil portion of overburden
- During excavation of drilled shaft bottom shall be cleaned and checked properly to avoid and any unexpected settlement
- The clear spacing between stirrups in the rebar cage shall be wide enough (at least 150 mm (6 inches) for the concrete to flow easily by gravity.
- Upon the completion of the excavation for a shaft, concrete shall be poured within 12 hours to avoid expansion of material at the bottom.

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Case History of an Anchored Earth Retention System for Stabilizing a Bridge Abutment during Construction

By

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ABSTRACT: The design of anchored earth retention systems for providing lateral support of adjacent ground is a highly empirical approach involving geometry and stiffness characteristics of the wall support system and retained earth. Typically, when designing an earth retention system, simplifying assumptions are required to reduce an indeterminate soil-structure interaction problem into a simple structural system for analysis, whereby movements of the wall and surrounding ground are limited within acceptable levels. This paper presents a case history of an anchored earth retention system for stabilizing a bridge abutment during construction. Factors considered in the overall design and construction of the system and the impact of these on ground movement are presented.

INTRODUCTION

The design of anchored earth retention systems for providing lateral support of adjacent ground is a highly empirical approach involving geometry and stiffness characteristics of the wall support system and retained earth. Typically, when designing an earth retention system, simplifying assumptions are required to reduce an indeterminate soil-structure interaction problem into a simple structural system for analysis, whereby movements of the wall and surrounding ground are limited within acceptable levels. In order to confirm these assumptions and reduce the risk of excessive ground movement, field monitoring is an important means of providing feedback during construction. Observed performances of excavation support systems have been reported by many (Peck 1969; Mana and Clough 1981; O'Rourke 1981; Clough and Reed 1984; Finno et al. 1989a, b; Clough and O'Rourke 1990; Whittle et al. 1993; Ng 1998, 1999; Ou 1998, 2000; Long 2001).

Early work by Peck (1969) summarized and reported field observations of ground surface settlements around several excavations in a graphical form as shown in Figure 1. Data were derived from excavations supported by sheet piles, soldier piles with lagging, and unsupported spans of sheet piles exceeding 10 to 15 ft, and surface settlements were defined by three categories of behavior, with the smallest movements indicated for sands, stiff clays, and soft clays of nominal thickness and average workmanship (Category I). The maximum movements in this category near the wall were one percent of the excavation depth. However, observations of well designed and constructed walls (Long 2001) suggest improvement over this empirical relationship. Peck's work also included data for excavations in soft clays where basal stability was an issue and the thickness of the clay below the line of excavation was large (Categories II and III). Movements in

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these conditions exceed those in Category I because plastic yielding occurs beneath the excavations. Control of movements for this type of behavior is more difficult than for Category I. Category II was intended for very soft to soft clays of limited depth below the base of the excavation, and Category III was also intended for very soft to soft clays but to a significant depth.

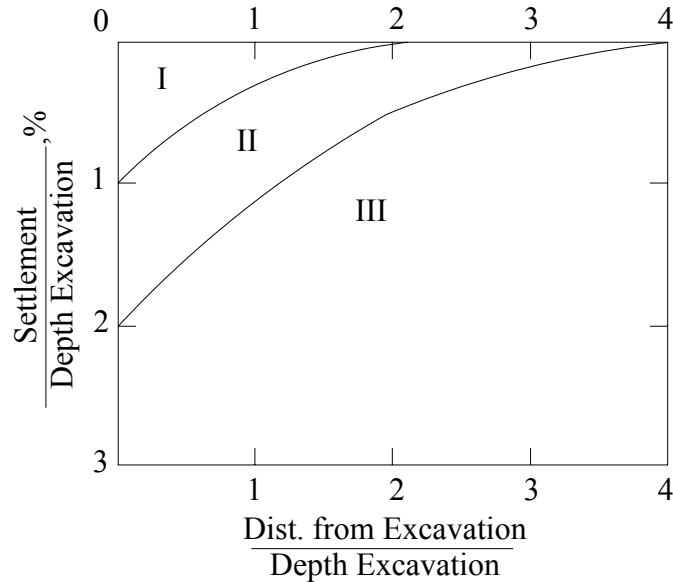


Fig. 1. Summary of soil settlements behind in-situ walls (Peck 1969)

Peck's work has been largely superseded by an approach described by Clough and O'Rourke (1990), which relates deformations to system stiffness and basal stability. 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), can be specified to control deformations within tolerable limits. Once these elements have been specified, the completion of the design becomes a relatively simple problem of structural analysis. 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:

$$(EI)/(\gamma_{ws}^4)$$

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 2 illustrates the effect of system stiffness, basal stability and predicted lateral wall movement.

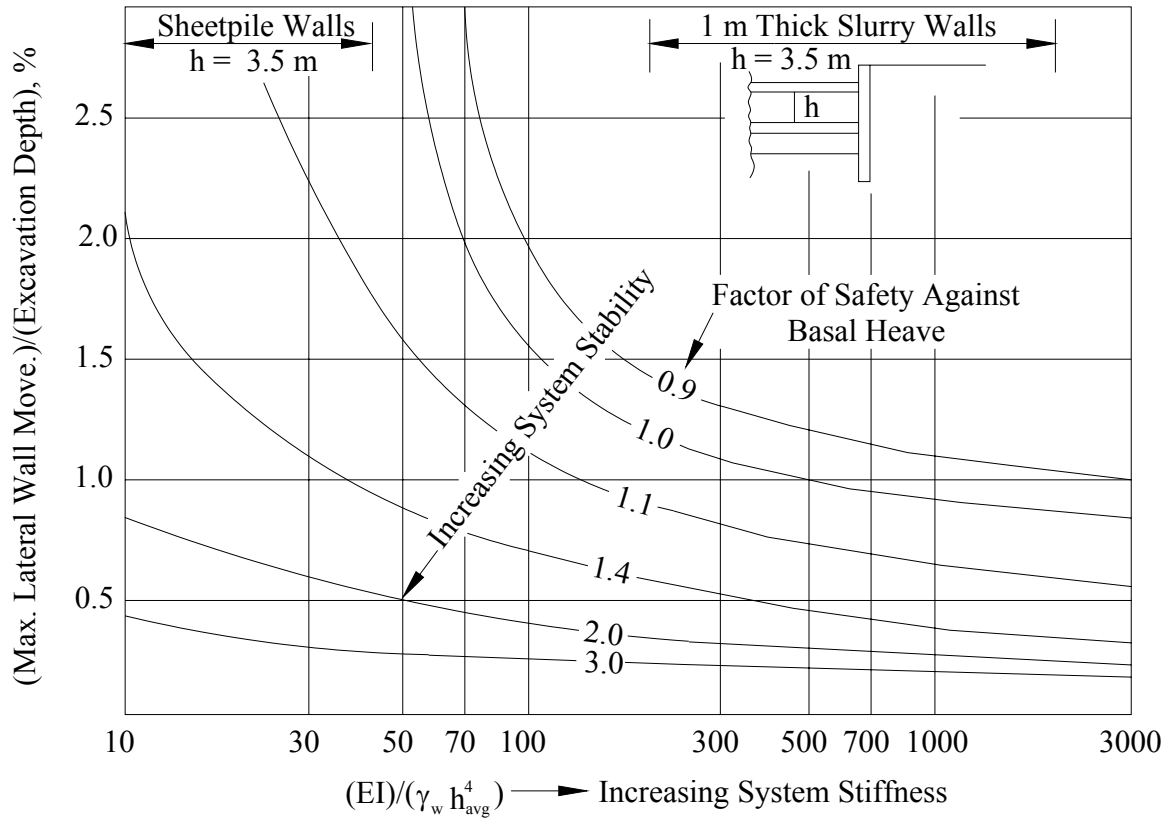


Fig. 2. Chart for predicting maximum lateral wall movements (Clough et al. 1989)

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 approximately 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.

The approach proposed by Clough et al. (1989) for estimating lateral deformations, together with the requirements to satisfy basal stability, are used for the design of an anchored earth retention system to stabilize a bridge abutment that had experienced movement during construction. Factors considered in the overall design and construction of the system and the impact of these on ground movement are presented.

BACKGROUND

During the reconstruction of a bridge along Fulton Avenue in Evansville, Indiana, movement of the north abutment had occurred in the Fall of 1999. Reconstruction of the bridge was part of an infrastructure improvements project to increase the hydraulic capacity in the vicinity of Pigeon Creek and upgrade the structural components. In this area, the creek flows in a west by northwest direction, and given the location, the new structure crosses the creek along the same horizontal alignment as the original bridge, i.e., a 21-degree skew. The original abutment and earth fill had been removed to accommodate a new vertical alignment and in-fill slope. The new abutment was of an integral type and located in the same general vicinity as the original abutment. To accommodate the new vertical alignment, the roadway profile was raised approximately 7 ft, while approximately 11 ft of new fill was placed for the new approach and in-fill slope. Refer to Figure 3 for a schematic representation of the bridge.

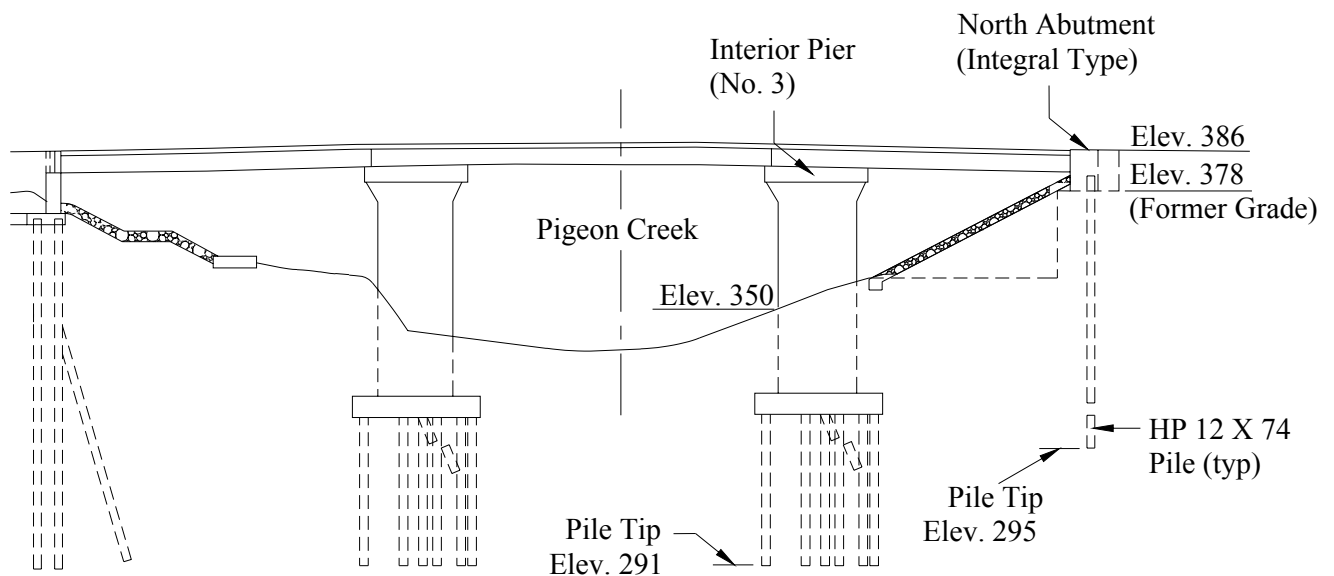


Fig. 3. Schematic representation of bridge

Concurrent with the placement of new earth fill, excavations were being made within the creek channel to construct two new interior piers for the three-span bridge. To facilitate construction of the piers, cofferdams were utilized whereby steel sheet piling was driven into the underlying subsoils. Wales consisting of structural steel elements were attached to the piles to provide continuity of the sheeting in an attempt by the contractor to limit deflections from earth pressures due to the resulting excavation only. Review of a photograph taken during construction indicated that the sheeting and wales had moved inward on the north side of the excavation, i.e., the side closest to the north abutment. Although no significant movement was observed at the crest of the new approach, construction of the foundations for the north abutment and interior piers continued via the installation of structural steel piling (i.e., driven to rock approximately 90 ft below grade). After completing the construction of the interior pier closest to the north abutment, the cofferdam was removed, and the composite prestressed concrete “I” beams for support of the bridge deck were then placed. This included casting intermediate diaphragms and the beams and steel piling together at the north abutment to form the integral-style element.

Noticeable movement of the interior pier (No. 3) closest to the north abutment was observed shortly after the previous elements were constructed. Refer to Photograph 1. By the time the author was involved with the project, the interior pier was 8 in. from vertical, and the abutment and beams were collectively skewed to the west less than $\frac{1}{4}$ of one degree over an 80-ft long span. The “I” beams were approximately 3 in. from the edge of the pier cap, i.e., in the closest direction, and the elastomeric bearing pads were racked. The skew was such that the beam pockets of the interior pier were spalling, and the length of the bridge had shortened by approximately $2\frac{3}{4}$ in.



Photo. 1. Interior pier (No. 3), looking west (taken after temporary support of the pier was provided)

COURSE OF ACTION

Often the events or sequence of events prior to an incident have a direct correlation with the outcome. Under the presumption that the cause of the problem was associated with the lack of adequate strength and deformational characteristics of the soils located within the zone of the in-fill, the solution of choice (from a constructability viewpoint) was to install a vertical wall at the north abutment to allow an excavation to be made adjacent to the interior pier thus relieving the earth pressure on the pier stem and allowing observation of the pier cap. This solution would not utilize a traditional abutment but a non-gravity tied-back wall with a one-sided cast-in-place concrete face

constructed immediately in front of the north abutment. Refer to Figure 4 for a schematic representation of the proposed solution.

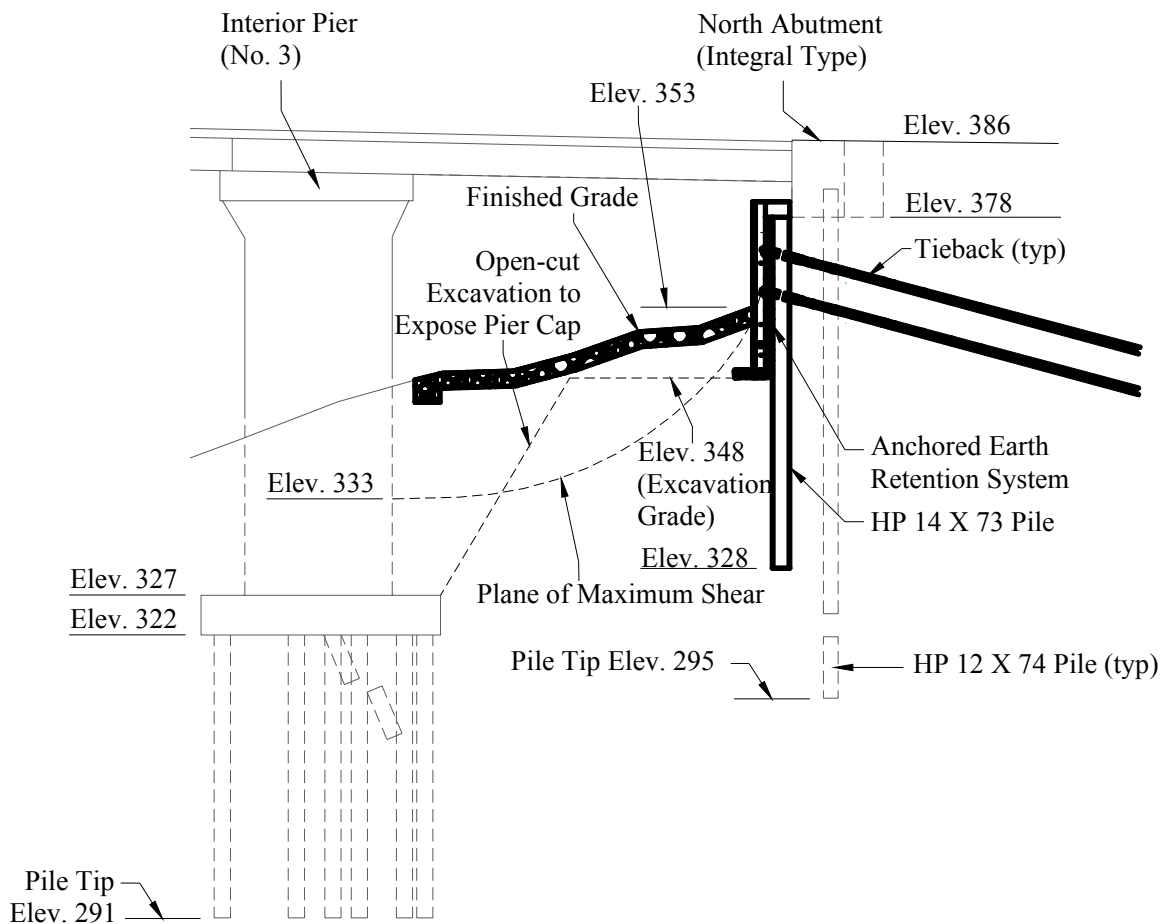


Fig. 4. Schematic representation of anchored earth retention system for stabilizing north abutment

To facilitate construction of the wall, temporary support, as shown in Photograph 1, of the second and third spans would be required to remove the dead load of the beams from the interior pier and provide a safe working environment for those who would be performing any future work below the beams. Structural steel piling (i.e., soldier piles) would also be required to be placed between the existing beams (6-ft on center) of the north abutment to facilitate a connection of the concrete face and provide support of the excavation to evaluate the integrity of the pier and stabilize the new approach embankment. Furthermore, tiebacks would be required to provide the necessary reaction at the north abutment and limit deflections of the retention system as well as avoid additional bending stresses of the piling that provide support of the integral abutment. Once temporary support of the second and third spans was provided and the soldier piles were in place, excavation could then commence utilizing a “top down” approach,

whereby the in-fill was removed to a safe vertical height in order to install tiebacks through a built-up pile section and taking into consideration the wall system stiffness.

Once the earth retention system was in place, an excavation could then be made to expose the pier stem/pile footing cap and relieve the earth pressure generated on the pier. In the event that the pier did not return to the vertical, a jacking bracket would be attached to the bottom of the beams (i.e., of the second span) and, subsequently, be used to re-plumb the pier.

REGIONAL GEOLOGIC SETTING

Surface Geology

Reconstruction for the bridge is located within the Ohio River valley near its major oxbow at the Indiana/Kentucky border and over Pigeon Creek, a major tributary to the Ohio River. The site is located in the “Tri-State” area where Indiana, Kentucky, and Illinois meet; a region characterized by alluvial and glacial geomorphology. The Evansville area lies at the southern limits of Pleistocene glaciation which left terminal moraine glacial ridges to the north of Evansville. As glaciation waned, glacial meltwater created a complex terrain of glacial lakes and outwash plains. Quiet sedimentation in the lakes (glaciolacustrine deposits) consisted primarily of silt and clay deposited sometimes in rhythmic varved beds representing warmer and colder annual cycles with more and less melting to the north during the summer and winter seasons, respectively. Post-glacial evolution of the area has been characterized by mature stream and river valley development and associated alluvial deposition of sand and fine-grained overbank clays and silts, and by erosion of upland areas.

Bedrock Geology

Bedrock does not crop out in the vicinity of the bridge, but it is present beneath the glaciolacustrine deposits in the general area and is believed to generally consist of Late Paleozoic (Pennsylvanian) limestone, sandstone, shale, and coal beds. Underground coal mining has been a prominent industry in the region. Bedrock in the general area is about 90 ft below the ground surface based on exploratory test boring data from this and nearby projects.

Seismicity

Evansville lies in a seismically active area and could be influenced by the New Madrid Seismic Zone that is centered to the southwest in Missouri. The New Madrid Seismic Zone’s Northeast Extension extends close to southern Indiana to the west and southwest of Evansville and mapped faults occur within 5 to 10 mi of the project alignment. Recent research also suggests that the Evansville area may be more influenced by the Wabash Valley Fault System in southeastern Illinois. The anticipated seismicity of the site area is believed to generally consist of the potential for seismic shaking from low to moderate intensity earthquakes.

The glaciolacustrine soils along the alignment may be susceptible to seismic settlement as a result of cyclic dynamic loading by earthquake shaking. The Pigeon Creek area specifically is considered by researchers to be at high risk to experience liquefaction-induced damage. Results of recent studies and hazard analyses have identified areas with a high seismic hazard potential that had not previously been identified in earthquake zonation studies.

SUBSURFACE CONDITIONS

Within the depth interest, the soil types near the north abutment consisted of man-made fill for the approach embankment overlying naturally-occurring soils consisting of silty clay loam and silty loam. The soil fill extended to depths of 17½ to 32 ft below the ground surface, corresponding to about Elevations 349 to 356. In general, the consistency of the fill varied (i.e., soft to hard) based on hand penetrometer readings. Like the consistency, the moisture contents of the fill also varied and generally ranged from 15 to 29 percent. A comparison of natural moisture contents and results of Atterberg limits suggest that the soil fill is of low to moderate plasticity and little compactive effort was used to control the density and moisture characteristics of the fill.

Beneath the fill, alternating layers of silty clay loam and silty loam (glaciolacustrine) were observed to the maximum depths explored. Occasionally, a relatively thin layer of clay was also observed as well as slickensided planes within these layers, suggesting previous movement. These planes were observed near depths of 23½ and 38½ ft below the ground surface, corresponding to about Elevations 343 and 346, respectively. The consistency of these soils typically varied from medium to very stiff based on hand penetrometer readings; however, at some depth intervals but less frequent, the consistency was observed to be slightly lower (i.e., soft) or greater (hard) than that previously noted. Additionally, moisture contents were typically within the range of 20 to 30 percent, with the higher values typically located near the zone(s) of anticipated shear or where the silt content was more appreciable (e.g., within silt seams). A comparison of natural moisture contents and results of Atterberg limits suggest that these soils are of low to moderate plasticity (with indices of 30 to 40 percent for the liquid limit and 8 to 15 percent for the plastic index) and slightly overconsolidated. Results of isotropically consolidated undrained compression (CIUC) tests with pore pressure measurements suggested a drained angle of internal friction of 30 to 32 degrees and an effective cohesion of less than 100 lbs/sq ft. Additionally, the groundwater level was observed to correspond with the average low water level of the creek (i.e., Elevation 342).

EARTH RETENTION SYSTEM - Design Rationale

Based on information from the exploratory field program and initial field performance monitoring, the solution of choice, as mentioned previously, was to construct a vertical wall at the north abutment to allow an excavation to be made adjacent to the interior pier to relieve the earth pressure. This solution would utilize a non-gravity tied-back wall with a one-sided cast-in-place concrete face constructed immediately in front

of the north abutment. Initial field performance data obtained from an inclinometer located within 40 ft north of the interior pier suggested a plane of shear located approximately 6 ft above the top of the pile cap (i.e., corresponding to Elevation 333) and extending to the north terminating within the in-fill adjacent to the north abutment. Refer to Figure 4.

Considering the nature of the framing at the north abutment, it was necessary to utilize a system with sufficient stiffness to limit deflections thus avoiding additional bending stresses of the abutment piling. Although no documentation of movement for specific elements was made prior to the author's involvement, it was estimated (via the design plans, verticality of project elements and equations of mechanics) that 3 in. of additional movement would be viable without overstressing the piling. Considering the approach by Clough et al. (1989), a dimensionless system stiffness parameter of 15 was estimated for a limiting wall movement of 3 in., an excavation depth of 40 ft, and a basal stability of 2. A depth of 40 ft was chosen to facilitate a 20-ft deep open-cut excavation adjacent to the interior pier for the purpose of relieving the earth pressure on the pier stem, allowing observation of the pier cap and re-plumbing of the pier, if necessary.

Since a tied-back wall system is subject to little prestress and is not exceptionally stiff, especially if soldier piles and lagging are used, enough movement is likely to occur to justify the use of active earth pressure loadings. Using case histories from literature, Clough and O'Rourke (1990) showed that the horizontal movements tend to average about 0.2 percent of the excavation depth for soils of similar stress history. The data also suggest that in a stiff soil environment variations in soil stiffness have a more profound effect on wall behavior than system stiffness as opposed to a soft soil environment where basal stability may be an issue. Consequently, in the design, total earth loads were used for the tiebacks using classical earth pressure theory (i.e., Rankine). Allowances were made to accommodate curved failure surfaces which yield higher resultant loads than the straight failure surfaces assumed in classical theory. Additionally, the resultant was empirically located near the mid-height of the wall instead of at the lower third point as in a triangular pressure diagram.

Structural design of the soldier piles (having a thru-pile connection) was concerned chiefly with flexural stress produced by either positive moments between points of support (i.e., the tiebacks) or negative moments at points of support. Design loads for checking shear and flexural stresses in the support wall can differ from those used for tiebacks. The tiebacks inevitably have to accommodate the total maximum horizontal load on the wall which was estimated using classical theory. However, soil arching shifts pressures away from the point of maximum deflection and reduces these moments from values computed on the basis of uniformly distributed pressures. To compute the forces acting on the soldier piles, timber lagging, and concrete facing, an apparent loading diagram was used after Terzaghi and Peck.

Based on a system stiffness parameter of 15 and an average spacing between tiebacks of 13 ft, a flexural rigidity, EI, of 3.2×10^5 kips-sq in./in. was required of the soldier beams. Considering that the beam-pile spacing for the integral abutment was 6 ft, it

was necessary to use the same spacing of 6 ft for the soldier piles. Consequently, an HP shape with a moment of inertia of 729 in.⁴ (i.e., HP 14 x 73) was chosen, and no consideration was given to the “shielding effect” of the existing piling for the abutment or frame action created by the interior pier/temporary supports. Additionally, piles were driven to Elevation 328, and the embedment depth was chosen based on an interaction approach between the piling and soil. Furthermore, two rows of tiebacks were utilized, and design loads were on the order of 65 to 80 kips.

PERFORMANCE MONITORING RESULTS

Performance monitoring of the interior pier and north abutment included establishing an array of survey points across the top of the pier stem and abutment. Monitoring began in June 2000 and continued through April 2001. Initial field performance monitoring was performed via inclinometers, as stated previously; however, data gathered at that time was used primarily for locating the plane of maximum shear within the in-fill and determining a reactionary force for design of a jacking bracket, in the event that it was necessary to re-plumb the pier. The following presents a timeline of the construction events, and Figures 5 and 6 reflect the movements with respect to these events.

Event	Period
Install temporary support for beams over interior pier	June 2000
Perform exploratory field and laboratory testing programs	July 2000
Prepare preliminary design of earth retention system	July – August 2000
Drive soldier piles for earth retention system at north abutment	August 2000
Prepare final design of earth retention system	September 2000
Perform top-down construction of earth retention system	Oct. – November 2000
Earthquake: M_L (Local Magnitude) = 5.4	December 7, 2000
Creek level too high to perform work	Dec. 2000 – Feb. 2001
Make open-cut excavation to expose cap of interior pier	March – April 2001

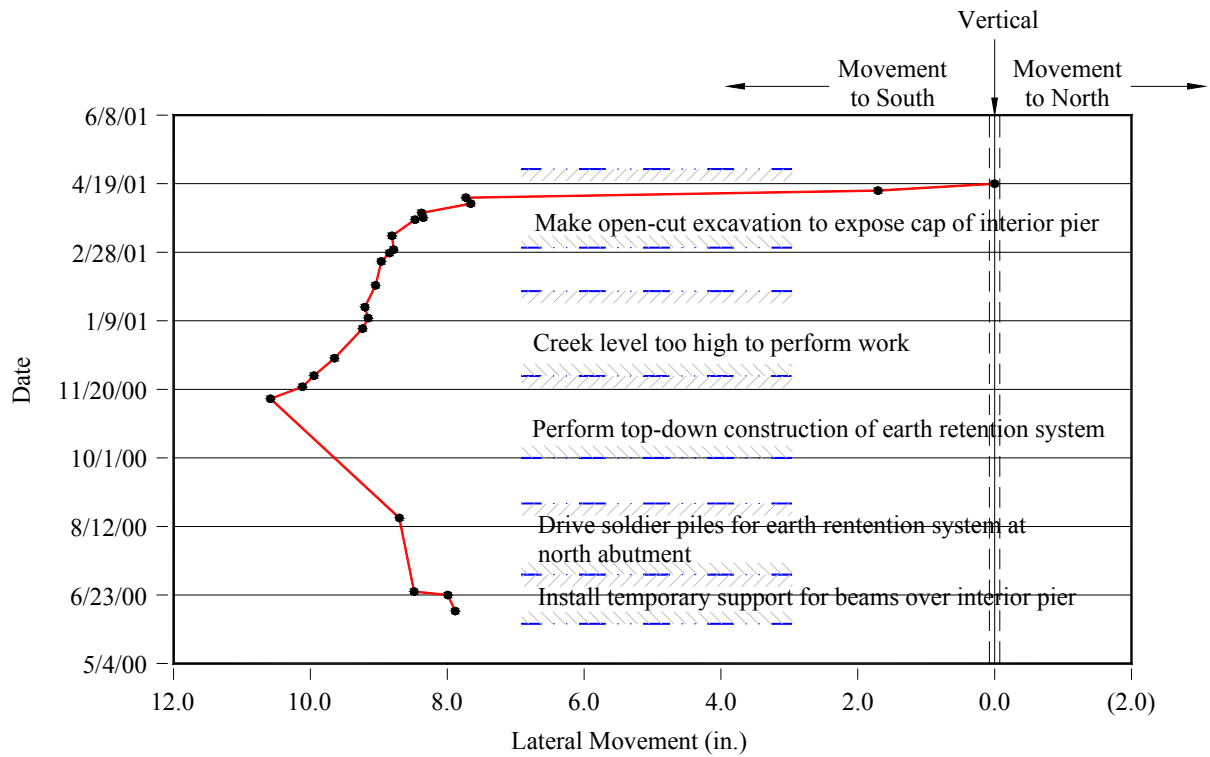


Fig. 5. Lateral movement measured from vertical at top of interior pier

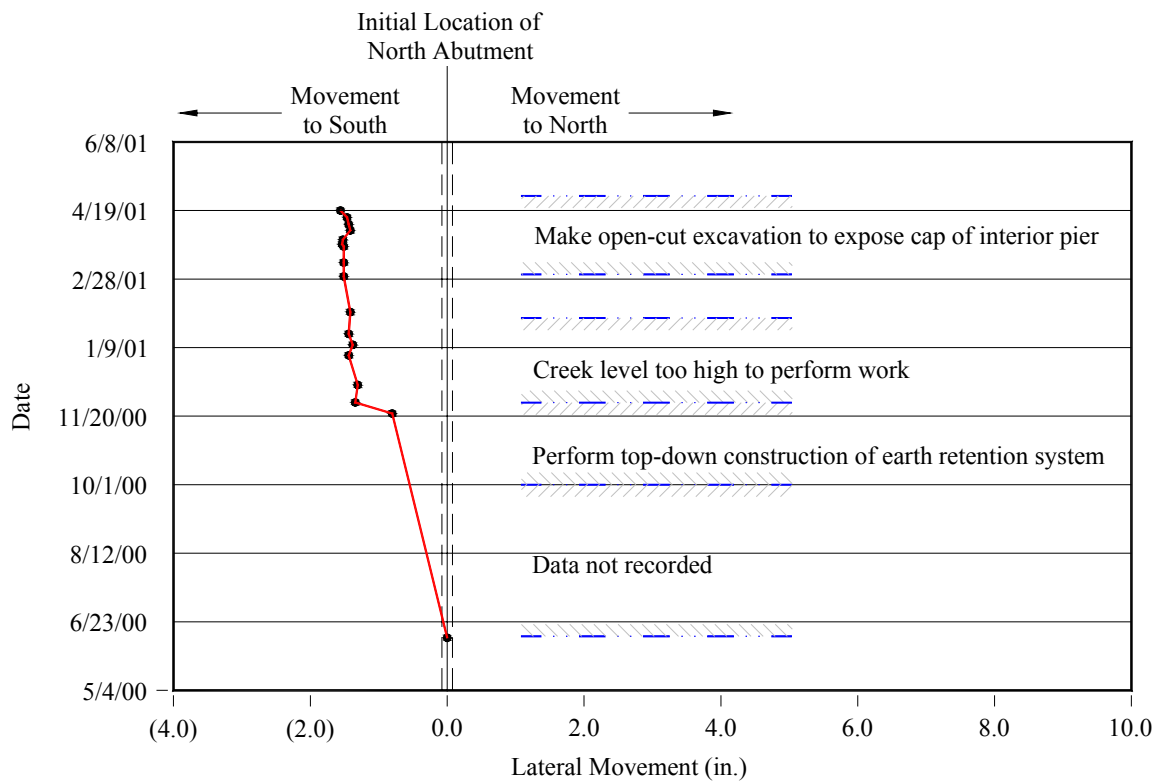


Fig. 6. Lateral movement measured at the top of north abutment from initial location

DISCUSSION AND CONCLUSIONS

Results of the performance monitoring indicated an additional movement of about 1¾ in. at the north abutment as compared to 3 in. predicted from an approach by Clough et al. (1989). As mentioned, consideration was not given to the “shielding effect” created by the piling for the integral abutment or frame action of the interior pier/temporary supports. Because of these unknowns, it is almost impossible to say with sufficient accuracy exactly what contribution could be gained by their effect. Therefore, a conservative approach in the design of the retention system would be to neglect their effect.

Considering average workmanship, other factors that may have influenced the observed response could include a stiffer soil response than predicted. Although not presented herein, shear strength test data from CIUC testing revealed little development of excess pore pressures, and when excess pore pressures did develop, they peaked at axial strains less than 1½ percent in comparison to the deviator stress which continued to gain strength with increasing axial strain. Considering that the predominant soils in the general vicinity are glaciolacustrine in origin, the preferred particle orientation can cause an inherent anisotropy which leads to changes in modulus, pore pressure response, and undrained and drained strength parameters as the direction of the major principal stress varies during undrained shear. Additionally, the effects of lateral drainage on consolidation rates have also been neglected potentially creating a stiffer soil response.

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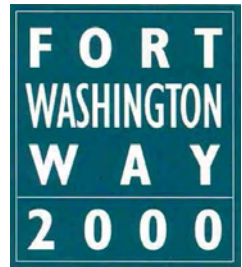
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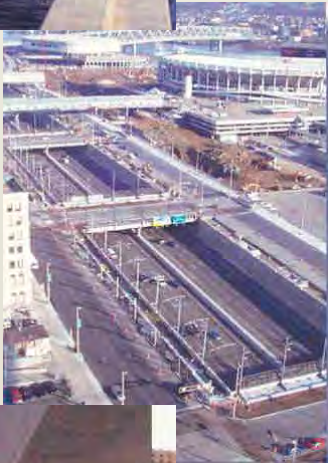
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FORT WASHINGTON WAY A GEOTECHNICAL ODYSSEY



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Presented by
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FORT WASHINGTON WAY- A GEOTECHNICAL ODYSSEY

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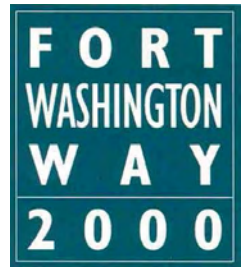
ABSTRACT

Constructed in the early 1960s, Fort Washington Way (FWW) is one of a few major highway connections bridging downtown Cincinnati's east and west sides. At over 400 million dollars, the FWW reconstruction project has been billed as one of the most expensive interstate highway construction projects undertaken in Ohio. The Fort Washington Way reconfiguration program consisted of a system of streets, access ramps, bridges, retaining walls, drainage, landscaping, and urban improvements in the Cincinnati Central Business District (CBD).

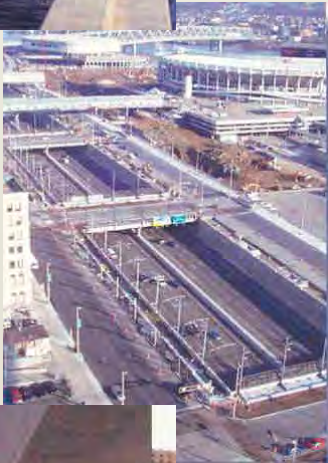
There were 42 new bridges on the project including rolled steel beam bridges, steel curved plate girder bridges, steel box girder bridges, post-tensioned cast-in-place concrete slab bridges and post-tensioned concrete box girder bridges with at least three "firsts" in the state of Ohio for bridge types. The FWW project also included a \$9.2 million Combined sewer Overflow (CSO) construction.

The complex Ohio River valley geology is the result of past glaciations erosion and deposition of a wide variety of soils, each with unique engineering properties. The wide variety of soil types and their inherent challenges along with the urban environment and an accelerated construction schedule resulted in the need for a very comprehensive geotechnical program. The geotechnical design and construction complexities required a wide array of solutions and creative engineering.

A variety of foundation types including spread footings, driven H-piles, steel pipe piles and large diameter drilled shafts were used. An extensive testing program including static and dynamic load tests, cross-hole sonic testing and the use of a pile installation recorder were undertaken. The project also involved several types of retaining walls requiring innovative engineering solutions. Several ground improvement techniques including stone columns, geofoam, geogrids, and chemical stabilization were also used.



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PROJECT DESCRIPTION

Constructed in the early 1960s, Fort Washington Way (FWW) is one of a few major highway connections bridging downtown Cincinnati's east and west sides. When it opened, Fort Washington Way could safely accommodate 90,000 vehicles per day. As downtown businesses and entertainment venues began to grow and flourish, so too did traffic in and out of the city.

Up until July 1998, this major downtown artery was exceeding its original construction limit by supporting more than 120,000 vehicles each day. Heavy traffic congestion, dangerous weaves from nearby interstate off-ramps and pedestrian safety were all quickly becoming concerns during rush hour and times of special downtown events.

Fort Washington Way reconstruction design began in 1995 when the City of Cincinnati asked the Ohio-Kentucky-Indiana Regional Council of Governments (OKI) to provide a solution. The reconstruction of Fort Washington Way became a sub-study of the OKI I-71 Corridor Transportation Study. In January 1997, OKI produced a plan for the revamping of Fort Washington Way, and the City approved this plan two months later. Parts of Fort Washington Way went from concept to design and into construction in a very short amount of time.

The Fort Washington Way reconfiguration project was named FWW 2000 and consisted of a system of streets, access ramps, bridges, retaining walls, drainage, landscaping, and urban improvements in the Cincinnati Central Business District (CBD). The project reconnected the CBD, as well as the CBD's of Covington and Newport, Kentucky, with the Cincinnati riverfront and served as a catalyst for urban renewal of the downtown area along the Ohio River. Considered the most significant highway project in a generation, it reclaimed 15 acres of riverfront real estate. The reconfiguration upgraded traffic control and management, increased safety and efficiency from poor to excellent, enhanced access into downtown Cincinnati, and improved traffic conditions on adjoining City streets. Significant urban design improvements were incorporated into the project, and it was designed to accommodate future transit and intermodal station possibilities.

Features of FWW 2000 include:

1. Increased number of lanes, while reducing the overall width of the highway;
2. Eight lanes of through, direct traffic for I-71 and U.S. Route 50;
3. A new Second Street and renovated Third Street as grand boulevard entrances into downtown Cincinnati;
4. Safer access ramps into downtown;
5. Safety for pedestrian access to, through and around the Fort Washington Way area;
6. Reclamation of 250 to 300 feet of the current Fort Washington Way between Elm and Sycamore streets, making this land available for new development;
7. Flexibility for development south of the new Fort Washington Way;
8. A new intermodal transitway to accommodate parking, buses and future rail transit;
9. Combined Sewer Overflow (CSO) system, a new sewage relief system, to dramatically reduce occurrences of combined sewer overflows into the Ohio River;
10. Third Street Viaduct;
11. Connectors from the Clay Wade Bailey and Taylor Southgate bridges;
12. A total of 42 bridges supported on various types of piles and deep large diameter caissons;

13. Several Mechanically Stabilized Earth (MSE) walls, 6000 feet. of pile-supported retaining walls, stone columns and other ground modification solutions;

14. A pile supported floodwall, with portions containing a soft slurry wall for flood protection.

The project includes many unique bridges on the Fort Washington Way. The project had at least three "firsts" in the state of Ohio for bridge types- the first steel box girders, the first cast-in-place concrete box girder, and the first concrete integral piers. There were 42 new bridges on the project, with almost every type of bridge represented. There were four bridges of steel rolled beams for a total bridge length of 797 feet; 16 steel curved plate girder bridges with a total length of 11,400 feet; and five steel box girder bridges with a total length of 2705 feet. There were also seven cast-in-place post-tensioned concrete slab bridges with a total length of 974 feet, as well as three cast-in-place post-tensioned concrete slab bridges with a combined length of 2690 feet. There was also one cast-in-place slab bridge (125 feet), and one prestressed concrete "I" Girder Bridge with a total length of 348 feet. The project also included two prestressed concrete adjacent box beam bridges for a total length of 579 feet, and two cable-stay pedestrian bridges (336 feet). The two cable-stay bridges were the landmark bridges of the project. The project location map is shown on Figure 1.

GEOLOGIC HISTORY

Major valleys in Southwestern Ohio and Northern Kentucky have their origins deep in the geologic past. They were originally carved millions of years ago by principally north and northwestward flowing waters termed the Teays Drainage System. These preglacial valleys were subjected to major modifications accompanied by changes in water flow due to the invasion of at least three continental glaciations.

Prior to glaciation, the old valleys were wide and had only down-cut around 150 feet (45.7m) below the tops of the surrounding peneplain hills. The first ice advance, the Kansan (approximately 1.5 million years ago), ponded the north-northwest flowing streams, causing the formation of extensive lakes, which deposited clays and silts,

partially filling the old valleys. As the ice sheet retreated, large volumes of meltwater, plus some possible uplift of the land, produced new and vigorous down-cutting in the valleys. The stream axis of the Teays Age Licking in the Cincinnati area (the Fort Washington Way Project is located in this valley) was cut well into bedrock, over 450 feet (137.2m) below the top of surrounding hills. This new, more vigorous drainage system is known as the Deep Stage. The Ohio River, or Deep Stage Ohio, at that time turned north up the Little Miami River Valley and joined the Deep Stage Licking near Elmwood Place, and then proceeded as one stream up to Hamilton, turning south down the Miami-Whitewater Valley, and then west in its present bed.

Two additional glacial advances, the Illinoian and Wisconsin, each brought additional disruption to the Deep Stage Drainage System, causing the formation of additional lake sediments, and clays from ice advances, with granular deposits from meltwaters of the retreating ice masses. These deposits almost completely filled the ancient wide valleys with interbeds of sand, gravel, silt, and clay. It was during the Illinoian Stage that the ponded waters overflowed the high cols in the area, resulting in the joining of the Ohio and Licking Rivers at downtown Cincinnati, and the now present westward course of the Ohio in its present bed at Cincinnati. Post-glacial erosion during the temperate interglacial periods, and during the past 15,000 years since the departure of the Wisconsin ice sheet, has removed much of the valley glacial deposits and produced today's topography. Valley fill remnants of the glacial periods exist as terraces along the valley walls, with Illinoian age terraces at approximately 650 feet (198.2m) elevation (Terrace Park is located on such a terrace), and Wisconsin age terraces at approximately 550 feet (137.7m) elevation (the City of Cincinnati is located on a large terrace of this age). Major streams continue to down-cut through the glacial valley fill, seeking the original floor of the ancient Deep Stage Drainage System.

Fort Washington Way is located just somewhat down-grade of the crest of Cincinnati's Wisconsin terrace, with top of ground ranging around 495 to 522 feet (150.9m to 159.1m) elevation, yet close enough to explore its composition, and the composition of underlying Illinoian and Deep Stage sediments. Test borings have penetrated these Ice Age remnants, with some borings penetrating sufficiently to locate the rock floor of the ancient Deep Stage at around 385 to 400 feet (117.4 to 122.0m) elevation.

Sediments encountered by the test borings are typical of the physical activities that produced them, i.e., silty clays of modern floodplain alluvium, silts and clayey silts of still water pondings, sands and gravels of meltwater flow, clays of ice intrusions, and granular soils from ancient river activities. This interbedding is not always neat and conforming to the history of the area because of the complexity of events during ice advances, retreats, interglacial temperate periods with erosion, and finally post-glacial erosion. But the signatures of these past drastic climatic changes are all there, though sometimes scrambled and defying precise interpretation.

The encountered subsurface conditions represent depositional characteristics typical of the geology of the area. The site lies in the transition area between the Wisconsin outwash terrace and the Ohio River flood plain. The several periods of glaciation and major drainage changes have resulted in a subsurface profile in today's major river valley areas that has complex stratigraphy, offering materials of water and ice deposition that vary considerably in their physical properties from location to location, indeed sometimes within the same location.

The soils encountered at this site can be characterized in five broad categories:

1. Manmade fills, of course, reflect the effort of present and past generations to reform an existing topography.
2. The natural clays most often reflect the location of ice lobe intrusions into the main valley areas; these are sometimes capped by alluvial soils of flood plain deposition.
3. Silt beds mark the ancient still water deposits, lakes formed by ice damming the main valley flow.
4. The sands and gravel mark two activities, the very basal portion representing Deep Stage river sediments, with the upper and much greater proportion being the result of departing glaciers, their meltwaters carrying tremendous amounts of granular sediments into the old drainage valleys. Post glacial erosion has removed some of these deposits.

5. The sand and gravel ranges in thickness from 50 feet (15.2m) to greater than 90 feet (27.4m) depending upon location within and beyond the ancient Licking River channel. The granular strata are intermixed with layers and seams of silt and clay, occurring rather erratically. Bedrock lies at depths ranging from 78 feet (23.8m) to 127 feet (38.7m).

It should be noted that this area of downtown has been heavily worked by man. Thus, cisterns, dry wells, privies, etc. were expected within the construction area. Past work on downtown sites has shown cisterns typically to be 3 to 6 feet (0.9 to 1.8m) in diameter, which can extend as deep as 40 feet (12.2m) below grade. Remnant foundation walls of buildings, were also anticipated to be encountered during excavation.

SUBSURFACE EXPLORATION

H. C. Nutting Company (HCN) has performed geotechnical explorations in the downtown area since the early 1940's. Nutting has studied most of the structures along the river in the vicinity of the FWW project. Hundreds of archive borings were reviewed to develop a comprehensive new boring plan for the 42 bridge structures, several thousand feet of retaining walls, flood wall, transit center, CSO structure, roadways and other miscellaneous structures. The boring program generally satisfied the minimum requirements of Ohio Department of Transportation (ODOT) investigative guidelines. All relevant geologic maps, soil survey, and published geologic and hydrogeologic resources were reviewed. The exploration program consisted of over 150 Standard Penetration test borings varying in depth from 60 to 125 feet. Relatively undisturbed Shelby tube samples were recovered from many borings to establish the compressibility and shear strength characteristics of all the soil strata. Rock coring was performed in many locations to determine the lithology and strength characteristics of bedrock. Cone penetrometer testing (CPT) was performed at some locations. Due to the upper random nature of fill (presence of bricks and other construction debris) and possible buried foundations, the CPT was not used more commonly though the use of CPT can provide very useful data for pile design and engineering properties of soil. Pressuremeter testing was also performed primarily in the upper soft shale bedrock. The primary purpose of the pressuremeter testing was to aid in the design of large-diameter, high capacity drilled

shafts. A large number of laboratory tests were performed to classify the soils and determine their engineering properties.

HCN mobilized several types of drill rigs (truck, ATV, and skid) with as many as 8 drill crews drilling simultaneously. Due to traffic issues and the rush nature of the project, drilling was performed continuously on a 24 hr. basis. Many of the borings were performed in the night to avoid traffic disruption.

BRIDGE FOUNDATIONS

The complex Ohio River valley geology, further disturbed by Man's activities and the restrictive urban environment provided a challenge during foundation selection. The choice of deep foundations to support the bridge structures was made early in the project design phases, based on the considerable experience of HCN over the past 7 decades in the downtown area. The upper heterogeneous fill and alluvial soils are known to be moderately to highly compressible. The foundation selection had to consider the following key factors:

- Load carrying capacity (axial compressive, uplift and lateral),
- Geologic conditions and their variability,
- Bridge structure type
- Space limitations due to urban setting,
- Presence of old and existing foundations/obstructions,
- Material and installation related foundation costs,
- Availability of materials,
- Installation time,
- Vibration and its impact on downtown offices and other socio-economic factors.

Based on the above-mentioned factors, a comprehensive matrix of foundation types, capacity and cost per unit capacity, along with their advantages/disadvantages was developed. At each structure, the data from the foundation selection matrix along with the several factors listed above were used to select the most appropriate foundation. The foundation selection was performed in consultation with the entire design team

(Geotechnical and Structural Engineers, City, ODOT and Construction Engineers). This logical approach to foundation selection led to the use of several foundation types on this project:

- Steel HP 12X53 piles driven to refusal on bedrock (Grade 36)
- Steel HP 14X73 piles driven to refusal on bedrock (Grade 36)
- Steel HP 14X89 piles driven to refusal on bedrock (Grade 36)
- 14" diameter closed-end, cast-in-place steel pipe piles (CIP)
- 16" diameter closed-end, cast-in-place steel pipe piles (CIP)
- Large diameter drilled shafts socketed in shale and limestone bedrock

Both vertical and battered piles were used. Over 10,000 piles were driven for the various bridge and retaining wall structures. The pile lengths also varied greatly based on the capacity and pile group size limitations. It is typical practice of ODOT to limit the allowable structural capacity of HP piles to 0.25 to 0.33 F_Y (9 to 12 ksi). The project design team requested a variance from ODOT and FHWA to use high capacity piles by proposing to increase the allowable structural capacity to 15 ksi (Grade 36 steel). The variance was granted and it was decided to increase the allowable stress of HP 12X53 piles to 15 ksi and HP 14X73/HP 14X89 piles to 12 ksi. The primary concern with the higher allowable capacities was the large elastic compression and the creep potential of the Ordovician shale bedrock under sustained loading. The HP piles were all driven to practical refusal on bedrock and varied in length from 60 feet to in excess of 110 feet. The structural capacity generally governed pile design and the geotechnical capacity was confirmed through static and dynamic load tests.

The pipe piles (CIP piles) were designed to bear in the medium dense to dense outwash sand and gravel deposits. The piles derived their capacities through a combination of side resistance and end bearing. The proportion of side resistance and end bearing was highly variable from one structure to another. The pile lengths varied from 40 to 85 feet. On average, the side resistance component was 30 to 60%, with the end bearing being in the range of 40 to 70%.

DOWNDRAG ON PILES

The presence of compressible alluvium, overlain by variable depths of uncontrolled man-placed fill was a critical element impacting pile lengths and capacity. Many bridge structures included MSE walls up to 30 feet in height. The new soil load on moderately compressible foundation soils resulted in large settlements and downdrag forces on the piles. Downdrag (negative skin friction) forces on deep foundations are a well-known and complex problem. Peck et al. (1974) indicated the importance of dealing with this problem when they state, "Several examples of unexpected settlement of large magnitude have been attributed to neglect of negative skin friction." Piles will be subjected to downdrag when the soils in contact with the upper portion of the foundation move downward relative to the pile and literally drag the pile down. Downdrag is a function of:

- Settlement of the soil surrounding the pile
- Settlement of the pile element
- Relative movement between the surrounding soil and the pile.

Research and field tests have shown that extremely small relative movements on the order of 0.10 to 0.25 inches mobilize frictional resistance between the soil and pile.

A detailed analysis of pile settlement and surrounding soil settlement was performed to determine the relative settlement profile along the pile length and find the theoretical neutral point (the point above which negative skin forces are mobilized). The neutral point is defined as the point along the pile at which the relative pile-soil movement is zero, that is to say the settlement of the soil is equal to the downward movement of the pile. Several options were considered to mitigate the downdrag forces. The accelerated construction schedule would typically not allow for pile driving after completion of the majority of fill-induced settlement (which is the preferred option). This construction sequence would have reduced/eliminated the consideration of downdrag forces, but would add time to the construction schedule. The use of bitumen was discussed but was not considered due to several handling and constructability issues associated with the

use of bitumen. The options used on this project to reduce or eliminate downdrag included:

- Pre-drilling in the zone above the neutral point. This would reduce the downdrag loads (50% reduction of the theoretical computed downdrag force),
- Use of plastic sleeves in the zone above neutral point,
- Use of large capacity piles (satisfy structural capacity) or additional piles in the group, designed to include full downdrag forces,
- Both geotechnical and structural capacity has to be satisfied, hence, sometimes piles had to be driven deeper (to satisfy geotechnical capacity).
- At some very isolated locations, the settlement due to new fill placement was allowed to occur before pile driving.

LATERAL LOADS

Lateral load controlled many pile group designs. HCN assisted the structural design teams with detailed lateral load designs using advanced analysis methods using LPILE and GROUP computer programs. Lateral analysis is a complex soil-structure interaction problem and involves several iterations and input from the structural designers. The modeling of complex boundary conditions and load transfer from the superstructure to the foundation substructure requires fundamental understanding of soil-structure behavior and experience with modeling issues. The load-transfer curves unique to the various soil types were developed through limited field and extensive laboratory testing and our long-term experience with the various soil and rock types. Battered piles (4:1 to 6:1 batter) were used to address lateral loads. The use of closely spaced piles in congested urban areas due to a requirement of small pile caps had to be considered in pile design and the group efficiencies for axial compressive and tensile and lateral loads. The pile group spacing and geometry require the use of group effects and influence the selection of p-y group modification factors. Passive lateral earth pressures are often conservatively ignored due to concerns related to inadequate compaction adjacent to the pile caps. A significant effort was made to consider full passive capacities of the soil surrounding the pile caps as they have a pronounced effect in producing efficient pile cap sizes. Careful quality control specifications were developed such that the soil

adjacent to the pile caps was well compacted. Typically a 5 ft zone around the pile cap was compacted to a minimum of 98% Standard Proctor density.

PILE DRIVING VIBRATIONS

Vibrations caused by the installation of pile foundations are often a cause of concern to the contractor and property owners. Predicting the effects of pile driving vibrations requires an understanding of the effects of vibrations on humans and structures along with an understanding of how vibration energy is generated by the pile and propagates through the ground. Vibration energy from pile driving can have three potential adverse effects:

- Vibrations transmitted through the ground to nearby structures can cause structural damage or annoyance to persons.
- Vibrations transmitted through the air can cause annoyance to persons.
- Repeated cycles of vibration in the ground can lead to soil subsidence, damaging nearby structures.

Peak particle velocity (PPV), as opposed to displacement or acceleration, has been found to be the best predictor of the effects of vibrations on humans and structures. The frequency content of the vibratory motion is also an important factor in predicting how vibrations will affect humans and structures. Generally, humans will perceive vibrations greater than 0.01 in./sec; vibration levels greater than 0.1 in./sec are strongly perceptible and possibly disturbing, and vibration levels greater than 1 in./sec are likely to be disturbing. Structure damage from vibrations is a function of properties and condition of the structure and the nature of vibration energy, specifically, the vibration level, the frequency content, and the number of cycles. Chae (1978) represents a somewhat conservative summary of PPV thresholds (see Figure 2).

The vibrations associated with pile driving were certainly a factor to consider during design and construction. Pile driving in congested downtown areas included both considerations of a perception human factor and impact to structures. Some of the pile driving occurred very close to office buildings. The age and type of structure had to be considered in setting threshold limits. A detailed and well-coordinated vibration-

monitoring program was undertaken to ensure that the measured vibrations due to pile driving were within prescribed limits. All major structures were instrumented and a pre-construction survey was also performed. A pre-construction survey was performed of critical structures before pile driving. An action plan was developed to address concerns if the vibration levels were exceeded. Though there were several complaints from nearby offices during construction, the measured vibrations levels were well below the threshold limits. No structural damage due to pile driving was reported on this project.

PILE LOAD TESTING

Due to the wide variety of pile types and loads and the variability of soil types, a comprehensive pile load-testing program was undertaken. Pile load testing consisted of static load testing (both axial compressive and tensile) and dynamic load testing. The data from the load test program was used to:

- Confirm pile capacity (axial compressive and tensile)
- Determine pile settlement (short-term and long-term) under working loads
- Determine creep characteristics of shale bedrock under sustained loads
- Develop an optimized soil model based on data from pile load tests from earlier contracts
- Assist in hammer selection, hammer performance and development of production pile driving criteria.
- Determine the magnitude of pile setup/relaxation and account for this behavior in determination of the pile capacity during pile driving.

A total of 16 static pile load tests were conducted for the various contracts. A significant number of piles in each contract were also dynamically tested as will be discussed later in this paper. The static pile load tests consisted of 10 axial compressive load tests and 6 axial tensile load tests. Load tests were performed on various soil stratigraphies and pile types. The static load tests were performed in conjunction with dynamic load tests performed during the driving of piles. Restrike testing was performed on some of the test piles. The results of static and dynamic load testing were used to determine the production pile driving criteria. The results of the static load tests were considered to be

extremely useful for future pile designs and have resulted in significant savings to the overall project.

SOIL/PILE SET-UP

Soil/pile set-up (time-dependent increase in pile capacity) has long been recognized and can contribute significantly to long-term pile capacity. Accounting for set-up in pile design offers substantial benefits, such as the use of smaller hammers, smaller pile sections, shorter piles, higher capacities, and therefore more economical installations than otherwise possible. Set-up is primarily related to an increase in shaft resistance and is related primarily to dissipation of excess pore water pressures within, and subsequent remolding and reconsolidation of, soil which is displaced and disturbed as the pile is driven. Depending on the permeability and the amount of disturbance, dissipation of excess pore water pressures can be non-uniform for some time after driving. Subsequently, excess pore water pressure dissipation is uniform with respect to log of time.

The upper soil profile along the majority of the project consists of fine-grained alluvial soils where the potential of significant set-up was anticipated. During the initial contracts (Contract 8), the pile installation would consist of driving an initial 50 feet long section, and the next section would be welded and then after a 12 to 24 hour wait, the pile would be driven to completion. It was observed that, after the 12 to 24 hour wait, the resistance to driving was substantially greater than during initial driving. Dynamic load testing was performed on piles during initial driving (EOID) and restrike testing and an increase in capacity of 20 to 100 kips was observed. The majority of the observed set-up was found to occur in the upper alluvium. At later contracts, it was decided to design and execute a project specific dynamic load test program (on production piles earlier in the Contract) to document the magnitude of set-up and incorporate that in the driving criteria. Piles were dynamically tested during initial driving and restrike tested at 1 day and 3 days after initial driving. The data from the load testing was used to determine the total capacity gain and the rate of capacity increase with time. It was observed that the majority of set-up occurred within 24 hours and minimal gain was observed beyond the first 24 hours. The fast-track nature of the project served as a disadvantage to implement a test program in advance and study the data in detail. However, based on the available test

data from previous contracts and limited restrrike testing it was determined that a set-up of at least 20 kips (conservative estimate) was occurring and the driving criteria of all future production piles incorporated this strength gain of 20 kips. This resulted in reasonable savings in pile lengths and also time to drive the piles.

DYNAMIC TESTING

High strain dynamic testing has become an established pile testing method to compute the static bearing capacity of piles. Although high strain dynamic testing refers specifically to the field-testing phase where real time measurements are taken on the pile while it is driven, the dynamic testing process really encompasses three components. These components include 1) wave equation analysis (WEAP), 2) field testing utilizing specialized equipment such as the Pile Driving Analyzer® according to the Case Method, and 3) CAPWAP analysis. These three elements of testing and analysis lead to a comprehensive evaluation of hammer performance, pile drivability and pile capacity determination.

Dynamic testing was performed for 9 separate contracts on a total of 114 piles. As indicated in Table 1, the piles driven included 14 and 16 inch O.D. steel pipes with wall thickness ranging from 0.25 to 0.375 inches, HP12x53, HP14x73 and HP14x89 H-pile sections. The ranges of the design capacities are given in Table 1 for each respective contract and pile type and size.

The hammers used were either single acting or double acting diesel hammers. The most common hammer used was the double acting ICE 640, which was used in 6 of the 9 contracts. Other double acting diesel hammers included the LinkBelt 520 and the ICE 520. Single acting diesel hammers included the Delmag D25-32, APE D19-32, APE D30-32 and the ICE 60-S.

Normally, single acting diesel hammers are monitored by the resulting stroke since the ram is visible to the eye. The ram of a double acting hammer is within a closed cylinder and therefore is not visible. The performance of the double acting diesel hammer was monitored by the resulting bounce chamber pressure (the peak pressure inside the top of the cylinder, which results during the ram up-stroke). This pressure is monitored

through a pressure gage, which is connected to the bounce chamber via a hose. Available charts from the hammer manufacturer relate the peak bounce chamber pressure to the available energy.

Table 1 summarizes results from the dynamic testing at all 9 contracts where dynamic testing was performed. Among other information are ranges of measured compressive stresses, hammer transferred energy, transfer efficiency (defined as the ratio of transferred energy to the rated energy of the hammer), and ranges of design and mobilized capacities.

Among all pipe piles tested, the maximum compressive stress (stress measured near the pile top at the location of the gages) ranged from 17 to 39 ksi. Among all H-piles, the maximum compressive stresses ranged from 23 to 34 ksi. These relatively wide ranges were a result of a combination of variable hammer performance, hammer size and variable soil resistance. Federal Highway Administration guidelines specify limiting compressive driving stresses to 90% of the steel yield strength. The pipe piles were typically ASTM A252, Grade 2 which have a minimum specified yield strength of 35 ksi. The H-piles were typically manufactured from A36 steel, which has a minimum yield strength of 36 ksi. Therefore, the FHWA recommends limiting compressive stresses to 31.5 ksi for the steel pipe piles and 32.4 ksi for the H-piles. With the exception of two cases, the compressive stresses were within these recommended limits reducing the risk of pile damage.

Among all piles tested, the energy transferred to the piles (and therefore the transfer efficiency) varied significantly especially when using the double acting diesel hammers. With this relatively large variability in hammer performance, it can be difficult to assign a stop criterion during production pile driving. To some extent, the same may be said about the single acting diesel hammers since the stroke can also be variable. To aid the inspectors, refined wave equation analyses (i.e., Constant Capacity Analyses) were performed in order to determine the required blow count for various strokes (for single acting diesel hammers) or for various bounce chamber pressures (for double acting diesel hammers) to mobilize the required capacity. Sample output of these analyses is shown in Table 2. With this information, the pile inspector was able to determine at what

blow count to stop driving based on either the observed stroke or observed bounce chamber pressure.

For capacity determination, dynamic testing was performed both during initial driving and restrike (re-test after a waiting period to assess time dependent soil strength changes). Unfortunately, many of the restrike tests were performed on piles that were not tested during initial driving. Sometimes, especially when using the smaller Linkbelt 520 and ICE 640 double acting diesel hammers, the required capacity could not be mobilized during the dynamic test. In some cases, the capacity for these piles was then confirmed by restriking with a larger hammer (ICE 60-S). During the course of performing restrike testing, the results generally indicated increases of capacity due to time dependent soil strength increases (i.e. soil set-up). These increases in soil strength (mainly from friction) ranged from approximately 50 to 100 kips (222 to 445 kN) after a 1 to 2 day waiting period. In some cases where the capacity could not be obtained at the anticipated depth during initial driving, the piles were stopped at a lower capacity than required (the revised wave equation analysis results were used to determine the appropriate blow count) with the anticipation that the required capacity would be obtained after the expected frictional increase due to soil set-up.

Figure 3 shows a sample output from the initial drive testing of a pile driven from 36 feet (11 m) to 81 feet (24.7 m). This figure summarizes results of maximum compressive stresses near the pile top at the location of the measurement gage (CSX), The computed compressive stress at the pile toe (CSB), the hammer transferred energy (EMX) and hammer stroke (STK), the mobilized capacity (RX7) and the observed blow count. Note the excellent correlation between the capacity and the observed blow count. Note also that the stroke (single acting diesel) and transferred energy are variable emphasizing the need for the wave equation constant capacity analysis.

PILE INSTALLATION RECORDER

Introduction

For driven piles, the hammer operates as both the installation device and also the inspection tool. Engineers relate observations of blow count or set per blow to pile capacity. Hammer performance affects the installation. Inspecting driven production

piles involves as a minimum the counting of blows per unit penetration (e.g. blows per foot). Ideally, information about the hammer's performance is observed and recorded. In some cases inspectors may not understand the project needs or even how the hammer works. Unfortunately, the inspection of driven piles is also a monotonous assignment leading to errors in logs recorded by human inspectors. The inspector may have other distractions resulting in missed information or errors. While inspection is critical in a good installation, it is sometimes assigned a low priority and often technicians with little experience serve as inspection personnel. In some cases inspection is totally absent for lack of available on-site inspection personnel. These situations may make automated inspection including hammer performance monitoring desirable.

Automated Monitoring - Driven Piles

The equipment for the Pile Installation Recorder for Driven Piles (PIR-D) requires several components depending on hammer type. All systems require a depth indicator that spools a cable, which is attached to the hammer that follows the pile motion during installation. As the hammer moves, the cable goes over an encoder wheel and the distance traveled or depth of penetration is then recorded.

The Control Unit (see Figure 4) is a small unit with a numerical display of results (depth, blow count and hammer performance). At the beginning of the project, the Control Unit was placed inside the crane within reach of the crane operator. The operator would enter the pile name on an interactive touch screen and the starting pile depth, and finished the monitoring by pressing the end key on the Unit when driving was completed. In later contracts, the Control Unit was placed outside the crane and operated by the oiler or foreman.

Case History for PIR-D

The Fort Washington Way project had multiple pile drivers operating simultaneously. On parts of the project, H piles were driven to rock. On other sections, pipe piles were driven to a sand-bearing layer. In all cases the piles required splicing and were driven in sections. The design team and ODOT decided to request that four pile driving

contractors equip several rigs with Pile Installation Recorders (PIR-D). Most hammers were closed end diesels, while one contractor used an open-end diesel. The contracts and work were staggered, but as many as four PIR-D systems were in simultaneous operation. The first projects monitored by PIR-D began in August to October of 1998 and the final contract was concluded in May of 2000, a total time of about 1.5 years of pile driving activity.

All PIR-D systems were equipped to detect blows by the sound sensor method. The closed end diesel hammers were instrumented beginning in December 1998 with pressure sensors for monitoring the bounce chamber pressure. Stroke was calculated for the open-end diesel from the hammer's blow rate (blows per minute). In all cases the results from every section were printed in the field for a permanent record (Figure 5.) Data was also recorded electronically on a memory card.

Because this was the first major production project for the PIR-D, during the initial phases of this project the PIR-D was further improved based on feedback from the job site. During this time many comparisons with visually obtained driving logs were made. A typical comparison is shown in Figure 6. Differences are generally due to imprecision of the manual inspection as with both the depth and blow counting is highly accurate, the automated blow count per unit penetration must be correct. After the initial three months trial period (and some program upgrades) the system was proven to be reasonably accurate. The PIR-D units then operated practically continuously with only a couple of minor interruptions (e.g. cable accidentally cut). The final contractor to begin PIR-D operation had only one interruption near the beginning of their 8-month use and reported at the end of their work that they "used the Pile Installation Recorder for driving 50,000 lineal feet of tube piling on Contracts 6 and 7 for the Fort Washington Way project."

The precision of the manual mechanical bounce chamber box readings was questioned as the "accuracy" of reading a moving needle on a pressure dial gage is difficult. Most inspectors did not record the bounce chamber pressure for every foot, and even when recorded was only a visual average over a few blows at most. Further, the calibrations of these field pressure monitor boxes were admittedly practically non-existent, while the PIR-D electronic pressure measurement had traceable calibrations and peak results were obtained for each blow and averaged over the penetration increment.

DRILLED SHAFTS

Drilled shafts were the preferred foundation choice for the " Ramp R over Eggleston & Second Street". Being in an urban congested area with several building structures and other ramps in its vicinity, there was insufficient room to accommodate the large pile caps needed for bridge pier support. Drilled shafts were determined to be effective in avoiding conflicts with existing foundations.

The loading varied from pier to pier but design loads were typically on the order of:

Dead + Live Load (kips)	Shear (kips)	Moment (ft-kips)
2250 to 5000	400 to 750	2000 to 9000

A total of thirteen borings were performed at each bridge pier location. The borings were extended to top of bedrock and sufficient rock coring (30 to 40 feet) was performed. The top of bedrock was generally consistent being 120 to 124 feet below existing ground surface elevation. The overburden profile consisted of existing fill, alluvium, lakebed and granular outwash with the static water table being approximately 40 feet below ground surface. Due to the height of the bridge piers, significant lateral loads and bending moments were also anticipated. HCN performed detailed lateral analyses working in close coordination with the structural engineer. There were 12 shafts 8 feet in diameter, 2 shafts 7 feet in diameter and 2 shafts 5 feet in diameter. The drilled shafts were typically 103 to 115 feet long and founded in shale and limestone bedrock. The shafts were socketed 8 to 15 feet in bedrock. Pressuremeter testing along with unconfined compression testing, point compression tests and slake durability tests were performed on the bedrock to aid in determination of rock properties. The unconfined compression strengths varied from 40 tsf to 500 tsf, typically being in the range of 150 to 200 tsf. The Ordovician age bedrock belongs to the very bottom of the Kope formation. Drilled shaft design capacities are very dependent on construction methods. The shafts were designed for both side resistance and end bearing within the bedrock socket. The soil-structure interaction and load distribution versus movement to mobilize side resistance and end bearing were studied. Based on detailed analyses the following design capacities were used:

Allowable End Bearing	Allowable side resistance
100 ksf	4 ksf

HCN performed vibration monitoring of nearby structures during the installation of steel casing for the drilled shaft vibration. Due to increased traffic activity during the drilled shaft construction, vibration effects on nearby structures due to traffic was also monitored.

Crosshole Sonic Logging

Due to the significant loads supported by the shafts and shallow groundwater HCN recommended that Crosshole Sonic Logging (CSL) testing be performed on all production shafts. The CSL test is a non-destructive test that measures the time it takes for an ultrasonic pulse to travel from one access tube in a receiver to another access tube. This test is primarily intended to measure the integrity of shaft concrete. In uniform, good quality concrete, the travel time between equidistant tubes will be relatively constant and correspond to a reasonable concrete pulse velocity from the bottom to top of foundation. Longer travel times and lower amplitude/energy signals indicate the presence of irregularities such as poor quality concrete, void, honeycomb and soil intrusions. The signal will be completely lost by the receiver and CSL recording system for more severe defects such as voids and soil intrusions.

A total of 8 tubes were installed in each shaft at a 45 degree spacing. The tubes were 2" inside diameter schedule 40 steel pipe. The pipes were fitted with a watertight shoe at the bottom and a removable cap at the top and were securely attached to the interior of the reinforcement cage with a minimum cover of 6 inches.

Only two of the sixteen shafts tested indicated some anomalies. At Pier 1R, two anomalous zones were reported from the CSL testing. The first region was from 20.7 to 25.3 feet from the top of the shaft and the second region was 31.1 to 35.1 feet from the top of drilled shaft. The first region was localized around CSL tube 1 with the concrete strength reported at 1680 psi in this region. The region affected by the poor concrete accounted for only 10 to 15% of the drilled shaft cross-sectional area. The second region

was more widespread, however it still only accounted for 20 to 25% of the total cross-sectional area. The strength in this area was estimated to be 520 psi, with the remaining cross-section having full concrete strength. After extensive analysis by the structural engineer and lateral analyses by HCN it was determined that no structural problems would result from these anomalous regions and no remedial action was necessary.

Pier 1R right shaft showed that the bottom 5 ft contained poor concrete or possibly mud mixed with concrete. The remaining shaft appeared competent. Due to the significant defect in the bottom, the end bearing could not be relied upon and the contractor decided to abandon the shaft. After further discussions with the design team, it was determined that the shaft was capable of handling the lateral loads and moments, since the maximum moments were in the upper 30 ft of the shaft. To carry the axial compressive loads, it was decided to drive H-piles (driven to practical refusal on bedrock) around the perimeter of the shaft. A pier cap was then cast between the piles and over the drilled shaft.

GROUND IMPROVEMENT

The presence of existing heterogeneous fill and upper moderately weak alluvium required both shallow and deep ground improvement. This area of downtown has been occupied and worked by man for many decades. Thus, cisterns, dry wells, privies, etc. were expected and encountered within the construction area. Past work on downtown sites has shown cisterns typically to be 3 to 6 feet (0.9 to 1.8m) in diameter, which can extend as deep as 40 feet (12.2m) below grade. Typically when cisterns were encountered, they were grouted full. Remnant foundation walls of buildings were also anticipated and encountered during excavation. The common methods of ground improvement included the undercutting of shallow unsuitable soils and replacement with new structural fill. Chemical stabilization using lime was also used. In some circumstances, where higher bearing capacity was desired, two or three layers of biaxial geogrid were also included to create a soil foundation mattress. Many of the MSE walls (15 to 35 feet high) required some minor undercutting, increasing the embedment of the bottom of the MSE wall, and increases in the minimum length of reinforcements.

However, for the 50EB-1 MSE wall, west of the Lytle Place underpass, slope stability analyses indicated unsatisfactory factors of safety against long-term and short-term global stability. The lower global stability results were primarily due to the large exposed wall heights (20 to 31 feet), weak lakebed clays encountered below the existing controlled fill soils, and the existing 2H:1V slope located below the proposed wall. Several wall heights ranging from 10 to 36 feet were evaluated in increments of 5 feet. The results are presented on Figure 7. The analyses indicated that wall heights less than 15 feet were stable. The foundation soils were somewhat variable, so some limited ground improvement including minor undercutting with or without geogrid reinforcement was recommended.

The stability analyses indicated that the failure surfaces were relatively deep-seated (greater than 10 feet (3.1m) below base of wall). As a result, shallow soil improvements will not substantially increase the global stability. Therefore, deep soil stabilization/reinforcement of the foundation soils was recommended for the 50EB-1 MSE wall construction west of the proposed Lytle Place underpass. The deep soil stabilization/reinforcement consisted of stone columns.

Vibro-stone columns have been used for the past 30 years for in-situ ground reinforcement. The term vibro-stone column is used to emphasize the vibratory nature of the installation method.

Vibro-stone column construction is accomplished by down-hole vibratory methods, typically referred to as vibro-replacement and vibro-displacement. A vibratory probe penetrates the ground, either under its own weight or aided by water jetting. Horizontal vibrations are produced close to the base of the vibrator, induced by rotating eccentric weights mounted on a shaft driven by a motor housed within the casing. The vibrations produced by this unit are generated close to the tip of the probe and emanate radially in the horizontal plane away from the unit. Stone backfill is introduced in controlled lifts, either from the surface down the annulus created by penetration of the probe (top feed) or through feeder tubes directed to the tip of the probe (bottom feed). Repenetration of each stone backfill lift forces the stone radially into the surrounding in-situ soil, forming a

vibro-stone column that is tightly interlocked with the soil. The vibro-stone column and the in-situ soil form an integrated system having low compressibility and high shear strength.

The degree of densification achieved is primarily a function of soil type, silt and clay content, soil plasticity, and pre-densification relative densities. In granular soils, typically with fine content ranging from 15 to 25 percent, stone backfill enhances densification and drainage. In more cohesive soils, the stone backfill in its densified state performs as a structural reinforcing element to increase the bearing capacity of the mass and greatly reduces total and differential settlements.

HCN performed the design of the stone column ground improvement. The design was based on 6 rows of stone columns, which were centered under the centerline of the MSE wall. To achieve global safety factors of 1.2 and 1.3 for short-term and long-term, respectively, or higher, we recommended that the design be based on 25 ft long, 12" diameter stone columns using a triangular spacing of 6.6 ft in each direction (see Figures 8 and 9). Ninety stone columns would be required to stabilize a length of 200 feet

Columns would be formed of No. 57 crushed stone. Columns are usually paid on the basis of weight per lineal feet of installed stone column. Sometimes heave is experienced when installing stone columns in cohesive soils. Approximately 1100 tons of No. 57 stone would be required to reinforce this area with stone columns. HCN developed the stone column specification and performed quality control during construction. Inclinoimeters were installed downslope of the wall to monitor deep-seated movements during the MSE wall construction and in the long-term. Based on data during construction and one year after construction, the inclinometer data showed lateral movement of foundation soils of less than 0.5".

RETAINING WALLS

The urban construction setting, right-of-way limitations, and the need to optimize land use necessitated the use of numerous retaining walls. Again, due to the significant length of these walls and the costs to construct these walls, a very methodical approach

to study the various wall types was performed. A matrix was developed of various wall types, cost per unit area, advantages and disadvantages, time of construction, constructability issues, and aesthetics. More than 25,000 lineal feet of walls were constructed on this project, varying in height from 6 feet to 35 feet. Some of the wall types included:

- MSE walls
- Pile supported cantilevered cast-in-place walls
- Soldier pile walls with rock anchors
- Soldier pile walls with anchors in granular outwash
- Cantilevered soldier pile walls (temporary and permanent)
- Temporary soil nail walls
- Temporary wire faced MSE walls

The Fort Washington Way Trench is delineated with a cantilevered retaining wall between Fort Washington Way and Third Street on the north and Fort Washington Way and Second Street on the south. These walls are supported on pile foundations. Several pile configurations and spacing were initially evaluated to determine pile loads and their relative distribution. The structural engineer provided the preliminary load distribution for the two-pile group to us. We used the computer program "Group" to analyze the behavior of the two-pile group arrangements subjected to both axial and lateral loads. This program is capable of computing the distribution of loads (vertical, lateral, and bending moment) from the pile cap to the piles in a symmetrical pile group. The method of solution is finite difference. Both vertical and batter piles with different boundary conditions (fixed, pinned, or elastically restrained) can be modeled. The non-linear response of the soil, in the form of T-Z curves for axial loading and P-Y curves for lateral loading, was generated and used in our analysis. In consideration of the two pile rows, an elastically restrained boundary condition was used in our analyses. Several preliminary cases were evaluated to compute an appropriate rotational spring restraint. Some passive resistance (using at-rest pressure coefficients) was considered along the face of the pile cap. An at-rest pressure coefficient was used since minimal movement of the pile cap into the soil was expected.

Wall backfill consisted of bank run sand and the walls were inclined some 10 degrees on both faces and were faced with a pre-cast concrete panel system for visual effect. The wall is supported on two rows of vertical and battered steel pipe piles. The pile lengths are typically on the order of 50 to 75 feet long. Due to space limitations, it was decided to keep the pile cap as small as possible and support the 32 feet high wall on no more than 2 rows of piles. The vertical piles were designed to be in permanent tension. Typically, it is ODOT practice not to allow design of piles to be in permanent tension. However, several tension load tests were performed to confirm load carrying capacity. Using the short-term load test data, some extrapolations were performed to determine the load holding characteristics of the pile over the design life.

The typical wall design consists of two rows of piles with a spacing of 7.84 feet (2.8m) between the centerlines of each row. The total width of the pile cap was 12.14 feet (3.7m). The front row of piles was driven at a 1H:4V batter, spaced approximately 4 feet (1.2m) apart. These battered piles will be in compression and resist a majority of the lateral shear forces through their horizontal force component.

The back row of piles was driven vertically, spaced approximately 12 feet (3.7m) apart and will act in permanent tension. Our analyses indicate that the majority of the loads will be transferred as a force couple to the two pile rows. Relatively low bending moments would result. The computed maximum bending moment at the top of piles was approximately 45 feet-kips (61m-kN). Our analyses using the computer program "Group" yielded comparable results for pile capacities and other reactions as computed by the Structural Engineer.

The maximum compressive load on the battered piles has been computed to be 120 kips (534 kN) per pile. The maximum tensile load on the vertical piles was computed to be 20 kips (89 kN) per pile. However, during construction, the back row of vertical piles will need to resist 151 kips (673 kN) of compressive loads. These temporary construction loads are primarily due to dead load of the retaining wall structure and pile cap with no backfilling (hence, no lateral earth pressure), as the tensile load will not yet be developed. The pile lengths have been designed for an allowable compressive load capacity of 120 kips (534 kN) with a factor of safety equal to 2.0. During construction, the anticipated load on the vertical back row of piles will be 151 kips (673 kN) (126.1% of

the allowable load of 120 kips (534 kN)). Since the construction loads are applicable only in the short-term, we were of the opinion that the piles will be able to resist the approximate 26% overload without significant settlement. However, it was recommended that the wall be backfilled within a relatively short period (perhaps no more than 4 weeks). The long-term loads on the vertical piles after the wall was backfilled were computed to be a maximum tension load of 20 kips (89 kN). An appropriate tensile connection was designed between these piles and the pile cap.

One of the challenging and interesting walls included the floodwall on the riverside of the FWW trench. The floodwall is necessary to protect the FWW trench and downtown from the 100-year flood. The floodwall functions in a dual role by providing flood protection for FWW and it retains the embankment between FWW and Second Street for landscaping. The top of the floodwall was near elevation 512.0 feet. The soil profile along the floodwall alignment consists of variable fill underlain by alluvium, granular outwash, lakebeds and bedrock occurring some 80 to 100 feet below existing grade. The initial concept was to construct a soft slurry cutoff floodwall extending all the way into bedrock. The slurry wall was studied in detail and it was deemed to be very expensive. As part of earlier flood protection, a deep undercut and cut-off trench was constructed over a portion of the alignment, which was deemed to have been effective during past high water events. A relatively impermeable lake clay zone was present 40 to 60 feet below existing grade. It was decided to drive interlocking steel sheet piles (with special design at the interlocks to reduce the flow of water) below the concrete floodwall to act as a cutoff. Due to the compressible nature of foundation soils and large estimated settlements, it was decided to support the approximate 30 feet high wall on pile foundations. The floodwall was located 25 feet south of the south FWW trench retaining wall A (see Figure 10). The trench walls were designed using an equivalent fluid of 32 pcf, based on granular backfill with the active wedge zone. The 10-degree batter into the wall backfill and the friction between the wall face and backfill was taken into account in determining the earth pressures. The south FWW trench wall had already been constructed before the floodwall. Due the excess quantity of fill available from the earlier contracts HCN was asked to evaluate the use of on-site cohesive backfill for the floodwall. The use of on-site backfill would impart much higher earth pressures on the wall and the south FWW wall had already been constructed. The piles supporting the wall could not support the additional load without increased settlement and a lowered

factor of safety, which was not deemed acceptable to the City and State. An innovative solution (and likely the first use in Ohio for a major project) was designed to reduce the earth pressure on both the floodwall and the south FWW wall. The earth pressure reduction concept was based on the use of compressible Expanded Polystyrene (EPS) geofoam inclusion against the wall face. Detailed numerical analyses using finite elements were performed to determine the thickness and engineering properties of the compressible inclusion.

The traditional approach with dealing with at-rest earth pressures is to design a structure that has sufficient strength in its constituent materials to withstand the stresses. A potentially more cost-effective solution would be to place a compressible inclusion behind the wall to create a condition called "controlled yielding" within the retained soil. This means that even if the retaining structure were rigid and non-yielding, the compression of the compressible inclusion would be sufficient to allow the retained soil to strain and mobilize its strength to achieve the active state. To reduce the earth pressures below the "active state," layers of tensile reinforcements can be considered. A reinforced mass of soil is ineffective in reducing lateral pressures by itself because the mass must be able to deform laterally to strain the reinforcement and allow the overall mechanism of mechanically stabilized wall earth (MSE) to develop. The presence of a properly designed compressible inclusion can allow sufficient movement so that a largely self-supporting MSE mass develops and lateral earth pressures on the retaining structure are reduced well below the active level.

The design required the use of geogrid reinforcement within lower third of the wall where the earth pressures were the greatest. Tensar UX 1700 HS geogrid reinforcement at 1.5 to 2.0 ft spacing was used. The compressible inclusion consisted of Type X1 EPS (0.75 pcf), Geotech systems Terraflex or equivalent, elasticized, and treated with "perform guard" for insect resistance. The geofoam was used for the entire wall height with the geogrid reinforcement included in the lower third or so of the wall. The geofoam thickness varied from 9 to 14 inches in the reinforced zone and 6 to 9 inches thick in the unreinforced zone. A Mirafi 6000 drainage panel was applied to the geofoam panel.

CLOSING

The unique combination of the challenging and varied geology, urban environment, accelerated construction schedule, and wide variety of structures truly classifies this project as a "geotechnical odyssey". This paper provides a brief overview of this landmark transportation project and highlights the major geotechnical challenges and innovative design and construction methods employed to make this project a huge success. The author would like to thank and acknowledge the efforts and contributions of several HCN staff and other design team members including ODOT, and City of Cincinnati personnel to make FWW 2000 a reality. The author would also like to thank Dr. Mark T. Bowers for his review of this paper and valuable input.

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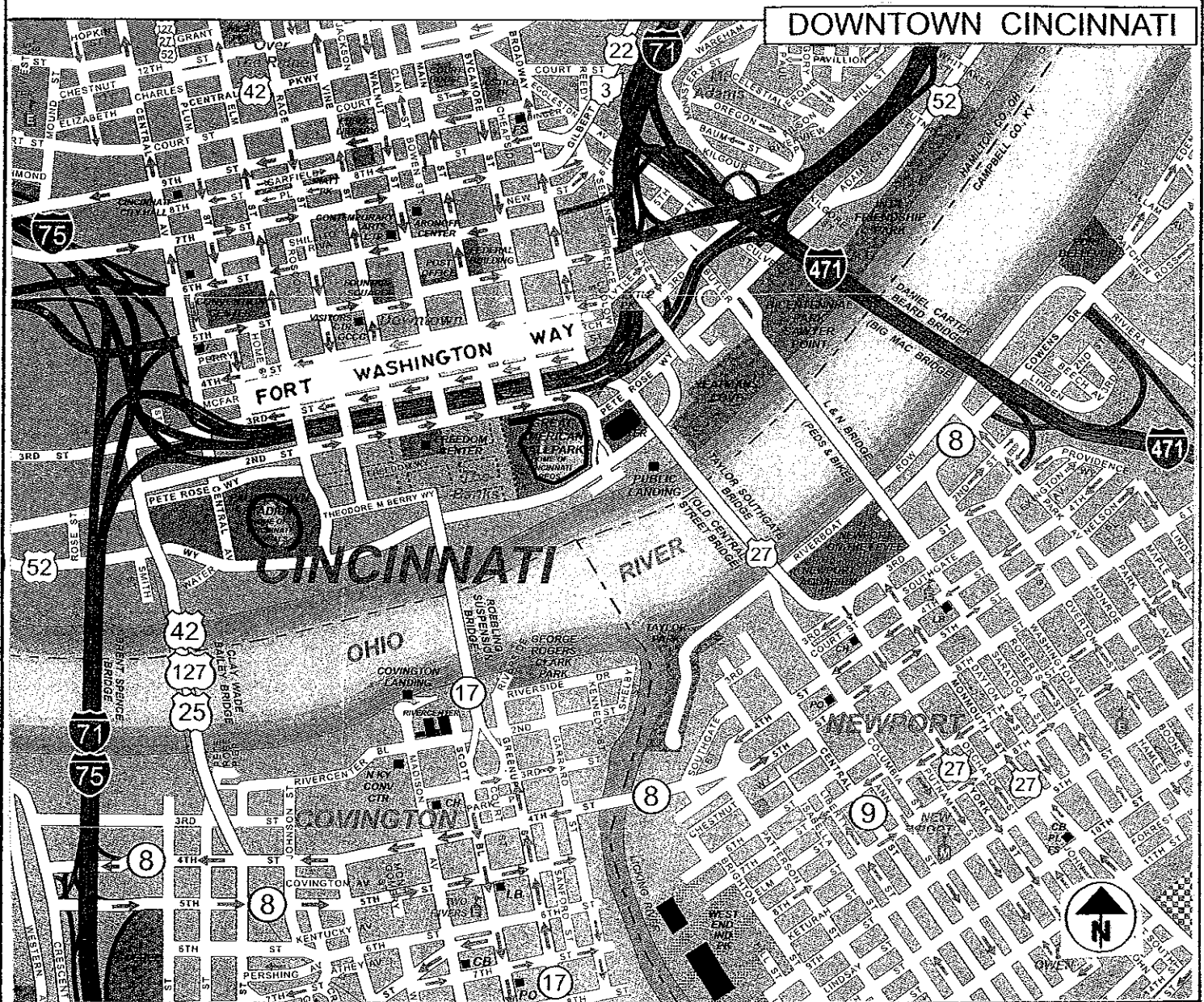
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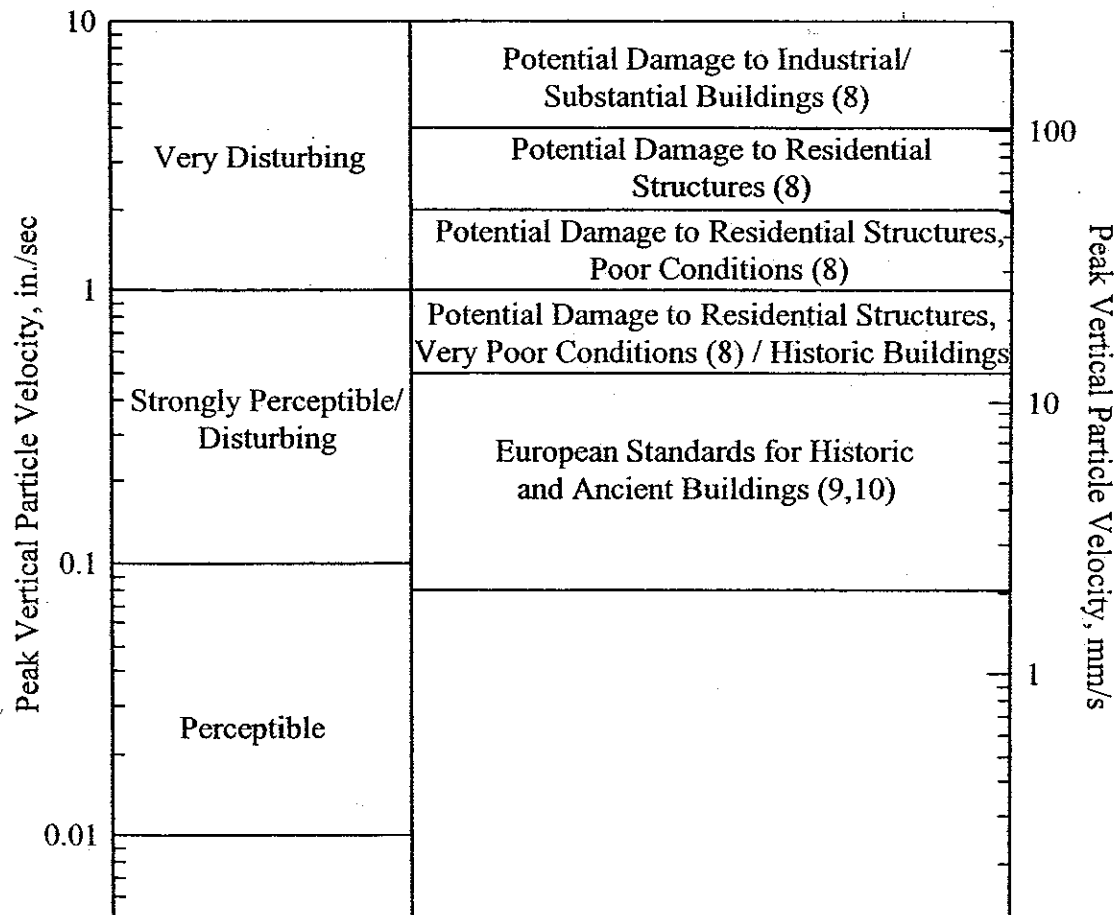
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Peck, R.B., W.E. Hanson, and T.H. Thornburn (1974). Foundation Engineering, Second Edition, John Wiley & Sons, Inc., New York.



FORT WASHINGTON WAY 2000

FIGURE 1



a) Human Perception
Thresholds

b) Structural Damage Thresholds

Chae, Y.S., "Design of Excavation Blasts to Prevent Damage," Civil Engineering, ASCE, Vol. 48, No. 4, Apr. 1978, pp. 77-79.

GRL & Associates, Inc.

2000-Mar-23

FWW-Cont #14, Pier 5D-4, Pile #39, APE D19-32

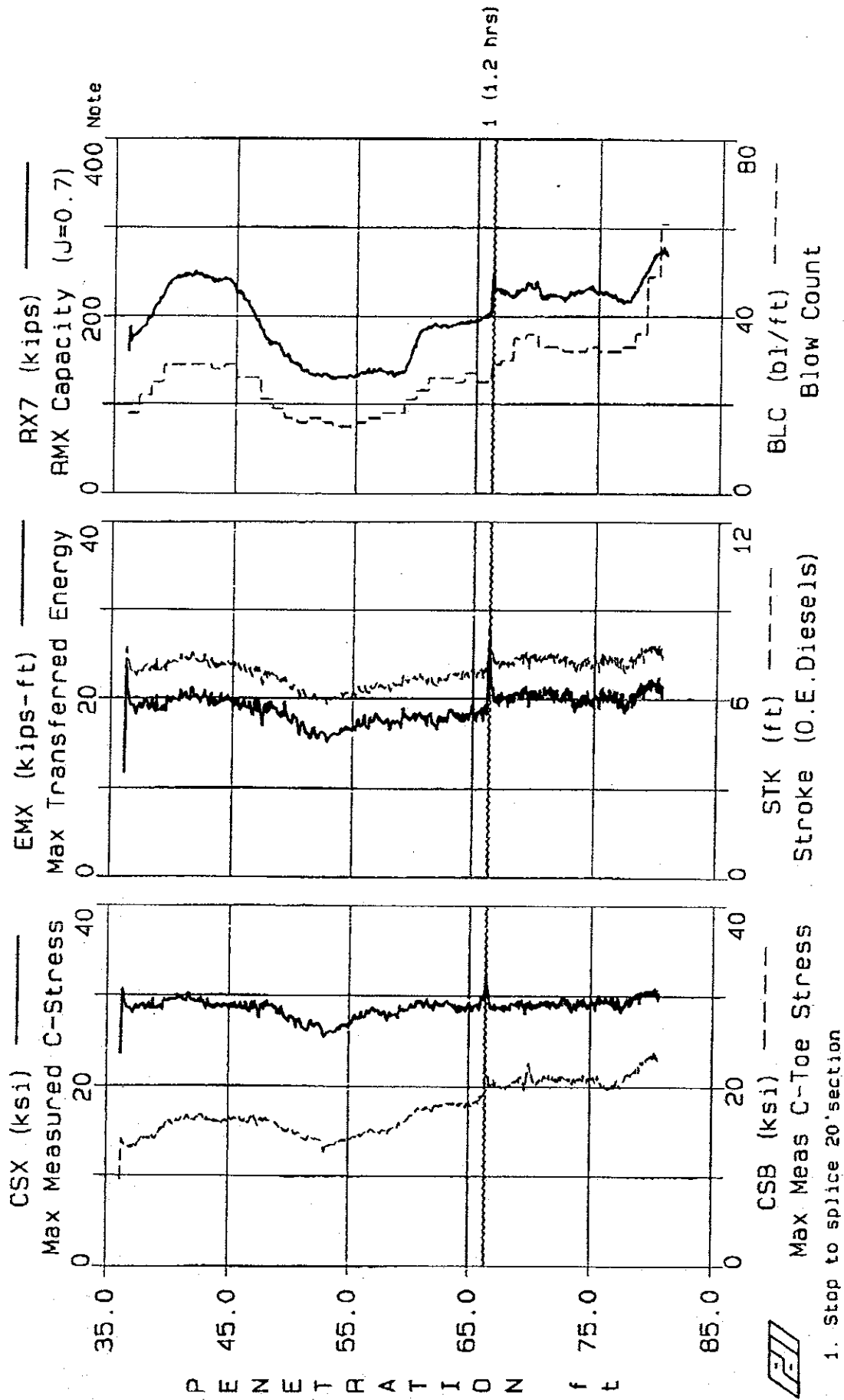
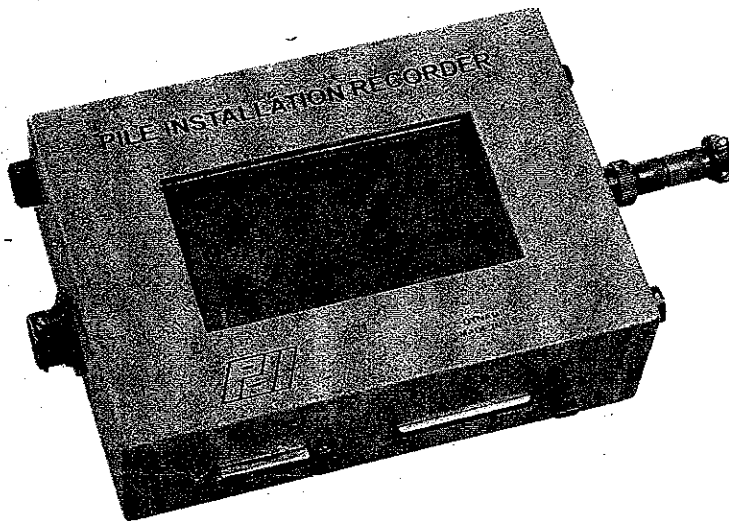


Figure 3



The "touchscreen" can display many different menus for data entry and inspection. Request are made by pressing the "box" with the name of the desired function. The display changes to a new screen with the appropriate requests.

CFA PIR Main Menu	
Project Info	Review Data
Configuration	Collect Data

By touching indicated locations on the screen, the operator easily enters information about the project site, crew and pile.

Project: CLEVELAND	Auger Dimensions: 50.00ft x 4in
Operator: CHRIS	Shaft Diameter: 18.00 Inches
Ris: HITACHI #12	Storage Interval: 1.00 Foot
Pump Volume: 0.500 ft ³	Min. Grout Ratio: 1.20
Note: TESTING	EXIT

The drilling depth and drilling rate are displayed in large, easy-to-read numbers.

DEPTH (ft.): 32.6	ELAPSED TIME 00:04:47
AUGER TORQUE PRES. (PSI): 1973	DONE?
AUGERING SPEED (ft./min.): 12.3	AUGERING
	ZERO DEPTH
	PILE NAME: TP2

During the grouting phase, the pumped volume per unit depth is graphically displayed as the auger is withdrawn. The operator observes the volume ratio and withdraws the auger so that a minimum volume per unit depth is maintained (dashed line).

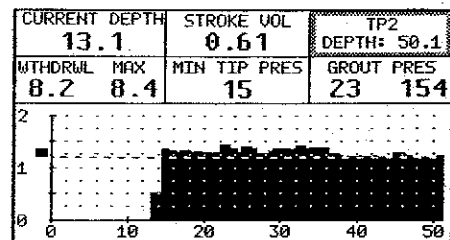


FIGURE 4

Project: FW WAY
 Date: 1998-11-13 16:03:28
 Pilename: B575

section #1

pen (ft)	bl/ft	bpm
1.00		
2.00	5	80.0
3.00	3	71.7
4.00	6	64.2
5.00	12	81.7
6.00	4	79.3
7.00	13	84.1
8.00	4	83.1
9.00	10	88.1
10.00	16	87.2
11.00	16	88.9
12.00	17	88.0
13.00	17	87.0
14.00	18	88.0
15.00	18	87.6
16.00	19	87.6
17.00	20	87.6
18.00	24	85.9
19.00	26	86.9
20.00	27	86.1
21.00	28	86.2
22.00	29	85.7
23.00	33	86.0
24.00	37	85.4
25.00	47	85.0
26.00	50	84.9
27.00	28	83.4
28.00	26	83.6
29.00	24	83.8
30.00	24	84.2
31.00	24	83.9
32.00	28	83.1
33.00	34	83.6
34.00	32	83.2
35.00	31	83.8
36.00	29	82.8
37.00	27	84.1
38.00	27	83.2
39.00	27	83.9
40.00	24	84.4

41.00	25	84.3
42.00	21	84.8
43.00	19	85.3
44.00	22	83.7
45.00	30	83.8
46.00	23	84.7
47.00	16	86.0
48.00	14	82.9
49.00	13	86.6
50.00	15	85.8
51.00	16	86.5
52.00	14	86.4
53.00	16	86.1
54.00	19	84.7
55.00	20	84.1
55.28	6	84.7

finished at 16:18:39
 final blowcount: 1.7 bl/in.
 (over the last 6.00 inches)
 set (last 10 blows): 5.61 in.

Project: FW WAY
 Date: 1998-11-14 12:31:06
 Pilename: B575B

section #1

pen (ft)	bl/ft	bpm
56.00		
57.00	30	78.1
58.00	25	82.1
59.00	26	63.3
60.00	23	58.9
61.00	26	74.6
62.00	26	83.3
63.00	25	82.3
64.00	31	83.5
65.00	31	81.6
66.00	33	82.3
67.00	36	83.2
68.00	34	82.9
69.00	35	82.5
70.00	68	82.2
70.69	76	82.0

finished at 12:38:43
 final blowcount: 10.6 bl/in.
 (over the last 6.00 inches)
 set (last 10 blows): 0.60 in.

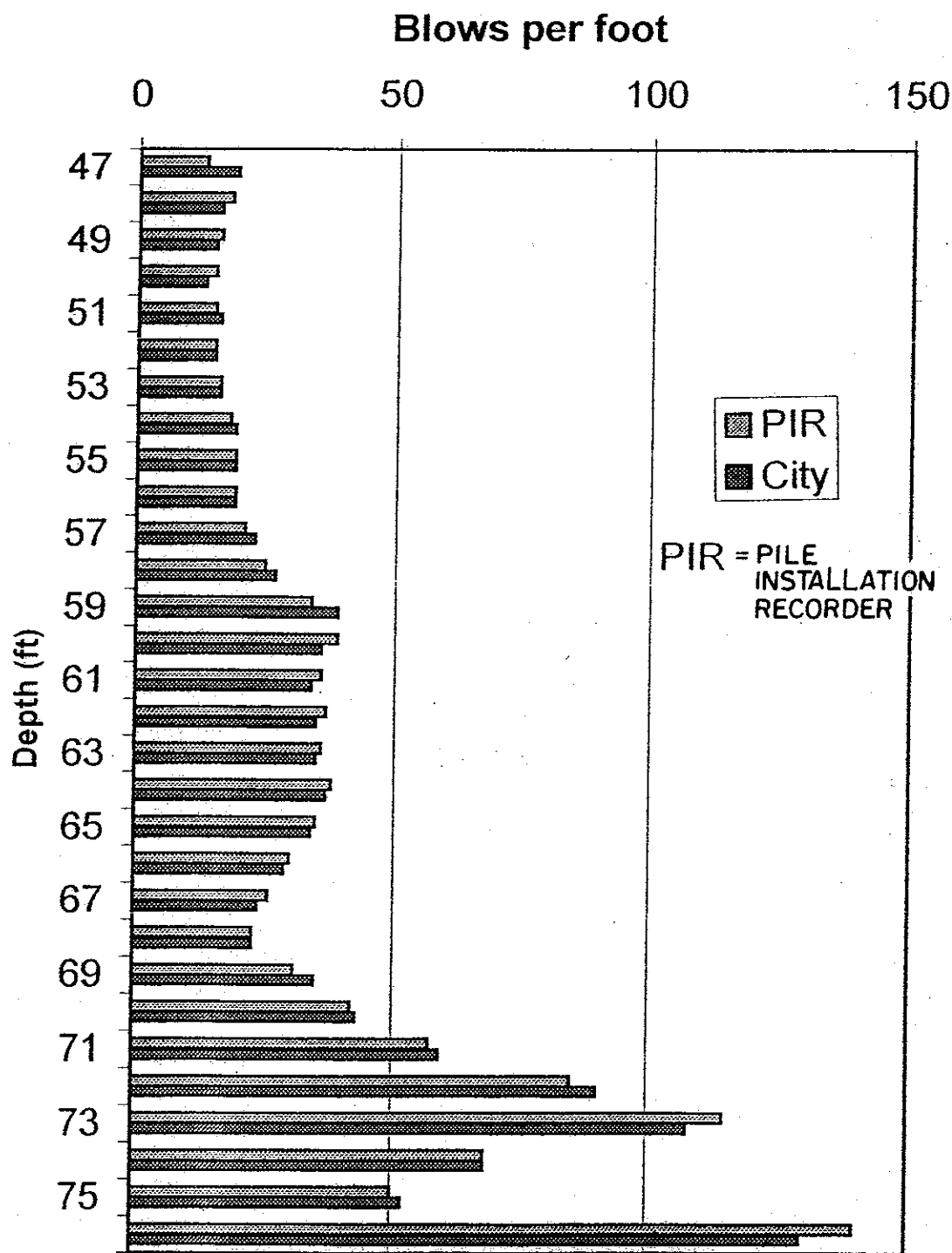


H. C. Nutting Company

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FIGURE 5

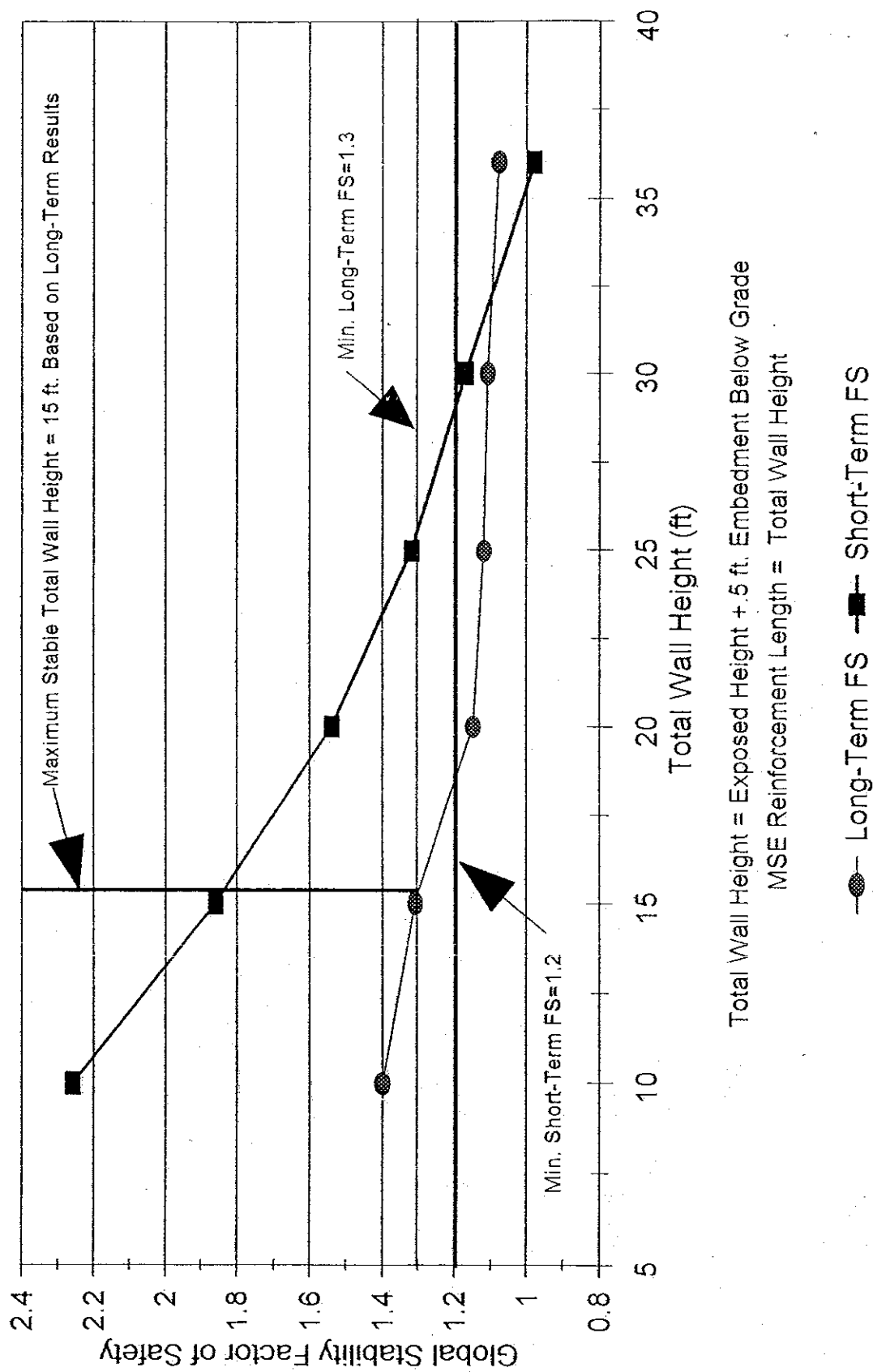
Pile 716 Site 8 FWW, 30-Dec-1998



MSE Wall 50EB-1

East of Lytle Underpass

No Soil Improvements



Total Wall Height = Exposed Height + 5 ft. Embedment Below Grade

MSE Reinforcement Length = Total Wall Height

● Long-Term FS ■ Short-Term FS

FIGURE 7

PROPOSED RAMP L

C

REINFORCEMENT TYPE, LENGTH AND ELEVATIONS TO BE
DESIGNED BY SPECIALTY CONTRACTOR

MSE RETAINING WALL

REINFORCED GRANULAR BACKFILL

NEW
COMPACTED
FILL

11m
(36 FT.)

2 APPROX.

460mm GRANULAR FILL

EXISTING
SOILS

7.6m
(25.0 FT.)

0.9m (3 FT.) DIA.
STONE COLUMNS

B

B'

2m
(6.5 FT.)

0.9m
(3 FT.)

SECTION A-A

CONDITION FOR MAXIMUM
11m HIGH MSE WALL

0 1 2

APPROXIMATE SCALE IN METERS



H.C. NUTTING COMPANY

CORPORATE OFFICE - 4120 AIRPORT RD.
CINCINNATI, OHIO 45226
(513) 321-5816

EMPLOYEE OWNED

GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS

STONE COLUMN SUPPORTED RETAINING WALL
RAMP L-STATION 414+414 TO 414+474

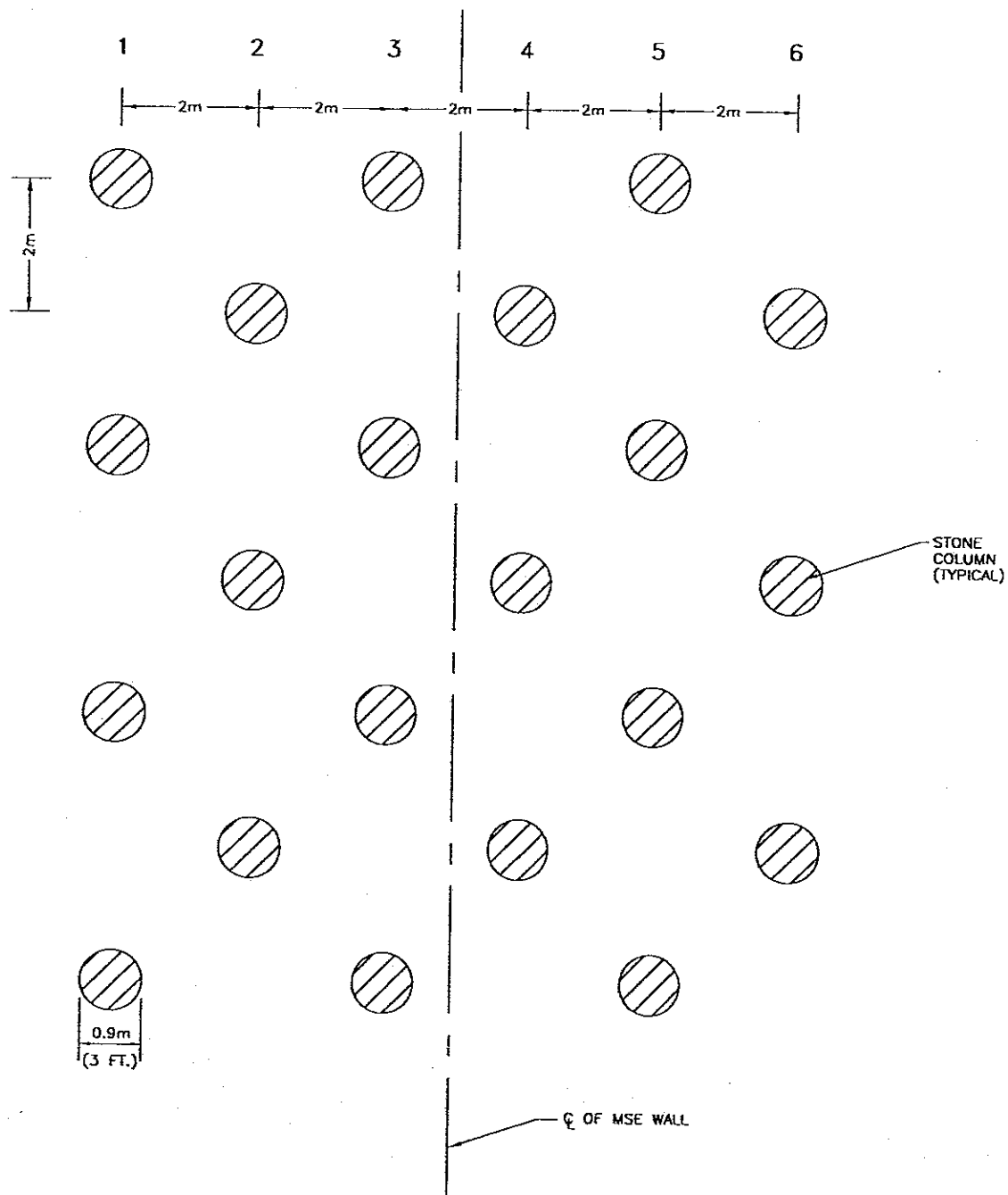
PARSONS BRINCKERHOFF OHIO, INC.
FT. WASHINGTON WAY RECONSTRUCTION
CONTRACT NO. 5-EAST FWW STRUCTURES
CINCINNATI, OHIO

FIGURE 8

SCALE: AS SHOWN

JUNE 4, 1998

DWG NO: 10974.014
DWG ID NO. GEOPIER-KM



SECTION B-B

CONDITION FOR MAXIMUM
11m HIGH MSE WALL

0 1 2
APPROXIMATE SCALE IN METERS

	H.C. NUTTING COMPANY CORPORATE OFFICE - 4120 AIRPORT RD. CINCINNATI, OHIO 45226 (513) 321-5816	
	EMPLOYEE OWNED	
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS STONE COLUMN SUPPORTED RETAINING WALL RAMP L-STATION 414+414 TO 414+474		
PARSONS BRINCKERHOFF OHIO, INC. FT. WASHINGTON WAY RECONSTRUCTION CONTRACT NO. 5-EAST FWW STRUCTURES CINCINNATI, OHIO		
FIGURE 9		
SCALE: AS SHOWN	JUNE 4, 1998	DWG NO: 10974.014 DWG 10 NO. GEOGRID-KM

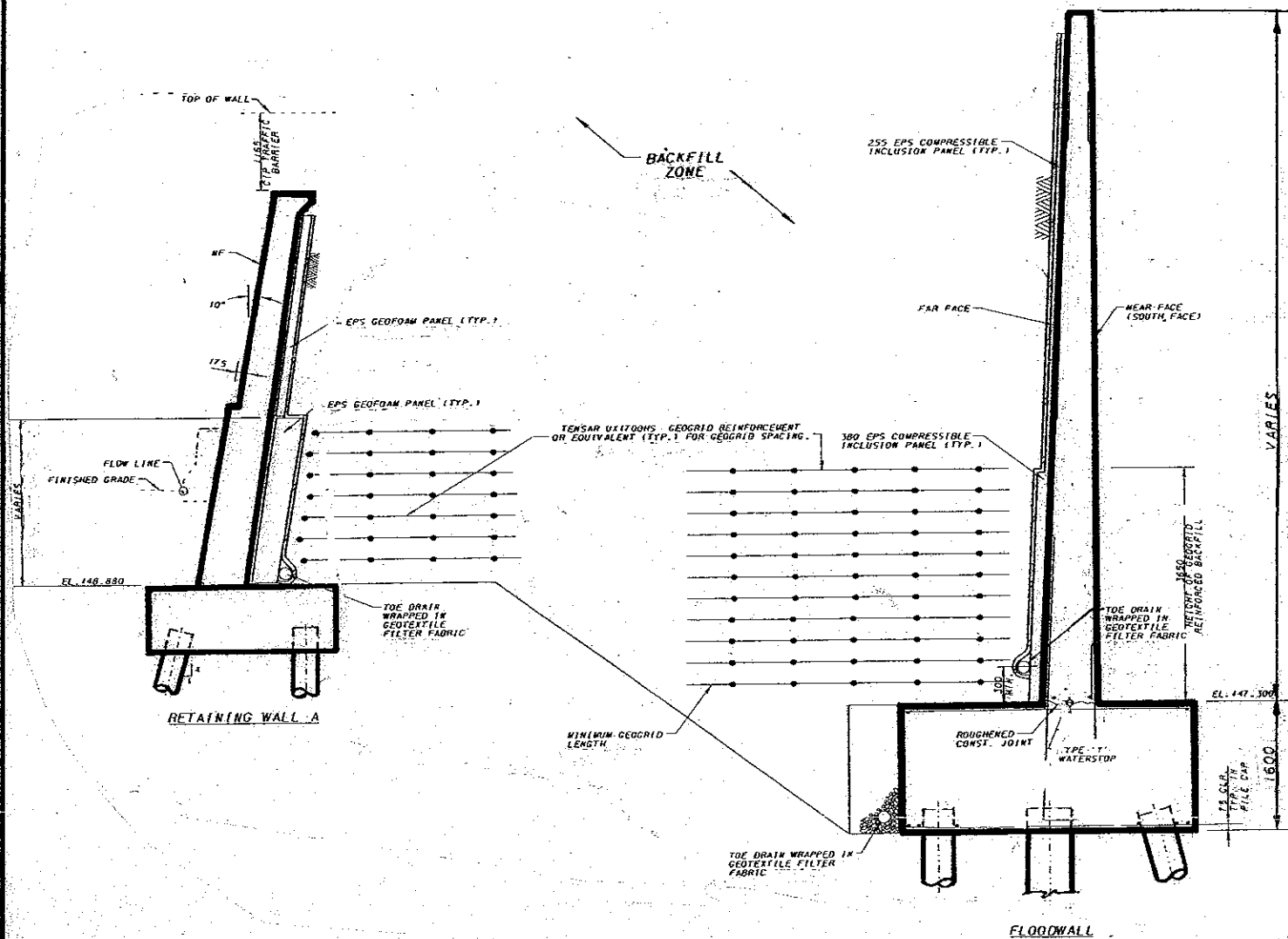


FIGURE 10

Fort Washington Way Dynamic Pile Testing Summary

Contract Number	Number Of Piles Tested	Pile Type	Penetration Depth	Hammer Utilized	Rated Hammer Energy (kip-ft)	Transferred Energy (kip-ft)	Transfer Efficiency (%)	Compression Driving Stresses (ksi)	Required Ultimate Capacity (kips)	EOD Mobilized Capacity (kips)	BOR Mobilized Capacity (kips)
#6	12	14" O.D. x 0.25" W.T.	48-101	Linkbelt 520	26.31	11 - 19	43 - 71	17 - 26	275 & 290	165 - 300	170-300
#5	2	HP14x73	96	ICE 640	40.62	15 - 20	36 - 48	18 - 24	540	390 - 410	-
#9	5	14" O.D. x 0.25" W.T.	55-70	Linkbelt 520	26.31	9 - 14	36 - 51	17 - 24	240	100 - 225	165 - 250
	9	14" O.D. x 0.25" W.T.	58-76	ICE 520	30.37	10 - 14	31 - 46	20 - 24	240	200 - 245	210 - 235
	15	14" O.D. x 0.275" W.T.	54-74	ICE 640	40.62	13 - 15	31 - 38	20 - 23	240 & 290	260 - 300	195 - 345
	3	16" O.D. x 0.335" W.T.	74-82	ICE 640	40.62	17	43	22	300	300	315 - 360
	5	HP14x73	44-50	ICE 640	40.62	18 - 19	43 - 46	23	513	500 - 520	-
#15	6	HP12x53	90-96	ICE 640	40.62	20 - 22	49 - 55	22 - 26	440	400 - 450	-
	7	HP12x53	93-116	APE D30-32	70.07	31 - 36	44 - 52	31 - 34	360	405 - 550	-
#7	14	14" O.D. x 0.318" W.T.	63-80	ICE 640	40.62	16 - 27	39 - 66	19 - 24	340*	240 - 325	270 - 345
#12	4	14" & 16" O.D.	68-80	ICE 640	40.62	20 - 23	49 - 57	19 - 22	340* & 400**	320 - 360	260 - 450
	3	14" & 16" O.D.	69-82	Delmag D25-32	61.49	26 - 28	42 - 46	26 - 28	340* & 400**	330 - 350	440
	7	14" & 16" O.D.	59-80	Delmag D25-32	61.49	Restrikes Only			340* & 400**		
#6B	4	14" O.D. x 0.318" W.T.	25-40	ICE 640	40.62	14 - 21	35 - 52	28 - 39	225	325 - 420	-
	3	16" O.D. x 0.375" W.T.	86	APE D19-32	44.41	20 - 22	45 - 50	32 - 33	362	390 - 400	-
#10/22A	1	HP14x89	98	ICE 60-S	72.94	31	42	30	624	640	-
	1	HP14x73	38	ICE 640	40.62	18	43	27	513	535	-
	2	HP12x53	51-53	ICE 640	40.62	13 - 15	33 - 37	23 - 24	513	400-430	-
	1	HP14x73	75	ICE 60-S	72.94	Restrikes Only			513		555 - 625
#14	4	HP12x53	52	ICE 60-S	72.94	26	35	27	513	555	-
	6	14" O.D. x 0.318" W.T.	66-96	APE D19-32	44.41	18 - 21	40 - 48	28 - 31	340	230 - 280	275 - 375

9 Contracts 114 Piles 10 Pile Types/Sizes 7 Hammer Models

* - Installed until capacity exceeded 300 kips anticipating additional long-term strength gains
 ** - Installed until capacity exceeded 350 kips anticipating additional long-term strength gains

Table 1

Table 2

Goble Rausche Likins & Associates, Inc.
FWW Contract #12 - ICE 640 - 14" O.D.

29-Sep-1999
GRLWEAP(TM) Version 1998-1

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Bounce Chamber Pressure psi	Energy kips-ft
300.0	18.484	1.801	669.0	15.70	11.94
300.0	18.974	1.813	410.9	17.05	12.37
300.0	19.460	1.823	295.1	18.50	12.87
300.0	19.947	1.833	228.4	20.08	13.53
300.0	20.724	1.837	176.6	21.78	15.55
300.0	21.355	1.816	147.5	23.64	16.93
300.0	21.941	1.776	127.4	25.66	18.21
300.0	22.497	1.741	112.2	27.87	19.50
300.0	23.387	1.704	98.5	30.30	20.81
300.0	24.159	1.674	90.8	32.02	21.66

Goble Rausche Likins & Associates, Inc.
FWW-Contract #14, Pier 5D-2, APE D19-32

13-Mar-2000
GRLWEAP(TM) Version 1998-2

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
250.0	22.590	0.809	83.5	5.00	14.30
250.0	24.299	0.805	63.1	5.62	16.21
250.0	26.023	0.736	49.8	6.24	18.23
250.0	27.846	0.656	42.4	6.87	20.05
250.0	29.606	0.591	37.4	7.49	21.80
250.0	31.365	0.552	33.3	8.11	23.62
250.0	33.071	0.483	30.5	8.73	25.25
250.0	34.705	0.422	28.0	9.36	27.00
250.0	36.197	0.352	26.1	9.98	28.65
250.0	37.515	0.303	24.2	10.60	30.46

XXXVI Ohio River Valley Soils Seminar

POST-TENSIONED RETAINING WALLS FOR THE DALLAS HIGH FIVE PROJECT



FOSTER • GEOTECHNICAL

A Division of L. B. Foster Company

by

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INTRODUCTION

In 2001, the Texas Department of Transportation (TXDOT) awarded a \$261-million contract – at the time the largest in the state's history – for the replacement of the three-level Highway 635/US Highway 75 interchange with a five-level interchange designed to link Highway 635 (part of the beltway system around Dallas) and north/south US75 to peripheral routes traveling in four directions. The five-level interchange resulted in the project being dubbed the "Dallas High Five".

Construction of this complex project was originally anticipated to take five years, but is currently expected to be substantially complete by the end of 2005 – a year ahead of schedule. A major reason for the early completion has been due to the willingness of the owner to consider and approve creative changes (Landers, 2005). One of the areas where creative changes proposed by the contractor had an impact on overall schedule is the retaining walls for the project. This paper provides a brief overview of the types of retaining structures that were adopted and presents design details and measured field performance of precast concrete, post-tensioned elements used for retaining walls in fill situations.

RETAINING WALLS

As-Designed Walls

The contract documents called for the construction of some 75 mechanically stabilized earth (MSE) retaining walls totaling more than 265,000 square feet.

Although MSE walls have been widely used in the Dallas area for many years, there were a number of issues that made their use on the Dallas High Five project unattractive. About one-quarter of the total area involved walls in cut situations. The use of MSE walls in these situations would have severely restricted existing traffic flows by requiring additional lane closures to accommodate the width of the reinforced soil zone. Because of space limitations, MSE walls in cuts would have also required significant amounts of temporary shoring. There was also a need to simplify construction in both cut and fill situations, reduce the potential impact on traffic flow, and minimize the number of wall panels that would have to be handled in the field where access is limited.

While MSE walls are generally the most economical retaining wall system in fill situations, experience in the Dallas area indicated instances where there was difficulty in maintaining wall alignment tolerances during construction. One of the ways in which alignment may be improved is through the use of cement-stabilized backfill material; however, this adds considerably to the costs of MSE construction and may have become an even greater cost factor given recent widespread shortages of cement.

The issue of MSE wall alignment was also viewed as an important factor because of the unique artwork to be incorporated in the facing panels. The reason for this is that much of the artwork spans several MSE wall panels, requiring precision in liner fabrication, panel casting and, most important, erection tolerances in the field. Finally, the need for high-quality granular backfill, meeting strict electrochemical criteria, would have further complicated wall construction by having to place both MSE backfill and common embankment at the same time.

Alternative Retaining Walls

Following contract award, the owner accepted a no-cost-change proposal from the contractor involving the use of two different innovative retaining wall systems instead of the as-designed MSE walls. In cut situations, the alternative wall system comprised permanent tied-back soldier pile and lagging walls with 10-foot wide, full-height precast concrete fascia panels. (Because of concerns about the long-term performance of pressure-treated timber, the lagging for all permanent soldier pile walls consisted of cast-in-place reinforced concrete.) In fill areas, the retaining wall system proposed consisted of 10-foot wide, full-height precast concrete units post-tensioned to cast-in-place reinforced concrete strip footings. By adopting the same overall panel width, completed walls have a consistent appearance, whether in cut or in fill. Because of the range of design heights (up to 29 feet), the precast, post-tensioned wall elements were fabricated in three basic types referred to as L-panels, Double-T panels, and Triple-T panels.

Both wall systems were accepted by the owner on an experimental basis, with release from experimental status being conditional upon field demonstration of performance meeting or exceeding that typical for other commonly used retaining wall systems.

DESCRIPTION OF WALL TYPES

Soldier Pile Walls

For the more than 70,000 square feet of retaining walls in cut situations, the system adopted comprised 2-stage soldier pile walls with full-height precast concrete fascia panels. Depending on design height, the steel soldier piles were either cantilevered or

supported by one or more levels of permanent tiebacks. Fig. 1 shows a section through a soldier pile with two levels of tiebacks. The permanent structural facing consists of cast-in-place concrete lagging (either 4 or 5 inches thick) placed in vertical lifts as the ground in front of the wall was removed. Fig. 2 shows a section of wall with two lifts of concrete lagging already in place.

The full-height fascia panels were precast in 10-foot wide sections up to about 30 feet high. A typical panel with the appropriate artwork is shown in Fig. 3. As illustrated in Fig. 1, the full-height panels were supported on an unreinforced concrete leveling pad, similar to that used in MSE walls. The vertical slot between the soldier pile and lagging wall and the fascia panels was not backfilled, except to the level of the top of the traffic barrier in front of the finished wall. A simple double angle attachment device was used to connect the full-height panels to the front of the steel soldier pile sections.

The soldier pile walls as well as the cast-in-place concrete lagging were designed in accordance with TXDOT's Geotechnical Manual and Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems, Report No. FHWA-IF-99-015.

Since there is considerable experience in Texas regarding the performance of permanent soldier pile and lagging walls, no monitoring of displacements either during or after construction was considered necessary.

Post-Tensioned Walls

In fill sections, the MSE walls specified in the contract documents were replaced by a system comprising three types of precast concrete elements post-tensioned to cast-in-

place strip footings. Each type of element was 10 feet wide, allowing the front of all completed walls to have a uniform appearance, regardless of whether they are in cut or in fill; in many situations, a single wall, consisting of both a soldier pile and lagging section with full-height fascia panels and a post-tensioned element section, spans both cut and fill areas.

In order to accommodate wall heights ranging from about 2 feet to almost 30 feet, three types of precast concrete wall sections were developed; these are referred to as L-panels, Double-T panels, and Triple-T panels.

As the term suggests, L-panels are L-shaped in cross section and are used for nominal panel heights up to 70 inches, measured from the top of the supporting footing. The L-panels are fabricated in 10-foot wide units and are secured to the cast-in-place footing using 1-inch anchor bolts; see Fig. 4.

For nominal panel heights in the range 75 to 230 inches, 10-foot wide double-T elements, consisting of a 5¼-inch fascia panel cast monolithically with two vertical stems, were used. As illustrated in Fig. 5, each stem is post-tensioned to the footing using high strength all-thread bars. The anchor plate and locating nut are cast into the footing and a corrugated metal sleeve in each stem allows the post-tensioning bars to be threaded into the securing nut after the panel has been placed and dry-packed with non-shrink grout. The access holes for the post-tensioning bars in the vertical stems can be seen in Fig. 6 while Fig. 7 shows a series of double-T panels that have been set in place, dry-packed, and post-tensioned.

Finally, triple-T panels, similar to the double-T elements but consisting of three vertical stems (see Fig. 8), were used for nominal panel heights in excess of 235 inches.

Erection of all three panel types in the field is relatively straightforward, although accuracy in placement of the anchor plates in the cast-in-place concrete footings is critical to overall efficiency of wall placement. The necessary accuracy was achieved through the use of specially designed steel-framed templates; no modifications or remedial work was necessary due to inaccurate placement of anchor devices during the entire project.

After placement the panels were shimmed before being dry-packed with non-shrink grout. The post-tensioning bars were then installed through the preformed holes in the vertical stems and into the footing and threaded into the nut on the underside of the anchor plates. The required post-tensioning force was then applied and locked off before the entire assembly was grouted in place.

The reinforced concrete footings were designed by the wall system supplier. The footings were supported on generally stiff and very stiff silty clay (typically less than 20 feet thick) underlain by weathered limestone. Subsurface soil conditions were modeled as a two-layer cohesive system; allowable bearing pressures varied from about 3.9 to 4.5 ksf, depending on footing width.

MONITORING OF WALL DISPLACEMENTS

Since this type of post-tensioned retaining wall system had not been previously used in Texas in a similar application, it was necessary to monitor wall displacements during and

after backfilling; in some instances, measurements were continued for a period of at least six months under live traffic. The modest program of field monitoring consisted of regular measurement of vertical and horizontal displacements of reflector plate targets attached to the front face of wall panels. Depending on wall height, between one and three reflector plate targets were installed on a single vertical line on selected wall panels. Initial readings were taken after the panels were placed on the footings and post-tensioned. Subsequent readings were taken during and after the backfilling process.

Survey measurements were made on twenty-nine panels, ranging in height from about 3 feet to almost 29 feet, in six different retaining walls. Although there was some attrition of reflector plates due to construction activity, difficulties in gaining access at some locations as construction progressed, and very small (and sometimes erratic) movements of some L-panels and short double-T panels, there were sufficient data to demonstrate the adequacy of performance of the retaining wall system. No attempt was made to directly measure lateral earth pressures on the panels.

EARTH PRESSURES

The post-tensioned retaining wall system was developed specifically for the Dallas High Five project. One of the major factors that influenced the overall economy and viability of the entire system is the level of lateral earth pressures that should be assumed in design. The precast panels and the cast-in-place footings were assumed to constitute a monolithic system and, consequently, the structure was considered as a gravity retaining wall. Although the entire system appears to be relatively stiff, it was judged that there

would be sufficient lateral displacement due to translation and/or rotation upon backfilling to warrant the use of active earth pressures.

The amount of movement needed to develop the limiting active condition has been investigated experimentally and through the use of the limited element method. Clough and Duncan (1991) state that the results demonstrate:

- The movements required to reach either the active or passive condition are roughly proportional to the height of the wall.
- The movement needed to develop the maximum passive earth pressure is of the order of ten times that required to reach the minimum active earth pressure.
- The movements required to reach the extreme pressures are larger for loose, compressible soils than for dense, incompressible soils. For any cohesionless backfill Clough and Duncan (1991) suggested that the movement required to reach the minimum active condition is no more than 1 inch in 20 feet, or $\Delta/H = 0.004$. The movement required to reach the maximum passive condition is no more than about 1 inch in 2 feet, i.e. $\Delta/H = 0.04$. These criteria (1 inch in 20 feet and 1 inch in 2 feet) provide simple, easy-to-remember guidelines for the movements needed to reach the pressure extremes, and in most cases they are conservative. The results from a number of studies, summarized by Clough and Duncan (1991), are presented in Table 1.

Type of Backfill	Values of Δ/H^a	
	Active	Passive
Dense sand	0.001	0.01
Medium-dense sand	0.002	0.02
Loose sand	0.004	0.04
Compacted silt	0.002	0.02
Compacted lean clay	0.01 ^b	0.05 ^b
Compacted fat clay	0.01 ^b	0.05 ^b

^a Δ = movement of top of wall required to reach minimum active or maximum passive pressure, by tilting or lateral translation; H = height of wall.

^b Under stress conditions close to the minimum active or maximum passive earth pressures, cohesive soils creep continually. The movements shown would produce active or passive pressures only temporarily. With time, the movements will continue if pressures remain constant. If movement remains constant, active pressures will increase with time and passive pressures will decrease with time.

TABLE 1. Movements needed to reach minimum active or maximum passive pressure.

SUMMARY OF MEASURED DISPLACEMENTS

Typical examples of the observed horizontal and vertical displacements for all three panel types are presented in this section.

Fig. 9 shows that backfilling of a 35-inch tall L-panel produced a displacement of 0.03 feet away from the fill. This displacement appears to have been fully recovered within ten days after backfilling was complete. Similar behavior has been observed for other L-panels and some of the shorter double-T panels and may be associated with relaxation

of compaction-induced earth pressures. The backfill material was a low plasticity (CL) silty clay. Measurements of pressures on relatively rigid retaining walls backfilled with clayey soils (Carder et al. 1980; Symons et al. 1989) suggest that compaction-induced pressures can relax with time. Perkins et al. (2000) measured lateral earth pressures against a 30-foot high massive counterfort wall section backfilled with CL and CL-ML material. Their earth pressure cell data showed that significant stress relaxation occurred over the top 10 feet and that this relaxation took place within a few days after the wall was backfilled.

Assuming a horizontal translation of 0.03 feet, then the value of Δ/H for this 35-inch tall L-panel is 0.01, which is well in excess of the value considered necessary to create active earth pressure conditions.

Fig. 10 illustrates that no movements were recorded at any of the three reflector plates during placement of backfill to a level equivalent to 50 percent of the height of this 10-foot tall double-T panel. At the end of backfilling, wall displacements had reached $\Delta/H = 0.002$, assuming translation only, or about $\Delta/H = 0.003$ for rotation, based on observations of the top and bottom reflector plates.

Over the subsequent three months, some time-dependent movements, both horizontal and vertical, did occur, probably due to a combination of creep in the backfill and compression of the cohesive soil above the weathered limestone. At the end of the observation period, vertical movement at the front of the panel amounted to 0.08 feet and the overall outward tilt reached $\Delta/H = 0.008$, implying that an active earth pressure condition was reached.

The 165-inch tall double-T panel shown in Fig. 11 was located in area underlain by a considerable thickness of less competent soils above relatively unweathered limestone. As a result, the movements of panels in this area are the largest that have been observed on the entire project. At the end of backfilling, the panel had translated approximately 0.1 feet, corresponding to $\Delta/H = 0.007$, while the panel (and footing) had settled about 0.05 feet. Both horizontal and vertical movements continued to occur over a two-month period after the wall was backfilled, but at a decreasing rate. Some six months of traffic loading behind the wall between October 2003 and April 2004 resulted in little, if any, additional movements.

Although this particular double-T panel settled approximately 0.12 feet over the nine-month observation period (during which time total horizontal displacements of a similar magnitude took place) there has been no evidence of any structural damage to the panel (or top-of-wall treatment), impairment of function or any negative impact on the aesthetics of the wall.

The 200-inch tall double-T panel shown in Fig. 12 is located adjacent to a bridge abutment in an area where there are at least 20 feet of soil overlying bedrock. Backfilling at this panel location was completed in less than a week; the panel translated about 0.06 feet corresponding to $\Delta/H = 0.004$. Movements continued to occur for at least a month after backfilling, although by early September 2003, when the reflector plate targets were lost, the rate of movement had diminished.

The pattern of displacements for the triple-T panel is an excellent example of combined translation and rotation; see Fig. 13. At the end of backfilling, the value of Δ/H was about 0.004. Settlement at the front face of the panel was less than 0.05 feet. The

effect of traffic loading between October 2003 and April 2004 was to increase horizontal and vertical displacements by a maximum of 0.02 feet resulting in a final angular rotation of $\Delta/H = 0.005$.

CONCLUSIONS

The principal features of a retaining wall system, comprising precast concrete elements post-tensioned to cast-in-place footings, have been presented. The wall system was developed specifically for the Dallas High Five project as an alternative to the MSE walls included in the contract documents. The concept of the post-tensioned retaining wall was accepted on an experimental basis, subject to field data demonstrating satisfactory performance.

Twenty-nine L-shaped, double-T and triple-T panels, ranging in height from 3 feet to almost 29 feet, were monitored for horizontal and vertical displacements during backfilling; some panels were monitored for periods of up to six months under traffic loading.

In general, relatively small movements of 0.02 to 0.03 feet occurred upon backfilling L-panels. Assuming that the panels translated, the horizontal displacements are roughly equivalent to $\Delta/H = 0.004$. This is considered sufficient to create active earth pressure conditions, thereby validating an important design assumption. The measurements also indicate that some relaxation of outward movements occurred after backfilling, possibly as a result of release of compaction-induced lateral stresses.

Some of the shorter double-T panels exhibit movement responses similar to the L-panels with outward movements occurring during backfilling, followed by some inward relaxation. With increasing height panel movements become better defined, with a clear pattern of combined translation and rotation. At the end of backfilling, the overall average angular rotation of the panels reached or exceeded the threshold generally considered necessary to develop active earth pressure conditions.

The pattern of movement for the five triple-T panels is broadly similar to that for the tallest double-T panels. The maximum horizontal displacement near the top of a triple-T panel occurred at the end of backfilling and amounted to 0.1 feet. After a further six months under traffic loading, the horizontal and vertical displacements increased by only 0.02 feet.

The results from the rather modest monitoring program confirm that for all panel types and heights sufficient displacements occur during wall backfilling to create active earth pressure conditions. The movements that were measured were typically less than 0.1 feet near the top of the wall, even for the tallest panels. The wall movements that were actually measured are believed to be similar in magnitude to the post-construction displacements of conventional retaining walls, including MSE walls. The successful performance of the post-tensioned retaining wall system on this project resulted in the owner removing it from experimental status, allowing it to be considered for other projects.

REFERENCES

Carder, D. R., Murray, R. T., and Krawczyk, J. V. (1980). Earth Pressure Against an Experimental Retaining Wall Backfilled with Silty Clay. Transport & Road Research Laboratory Report LR946, Crowthorne, United Kingdom.

Clough, G. W., and Duncan, J. M. (1991). Earth Pressures. Chapter 6, Foundation Engineering Handbook, 2nd Edition, pp 223 – 235, Hsai-Yang Fang, Ed. Van Nostrand Reinhold, New York.

Landers, J. (2005). A High Five for Dallas. Civil Engineering, March 2005, pp 58 – 63 and 86.

Perkins, S. W., Miller, D. J., and Lindsey, A. (2002). Development and Relaxation of Compaction Induced Earth Pressure of a Clay Backfill at Tongue River Dam, Montana. Proc. ASCE Specialty Conf. on Performance Confirmation of Constructed Geotechnical Facilities, Geot. Spec. Publ. No. 94, pp 263 – 276.

Symons, I. F., Clayton, C. R. I., and Darley, P. (1989). Earth Pressure Against an Experimental Retaining Wall Backfilled with Heavy Clay. Transport & Road Research Laboratory Report 192, Crowthorne, United Kingdom.

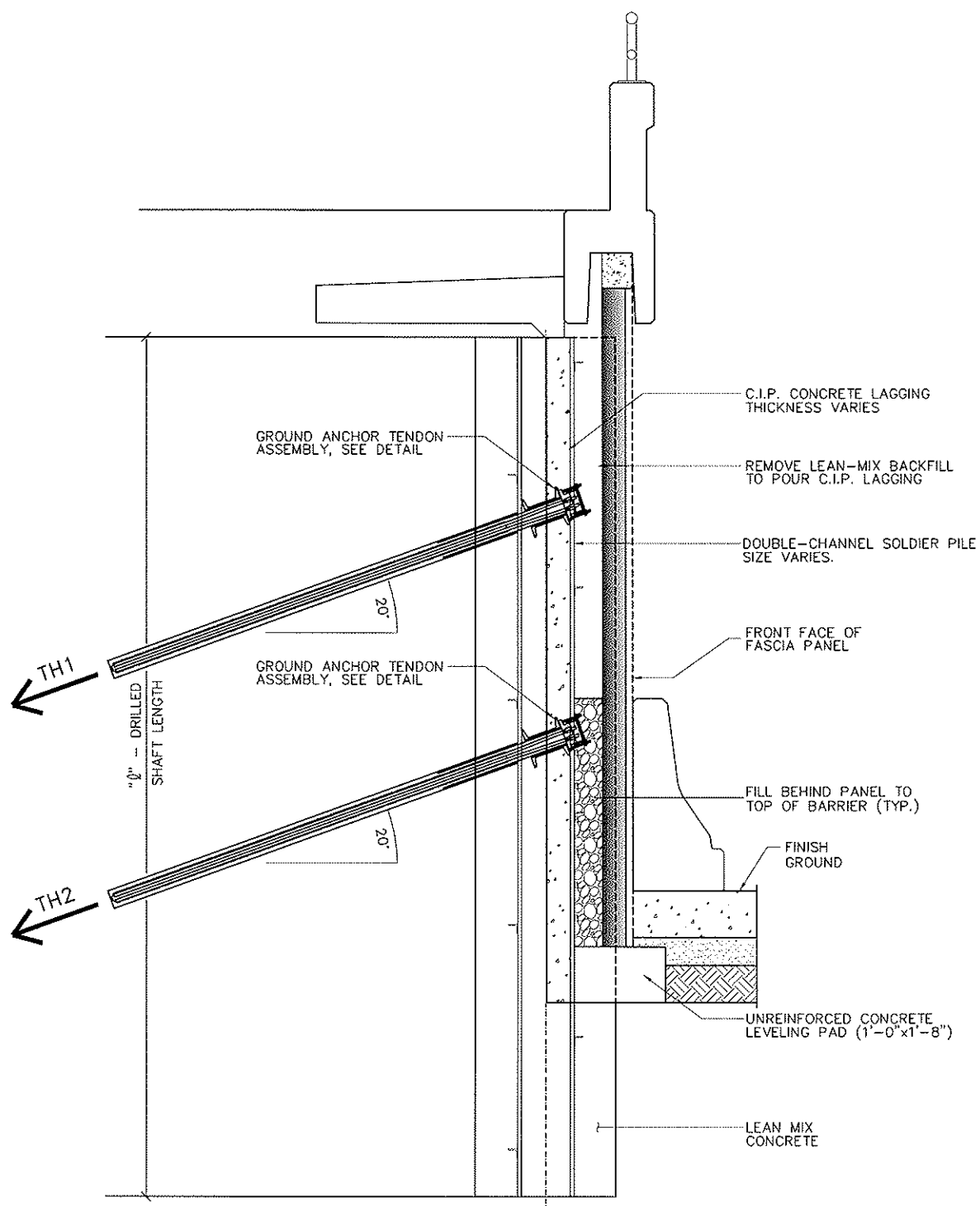


FIG.1 TYPICAL SECTION AT PERMANENT TIED-BACK
SOLDIER PILE WALL



FIG. 2 TIED-BACK SOLDIER PILE WALL SHOWING CIP CONCRETE LAGGING

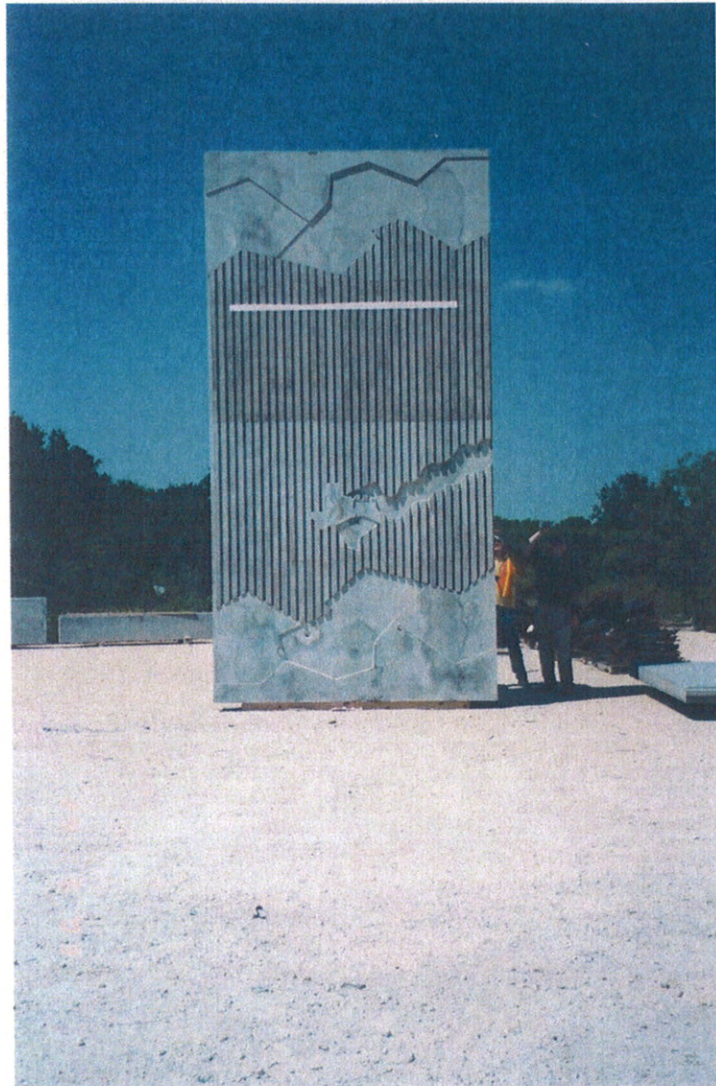


FIG. 3 FULL-HEIGHT FASCIA PANEL FOR SOLDIER PILE WALLS



FIG. 4 L-PANELS ON CIP CONCRETE FOOTINGS



FIG. 5 TYPICAL SECTION AT DOUBLE-T PANEL



FIG. 6 TYPICAL DOUBLE-T PANEL



FIG. 7 ERECTED DOUBLE-T PANELS



FIG. 8 TYPICAL TRIPLE-T PANEL

L35 / PANEL COLUMN 31

- 0 6/25/03
- 1 7/9/03 (±50% COMPLETE)
- 2 7/16/03 (100% COMPLETE)
- 3 7/17/03
- 4 7/25/03

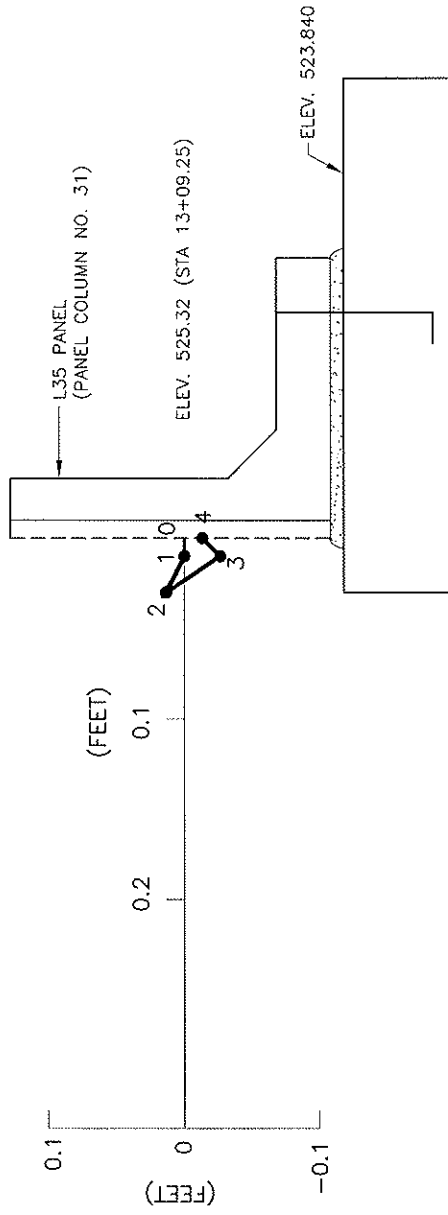


FIG. 9 OBSERVED DISPLACEMENTS OF 35-INCH HIGH L-PANEL (WALL 635C)

DT 120 / PANEL COLUMN 61

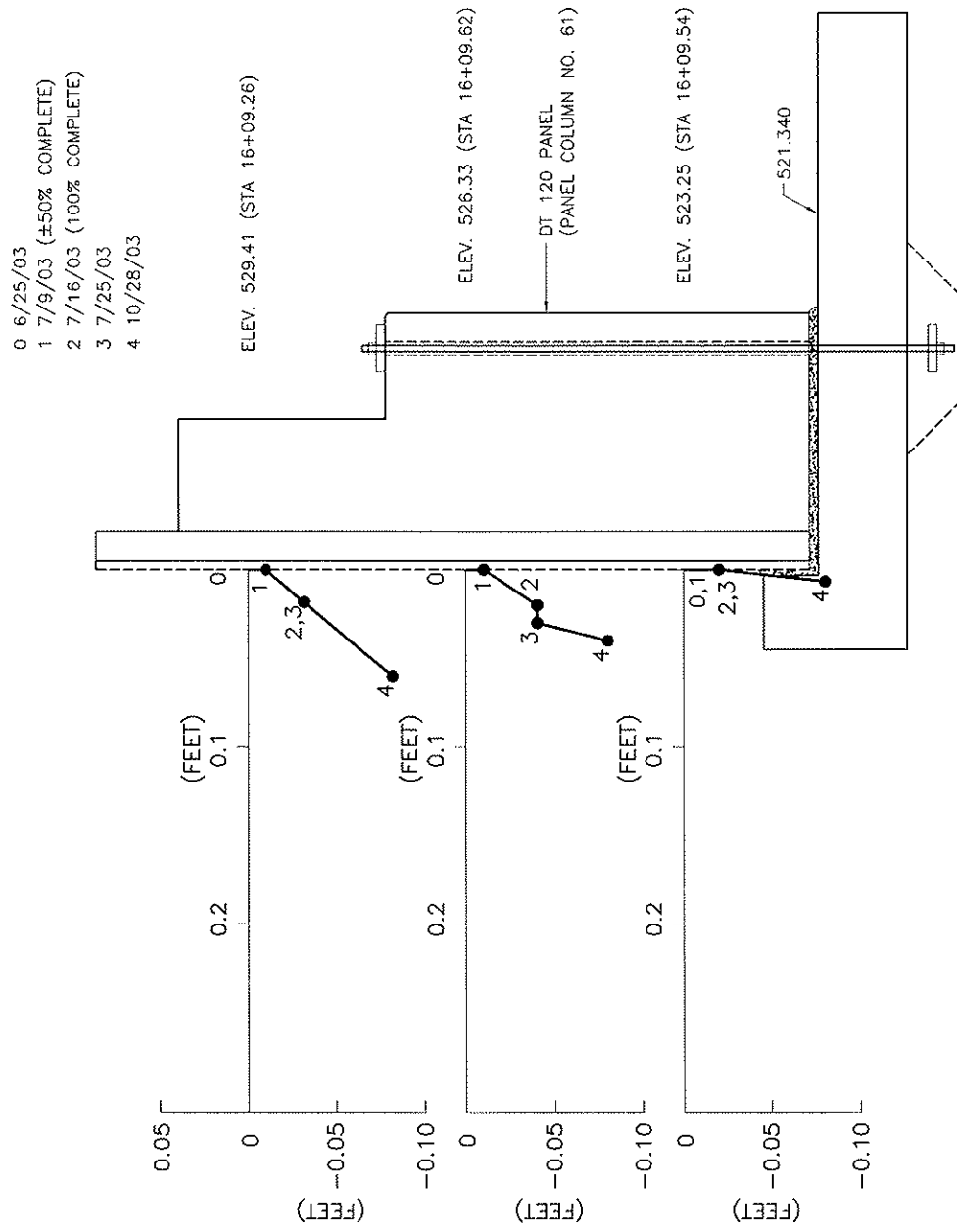


FIG. 10 OBSERVED DISPLACEMENTS OF 120-INCH HIGH DOUBLE-T PANEL (WALL 635C)

DT 165 / PANEL COLUMN 24

- 0 7/31/03
- 1 8/06/03 (±6" BACKFILL)
- 2 8/11/03(±12" BACKFILL)
- 3 8/12/03 (COMPLETE)
- 4 8/14/03
- 5 8/27/03
- 6 9/08/03
- 7 10/07/03
- 8 4/22/04

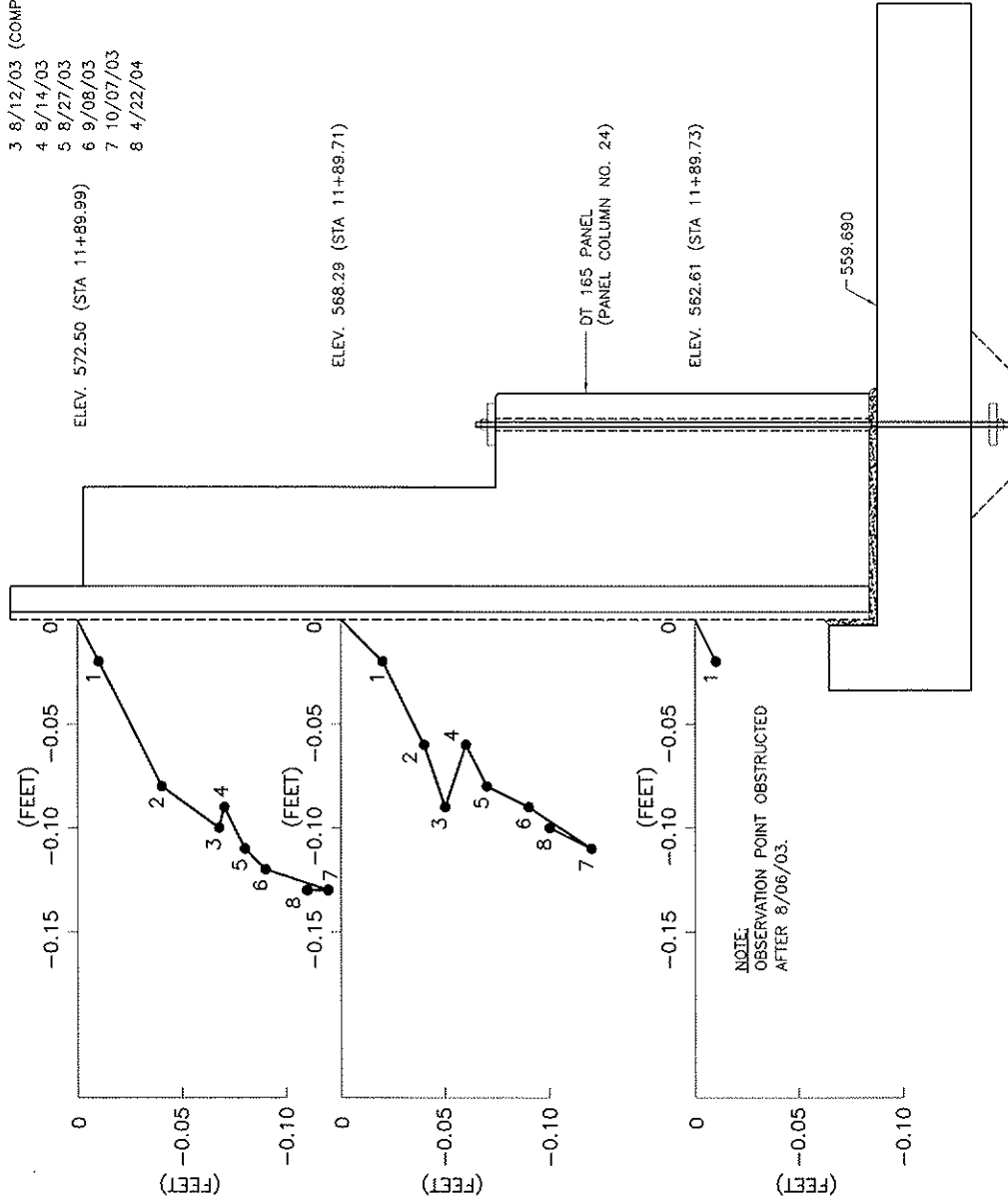


FIG. 11 OBSERVED DISPLACEMENTS OF 165-INCH HIGH DOUBLE-T PANEL (WALL 635Z)

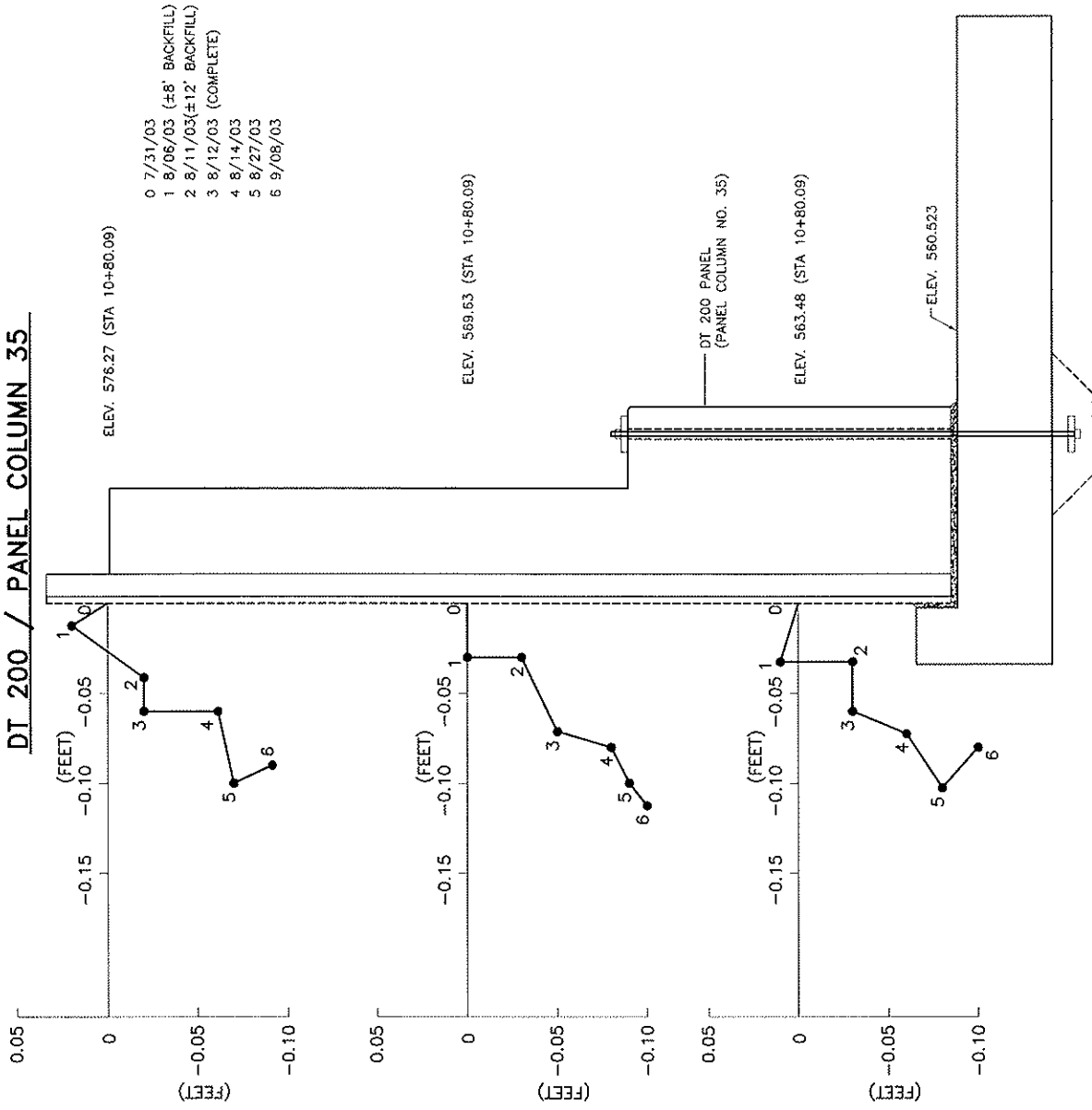


FIG. 12 OBSERVED DISPLACEMENTS OF 200-INCH HIGH DOUBLE-T PANEL (WALL 635Z)

TT 235 / PANEL COLUMN 10

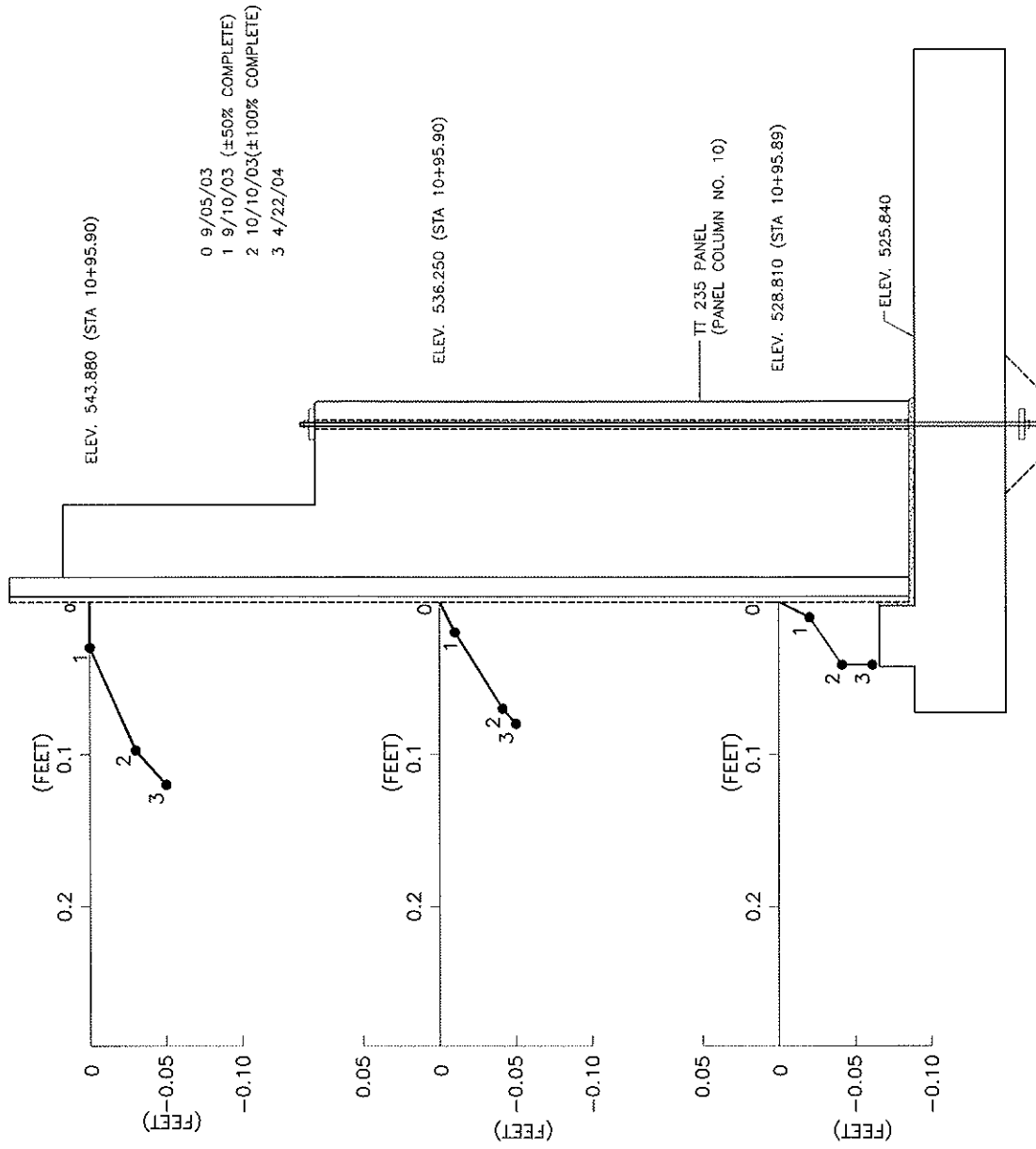


FIG. 13 OBSERVED DISPLACEMENTS OF 235-INCH HIGH TRIPLE-T PANEL (WALL 75J1)

Reticulated Micropile Walls for Landslide Stabilization – Snake River Canyon, Wyoming

John Wolosick¹, John Turner², and Robert Hasenkamp³

Abstract

Stabilization of the Blue Trail Landslide on U.S. Highway 26/89 was accomplished in 1997 by constructing three Reticulated Micropile Walls (RMP), each consisting of battered micropiles and tieback anchors attached to a reinforced concrete cap beam. The landslide and retaining walls are the subject of a Federally sponsored instrumentation and monitoring program. Objectives of the study are to determine the mechanisms of load transfer from the sliding mass to the structural elements of the wall and to evaluate the effectiveness of this type of wall in controlling ground movements. Instrumentation to monitor load transfer consists of strain gages on micropiles and load cells on tieback anchors. Ground and wall movement instrumentation includes tiltmeters, slope inclinometer casings, and ground surveying. Piezometers are being used to monitor variations in pore water pressures with time. Measured ground movements since construction of the walls have been minimal, indicating this is a highly effective technique for stabilizing highway slopes.

The Blue Trail Landslide

U.S. Highway 26/89 between Alpine Junction and Hoback Junction in northwestern Wyoming follows the scenic Snake River Canyon, which cuts through Paleozoic and Mesozoic sedimentary rocks of the Wyoming-Idaho Overthrust Belt. This physiographic region is characterized by northward trending low-angle thrust faults along which thousands of feet of marine sedimentary rocks were displaced eastward about 50-150 million years ago. The Blue Trail Landslide is located at M.P. 127, approximately 15 km east of Alpine Junction. Seasonal slide movements created ongoing maintenance and safety problems from the time the current roadway was constructed, between 1955 and 1960, until construction of the walls in 1997. The slide has been monitored since 1969, when WYDOT first conducted a site investigation of the slide area.

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Geological and geotechnical aspects of the slide are described by Dahill (1995) and Hasenkamp et al. (1996). The slide trends N70°W (35° right skew) and stretches from the edge of the Snake River, 61 m below the road, to approximately 300 m above the existing roadway. The headward zone is active approximately 6 m upslope of the roadway and the mass extends vertically beneath the highway to depths of 10 to 12 m. The portion of the slide above the roadway and active headward zone is currently stable. Figure 1 shows the slide area as viewed from the west.

The slide mass at Blue Trail consists of a heterogeneous mixture of detrital sandstone, siltstone, shale, and limestone with sand, silt, and clay. Slide debris is predominantly brecciated rock that has deteriorated to the consistency of clayey gravel, and varies in thickness from 3.5 m to 15 m with various perched water tables confined to permeable zones. Water levels appear to vary seasonally. The landslide debris is underlain by the Lower Cretaceous Bear River Formation, which consists of interbedded shales, siltstone, and thin limestone at a strike of N67°W and dip of 27°W. The eastern edge of the slide is very near the contact between the Bear River and underlying Gannet Formation. The Lower Cretaceous Gannet Group consists of siltstone, sandstone, and cross-bedded quartzite.



Figure 1. Blue Trail Landslide site, April 1997

The Blue Trail Landslide is a block-type failure with movement occurring along a relatively weak layer at the soil-rock interface. Observations indicate that movement is initiated through a combination of wet conditions in the slide and high water levels in the Snake River, both of which occur during maximum runoff in late spring or early summer. When the slide debris is at a high water content, driving forces are increased (higher weight) and resistance is decreased due to higher pore water pressure and lower effective normal stress on the failure surface. The annual spring melt leads to erosion of the toe of the slide, further reducing resistance. The combination of increased driving forces and decreased resistance is believed to be the mechanism for slide initiation.

Selection of the Stabilization System

Site Investigation and Analyses

Additional site investigations were conducted more recently by Chen-Northern (1989) and WYDOT (1995) for the purpose of selecting and designing remedial measures to control movement of the Blue Trail Slide. These investigations were used to identify the slide surface, establish depth to bedrock, and obtain samples for laboratory testing. Slope stability analyses were then conducted using the XSTABL program. Assuming that the slide was close to a state of equilibrium ($FS = 1.0$), stability analyses were used to back-calculate average effective stress strength parameters for the failure surface. Assuming there is no effective stress cohesion ($c = 0$), an average residual friction angle $\Phi = 14^\circ$ yields a global $FS = 1.0$. These strength parameters were used in subsequent analyses to assess various means of increasing stability of the slide.

Selection of Reticulated Micropile Wall

After considering a variety of approaches for stabilizing the Blue Trail Landslide (Hasenkamp et al., 1996), Wyoming DOT accepted a design-build proposal to construct three Reticulated Micropile Walls. Each wall consists of a row of battered micropiles installed through and attached to a concrete cap beam, as illustrated in Figure 2. For the Blue Trail Landslide, potential seismic loading required the addition of tieback anchors, also installed through and attached to the cap beam.

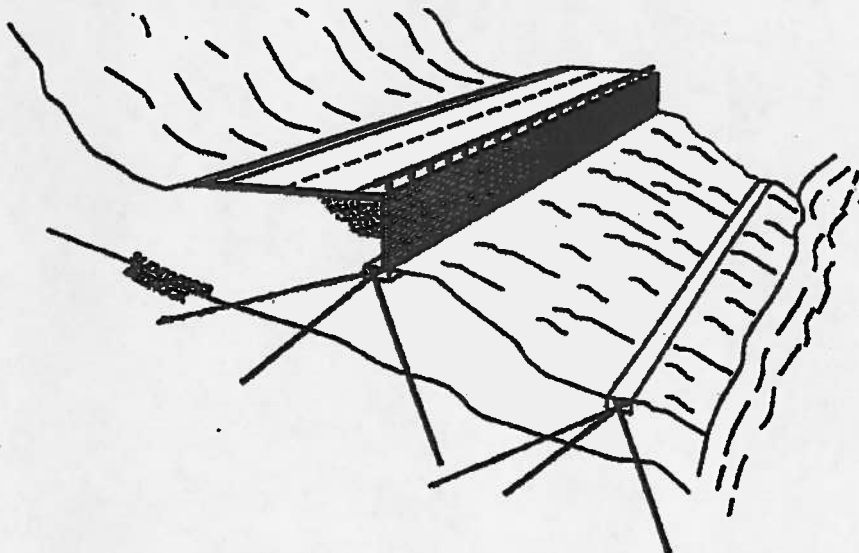


Figure 2. Conceptual drawing of Reticulated Micropile Wall with Tieback Anchors.

Design

The general steps in the design of a Reticulated Micropile Wall for slope stabilization are as follows (Pearlman et al., 1992):

1. Conduct slope stability analyses to determine the additional resisting force on the failure plane required to achieve the desired factor of safety.
2. Check the potential for structural failure of the micropiles due to loading caused by the moving soil.
3. Check the potential for failure by plastic flow of the soil around the micropiles.

For the Blue Trail Slide, slope stability analyses (Step 1) were conducted to estimate the driving and resisting forces for the existing conditions, for which it was assumed that the slope was in or close to a state of limit equilibrium ($FS = 1.0$). For the static case (i.e., no seismic loading) these analyses yield an approximate driving (and resisting) force of 5.6 MN/m (383 kips/ft). For the seismic case, assuming a pseudo-static seismic coefficient of 0.11 g, the driving force increases to approximately 7.86 MN/m (538 kips/ft). Design criteria for the global stability of the slope and wall system were established as follows: static $FS = 1.3$ and seismic $FS > 1.0$.

Resistance against sliding provided by RMP walls is based on analysis of the structural capacity of the micropiles (Step 2) and the pullout capacity of the tieback anchors. Each micropile consists of a high strength steel pipe, 114 mm diameter and 7.6 mm wall thickness, grouted into a 152 mm diameter drilled hole. Estimates of micropile capacity are based on the assumption that landslide driving forces are transferred to the micropiles as axial and lateral loads. The ultimate lateral resistance provided by a single micropile is estimated using the p-y method of analysis, in which the micropile is assumed to act as a free-headed pile at the slide surface. The program COM624 (Reese and Sullivan, 1983) was used for the lateral load analyses. Additional resistance derived from battering the piles with respect to the direction of movement is then estimated based on an elasticity solution given by Poulos and Davis (1980). The amount of axial load transferred to the micropiles is limited by the ultimate bond that can develop between the pile perimeter and the soil in the sliding mass. Resistance to sliding derived from the bending and axial capacities of micropiles which are battered alternately upslope and downslope are then combined and micropile spacing is selected to provide the desired additional resistance to meet the minimum factor of safety.

The length of the active zone of sliding at Blue Trail is too great to permit a single location of support to control the several possible modes of failure. A single wall at the center of the slide would not necessarily prevent further settlement of the roadway, while a wall directly beneath the roadway would not control slope failures downslope from the wall. The most efficient solution was found to be the use of two RMP walls, the first located 3m below the roadway ("upper") and the second located approximately midway between the road and the toe of the slide ("lower"). A third RMP wall ("middle") was added at the western edge of the project, 12 m below the roadway, to increase the factor of safety of the slope where it was being surcharged by the MSE wall used for roadway widening. Figure 3 is a plan view of the site showing the final wall locations.

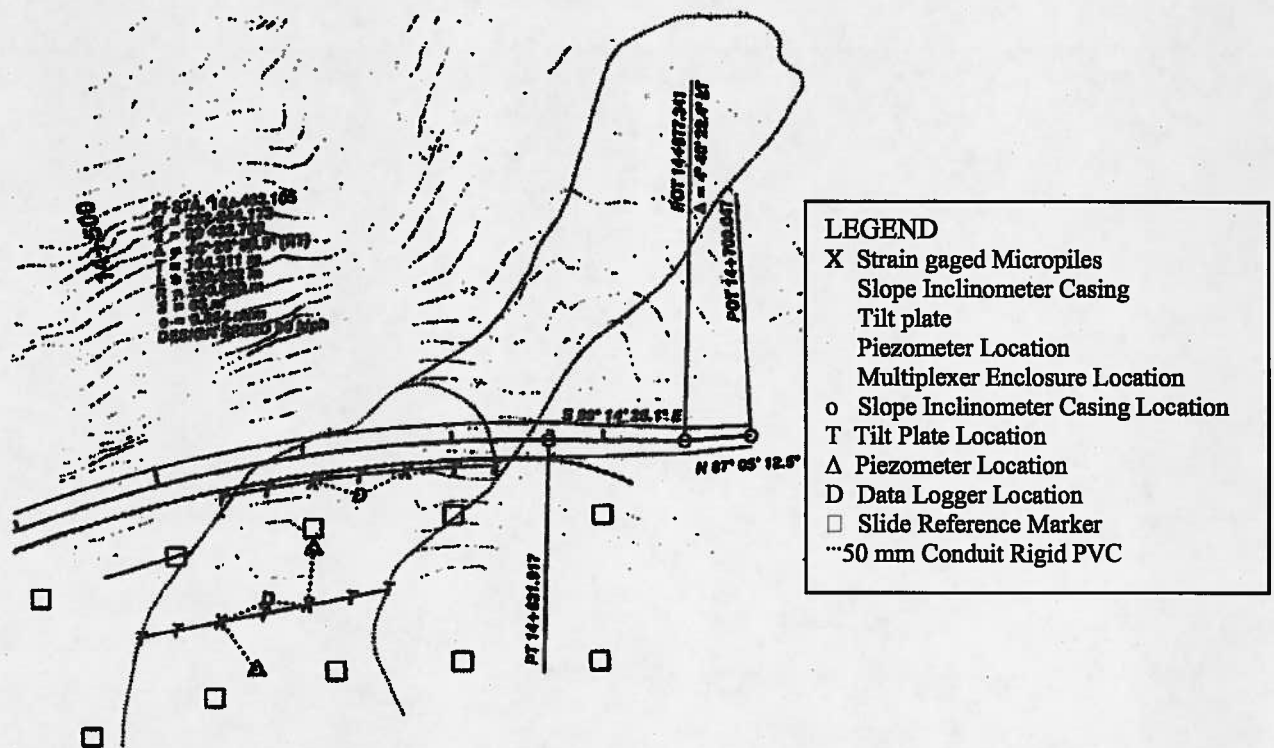


Figure 3. Plan of site showing slide, walls, and instrumentation locations.

Total resistance to sliding provided by the micropiles alone does not satisfy the minimum required factor of safety against sliding for the seismic loading case. Tieback anchors provide the necessary additional resistance and also act to reduce post-construction ground movements to minimal levels. Tieback anchors in the upper wall were designed for a capacity of 1.56 MN (350 kips) at a spacing of 7.6 m (25 ft), providing 205 kN/m (14 kips/ft). The anchors in the lower wall have a design capacity of 2.82 MN (630 kips) at a spacing of 3.8 m (12.5 ft), providing 740 kN/m (50 kips/ft). Tieback anchors are battered 30° from horizontal. In the slope stability analyses, effects of tieback loads on both normal force acting on the failure plane and the tangential component of force are taken into account.

Final design specifications for the two walls (upper and lower) are summarized in Table 1. Total resistance provided by the two walls yields a static FS of approximately 1.6 and a seismic FS of approximately 1.1. The three walls combined included 478 micropiles and 40 high-capacity tieback anchors. Figure 4 shows a typical section.

Table 1. Summary of Design Parameters for RMP Walls

Wall	Length m(ft)	Micropile Spacing m(ft)	Micropile Resistance kN/m(k/ft)	Tieback Capacities MN(kip)	Tieback Spacing m(ft)
Upper	94 (308)	0.51 (1.67)	742.5 (51)	1.56 (350)	7.6 (25)
Lower	88 (289)	0.38 (1.25)	837.4 (57)	2.82 (630)	3.8 (12.5)

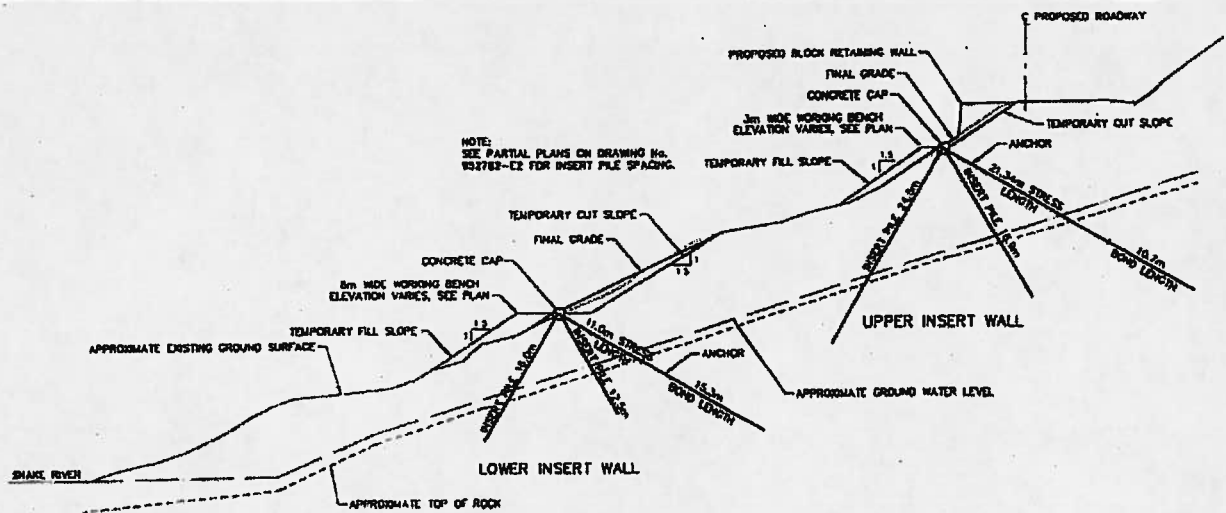


Figure 4. Typical section of Reticulated Micropile Walls, Blue Trail Landslide.

Construction

The walls at Blue Trail Landslide were constructed by Hayward Baker Inc. Construction began in April 1997 with excavation of access roads and working benches for the cap walls (Figure 1). Figure 5 shows the completed lower cap, which is a heavily reinforced concrete beam 1.2 m x 1.2 m in section. Blockouts consisting of 152 mm diameter corrugated plastic pipe were installed at 30 degree angles (alternating between upslope and downslope) to provide guides through which micropiles were drilled and grouted. Figure 6 shows the drilling operation, in which a 152 mm diameter hole is excavated using a crane-suspended, hydraulic powered rotary drilling rig. Drilling techniques included tri-cone roller bits and/or down-hole-hammer drilling tools. Typically, compressed air was used to flush drill cuttings. Drilling was often difficult where boulder fill and talus materials were encountered. Micropiles were installed by using the high strength pipe piles as drill tooling. The piles were then withdrawn, the drill bit removed, and the pile placed to the bottom of the hole. Neat cement grout, consisting of portland cement and water (w/c ratio of 0.45) was injected from the bottom of the hole, filling the drill hole and the inside of the pipe. The final stage of wall construction was

installation of the tieback anchors. Drilling and setting of steel casings was first done through the talus and overburden down to bedrock. Rock sockets, 12-17 m in length were then hammered through the casing. The strand anchors were placed down the hole and tremie grouted using neat portland cement grout. 10-strand anchors were used in the upper and middle walls, while those in the lower wall have 18 strands. Figure 7 shows the lower cap wall and tieback anchor strands. The roadway above the upper wall is supported on a MSE wall consisting of geogrid-reinforced backfill with Keystone block facing.

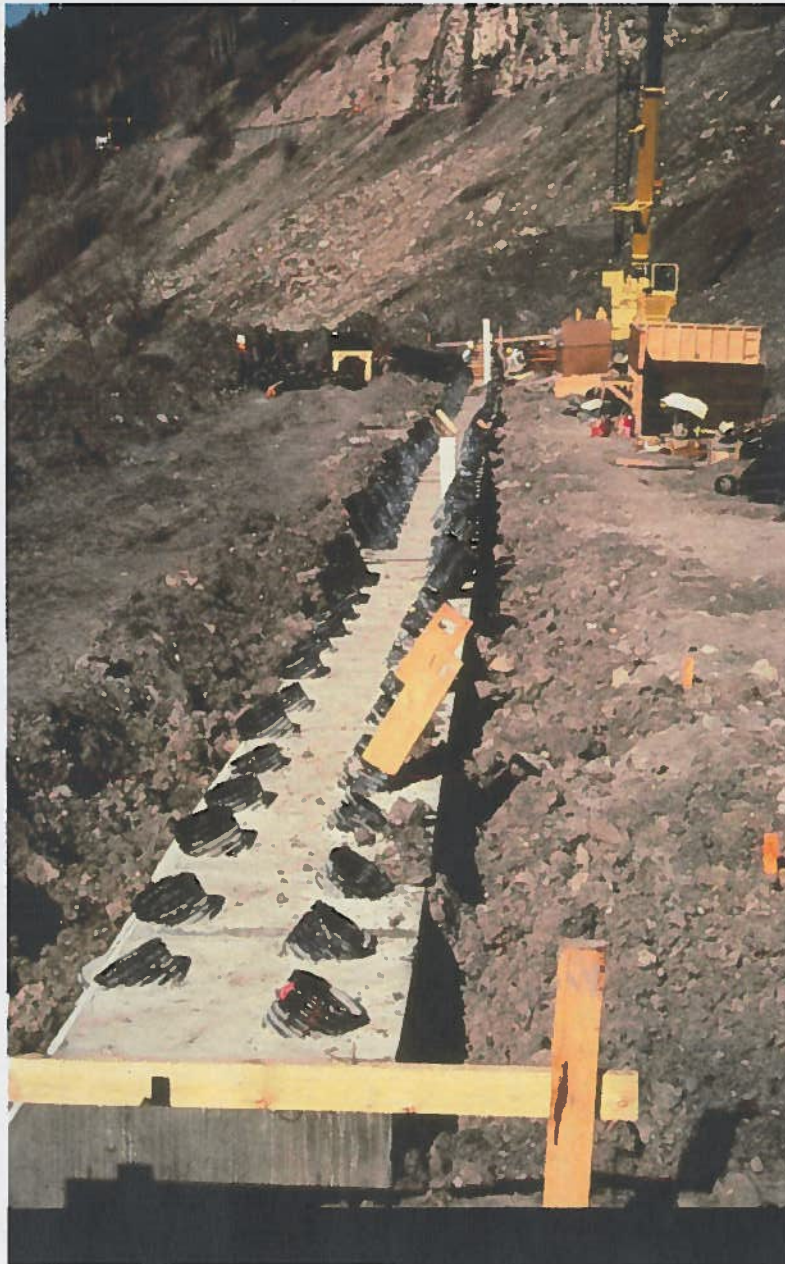


Figure 5. Lower cap wall before installation of micropiles and tieback anchors.



Figure 6. Drilling machine for micropile installation.

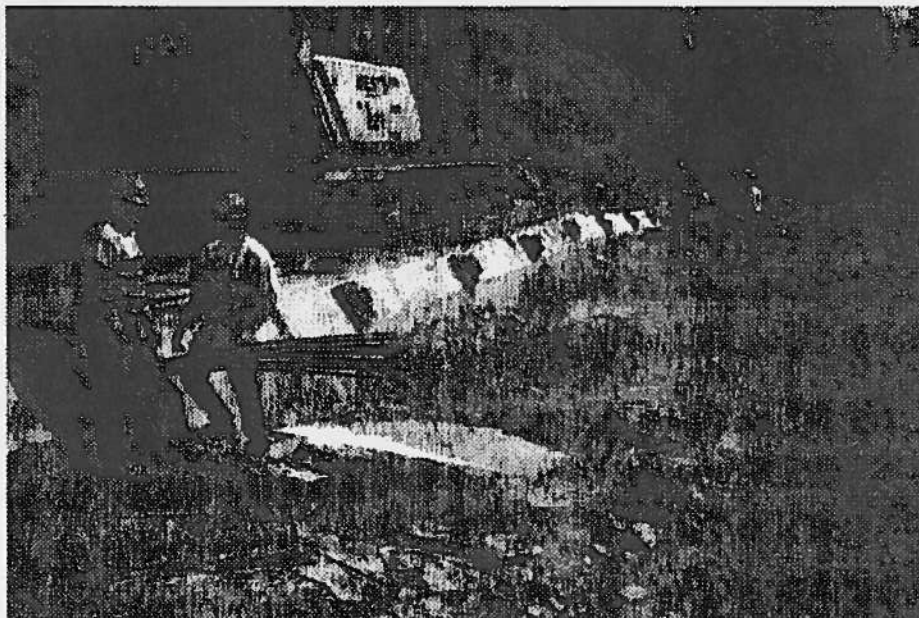


Figure 7. Lower wall cap and 630 kip tieback anchors.

Instrumentation and Monitoring Program

The RMP walls at Blue Trail are instrumented as part of a monitoring program being conducted by Wyoming DOT and the University of Wyoming. The principal objectives are to assess the effectiveness of this type of landslide stabilization method and to investigate the mechanisms of load transfer to the structural components (micropiles and tieback anchors) of the walls. Instrumentation includes strain gages on micropiles and load cells on tieback anchors for the purpose of monitoring loads, and inclinometers and tiltmeters for monitoring wall and ground movements. In addition, several piezometers were installed to monitor pore water pressures close to the assumed slide plane. Figure 3 shows the instrumentation locations. Four micropiles in each wall (upper and lower) are instrumented with vibrating wire strain gages. Strain gage depths were selected in an attempt to have two monitoring points above the assumed slide plane and two monitoring points below as shown in Figure 8. The four instrumented micropiles in each wall include two adjacent piles (one upslope and one downslope) at distances of approximately one-third the wall length from the east and west ends of the walls, respectively (Figure 3). At each monitoring depth, three strain gages were attached at a circumferential spacing of 120 degrees (Figure 8), providing strain readings needed to determine principal stresses and their orientation, which are then used to calculate axial load and bending moment. Permanent load cells were installed to monitor tieback anchor forces in two anchors of each wall (four instrumented anchors total). The instrumented tiebacks are also located at distances of approximately one-third of the wall length from the east and west ends of the two walls (Figure 3).

Inclinometer casings are placed at various locations, including several which extend through the cap beams as well as locations above and below the walls (Figures 3,9). Casing deformations have been monitored on a regular basis since the start of construction through the present (June 1998). Several of the casings underwent excessive shear deformations and were replaced during the construction period, before the wall provided sufficient resistance to limit further slope movements. Tilt plates which can be monitored with a tiltmeter are located on the top surface of the cap walls at fourteen locations (seven on each wall) to monitor wall rotations. Readings were made beginning in late July 1997, before installation of the tieback anchors, and during September 1997, following installation and prestressing of the tieback anchors.

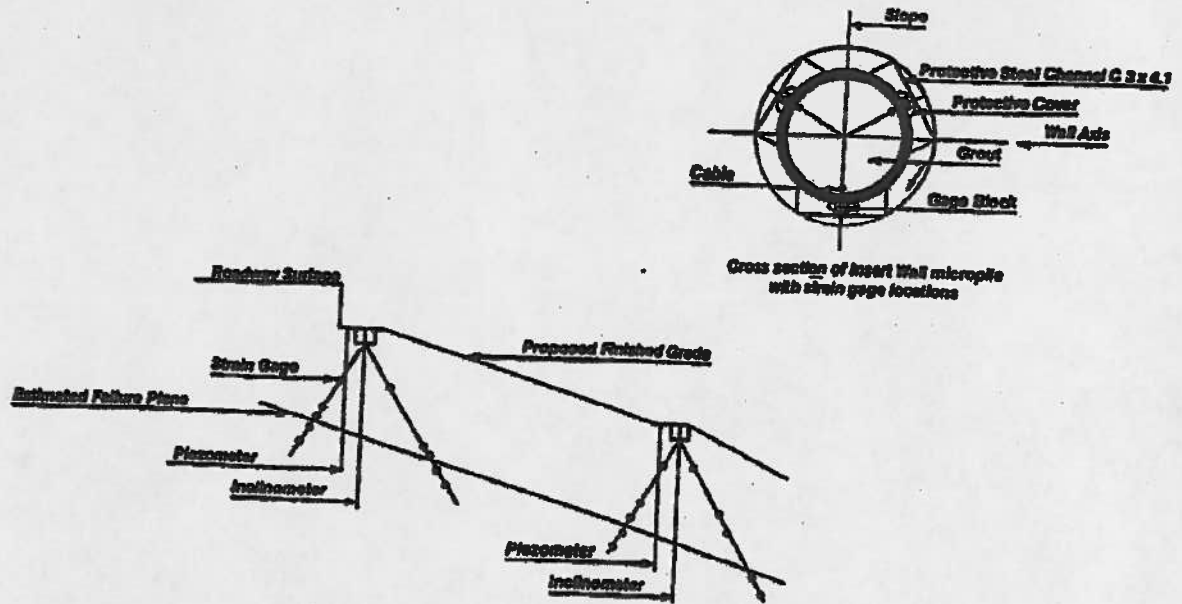


Figure 8. Details of strain-gaged micropiles



Figure 9. Taking inclinometer readings, lower wall cap.

Results and Interpretation

Wall and Ground Movements

The slide was active during the initial stages of wall construction. Figure 10 shows inclinometer readings at inclinometer No. 3, located in the center of the lower wall. The casing was installed on May 27, immediately after which movements occurred in the downslope direction. As the micropiles were being installed, the rate of movement decreased. As tieback anchors were installed, the direction of movement in the upper 3 to 4 meters below the wall began to reverse. When the tieback anchor closest to this inclinometer casing was installed and prestressed (Aug 28), the cap wall and inclinometer casing moved in the upslope direction. This behavior is typical of inclinometer readings in the lower wall and is a direct result of the high prestress loads applied to the tieback anchors, which were 2.82 MN (630 kips). Inclinometer readings in the upper wall also showed decreasing ground movements as the micropiles were installed, but did not show a reversal in direction with installation of tieback anchors. Prestress loads in the tieback anchors of the upper wall were 1.56 MN (350 kips). The force per unit length of wall for the tiebacks in the upper wall is 205 kN/m, compared to 740 kN/m in the lower wall.

Wall and ground movements since construction of the walls have been minimal, based on tiltmeter, slope inclinometer, and ground surveying measurements, indicating that the RMP walls have been highly effective in controlling the Blue Trail Landslide.

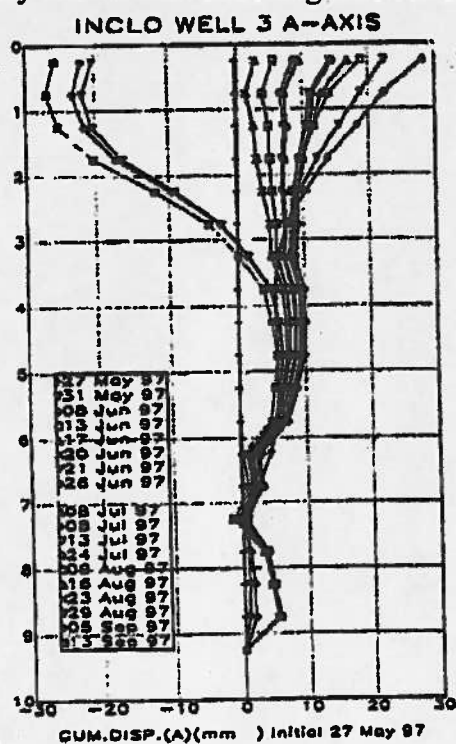


Figure 10. Inclinometer readings, center of Lower wall cap.

Load Transfer

Figure 11 illustrates axial load transfer in micropiles of the lower wall. Micropile 326 is battered in the downslope direction. After micropile installation, but before installation of the tieback anchor closest to this micropile (9/7/97 reading), axial load transfer in the upper portion of the micropile is minimal, while axial load transfer close to the assumed slide plane shows the pile to be in compression. This is the expected mode of axial load transfer in piles with a downslope batter resisting landslide driving forces. However, when the tieback anchor closest to this micropile was installed and prestressed (9/15/97 reading), the pile went into tension. Application of prestress load to the anchor applies an eccentric load to the head of the micropile at the cap wall. Similarly, the adjacent micropile which is battered upslope (Micropile 327) goes into compression. Axial load in Micropile 326 (downslope batter) has not changed significantly since installation of the tieback anchors, while the magnitude of compression loading in Micropile 327 has increased slightly.

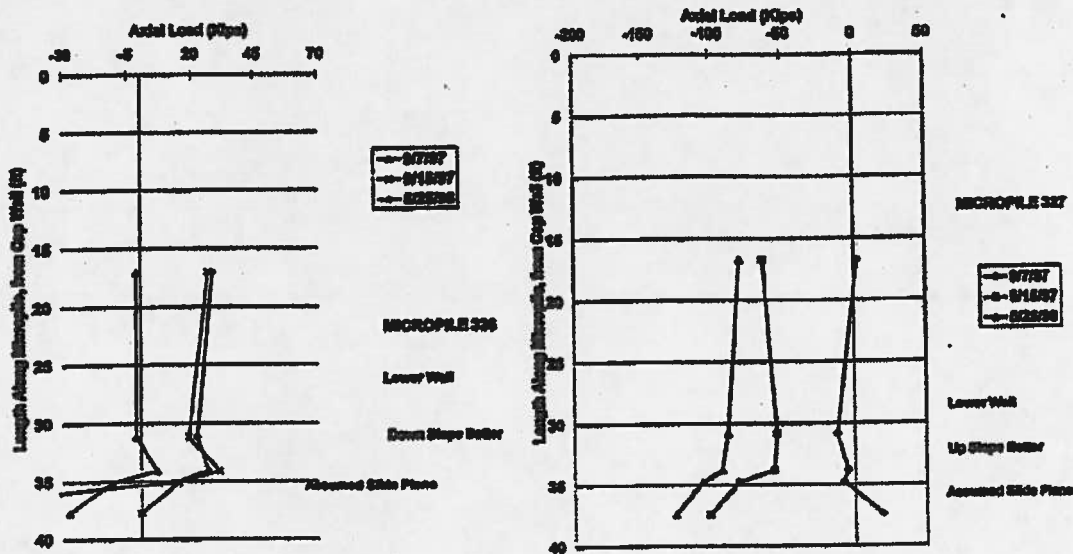


Figure 11. Typical axial load transfer for micropiles at lower wall.

Discussion

Reticulated Micropile Walls consisting of closely spaced battered micropiles and tieback anchors attached to concrete cap beams were used successfully to control ground movements at the Blue Trail Landslide. Monitoring of wall and ground movements show that this is a highly effective method for controlling ground deformations which had been causing roadway damage for thirty years.

Acknowledgments

The authors wish to thank the Wyoming Department of Transportation for their support of the instrumentation and monitoring of the walls at Blue Trail Landslide. Mike Hager and Jim Dahill of the WYDOT Geology Program provided invaluable assistance to the authors with the contribution of their knowledge and expertise. Brad Campbell of D'Appolonia was lead designer of the RMP walls, and John King of Seismic Imaging installed the instrumentation.

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INTERNALLY REINFORCED SOIL EMBANKMENT AND SLOPE DESIGN FOR ODOT PROJECTS IN WEAK SHALES OF SOUTHERN OHIO

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ABSTRACT

The LAW-607-0.25 and LAW-7-7.25 projects are part of an approximate 9-mile bypass of existing SR 7 around the Ohio River towns of Proctorville and Chesapeake, Ohio, located about 35 miles southeast of Portsmouth, Ohio (see Vicinity Map, Figure 1). The bypass is designed to alleviate local traffic congestion associated with east-west through-travel on SR 7, as well as provide improved access to the East End Bridge over the Ohio River into neighboring Huntington, West Virginia. The bypass traverses a steeply changing topography coupled with challenging subsurface conditions, comprised of weak alluvial deposits derived from Ohio River deposition, weak geologic conditions associated with the Conemaugh shale formation and weak colluvial deposits primarily derived from the weathering of the Conemaugh formation shales. Comparisons of proposed road grade and existing topography indicate the need for cut excavations and embankment fill heights as great as 140 feet.

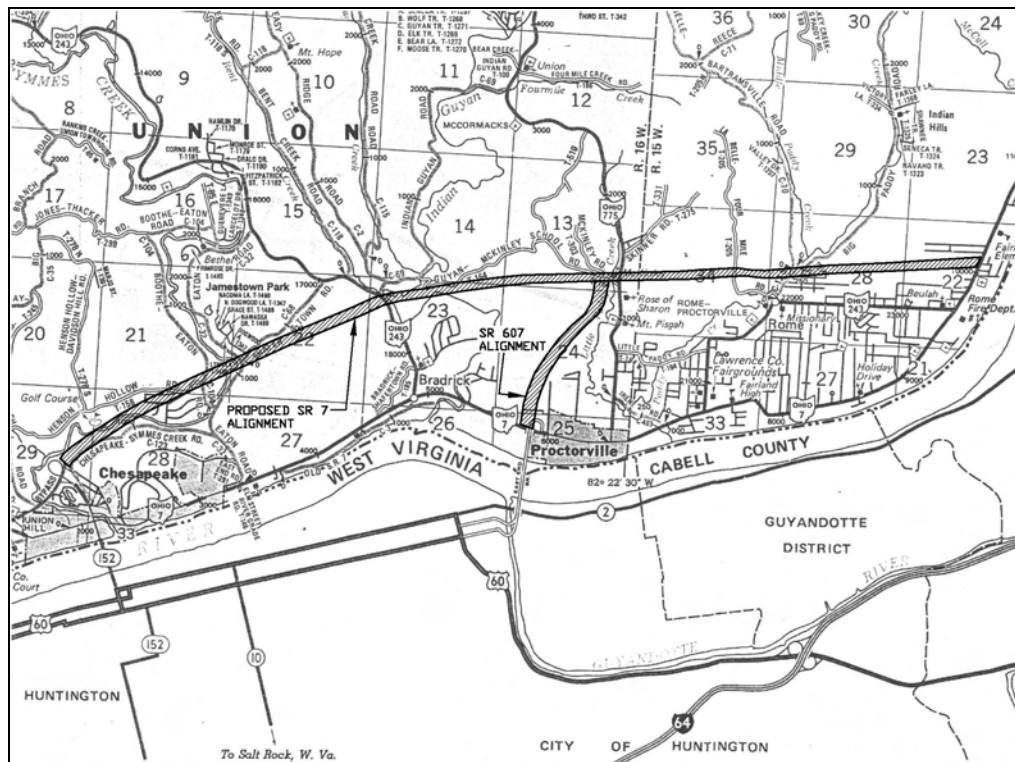


FIGURE 1: Vicinity Map

To the greatest extent possible, incorporation of the inherently weak on-site soils and rock materials was critical to maintaining a reasonable balance between earthwork cut and fill requirements and assisting project costing. However, the weak on-site materials posed a challenge to the design of stable embankments (free-standing as well as bridge approaches) and cut slope geometry within the traditional ODOT guidelines for such structures while maintaining minimal impact outside the desired roadway project right-of-way limits. The incorporation of internal geogrid reinforcement into selected embankments and sidehill fills proved essential in designing and building these projects with the on-site and locally available soils and rock, including the weak Conemaugh shale and colluvium. A description of the subsurface conditions and soil strength parameters used on this project will be presented along with a discussion of the design process, including the justification of geogrid reinforcement as an essential component of global and internal stability of the embankments and cut slopes. Selected designs and important design details will be discussed.

GEOLOGY AND PROBLEM DESCRIPTION

The project area is located in a portion of Ohio which has not been glaciated. The topography along the project corridor is characterized by upland hills, which effectively indicate the northern boundary of the ancient Ohio River valley, and the floodplain areas bordering the existing Ohio River. The hilly portions of the corridor are broken by several smaller floodplains created within valleys cut by the smaller watercourses.

Bedrock in the eastern portion of Lawrence County consists of the lower portion of the Monongahela Formation overlying Conemaugh Formation shale, siltstone, coal, limestone and sandstone of the Pennsylvanian rock system. Bedrock is generally shallow on the hills along the corridor, and numerous outcrops are present in the hills. Where these hills are cut by watercourses, the bedrock may drop relatively quickly and the depth to bedrock may vary significantly across the width of these cut valleys. Near and below the floodplain adjacent to the Ohio River, bedrock is known to be located at depths as great as 60 to 90 feet.

Significant portions of the Monongahela and Conemaugh Formation bedrock are known to be slide-susceptible. Many of the slide-prone materials within these bedrock formations consist of weak shales, underclays and claystones that are sometimes referred to as “red-beds.” These types of slide-susceptible strata may become unstable or be easily eroded, particularly when exposed in cut slopes or when subjected to moisture and temperature variation.

At the valley locations, geologic literature indicates that soils were likely deposited in a lake environment created when the Teays River and its tributaries were dammed by advancing glaciers. Below these lacustrine materials and often immediately above the bedrock, relatively clean sand deposits were encountered. Bedrock, generally consisting of shale and siltstone, was encountered at depths ranging from 20 to 50 feet below the stream levels. At some locations, soils derived from the weathering of the bedrock were encountered immediately above the bedrock proper and, as a result, the actual elevation of the bedrock is poorly defined.

In general, the stratigraphy encountered in the borings located in the cut slope areas correlated well with the information obtained from geological references. Only minimal soil cover was encountered in these cut areas, and this cover consisted of soils derived from weathering of the underlying parent bedrock. The predominant bedrock types encountered were shale, siltstone and sandstone. Significantly smaller thicknesses of limestone and coal were also encountered. As anticipated, a large portion of the shale was reddish-brown or reddish-gray in color and in a very-soft to soft condition. Much of the red shale was observed to be partly similar to hard soil and contained numerous fractures, fissures and slickensides. The sandstone was generally medium-hard and contained many to few fractures. Many of the sandstone layers also appeared to contain water.

The slickensides noted within the reddish-gray shale strata are evidence of past geologic movements. Specifically, the slickensides represent surfaces over which considerable displacement has already occurred and the shear strength of the shale along these surfaces is generally considered to be at or near residual strength levels, which may be significantly lower than the peak shear strength. Consequently, it was imperative that cut slopes designed within sections of the roadway which expose shale take this condition into account.

It was anticipated that the material used for the embankment fills would be composed of the soil and rock removed from the cut sections of the roadway. Based on boring information, this material consisted primarily of shale, sandstone and siltstone. In order to use these materials as fill, rock fragments (excluding shales) had to be broken down to a maximum particle size dictated by 2002 ODOT Item 203.06c. The shale, due to its generally weak structure, required reduction to a maximum particle size of 3 inches prior to further processing for use as fill material. Beyond simply breaking down the rock, the shale also required significant moisture conditioning. Moisture conditioning involves the repeated application of water to the shale stockpile, followed by plowing or discing to thoroughly incorporate the water throughout the material. Fill material derived from the relatively intact and unweathered shale strata typically possesses an in-situ moisture content considerably less than the optimum moisture content required for proper compaction. Accordingly, significant quantities of water and considerable compactive effort will likely be required to properly break down and moisture condition the shale for use in embankment construction.

To assure the stability of the constructed embankments using the shale and sandstone fill, it was preliminarily recommended that embankment slopes, no steeper than 3:1 (H:V) be incorporated into the design. Embankment stability is both a function of the fill material as well as the foundation soil properties. As such, the 3:1 side slope recommendation was prefaced upon the existence of adequate foundation conditions.

OVERVIEW OF DESIGN

Modeling

Using proposed SR 7 embankment cross-section information (provided by R.D. Zande & Associates, Inc.) as well as subsurface data obtained during the exploration program, schematic subsurface sections were developed and used as a basis for performing slope stability analyses at selected locations subject to, either, rapid drawdown conditions or requiring steepened slopes in order to minimize impact outside the desired roadway project right-of-way limits. These sections were analyzed utilizing an embankment/sidehill fill slope face geometry of 1.5:1 (H:V), 1.7:1, or 2:1. Based on available floodplain information, the 100-year flood event rapid drawdown condition was estimated to occur when the Ohio River rises to approximate Elevation 555 and then rapidly recedes.

Method of Analysis

Slope stability analyses were performed on selected embankment/sidehill fill geometries utilizing both non-internally-reinforced side slopes and slopes incorporating geogrid-reinforced soil. The STABL6H and GSTABL7 computer programs, which were developed by Purdue University, were used for these analyses to assist with identifying potential failure surfaces. A minimum target factor of safety against both deep-seated and internal slope failures of 1.3 (short term and long term – ODOT and FHWA requirement) was utilized for these analyses. BBC&M performed slope stability analyses on the desired embankment/sidehill fill geometry incorporating geogrid-reinforced soil parameters in accordance with FHWA Guidelines (Elias, Christopher and Berg, 2001).

Perceived advantages for using reinforced slopes are:

- 1) ability to construct the embankment with slope faces that are steeper than could otherwise be safely constructed with the same fill material in an unreinforced condition. This would provide the economic advantage of material and right-of-way savings, as well as reducing the potential environmental impact to the adjacent area.
- 2) provide improved lateral resistance during compaction at the edges of the compacted fill slopes, thereby decreasing the tendency for surface sloughing.
- 3) reduce local differential settlements to magnitudes that are less detrimental to roadway pavements.

Strength Parameters Used for Design

One of the most critical variables controlling slope stability for the Chesapeake Bypass was the shear strength of the bedrock and soil and, in particular, the shear strength of the very-soft to soft light-gray and red shales associated with the Conemaugh and Monongahela rock formations. A large portion of the shale encountered within the borings was reddish-brown or reddish-gray in color, very-soft to soft in hardness and frequently contained slickensides. The overall rock mass shear strength is often

dictated by the presence of slickensides, and consequently, standard direct shear laboratory tests often do not accurately predict long term shale strength (Wu, 1981). As such, a large number of past studies (including BBC&M projects) were reviewed and the suggested long term strength values were compiled (Bishop et. al., 1986; D'Appolonia et. al., 1972; Geiger and Graham, 1993; Hall, 1974; Hamel and Flint, 1967; Williams, 1982; and Wu et. al., 1987).

To further refine the strength values, back-analyses were performed for several of the cut slope cross-sections associated with the project. These analyses were executed with the presumption that the existing factor of safety was near unity, typical of such a geologic environment. Based on past studies, BBC&M project experience and the back-analyses, the shear strength parameters listed in Tables 1 and 2 were developed.

TABLE 1: Short Term Strength Parameters

Soil Parameter	New Compacted Fill	Very-Soft to Medium-Stiff Silty Clay and Clayey Silt - Colluvium	Stiff to Hard Silty Clay and Clayey Silt - Colluvium	Very-Loose to Loose Sand and Silt	Medium-Dense to Very-Dense Sand and Silt	Very-Soft to Soft Shale
Total Unit Wt. (pcf)	140	125-132	132	130-135	120-140	147
Saturated Unit Wt. (pcf)	143	130-135	135	132-140	125-144	150
Cohesion (psf)	50	50-750	1500-2000	0	0	3000
Friction Angle (deg)	26	0	0	26-30	26-32	0

TABLE 2: “Normal” Long Term Strength Parameters

Soil Parameter	New Compacted Fill	Very-Soft to Medium-Stiff Silty Clay and Clayey Silt - <i>Colluvium</i>	Stiff to Hard Silty Clay and Clayey Silt - <i>Colluvium</i>	Very-Loose to Loose Sand and Silt	Medium-Dense to Very-Dense Sand and Silt	Very-Soft to Soft Shale
Total Unit Wt. (pcf)	140	125-132	132	130-135	120-140	147
Saturated Unit Wt. (pcf)	143	130-135	135	132-140	125-144	150
Cohesion (psf)	50	0	0	0	0	0
Friction Angle (deg)	26-28	13-15	17-18	28-32	28-34	15-18

Primary Reinforcement (Type, Strength, Length)

- a. 1.5:1 (H:V) Side-Sloped Embankment From SR 7 Sta. 451+00 to Sta. 453+00

The portion of the proposed SR 7 mainline roadway embankment from Sta. 451+00 to Sta. 453+00, required a 1.5:1 (H:V) slope on the southern face of the embankment in order to prevent encroachment onto existing TR 205 (La Belle-Four Mile Road). The desired embankment configuration within this section generally required fill ranging in height from approximately 25 to 55 feet. The 100-year flood event rapid drawdown condition was not considered a factor at this embankment segment location.

The results of the slope stability analyses performed on the model representing the proposed SR 7 cross-section at Station 451+50 indicated that the long term factor of safety with respect to shear strength would be unacceptably low for 1.5:1 (H:V) *unreinforced* southern-facing side slopes (see Table 3). However, factors of safety exceed minimum acceptable values for both short term and long term cases for 1.5:1 (H:V) *geogrid-reinforced* southern-facing side slopes (see Table 4). For the cross-section representing the proposed embankment at Station 451+50, the plots of the ten most critical failure surfaces for the *unreinforced* and *geogrid-reinforced* cases are shown in Figures 2 through 4. In accordance with FHWA Publication No. FHWA-NHI-00-043, the embankment section

was divided into three zones to determine the optimum primary reinforcement spacing and strength. The geogrid-reinforced slope stability analyses at Station 451+50 were conducted using ten primary layers of moderate strength uniaxial geogrid (Long-Term Design Strength, $T_A = 3,300$ lb/ft) at a vertical spacing of 3 feet between El. 592 and El. 613, and a spacing of 6 feet between El. 613 and El. 625. Primary reinforcement lengths at this location ranged from 55 to 65 feet. The assigned geogrid design strengths (T_A) were provided to BBC&M by the ODOT Office of Geotechnical Engineering and were consistent with past project applications and strengths of geogrids manufactured by a variety of suppliers. The minimum factors of safety determined from the slope stability analyses for the embankment cross-section at Station 451+50 are summarized in Tables 3 and 4 of this report. It must be noted that these embankment slope stability analyses do not account for seismic effects.

TABLE 3: 1.5:1 (H:V) *Unreinforced* Southern-Facing Embankment Side Slope Minimum Factors of Safety

Embankment Location	<i>Unreinforced</i> Southern-Facing Embankment Side Slope	FACTOR OF SAFETY		Meets Both Target Ranges?
		Short Term Condition	Long Term Condition	
Sta. 451+50 (SR 7)	1.5:1 (H:V)	1.74	0.92	No

TABLE 4: 1.5:1 (H:V) *Reinforced* Southern-Facing Embankment Side Slope Minimum Long Term Factor of Safety

Embankment Location	<i>Reinforced</i> Southern-Facing Embankment Side Slope	FACTOR OF SAFETY	Meets The Target Range?
		Long Term Condition	
Sta. 451+50 (SR 7)	1.5:1 (H:V)	1.30	Yes

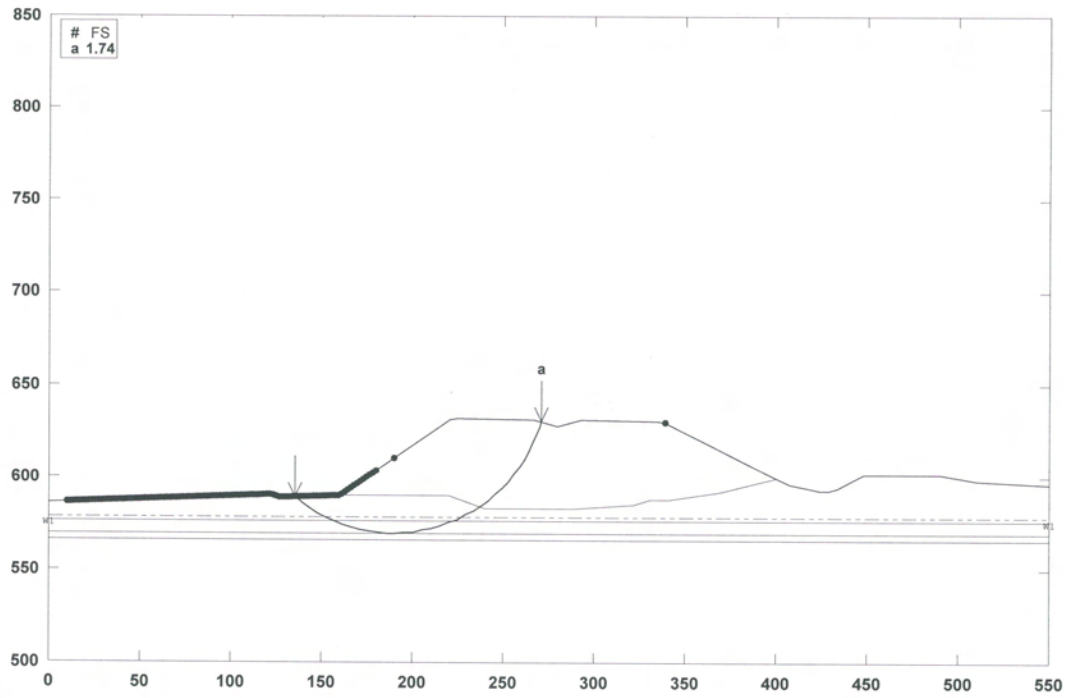


FIGURE 2: Sta. 451+50 (LAW-7-2.17) 1.5:1 Embankment - Short Term - Unreinforced

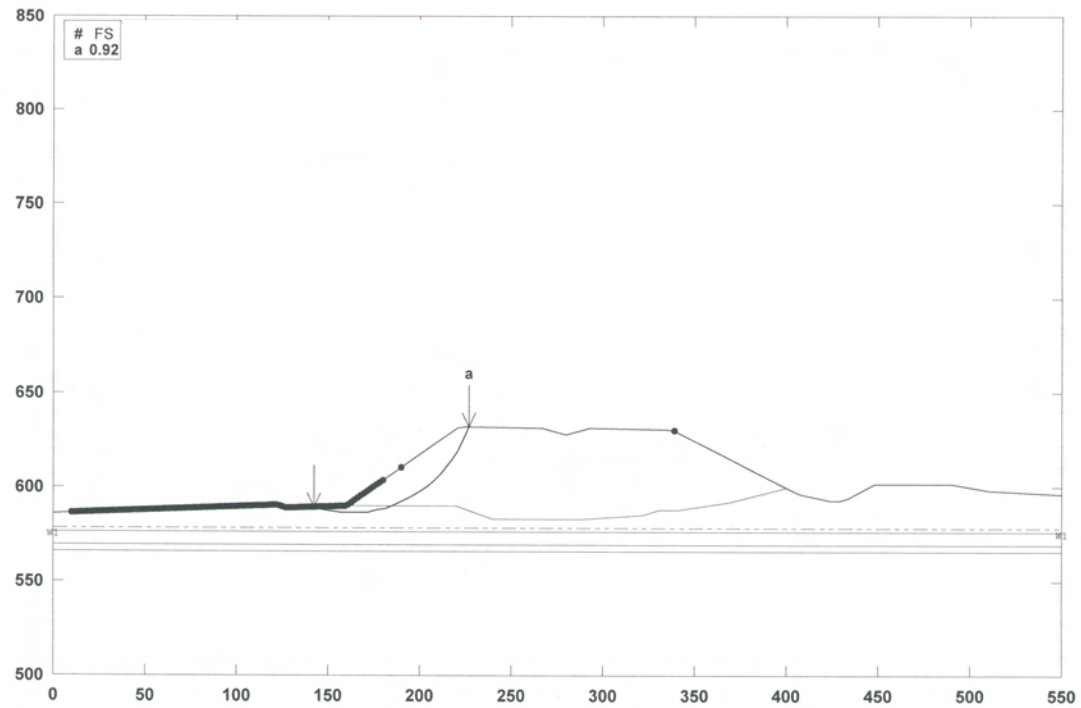


FIGURE 3: Sta. 451+50 (LAW-7-2.17) 1.5:1 Embankment - Long Term - Unreinforced

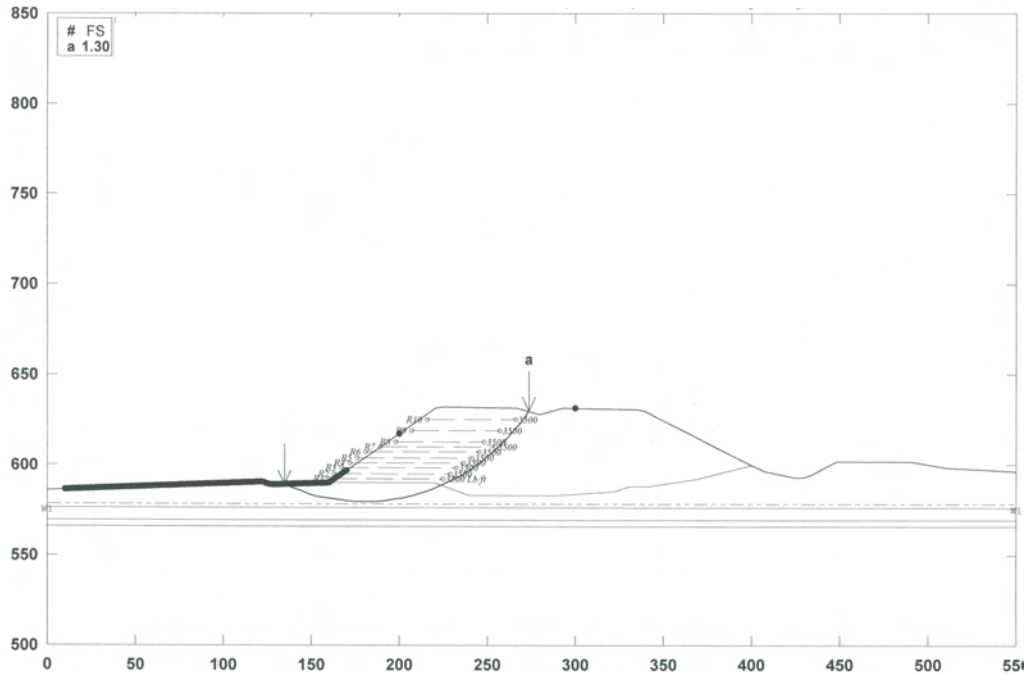


FIGURE 4: Sta. 451+50 (LAW-7-2.17) 1.5:1 Embankment - Long Term, Reinforced

b. 1.7:1 (H:V) Sidehill Fill Embankment From SR 7 Sta. 463+00 to Sta. 463+50

The proposed SR 7 mainline roadway section from Sta. 463+00 to 463+50 required sidehill fill ranging in height from approximately 55 to 70 feet to support a portion of the eastbound lanes, although the majority of the roadway section will be in cut. This segment also required a 1.7:1 (H:V) slope on the sidehill fill portion in order to include a required 10-foot buffer zone (environmental requirement) between the toe of the fill slope and Paddy Creek. The 100-year flood event rapid drawdown condition was not considered a factor at this embankment segment location.

The results of the slope stability analyses performed on the model representing the proposed cross-section at Station 463+50 indicated that factors of safety with respect to shear strength exceed minimum acceptable values for both short term and long term cases for 1.7:1 (H:V) *geogrid-reinforced* southern-facing sidehill fill slopes (see Table 6); however, the long term factor of safety would be unacceptably low for 1.7:1 (H:V) *unreinforced* southern-facing sidehill fill slopes (see Table 5). For the cross-section representing the proposed slope configuration at Station 463+50, the plots of the ten most critical failure surfaces for the *unreinforced* and *geogrid-reinforced* cases are shown in Figures 5 through 7. The geogrid-reinforced slope stability analyses at Station 463+50 were conducted using twenty-six primary layers of high strength uniaxial geogrid (Long-Term Design Strength, $T_A = 4,200$ lb/ft) at a spacing of 4 feet between El. 550 and El. 582, and a spacing of 2 feet between El. 582 and El. 616. In order to obtain a long term factor of safety of at least 1.3, additional excavation of the existing weak shale (and replacement with geogrid-reinforced compacted fill) beneath the proposed westbound lanes to a depth of approximately 50 feet, was

required in addition to the benching along the sidehill fill portion of the slope. The minimum factors of safety determined from the slope stability analyses for the cross-section at Station 463+50 are summarized in Tables 5 and 6 of this report. It must be noted that these slope stability analyses do not account for seismic effects.

**TABLE 5: 1.7:1 (H:V) *Unreinforced* Southern-Facing Sidehill Fill Slope
Minimum Factors of Safety**

Embankment Location	<i>Unreinforced</i> Southern-Facing Sidehill Fill Slope	FACTOR OF SAFETY		Meets Both Target Ranges?
		Short Term Condition	Long Term Condition	
Sta. 463+50 (SR 7)	1.7:1 (H:V)	1.04	0.69	No

**TABLE 6: 1.7:1 (H:V) *Reinforced* Southern-Facing Sidehill Fill Slope
Minimum Long Term Factor of Safety**

Embankment Location	<i>Reinforced</i> Southern-Facing Sidehill Fill Slope	FACTOR OF SAFETY	Meets The Target Range?
		Long Term Condition	
Sta. 463+50 (SR 7)	1.7:1 (H:V)	1.30	Yes

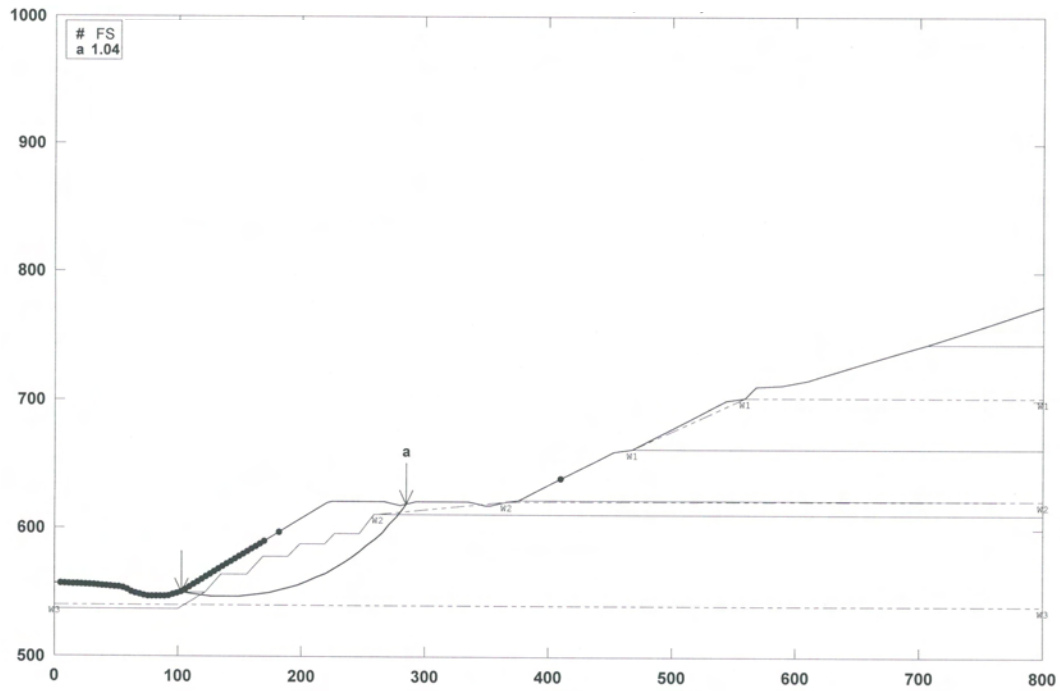


FIGURE 5: Sta. 463+50 (LAW-7-2.17) 1.7:1 Sidehill Fill - Short Term – Unreinforced

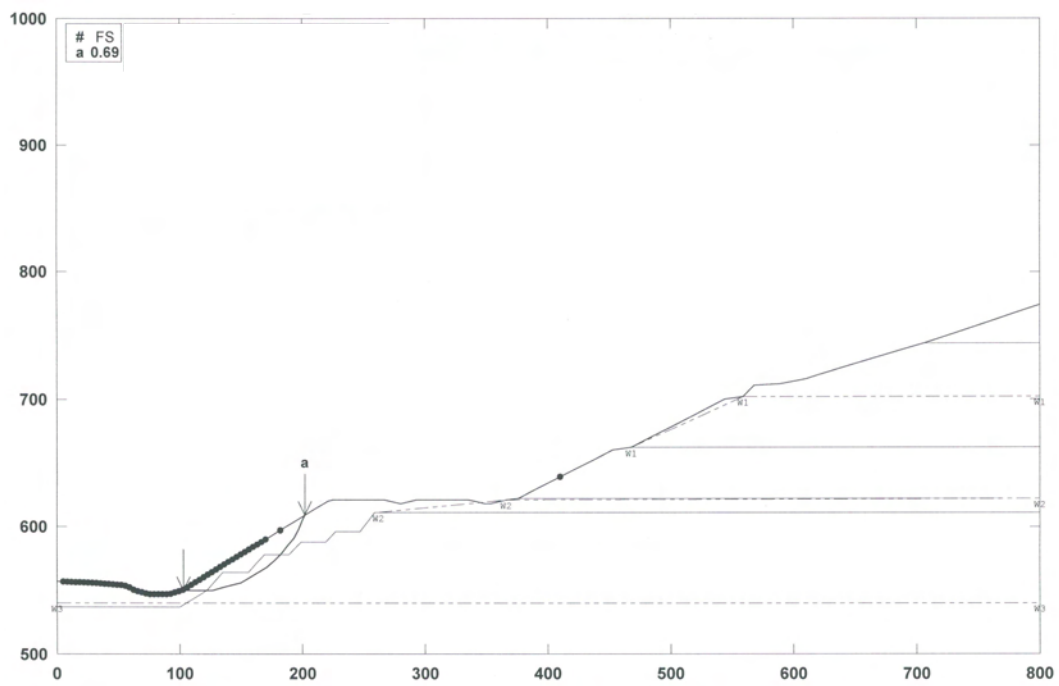


FIGURE 6: Sta. 463+50 (LAW-7-2.17) 1.7:1 Sidehill Fill - Long Term - Unreinforced

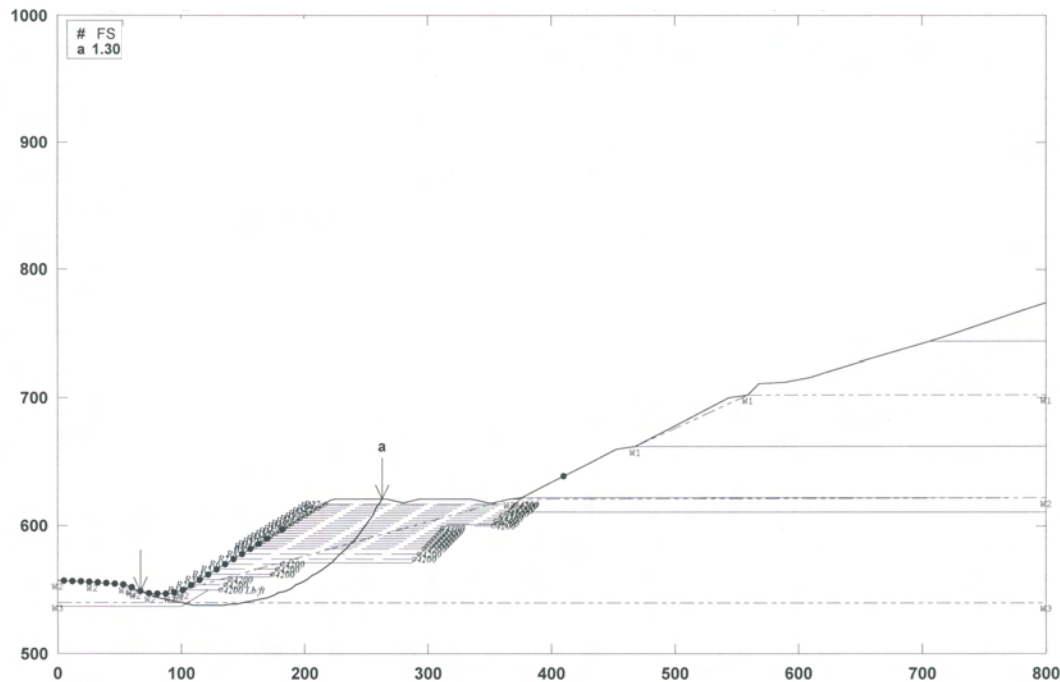


FIGURE 7: Sta. 463+50 (LAW-7-2.17) 1.7:1 Sidehill Fill – Long Term, Reinforced

It must also be recognized that the long term performance of any sidehill fill is dependent largely on the surface and subsurface drainage features of the embankment design. Sidehill seeps commonly occur in the sides of natural colluvium soil and shale bedrock slopes. Unmanaged groundwater decreases the stability of filled sidehill slopes with time through the additional weight caused by the saturation of the fill and the decreased effective strength of the soil and shale fill materials. Surface water can present problems through both surface erosion and infiltration into the sidehill fill material. Effective sidehill fill subsurface drainage techniques, in accordance with the ODOT Manual of Construction Procedures 203.05, include trench drains placed at the rear of each bench and granular outlet channels connecting the bench drains. Surface drainage should be designed to minimize both the amount of runoff over the face of the compacted fill slope and the amount of surface erosion.

c. 2:1 (H:V) Sidehill Fill Embankment From SR 607 Sta. 10+00 to Sta. 14+50

The proposed SR 607 ramp embankment connects existing SR 7 with the proposed SR 607 Connector at approximate Station 13+00, and required comparatively small amounts of fill ranging from several feet to as much as 20 feet in height. In order to minimize impact outside the desired roadway project right-of-way limits, BBC&M designed internally reinforced geogrid slopes so that planned 2:1 (H:V) sidehill fill embankment slopes could be constructed. The 100-year flood event rapid drawdown condition was a consideration for design at this embankment segment location.

The results of these analyses for the SR 607 embankment indicated that the factors of safety with respect to shear strength of the internally-reinforced embankment portion, incorporating 2:1 slopes, exceeded minimum acceptable engineering design values for this type of construction. In order to

achieve a computed factor of safety in excess of 1.0 in the rapid drawdown condition (the minimum acceptable engineering criteria), internal reinforcement of the sidehill fill was required. For the cross-section representing the proposed slope configuration at Station 13+00, the plots of the ten most critical failure surfaces for the *unreinforced* and *geogrid-reinforced* cases are shown in Figures 8 through 11. The geogrid-reinforced slope stability analyses for Station 13+00 were conducted using eight primary layers of moderate strength uniaxial geogrid (Long-Term Design Strength, $T_A = 3,300$ lb/ft) at a spacing of 3 feet between El. 538 and El. 559. The minimum factor of safety determined from the slope stability analysis for the cross-section at Station 13+00 are summarized in Tables 7 and 8 of this report.

TABLE 7: 2:1 (H:V) *Unreinforced* Southern-Facing Sidehill Fill Slope Minimum Factors of Safety

Embankment Location	<i>Unreinforced</i> Southern-Facing Sidehill Fill Slope	FACTOR OF SAFETY			Meets All Target Ranges?
		Short Term Condition	Long Term Condition	Long Term Rapid- Drawdown Condition	
Sta. 13+00 (SR 607)	2:1 (H:V)	1.12	1.31	1.02	No

TABLE 8: 2:1 (H:V) *Reinforced* Southern-Facing Sidehill Fill Slope Minimum Long Term Factor of Safety

Embankment Location	<i>Reinforced</i> Southern-Facing Sidehill Fill Slope	FACTOR OF SAFETY	Meets The Target Range?
		Long Term Rapid- Drawdown Condition	
Sta. 13+00 (SR 607)	2:1 (H:V)	1.31	Yes

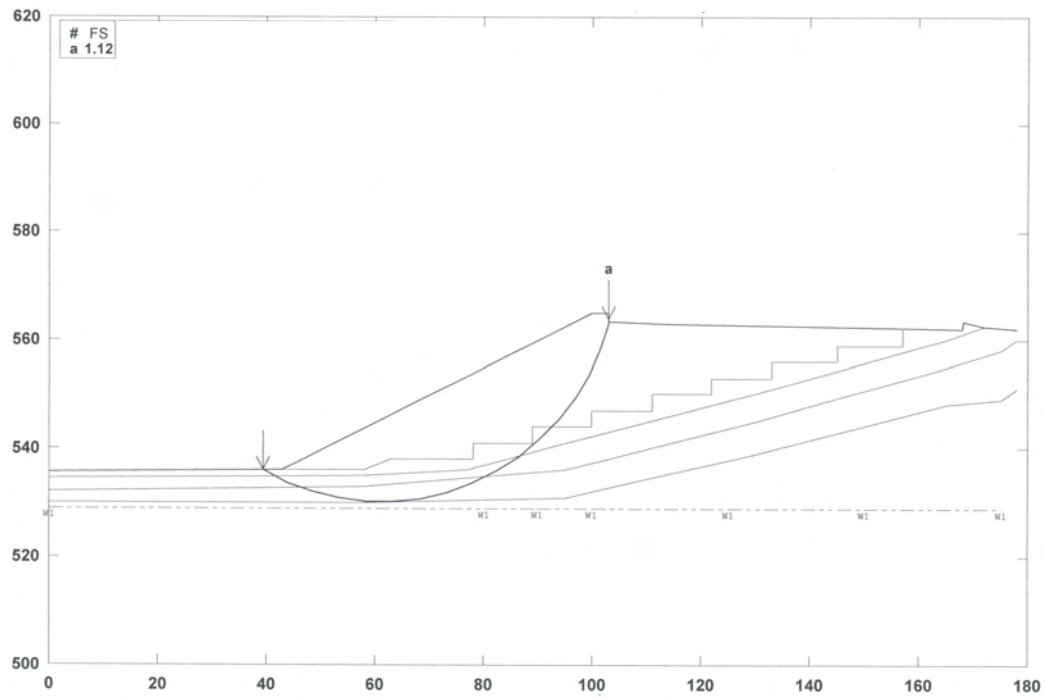


FIGURE 8: Sta 13+00 (LAW-607-0.25) 2:1 Sidehill Fill - Short Term - Unreinforced

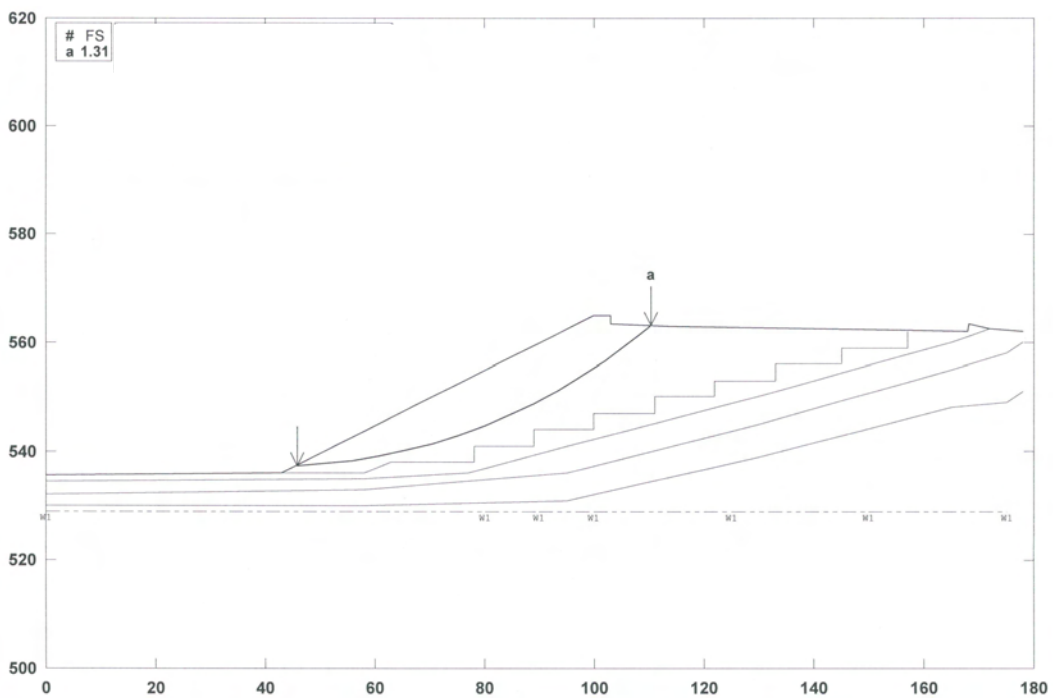


FIGURE 9: Sta 13+00 (LAW-607-0.25) 2:1 Sidehill Fill - Long Term - Unreinforced

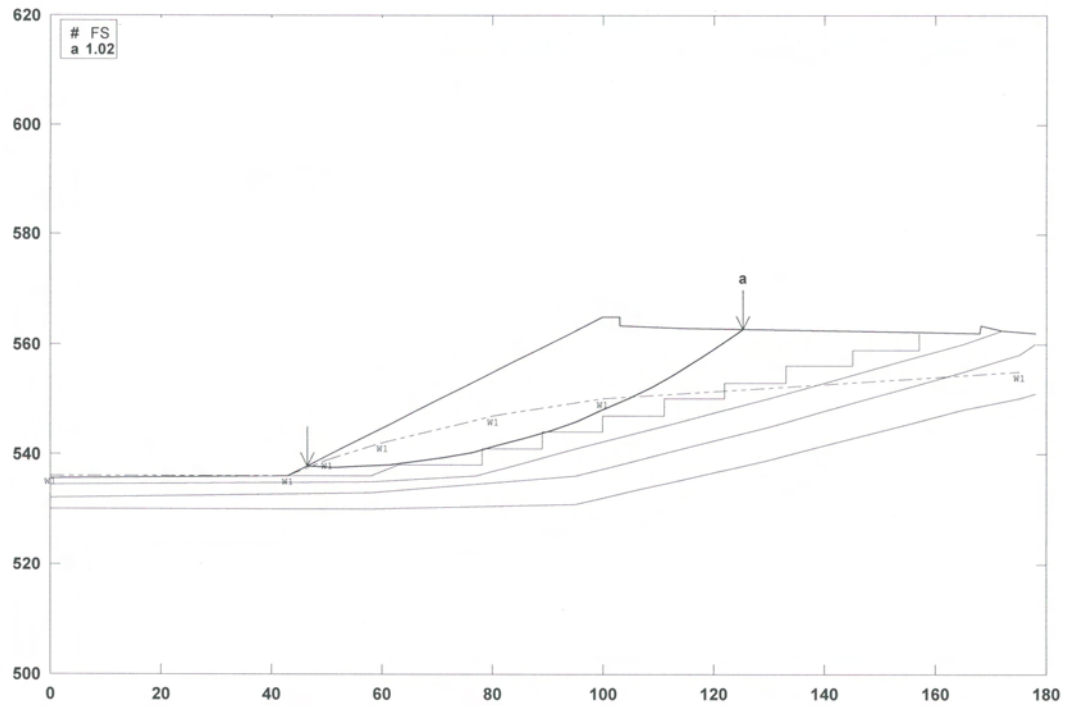


FIGURE 10: Sta 13+00 (LAW-607-0.25) 2:1 Sidehill Fill - Long Term, Rapid Drawdown

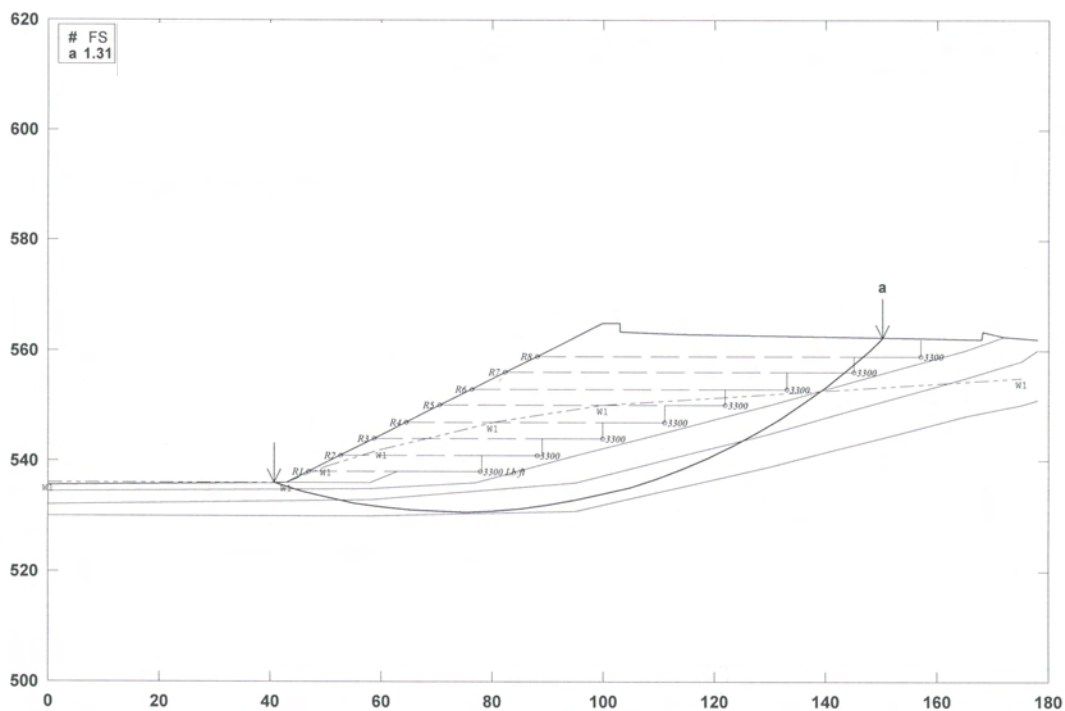


FIGURE 11: Sta 13+00 (LAW-607-0.25) 2:1 Sidehill Fill Reinforced - Rapid Drawdown

Secondary Reinforcement

Though not included in any of these analyses, secondary or intermediate internal reinforcement was included in the designs for the portions of the SR 607 and SR 7 embankments and sidehill fills incorporating primary internal reinforcement. Though secondary reinforcement is not required to achieve the necessary overall stability, incorporating secondary reinforcement in the engineered fill slopes augments resistance to surface erosion forces as well as provide additional lateral resistance during compaction at the edges of the fill slope. This increased lateral resistance allows compaction equipment to operate more safely near the edge of the slope and improve face stability.

The secondary reinforcement was placed in-between the layers of primary geogrid. Secondary reinforcement of one-half of a standard roll width (approx. 6 feet) was installed at the face of the fill slope and consisted of a biaxial geogrid of moderate to low strength (Long-Term Design Strength, $T_A = 1,300 \text{ lb/ft}$).

Settlement

2:1 (H:V) Sidehill Fill Embankment Along SR 607 From Sta. 10+00 to Sta. 14+50

Consolidation settlement estimates for the cohesive embankment foundation soils were based primarily on the results of consolidation tests performed on undisturbed samples of medium-stiff to stiff silty clay obtained from borings performed in this area. The results of computations indicated that as much as 6 inches of consolidation settlement was to be expected in these areas of the proposed SR 607 embankment.

It should be noted that the majority of the embankment foundation materials encountered in the borings performed in this area were granular soils and it was therefore anticipated that the majority of settlement would occur quickly, with most of the settlement occurring as the fill was being placed. However, settlements of the magnitude calculated may be detrimental to the new pavement and structures. It was therefore recommended that embankment settlement be monitored and that construction of the embankment only proceed after the rate of settlement has decreased to an acceptable level.

Long-term settlement of the broken down compacted shale fill embankment could become significant for the proposed embankment heights. Such long-term effects may be minimized with proper breakdown and moisture management of the shale fill materials during compaction operations. Secondary compression and settlement due to shear strain may still occur, even in well compacted fills. It was estimated that these settlements may amount to as much as 2 inches of additional settlement over a period of 15 to 20 years.

BBC&M additionally recommended that construction of the embankment sections for the LAW-607-0.25 project be performed at such a rate as to prevent embankment slope and/or foundation soils failures, since the weak alluvial deposits encountered in the borings performed at these locations were highly compressible and would likely undergo significant deformation under the new embankment load. The consolidation of such compressible soils may result in a low factor of safety if pore water pressure in the cohesive foundation soils and in the embankment fill itself is not allowed to sufficiently dissipate during embankment construction. The elevated pore pressures may compromise the strength and possibly result in local instability. It was recommended that piezometers be placed in the foundation soils prior to placing the embankment fill in order to assess whether an unsatisfactory buildup in the pore water pressure within the soil is occurring during construction, but budgetary issues prevented this from being done. Without piezometers, it was not possible to assess if the pore water pressure was dissipating, and whether or not it would be necessary to temporarily suspend construction of the embankment until the pore water pressure has dropped to a satisfactory level. No apparent issues rose during construction.

BBC&M also recommended that settlement platforms be installed at several locations along the embankment alignment so that embankment settlement could be monitored both during and after embankment fill placement. Monitoring of settlement platforms was used to determine when the rate of settlement had decreased to an acceptable rate. The results of the settlement platform installed at Sta. 13+50 of the SR 607 project are included in Figure 12.

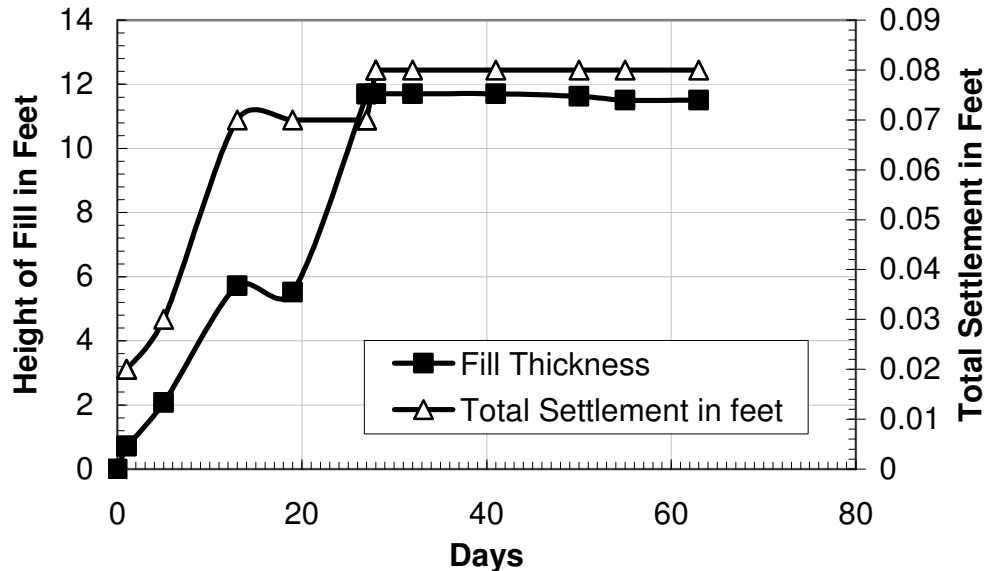


FIGURE 12: Sta. 13+50 Height of Fill and Settlement vs. Time

PERFORMANCE TO DATE

Construction of the LAW-607-0.25 project was completed in 2004 and the embankments and pavement appear to be servicing well. Embankment settlements, as measured with settlement platforms, tended to be somewhat less than anticipated and the rate of primary consolidation was greater than that computed on the basis of laboratory consolidation tests. In general, measured settlements appeared to be 65% of the computed primary consolidation settlement, and the time period required to attain 90% consolidation was on the lower bound of the time period predicted by EMBANK, using laboratory consolidation data. As is common with alluvial depositional environments, the positive influence of granular seams and lenses is difficult to quantify during design-stage computations and often results in better-than-expected performance during construction.

Construction of the LAW-7-7.25 project is on-going and scheduled for completion in 2006. Most of the earthwork for this project is complete.

ACKNOWLEDGEMENTS

The authors wish to thank R.D. Zande & Associates, Inc., the Ohio Department of Transportation District 9 and the Office of Construction Administration for their permission to present this information and their assistance in providing detailed performance records for use in preparing this document.

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AVOIDING POTENTIAL CLAIMS IN CONSTRUCTING A PERMANENT TIEBACK WALL

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Schnabel Foundation Company

Introduction:

The purpose of this paper is to describe the successful re-design and construction of a permanent tieback wall in Vandalia, Ohio, and to describe how potential construction problems and claims were avoided. A pre-designed soil nail and shotcrete wall was specified in the bid documents and was value-engineered by Schnabel Foundation Company. The value-engineering proposal of drilled-in soldier beams, wood lagging and permanent tiebacks with a cast-in-place concrete facing was accepted by the Ohio Department of Transportation and completed without delays or claims.

Background:

Located just south of the Dayton International Airport, the interchange of I-70 / I-75 in Vandalia, Ohio has been undergoing major reconstruction for the past few years. As part of the first phase of construction, a permanent soil nail retaining wall was shown in the drawings by the Ohio Department of Transportation. The retaining wall's purpose was to allow widening of a ramp between I-70 and I-75.

The length of the retaining wall was approximately 770 feet. The maximum height was 18 feet in the permanent condition and 25 feet for the temporary condition. There was also a backslope extending behind the wall at approximately an 8 degree angle from horizontal.

The borings behind the wall described the soil as a silty clay and sandy silt. Blow counts were in the range of 11 to 33. In the geotechnical report, pockets of sand and gravel containing perched water were noted. Below the silty clay layer was a shale layer which sloped downward throughout the wall length.

Original Design:

The original design showed permanent soil nails installed with a temporary shotcrete facing. A cast-in-place permanent facing was poured directly against the shotcrete. Figure 1 is a typical cross section showing the elements of the pre-designed wall. Soil nails ranging from 15 feet to 20 feet in length were installed on a four foot grid pattern. The planned nails were no. 6 or no. 7 threaded reinforcing bars with shop-grouted encapsulation for double corrosion protection. The soil nail bearing plates had four (4) headed studs welded to transfer the load to the cast-in-place reinforced permanent facing. The temporary shotcrete was four inches thick with welded wire mesh and both horizontal and vertical reinforcing bars at every soil nail location. Vertical drainage mats were placed in between the soil nails and behind the shotcrete to collect seepage and carry it to the bottom of the wall.

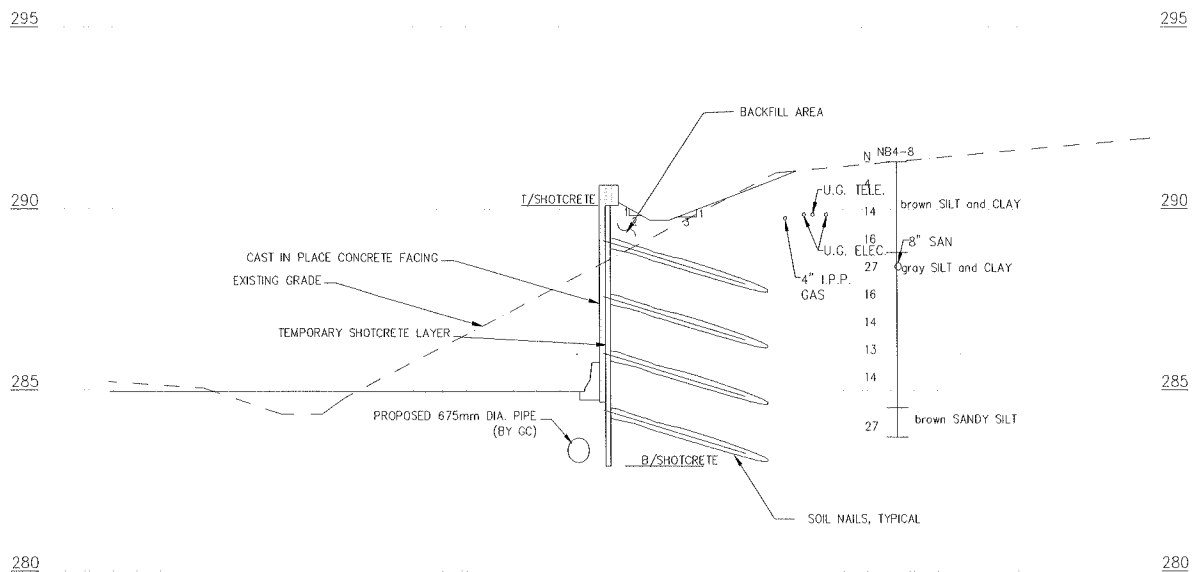


Figure 1 – Original Design - Typical Cross Section with Soil Nails and Shotcrete

As shown in Figure 1, some areas had up to five feet of the shotcrete wall being placed above existing grade, including the top tier of soil nails. This means that either shotcrete would be placed against temporary formwork and then have backfill compacted behind, or backfill would be compacted and then shotcrete placed. There are potential constructability problems for both of these construction sequences. First, when shotcreting against freshly placed backfill, local failures or sloughing could occur at the ground face. Second, when shotcreting against a temporary formwork and then backfilling, the compaction could cause deflection and cracking.

Due to these constructability problems, changing the wall to drilled-soldier beams, lagging and tiebacks was evaluated. It was concluded that a soldier beam and tieback wall would be both more constructible and less expensive.

On future projects, it is recommended that the owner give overall parameters or allow alternate designs in the bid documents. This allows each contractor to decide which design and construction procedure is most economical. A design-build approach benefits both owner and contractor by allowing the strengths of the contractor, such as specialized equipment or installation techniques, to be reflected in lower project costs and improved schedules.

Specialty Contractor Design:

Figure 2 shows a typical cross section using a soldier beam, lagging and tieback construction approach. The retaining wall was redesigned per Geotechnical Engineering Circular No. 4. The following soil parameters were given in the original bid documents and were used in the redesign:

$$\phi = 30^{\circ}; \quad \gamma = 126 \text{ pcf}; \quad Su = 2,000 \text{ psf}$$

An unsymmetric trapezoidal diagram was used to model the earth pressure. This unsymmetric diagram is based on the apparent earth pressure diagram derived by Terzaghi and Peck, and puts more load in the tieback due to pre-stressing. Global stability was checked for both the temporary and permanent condition. The temporary condition was the full height excavation for the proposed drain pipe construction. Undrained soil parameters were used for this temporary case. Drained soil parameters were used for the permanent condition when the final grade in front of the retaining wall was completed.

in between the soldier beams; therefore, using 4 inch boards above existing ground minimized potential problems with excess deflection of wood lagging due to backfill compaction.

Construction

The permanent retaining wall was constructed by drilling and setting soldier beams, installing wood lagging between the beams, and drilling tiebacks through the soldier beams. Where seams of gravel and water were present, the soldier beam holes were drilled with casing. After each hole was drilled, the soldier beam was placed and a lean concrete mix was tremied to fill the hole.

Bar tendons were used for the structural elements of the tiebacks. The permanent tendons were encased in a corrugated encapsulation and shop-grouted to provide required corrosion protection. The tiebacks were drilled with a small hydraulic powered, track mounted drill rig. Casing was used when the ground would not stay open. After inserting the tendons, grout was pumped into the hole and allowed to set up before testing. For most tiebacks, the bond length was regouted the next day after installation.

An advantage to constructing this project with tiebacks in lieu of soil nails was minimizing the potential for underground interferences. There was one tieback installed for every two hundred square feet of wall versus approximately eleven soil nails for every two hundred square feet of wall for the original design. This resulted in the tieback drilling being much less likely to conflict with present or future utilities.

As typical for a permanent tiedback wall, geocomposite drains were attached to the wood lagging boards between the soldier beams for drainage behind the cast-in-place wall as shown in Figure 3. Shear studs were welded on to the soldier beams for permanent load transfer to the cast-in-place facing.



Figure 3 – Placing drainage mats to full temporary height.

Performance of Tiebacks

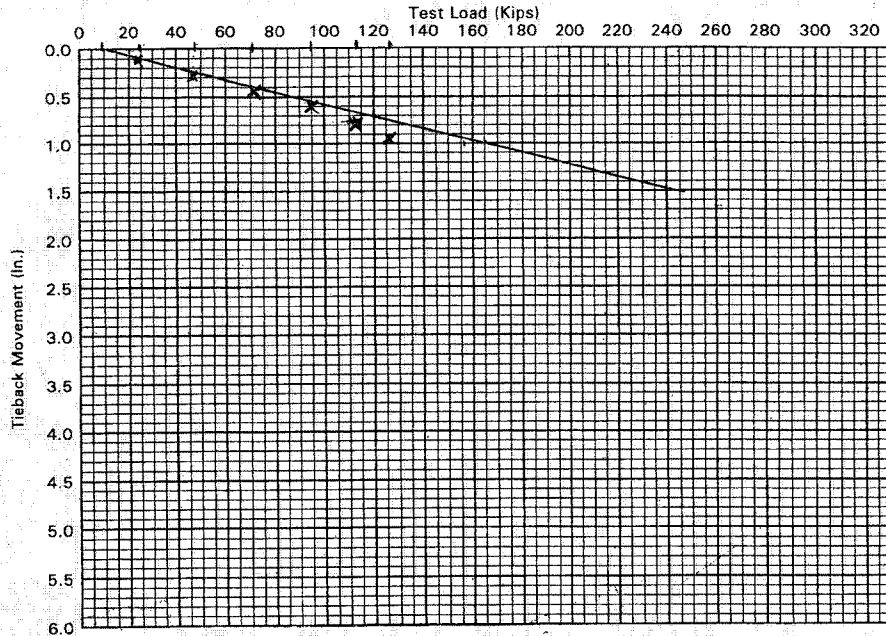
Proof, performance or extended creep tests were performed on all tiebacks. An extended creep test was performed on one of the tiebacks in order to check the long-term performance of the tiebacks. The creep test produced 0.032 inches of movement over the last log-cycle of time which was less than the allowable 0.08 inches. Also, the results of the proof and performance tests were within the allowable creep criteria of 0.04 inches for the 1 to 10 minute load hold at the maximum test load. The average creep movement during load hold was 0.004 inches and the maximum was 0.021 inches. A typical proof test is shown in Figure 4.

PROOF TEST SHEET

Ground Anchor No.: 57
Date Installed: 2-19-04

Job No.: 08-5445
Date Tested: 2-25-04

Tester's Initials: BDS
Engineer's Initials: CEK



PROOF TEST DATA

creep = 0.007

BASIS OF LOAD	LOAD (kips)	PRESSURE (psi)	MOVEMENT (inches)	BASIS OF LOAD	LOAD (kips)	PRESSURE (psi)	MOVEMENT (inches)
Alignment Load	9	750	0	1.33 x DL @ 2 min.	126	6650	0.976
0.25 x DL	24	1450	0.101	1.33 x DL @ 3 min.			0.976
0.50 x DL	47.5	2700	0.260	1.33 x DL @ 4 min.			0.978
0.75 x DL	71	3900	0.426	1.33 x DL @ 5 min.			0.980
1.00 x DL	95	5100	0.609	1.33 x DL @ 6 min.			0.981
1.20 x DL	114	6050	0.790	1.33 x DL @ 10 min.			0.983
1.33 x DL @ 0 min.	126	6650	0.929				
1.33 x DL @ 1 min.			0.976	LOCK OFF	71	5900	0.625

Figure 4 – Typical Tieback Proof Test Sheet

Design-Build Advantages

One of the benefits in design-build construction is that the specialty contractor determines the soil-to-grout bond capacity. If a tieback fails to meet the testing criteria, the contractor can decide if the bond stress was overestimated, if the installation procedure could be improved or if there were other factors involved. The tieback contractor is able to evaluate problems immediately and proceed with a solution promptly, thereby minimizing any delays to the project.

Another advantage of design-build is the ability to obtain the lowest cost for a project while fully utilizing different contractor's strengths in design and construction techniques. The best way to take advantage of this is to allow alternate designs or to provide design parameters at the project bidding stage.

Local and past project experience should be taken into consideration in deciding what construction procedures should be used. Shotcreting is a very sensitive procedure, which is most efficient and economical in near ideal ground conditions. Specialty contractors can use their previous experience to help decide which construction procedures will be best for a particular project. Different procedures can be advantageous and provide an equivalent final product.

Conclusions

For this project, a soldier beam, wood lagging and tieback wall was more economical than the original soil nail design proposed. The design-build approach generally results in contractors using their experience to construct projects with greater economy to cost and schedule. Consequently, the potential for claims and time delays due to contractors utilizing less efficient designs and methods are minimized.

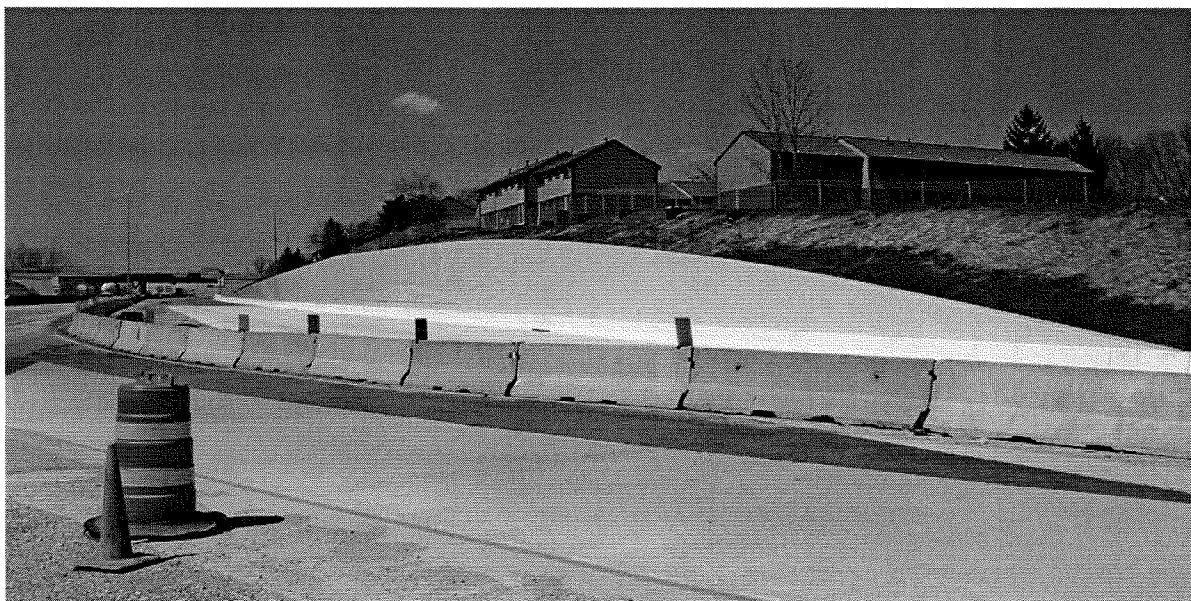


Figure 5 – Finished Permanent Tieback Wall with Cast-in-Place Wall Facing

Acknowledgements

The author would like to thank Kokosing Construction Company and the Ohio Department of Transportation for their help in making this project successful. Kokosing Construction Company was the general contractor on this project. Kokosing worked with Schnabel Foundation Company in proposing the design-build concept to the Ohio DOT, and also constructed the cast-in-place wall facing.

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Practical Considerations for the Use of Mechanically Stabilized Earth Walls for Transportation Applications

Abstract: This paper provides a discussion of the various components of Mechanically Stabilized Earth (MSE) walls used for transportation projects. Considerations for various factors such as ground settlement, earthquakes, impact loads and horizontal and vertical inclusions on the performance of MSE walls are discussed. Techniques used for MSE walls to directly support (abutment resting on a spread footing atop the MSE structure) and indirectly support (abutment on piles with the MSE structure supporting the fill) bridge structures are provided. Current practices regarding the use of MSE walls for integral bridge abutments, skewed bridges and bridges involving phased construction are also provided. Techniques to improve aesthetics are discussed as part of the wide appeal for appearance in these walls. Examples of MSE walls constructed for major transportation projects in the Midwest are provided to illustrate concepts described in this paper. The considerations mentioned in the paper may assist a geotechnical engineer in evaluating the feasibility of a MSE wall for transportation projects.

Introduction

Mechanically Stabilized Earth (MSE) walls consist of alternating layers of granular backfill and galvanized steel reinforcements connected to precast concrete facing panels (Figure 1). This paper discusses the design features of MSE walls in relationship with their use for transportation applications. Consideration of these design features may assist practicing geotechnical engineers to evaluate the applicability of the MSE walls for the specific transportation applications under consideration. The information provided in this paper will provide a framework for the geotechnical engineers to discuss MSE wall related issues with the other members of the project team.

Design of MSE Walls

In a MSE wall, the mechanical properties of a soil mass - in this case composed of granular backfill - are improved by discrete steel reinforcing elements aligned parallel to the orientation of its greatest strains. The friction between the granular soil and the steel reinforcements allows the backfill, which can withstand only compressive and shear stresses, to transfer tensile stresses to the reinforcements. The combination of soil backfill, steel reinforcing elements and a durable facing produces a composite material that is both strong and flexible. A description of the components is provided in the following sections.

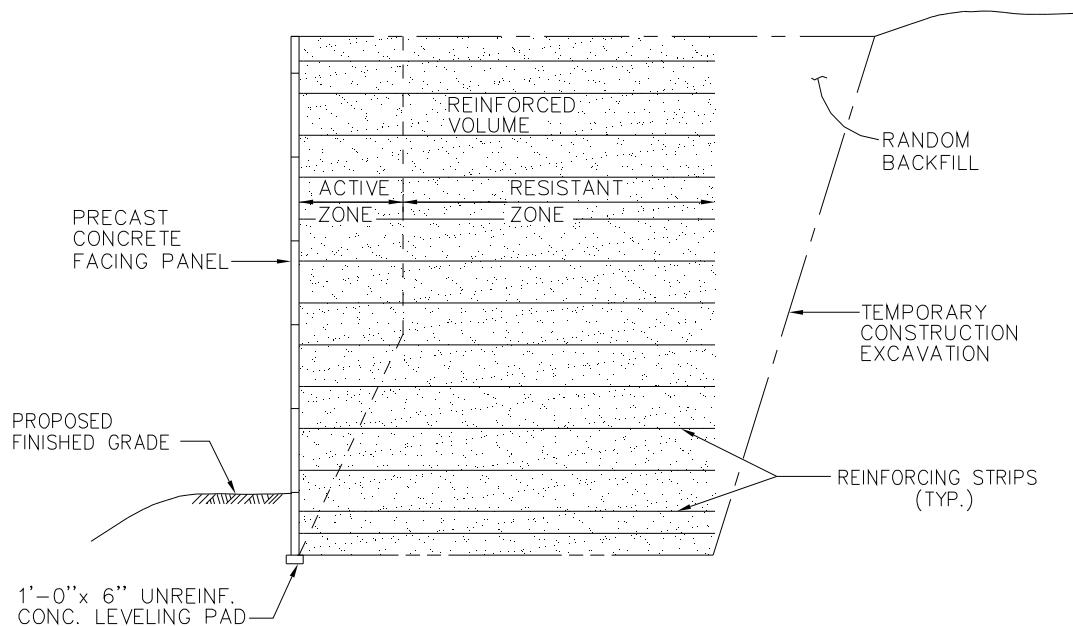


FIGURE 1
TYPICAL SECTION OF MSE STRUCTURE

Facing Elements

MSE walls for permanent applications (defined as a design life of 50 to 100 years) predominantly use precast concrete facing panels with open joints separating the panels. The flexibility of a MSE wall depends on maintaining the integrity of the open joints between the precast modular facing elements. The open joints allow the panels to move relative to each other, thereby making it possible to undergo significant deformation. The joint design consists of the panels' edge lips, geotextile fabric behind the joints and the 'bearing pads.' The 'bearing pads' are composed of elastomeric material that provides a bearing surface between the top of one panel and the bottom of the panel resting above. The bearing pads are typically $\frac{3}{4}$ - to 1-inch in thickness, varying based on the size of the panel and height of wall. The pad material is highly resistant to oxidation, resistant to corrosive attack and retains resiliency against cracking. The weight of the overlying panels as well as the effect of long-term creep usually results in a 10 to 15 percent compression of the bearing pads. The bearing pads provide the necessary balance between compressibility under increased loads and ability to maintain an open joint during the design life of the MSE wall.

Panel Size, Shape, Color and Texture: **Since** facing is the only visible feature of a MSE wall, selection of the right facing type (including size, shape, color and textures) is an important design decision. Options exist for improving the appearance of MSE walls through geometric variations or casting techniques. One approach is to vary the

thickness of the panels or create projections on the panels that create shadow lines giving the wall a unique appearance. Another is to use form liners to texture the concrete in ways that either emphasize or hide panel joints. Either approach is acceptable as long as it is applied consistently and fits the vision of the corridor (Ohio Department of Transportation, 2005). Square, rectangular and proprietary shapes are currently the industry standard for MSE walls supporting embankments, ramps and bridges along America's highways. Most State Department of Transportation specifications limit the maximum size of the standard panels to sixty (60) square feet. For relatively long walls, such as ramp walls, the use of larger panels may reduce installation costs. Square and rectangular panels are also more amenable to applying special mural work on panels. Complex images that spread over several panels can be easily cast using rectangular panels. The Interstate-70/Interstate I-75 (I-70/I-75) Interchange project in Dayton, Ohio used 5-ft. by 10-ft. rectangular panels. This project involved the creation of an image (Wright B Flyer) on the MSE wall panels. Sixteen Wright B Flyer images were cast on the MSE wall panels for this project. Rectangular panels sized to 50 sq. ft were an appropriate choice for this application, since it reduced the number of precast pieces as well as reduced installation costs.

Backfill

The current practice in the design and construction of MSE walls is to use durable granular materials as backfill with no aggregate greater than 4-inch diameter or more than 15 percent particles finer than 0.075 mm (US Sieve No. 200) [American Association of State Highway and Transportation Officials (AASHTO), 17th Edition 2002]. For a period in the early 80s, the Federal Highway Administration (FHWA) specification for MSE walls allowed up to 25% fines, and many structures were constructed in the United States with these finer materials. Quality control was a problem, however, with backfills significantly exceeding the 25% fines limit, making wall-facing alignment a problem for contractors. Better quality control, as well as the desire to reduce the backfill-induced wall deformations, favors the current specifications based on no more than 15 percent passing No. 200 sieve. The granular composition of the backfill within the reinforced volume enhances drainage, as well as ease-of-construction considerations at the work site.

Alternate backfill materials: There is significant interest in the construction industry to approve the use of alternate backfill materials for MSE applications. Indiana Department of Transportation (Indiana Department of Transportation, 2005) approves the use Air Cooled Blast Furnace Slag (ACBF) as the select granular fill for MSE walls in the State of Indiana. In 2003, ACBF was extensively used for MSE walls constructed along the I-80/I-94 corridor in Northwest Indiana. ACBF used for the I-80/I-94 project meets the electrochemical, shear strength and durability requirements for MSE select backfill. In 2000, Ohio Turnpike Commission (OTC) approved the use of foundry sand as the select backfill for the MSE walls constructed for the OTC Third Land Widening Project (OTC Project 77-99-01). At locations where such materials are readily available and meet

electrochemical and physical strength criteria, the use of materials such as ACBF and foundry sand provide significant savings to the overall cost of the MSE walls.

Steel Reinforcements and Connection of Facing Elements to Reinforcements

Detailed procedures for the design of inextensible (steel) reinforcements are provided in the Standard Specifications referenced by AASHTO (AASHTO, 2002). The specifications require that the tensile stress in the reinforcements will be less than or equal to allowable stress of steel during the service life of the structure. MSE walls are routinely designed for a 75-year service life, while structures supporting bridges are typically designed for a 100-year service life. The primary factor determining the service life of an MSE structure is metal losses of the reinforcements that, for a metallic reinforcement material, is closely related to backfill electrochemical properties. Research on buried galvanized steel, conducted by the National Bureau of Standards, Terre Armees Internationale, FHWA, and several state DOTs, confirms that the metal loss rates (Table 1) used in the design of MSE structures are conservative. The design loss rates accompany AASHTO specification criterion for steel soil reinforcements galvanized with 2 ounces per square foot of zinc and buried in backfill meeting the electrochemical requirements in Table 2. It is noted that evaluations of in-service MSE structures meeting the AASHTO electrochemical requirements actually have zinc loss rates at half the value listed in Table 1 (Anderson and Sankey, 2001).

Table 1 – Metal Loss Rates	
Material	Loss Rate
Zinc (first 2 years)	15 $\mu\text{m}/\text{yr}$
Zinc (subsequent years to depletion)	4 $\mu\text{m}/\text{yr}$
Carbon steel (after zinc depletion)	12 $\mu\text{m}/\text{yr}$

Table 2 – Electrochemical Requirements	
Backfill	Electrochemical Requirements
Resistivity	≥ 3000 ohm-cm at saturation
pH	Range 5-10
Chlorides	< 100 ppm
Sulfates	< 200 ppm

The carbon steel loss rate in Table 2 is twice the average loss of steel based on weight, so that the calculated loss is in proportion to the loss of tensile strength based on rupture of corroded samples (The Reinforced Earth Co., 1995). Since it is necessary to maintain the reinforcement in an allowable stress condition at the end of the service life, the loss rates in Table 2 determine the sacrificial thickness of steel which must be added to the load-carrying cross section to produce the design cross section. At the end of service life, the remaining steel will have a factor of safety of 1.8 against yield and 2.2 against rupture.

Length of Reinforcing Elements: The rule of thumb is to assume that the strip length is typically 70% of the wall height. Longer strip lengths are required when the MSE structure is supporting sloping surcharges, bridge abutment loads, and hydrostatic loads due to submerged conditions. Global stability analyses that model the loading and in-situ geotechnical conditions are suggested to develop the minimum strip length requirements for such conditions. Minor reductions in applied bearing pressures can also be obtained by increasing the strip lengths within the 70% to 100% range of the wall height. There is no significant reduction in bearing pressure from reinforcement lengths longer than the wall height (The Reinforced Earth Company Design Manual, 2000). For walls located on the crest of a slope, the global stability can be improved by using longer reinforcements at the lower portion of the wall.

Strip lengths less than 70% of the wall height are acceptable when the retained material has excellent shear strength properties, with such design usually performed on a case-by-case basis. A design using shorter strips for the lower half of the wall was performed for a 275-ft. long wall along Trunk Highway 52 in Rochester, Minnesota. The height of the wall at the tall section was 31-ft. The lower 13-ft. section of the wall was located within competent St. Peter sandstone (see Figure 2). A strip length that corresponds to 43% of the wall height was used for the lower section.

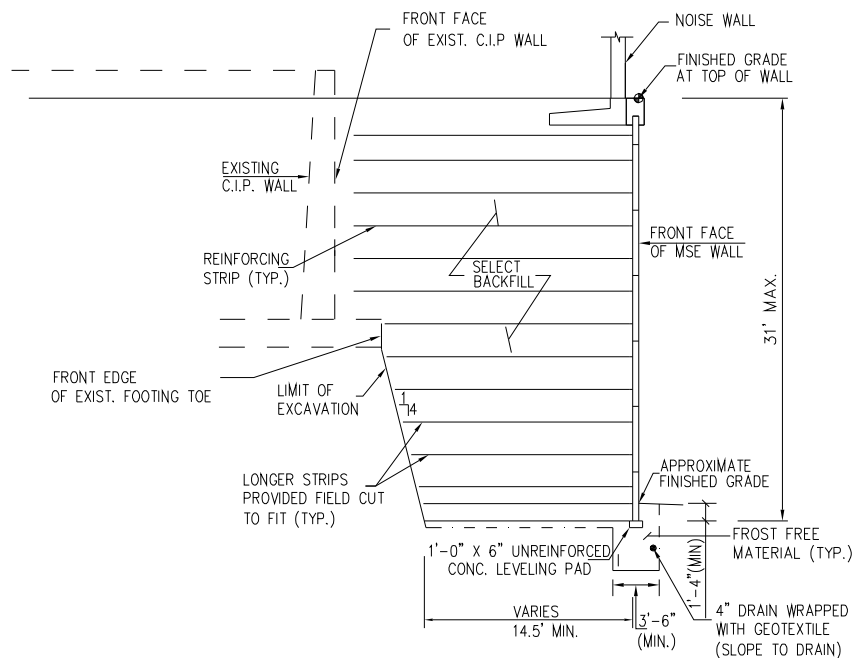


FIGURE 2
MSE STRUCTURE WITH
SHORTER STRIP AT BASE

Connection of Facing Elements to Reinforcements: The selection of a connection between the facing elements and the reinforcements is based on a detailed evaluation of the anticipated stresses at the connection and the ease of connecting the reinforcing elements to the facing panels during construction. A well-designed positive connection will not allow separation to occur between the facing and the reinforcing elements. A positive connection also prevents movement between the facing and the backfill in the event of deformation due to external loads.

Behavior of MSE Structures used for Transportation Applications

MSE systems were introduced to the American Transportation Industry in 1971 at the invitation of Federal Highway Administration. During the last 34 years, MSE systems have gained national acceptance and have been used widely for highway, railroad and waterway applications. The following sections summarize the design and construction issues that should be considered for the use of MSE walls for transportation applications.

Settlement Performance

The ability of MSE walls and abutments to withstand extreme settlements is a well-recognized benefit of this construction technology. MSE structures have accommodated total settlements of nearly 3 feet, as well as differential settlements greater than 1 foot in 100 feet (1%), without loss of structural function and without showing facing panel distress. Depending on the magnitude and rate of settlement, there are several methods to reduce the effect of settlement on MSE walls (Abraham and Sankey, 1999). Settlements less than 6-inches are typically accommodated by adjusting the height of the concrete cap (coping) placed on the top of the wall. If the expected total settlements are greater than 6-inches, the construction of the MSE walls are performed in several stages such that the upper row of panels and the drainage elements are installed after most of the foundation settlement is essentially complete. These techniques are feasible as long as the waiting periods can be accommodated within the construction schedule. Slower settlements that cannot be accommodated within the construction schedule must be accelerated or an appropriate ground improvement technology should be used in conjunction with the MSE wall.

One of the large Midwest MSE projects subjected to large settlement was the MSE walls supporting the Blue Water Bridge Plaza located on the U.S. side of the bridge over the St. Clair River between Port Huron, Michigan and Port Edward, Ontario (Lamberson and Bloomfield, 1996). The plaza was constructed 27-ft. above the surrounding grade and supported by 5400-linear feet of MSE walls and MSE bridge abutments. The plaza was constructed over 110-ft. of soft to firm clay which was in turn underlain by competent bedrock. Due to concerns with global stability, Michigan DOT chose to construct the plaza over a continuous band of 3,000 stone columns. Settlement readings taken during a period of 1840 days (5 years) indicated total settlement of 10-inches at one of the abutments.

Seismic Performance

Seismic performance is a design consideration in some of the Midwestern States due to the proximity to the New Madrid Earthquake zone. MSE walls have performed well during earthquakes due to their relative flexibility and due to their ability to withstand distress in the face of large horizontal and vertical accelerations. Twenty-one Reinforced Earth walls and two Reinforced Earth abutments experienced the 6.7 Richter magnitude Northridge earthquake in 1994, in the densely populated San Fernando Valley 20 miles northwest of Los Angeles. Buildings, bridges and freeways suffered severe damage, yet all MSE structures performed well, with only superficial cracking to a few facing panels of one wall. MSE walls have proven to be safe and flexible in the presence of seismic events throughout the world (Sankey and Segrestin, 2001).

Traffic Impact Loads

There are two types of traffic impact loads that need to be considered in the design of MSE walls for Transportation Applications. The first one is the impact on the barrier system mounted on the top of a wall. The second type is the impact from vehicles such as freight trains that run along the front side (base) of the wall.

Traffic Impact on a Barrier atop a MSE wall: The excellent performance of hundreds of miles of both precast and cast-in-place traffic barrier mounted on top of MSE structures is a testament to the ability of these structures to carry such loads without damaging the panels. The behavior of the MSE walls during traffic impact were verified by crash tests on barrier atop a Reinforced Earth wall. To withstand the vehicle impact stipulated by highway design codes, the parapet placed on top of an MSE wall normally has a horizontal slab attached to its base to resist the shear force and overturning moments generated during vehicular impact events. Traffic impact loads and allowable distribution length often vary from jurisdiction to jurisdiction, as do the designs used to meet such loadings. The objective of the design is **not** to transfer the high impact forces to the precast concrete panels. Rather, the horizontal impact force is transferred to the reinforced soil mass by soil shear stresses that develop beneath the barrier slab.

Severe Vehicular Impact: A MSE structure has the ability to absorb high levels of energy due to its tolerance for deformation. This ability to absorb impact was demonstrated when a freight train hit a MSE Bridge Abutment in near Robertson, New South Wales, Australia (made famous as the site of the movie “Babe”). The MSE system was supplied by VSL Corporation in February 1995 to replace an aging road overpass. The train was de-railed when the diesel engine rolled off the track and struck the wing wall, crushing the drivers cabin. The wing wall was constructed at approximately 30 degrees to the main abutment wall. One panel was badly damaged and several more sustained superficial damage. Generally, the wall was unaffected structurally and the bridge, which is supported directly on the MSE system without piles, remained quite

safe. Owners repaired the cracked panel but no work was necessary on the bridge superstructure (Gary Power, 2000).

Impact of Railway Traffic Loads and Stray Currents

Heavy rail is generally designed for Cooper's E-80 loading, which in the design of MSE structures is modeled as a uniform live load equivalent to 9.5 ft of soil. Light rail for transit systems are subjected to relatively light loads and may be modeled with as little as a 3 ft. soil surcharge. A typical application for this design approach is the MSE walls supporting the Hiawatha Light Rail in Minneapolis, Minnesota (Figure 3). MSE structures designed for static loads also safely accommodate the effect of vibrations from trains without wall deformation. This is due to the excellent damping characteristics of the MSE structures, or more accurately, the MSE backfill.

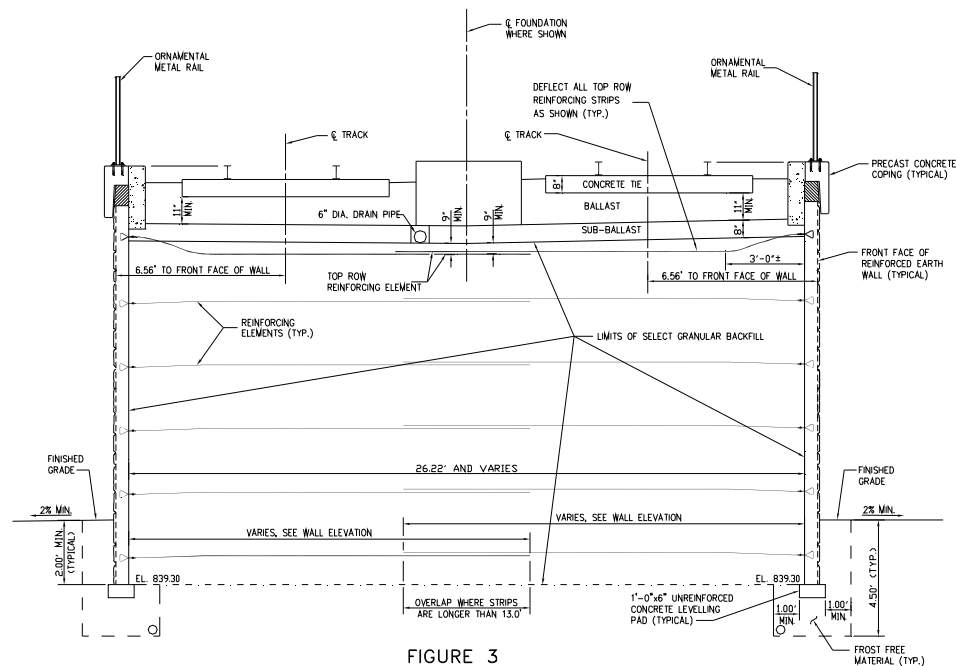


FIGURE 3
TYPICAL SECTION OF MSE
STRUCTURE SUPPORTING LIGHT RAILS

Horizontal and Vertical Inclusions

Design and functionality needs of a wall project may require consideration for horizontal and vertical inclusions. Such inclusions consist of inlets, manholes, and pipes running parallel and/or normal to the front face of the wall. Several state DOTs and major owners have developed various details for incorporating the horizontal and vertical inclusions within the MSE structures. MSE vendors are very experienced in the design and construction issues that should be considered in these inclusions.

Bridge Abutments

MSE technology has made possible new and economic designs for bridge abutments adaptable to the widest variety of superstructures and foundation soils. The use of MSE structures to support bridge abutments has increased rapidly in recent years as more and more owners and engineers become familiar with and develop confidence in this technology. In the United States, there are approximately 600 MSE-supported abutments (300 bridges) constructed annually (Anderson and Brabant, 2005).

There are two types of MSE abutments – true and mixed. In a true abutment, the bridge beams are supported on a spread footing bearing directly on the MSE structure. Figure 4 provides the design of the typical design of a MSE true abutment. A major advantage of true abutments is the limitation or elimination of the “bump” that otherwise occurs at the bridge approach when abutments are supported instead by piles.

For the MSE wall supporting bridge abutments at the I-70/I-75 Project, the anticipated long-term settlement needed to be limited to no more than 1-inch. Overexcavation of the poor subgrade soils and replacement with structural fill was required at some of the locations where anticipated long-term settlement exceeded bridge design limitations. The cost of the proposed over excavation and replacement was relatively minor and did not prohibit the use of the MSE abutments (CH2M Hill, 2002).

To prevent overstressing the soil of a true abutment, the beam seat is sized so the centerline of bearing is at least 3 feet behind the MSE wall face and the bearing pressure on the reinforced soil is no more than 4 kips per square foot. The bearing stresses beneath the seat are distributed into the reinforced soil, so soil reinforcement density is higher near the top of the structure and decreases with depth as the bearing stresses dissipate.

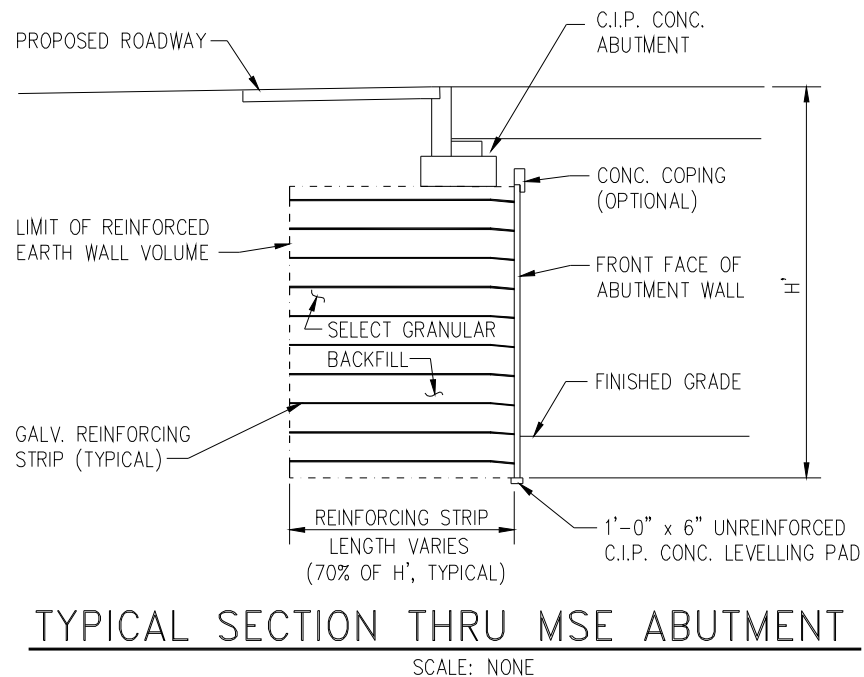
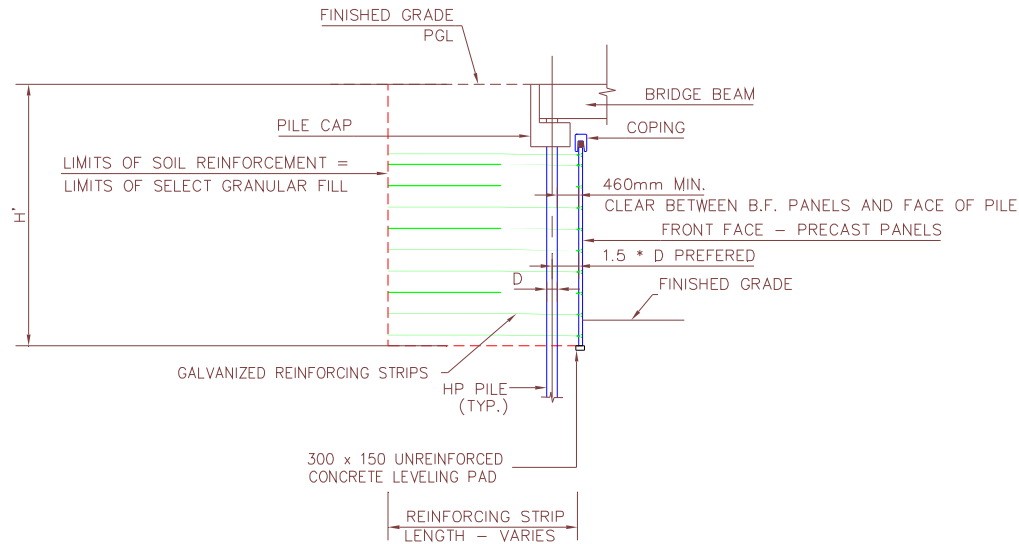


FIGURE 4

A mixed abutment, by comparison, has piles supporting the bridge seat (Figure 5), with the MSE walls retaining the fill beneath and adjacent to the end of the bridge. In some cases a portion of the lateral load on the pile-supported seat is transmitted to the MSE wall volume. This load can be resisted by MSE reinforcements in the wall or by reinforcements extending from the backwall of the seat.

Integral Abutments and MSE Structures: Integral bridge abutments, having neither joints nor bearings, are becoming popular in the United States. Since repair of joints and bearings on traditional bridges is costly and disruptive to traffic, a goal of integral bridge design is reduced maintenance, with nearly maintenance free substructures. Integral bridges are generally supported on piles, so pile deflection caused by thermal movements of the superstructure must be accommodated by the reinforced soil of the MSE structure. The additional horizontal forces due to the deflection of the piles can be accommodated in the design of the soil reinforcement. National Highway Institute (NHI) Course No. 132042 (National Highway Institute, 2001), provides design details representing the combination of integral abutments and MSE walls. The details include a minimum 18-inch horizontal clearance between the piling and panels, as well as the use of a casing around the piling where significant negative skin friction is anticipated. Integral Bridge Abutments with MSE walls are used by Indiana DOT for the Bridge carrying Interstate I-465 over 86th Street in Indianapolis, Indiana.



TYPICAL SECTION PILE SUPPORTED ABUTMENT

SCALE: 1 : 100

FIGURE 5

Skewed Bridge and MSE Structures: Skewed bridges benefit from the use of MSE walls. The use of MSE walls with acute corners less than 70° has generally been a standard practice since 1986. When two MSE wall segments are joined to form an acute angle, the standard design may be acceptable or, upon further analysis, a special design may be required. The need for a special design will depend upon the magnitude of the acute angle and the height of the walls (which determines the length of the reinforcing strips). Figure 6 shows the typical layout of the strips for acute corners of the MSE walls around the bridge for US Highway 53 over Hastings Way in Eau Claire, Wisconsin. The nose portion of the structure is designed as an internally tied bin structure with at-rest earth pressures from top to bottom. The reinforcing strips are designed as tension members only and are generally at low stresses due to the bin effect (soil arching). A conservative number of reinforcing strips are used with some extended from the nose portion to the main MSE wall volume and others tied across opposing walls.

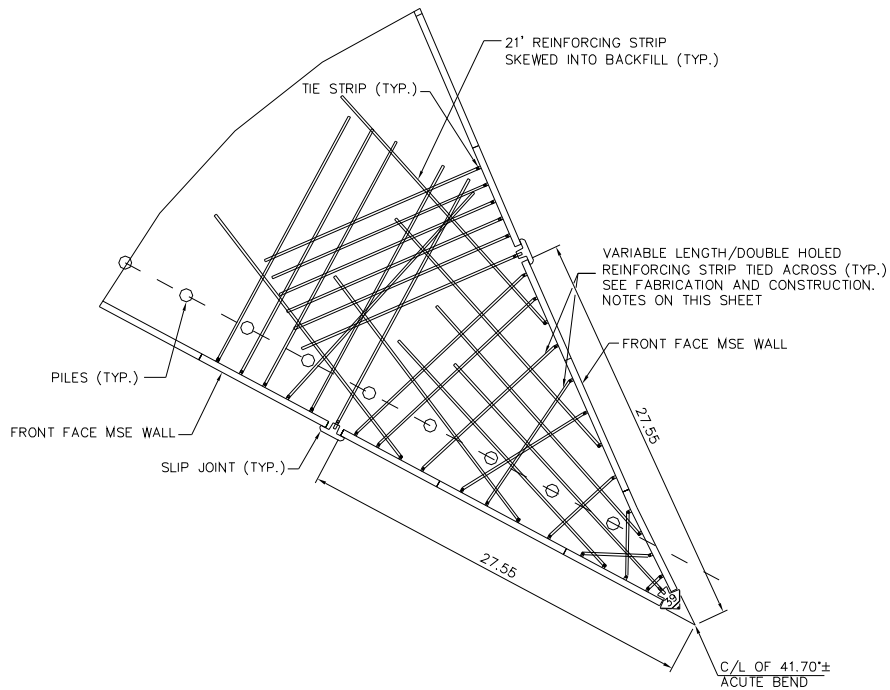


FIGURE 6
REINFORCING STRIP LAYOUT
AT ACUTE CORNER

Phased Construction of the Bridges: Maintenance of traffic requirements for most of the bridge improvement projects in urban areas are such that portions of the bridge (one or two lanes) are available for use when the construction is in progress. Temporary MSE walls using wire mesh facing are generally used for such cases to retain the fill. During the initial phase of construction, the temporary wall will be used to support the fill and the bridge dead loads. During the subsequent phases of construction, the traffic and the live load on the abutment loads (in the case of the MSE abutments) will be supported by the wire wall. The temporary wire wall is buried during the construction of the subsequent phases. A successful phased construction of the MSE wall requires that the design of the permanent wall fully incorporate the construction details of the temporary wall as well. In general, the construction tolerance requirements for temporary walls are not as stringent as that of a permanent wall. However, more attention to construction tolerances are suggested when a temporary wall is used at a bridge location. A significant consideration in such applications is the limited work space available for construction crews to properly install backfill. Proper controlled placement of fill is still critical to the function of the temporary wall.

Overall Lifecycle and Maintenance Costs

Several owners have observed that the overall costs for maintenance and inspection of a filled-in raised roadway of retained earth is significantly lower than that

of a steel or concrete bridge. In several parts of the country, existing steel bridges were replaced by MSE embankments (embankments with MSE walls on both sides) to reduce maintenance costs. This concept was used for the rehabilitation of the Chicago Skyway, which is a 7.8 mile long limited access toll on the east side of Chicago. The Skyway was built by the City of Chicago in the late fifties as an extension of the Indiana Toll Road into the heart of Chicago. The City of Chicago has currently undertaken an extensive rehabilitation project for the Skyway. According to Mr. Stan Kaderbek, a deputy commissioner and chief engineer for the Chicago Department of Transportation's Bureau of Transit and Bridges (quoted in Chicago Tribune on November 10, 2002), the overall objective of the rehabilitation program is to provide another 40 years of useful life for the Skyway. The second goal is to reduce overall maintenance costs by eliminating a lot of the Skyway bridge structure. According to Kaderbek half of the steel structures were eliminated and converted to a filled-in raised roadway of retained earth. By eliminating the steel structures, City reduced the costs of maintenance and inspections.

Life cycle and maintenance costs are also a consideration when MSE abutments are considered to reduce the bridge deck length. Reduction in the deck length will result not only in a reduction of the beam cost (reduced depth results in less expensive beams), but also in future maintenance costs.

Conclusion

MSE walls using modular facing panels, steel reinforcements and granular backfill are well suited for the construction of retaining walls and bridge abutments for America's highways, railways and waterways. It is acknowledged that the design of MSE walls should include a comprehensive geotechnical evaluation of the subsurface materials. During construction, monitoring and inspection should be used to verify the evaluations made during design and to modify the construction procedures based on the field data.

This paper provides a summary of the practical considerations that should be considered when these walls are evaluated for transportation applications. Knowledge of the various benefits and applications of these wall systems will enable a practicing geotechnical engineer to make a prudent choice based on the applicability of the wall types for the project under consideration. It will also help the engineer to make recommendations on the cost-effective use of MSE walls for a particular transportation project.

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MEGA-Trends Affecting The Geo-Community: What Does the Future Hold?

Ohio River Valley Soils Seminar: October 14, 2005
Jerry A. DiMaggio, Principal Geotechnical Engineer
U.S. Dept. of Transportation
Federal Highway Administration Wash. D.C.

Externally Driven “Mega-Trends” are beginning to affect the Geo-Community and it is becoming increasingly clear that our traditional organization and response vehicles may not be well suited to react to these changes.

MEGA-TRENDS

- 1. * *Accelerated Construction*
- 2. * *Innovative Contracting*
- 3. *Asset Management*
- 4. *Contact Sensitive Solutions*
- 5. *Safety*
- 6. *Risk Management*
- 7. *Cost Analysis*
- 8. * *Research Development Training*

INFLUENCE OF Mega-Trends

- These Trends have a strong and significant affect on our WORK: Scope, role in project development and delivery, fees, responsibility and authority, how work is performed and execution.

The Geo-Community has a significant stake in the extent and manner of implementation of these Mega-Trends.

ACCELERATED CONSTRUCTION

“Geotechnical features routinely are on a project’s critical path.”

- REAL INITIAL COST based on speed of construction.
- Recommend solutions that are speed friendly.
- Rediscover time in motion studies.
- Need greater acceptance of “quick” site investigation and Qa /Qc tools.
- Be more aware of overall projects features and construction sequence.

INNOVATIVE CONTRACTING

“Design Build will be my poster child”

NEED (MORE/SOME) GEOTECHNICAL INVOLVEMENT IN:

- Proposal development and evaluation.
- Warranty language.
- Site investigation.
- Design requirements and performance criteria.
- Quality Control and Assurance Programs.
- Contract Enforcement.

Research Development and Training

- Ever shrinking # and level of resources.
- Ever increasing # dedicated projects and programs.
- Increasing need for interdisciplinary solutions.
- Need to better combine basic and applied research tools.
- Ever increasing focus on technical training.

QUESTIONS

- Do we have a problem?
- If we do, then what should we do and what can we do?
- Who should do what, when and how?
- WHAT SHOULD I DO??? (BE INFORMED!
GET INVOLVED!)

Thank You For Your Attention Any Questions

THIS TOPIC WILL BE ADDRESSED AT AN
IMPORTANT NATIONAL CONFERENCE AT ACSE
HEADQUARTERS MAY 3 -5, 2006.

Have A GREAT DAY!

The Port Ivory Intermodal Facility Development and Site Improvements

On Staten Island, NY

Sudhir Sheth¹, M.Tech, M.CE, P.E.; Christopher Burke², Ph.D., P.E.; Peter Dunlop³, Ph.D., P.E.

ABSTRACT

The Port Ivory Intermodal rail facility is located adjacent to one of the most active container port terminals on the Eastern Seaboard, the Howland Hook Marine Terminal (HHMT), located on the Arthur Kill shipping channel on Staten Island, New York. The HHMT started operation in 1960 and is a vital facility for the shipment of containerized cargo through the Port of New York to East Coast and Midwest markets.

The Port Authority of New York and New Jersey (Port Authority) recently purchased Port Ivory, a 124-acre site, from the Proctor & Gamble Co. The facility was in operation from 1907 to 1991 mainly manufacturing soap products. A 37-acre parcel of the Port Ivory site known as Parcel 'A' is being developed as an intermodal facility, which will enable containers to be transferred directly from the HHMT to a freight rail line.

The present redevelopment program involves the construction of eleven rail tracks, rubber tired gantry crane (RTG) runways, pad areas, and roads. A simple span bridge over Bridge Creek connecting Parcel 'A' to the HHMT is also proposed. The total cost of developing the Port Ivory Parcel 'A' intermodal facility is estimated to be \$100 million.

The project site contains former lagoons (sedimentation basins) of lime sludge, which is a typical waste product of soap manufacturing. Compressible layers of organic clays and peats are present underneath the lime sludge, presenting a design challenge. This paper presents the results of a geotechnical investigation of the properties of lime sludge and compressible soils based on laboratory tests and field test fills. The interpretation of both the field data and lab data enabled the implementation of a rollover-preloading scheme with confidence to meet the short project construction schedule.

INTRODUCTION

The Howland Hook Marine Terminal and Port Ivory Intermodal Facility (under construction) are located on Staten Island, New York (Figure 1). The Arthur Kill, a major shipping channel provides container ship access to the HHMT. The HHMT is expected to experience significant future container cargo growth. To meet this increased demand, the facility is being upgraded to accommodate the next generation of deeper draft container ships, provide direct access to intermodal rail yard facilities and to expand upland container storage. Figure 2 shows the proposed development of Port Ivory Parcel

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‘A’ in relation to the existing HHMT.

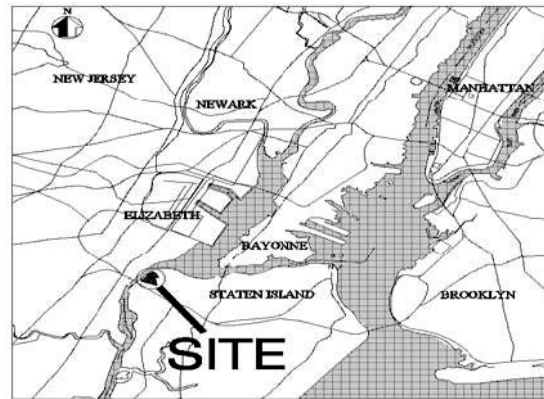


Figure 1 Site Location

Reference 1 describes the modifications and expansion of the marine terminal facility in detail. This paper will describe how the geotechnical challenges of subsurface conditions resulting from former industrial activity and underlying compressible soils were overcome. These challenges included constructing rail lines, rubber tired gentry (RTG) crane runway beams, and container storage areas with limited future secondary settlements.

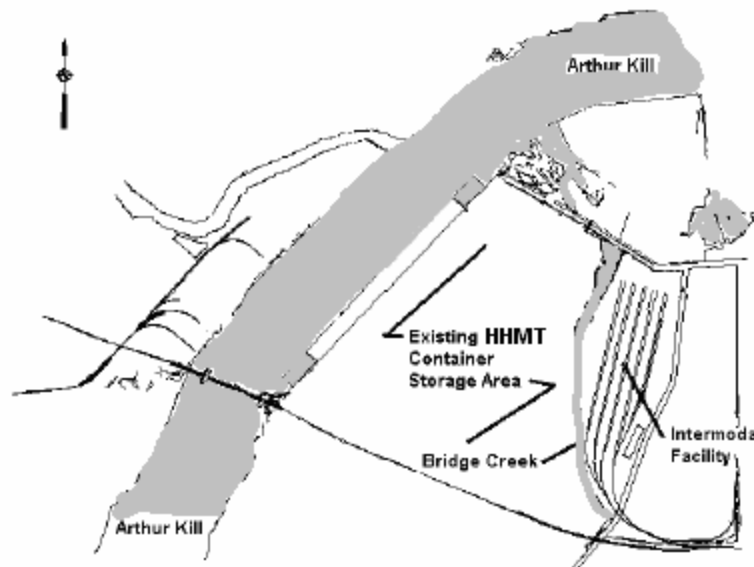


Figure 2 Howland Hook Marine Terminal (HHMT) Plan

The Port Ivory site has a total area of 124 acres, of which 62 acres are underlain by poor soils requiring improvement. Of the 37 acres of Port Ivory known as Parcel ‘A’, approximately 13 acres are underlain by lime sludge, peats and organic clays. Figure 3

shows the planned development of intermodal facility and areas of poor soils requiring improvement.

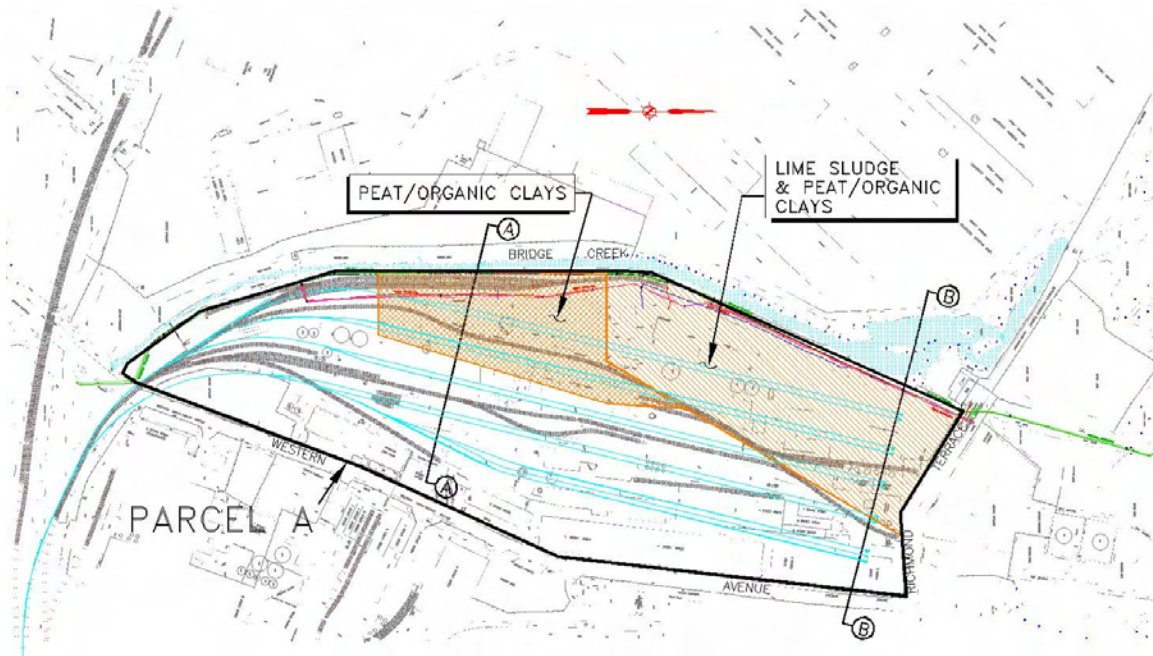


Figure 3 Areas of Poor Soils

This paper presents the geotechnical properties of lime sludge and compressible soils based on laboratory tests, field test fills, and initial results of the site construction. The interpretation of the field test data and monitoring during construction enabled the Port Authority to implement a rollover-preloading scheme with confidence and still meet the short project construction schedule.

The site, a prior industrial facility, contained former basins of lime sludge, a byproduct of soap manufacturing as well as underlying compressible layers of organic clays and peats. These poor soils were present over a significant portion of the site being developed for the intermodal rail yard. The compressibility of the lime sludge, organic clays and peats under the planned rail yard and container storage areas, required improvement to limit post-construction settlement.

Alternatives such as vacuum consolidation, preloading of compressible layers, stabilization with lime-cement, cement and stone columns to support anticipated rail-bed and container storage loadings were all considered. The feasibility of implementing vacuum consolidation over such an extensive area with variable soil conditions was immediately ruled out. An economic analysis, based on preliminary design, indicated that a conventional rollover surcharge scheme was the most cost effective improvement technique for this site.

PROJECT SCHEDULE

Five of the eleven rail tracks are to be operational by the end of 2005. To meet the anticipated increase in container volumes, tracks six and seven will be operational by 2007. The remaining four tracks will be constructed as future facility demands warrant.

SUBSURFACE CONDITIONS

The subsurface conditions are based on borings drilled within Parcel 'A' in 2002 and 2003. The soil conditions in Parcel 'A' vary, but typically consist of:

Sand Fill: The fill is composed of brown medium to fine sand with varying amounts of silt, gravel and cinders. The thickness of this layer ranges from 3 to 21 ft. and showed variable standard penetration test values.

Lime Sludge: Soft white lime sludge was encountered in the northwest area of the site. The material was detected either at the surface or overlain by 2 to 3 ft. of sand fill. The thickness of lime sludge varies from 3 to 18 ft.

Peats and Organic Clays: A tidal marsh deposit consisting of clay and peat underlies the fill and lime sludge. This material was found in the western and northern areas of the site. The thickness of this layer ranges from 5 to 11 ft. and disappears towards the east. Based on the standard penetration test, the material is soft to very soft.

Sand/Silty Sand: Medium dense to dense, coarse to fine sand/silty sand was encountered beneath the peats and organic clay in most of the borings. The thickness of this stratum ranges from 10 to 30 ft. This stratum was not encountered in the eastern portion of site.

Silts, Clays and Sands (Glacial Lake Deposits): The glacial lake deposits consist of red brown varved silty clays and clays with varying amounts of sand and gravel. The glacial lake deposits are over-consolidated and range in thickness from 8 to 32 ft.

Glacial Till: The glacial lake deposits are underlain by moraine deposits, which are primarily very stiff silty clays, mixed with sands and gravels. Decomposed bedrock fragments are also found within this stratum. The thickness of this layer ranges from 4 to 20 ft.

Bedrock: Some borings were drilled into red shale bedrock, which was encountered at a depth of 50 to 64 ft. below ground surface.

Figure 4 illustrates the stratigraphic relationship of these deposits.

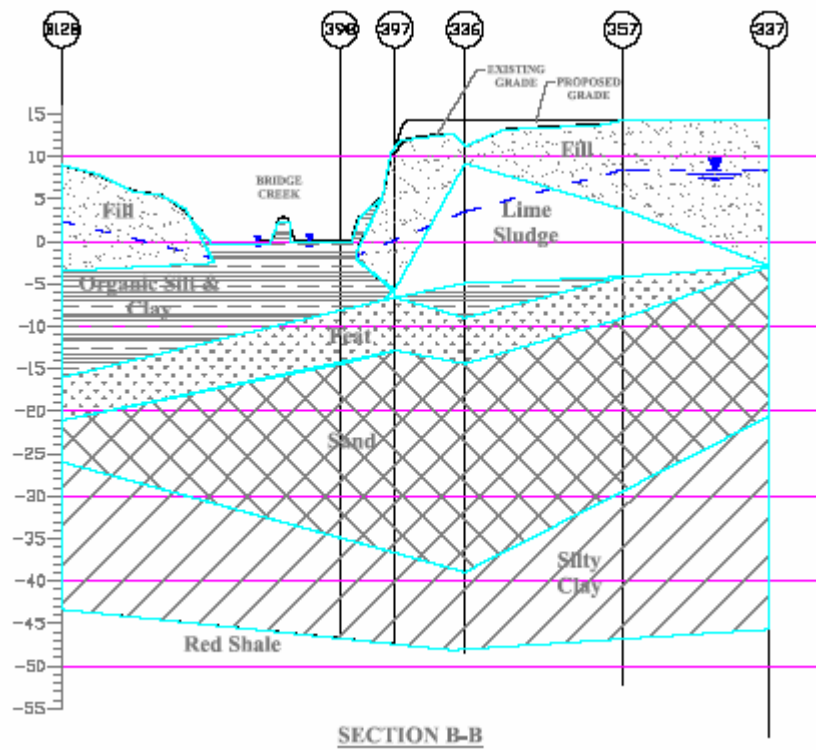
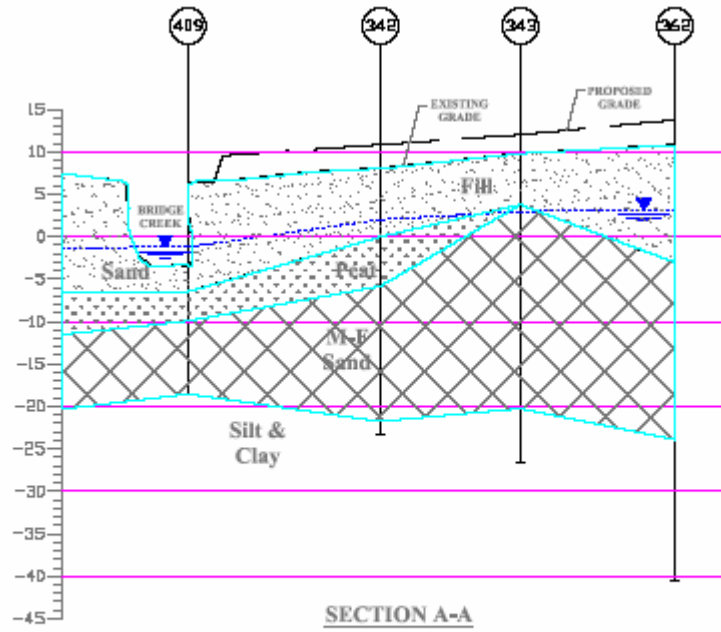


Figure 4 Geologic Sections

TEST SURCHARGE AREAS

An earlier study had been performed to determine the feasibility of developing this site as an intermodal facility. This study concluded that a maximum settlement of 30 inches would occur in the northwest area of the site under a container stack load of 600 pounds per square foot (psf) plus fill to grade. It was also estimated that 90% consolidation would take 2 years (using the $C_v = .02 \text{ ft}^2/\text{day}$ for the organic clays/peat layer).

Current plans call for approximately 4 ft. of fill in the northwestern portion of the site to reach proposed grade level (PGL). An additional design load for container stacking is specified as 1000 psf.

Settlement calculations were performed using a stratigraphy of 14 ft. of lime sludge and 8 ft. of peat/organic clays and using consolidation properties of normally consolidated organic clays at the HHMT site. Settlement of 9 and 19 inches were estimated for fill placement to PGL and loads imposed due to container stacking, respectively. Using an estimated value of $C_v = .05 \text{ ft}^2/\text{day}$, it would take about 18 months to achieve approximately 90% of settlement due to fill placement to PGL plus container loading.

In light of the large estimated settlements and time to complete primary settlement, along with the desire to expedite the project schedule for construction of the first five tracks, it was decided to construct test surcharge areas simultaneously with ongoing subsurface investigations and laboratory testing. These test fill areas were 75 ft. by 75 ft. in plan and 10 ft. high.

Test Area 1

Test area 1 was instrumented with two vertical extensometers and five settlement plates on the existing grade. Each extensometer had four spider magnets and a datum magnet. Extensometer #1 had three spider magnets in lime sludge and one in organic clays/peat, while extensometer #2 had two spider magnets in the lime sludge and two in the organic clays/peat. The surcharge was left in place for 260 days. Figure 5 shows the typical test surcharge and instrumentation in detail.

The test fill revealed that the consolidation of lime sludge was essentially immediate. Of the total settlement of 6 inches recorded at the center settlement plate, 2- to 2.5- inches was judged due to settlement of lime sludge, with the remainder due to the peat and organic clays. The thickness of lime sludge and organic clays was 16 ft. and 5 ft., respectively.

Test Area 2

Test area 2 was constructed at a location where borings revealed 6 ft. of compressible organic clays and peats overlain by 8 ft. of fill. Two spider magnets in one vertical extensometer and five settlement plates were installed. The surcharge was left in place for 270 days.

Total settlement of a little more than 8 inches was recorded. Data also showed that a significant amount of primary settlement was achieved.

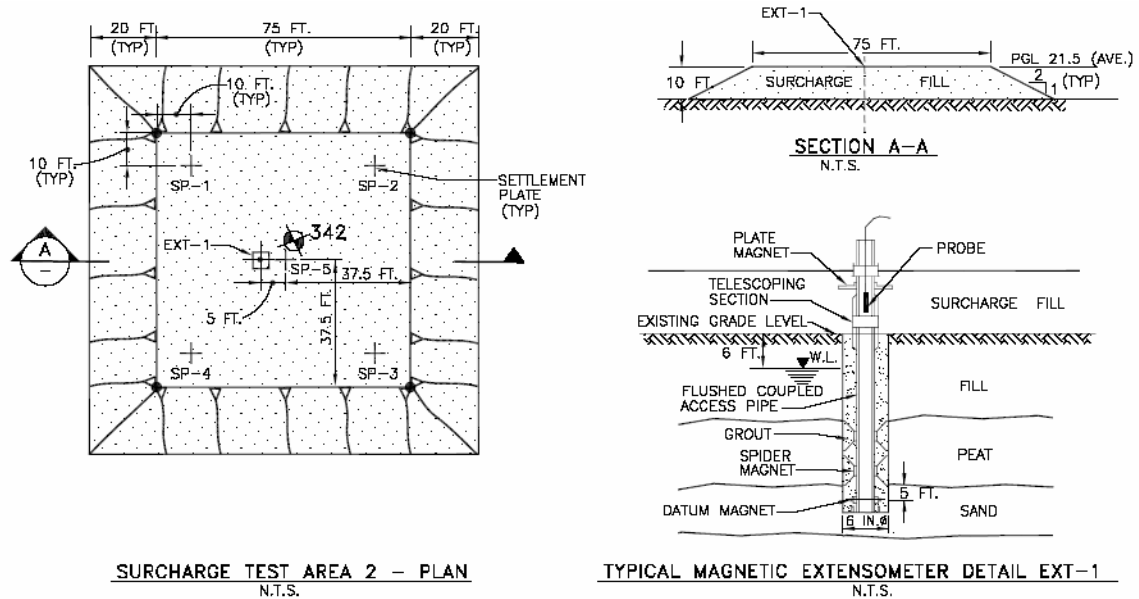


Figure 5 Test Area 2 Surcharge Details

COMPARISON OF LABORATORY AND FIELD DATA

Because of the desired project completion date, laboratory and field-testing programs were performed in parallel to investigate the compressibility of the peat, organic clay, and lime sludge.

One-dimensional consolidation and unconsolidated-undrained tests were performed on a number of representative peat and lime sludge samples. A total of 5 consolidation tests were carried out on the lime sludge, while 6 consolidation tests were carried out on the peat. The results indicated that the material is non-plastic and normally consolidated. The laboratory tests also revealed that lime sludge has a relatively high coefficient of consolidation (C_v) and insignificant rebound characteristics. Coefficient of consolidation values for the lime sludge averaged $0.671 \text{ ft}^2/\text{day}$, with values ranging from $0.165 \text{ ft}^2/\text{day}$ to $1.185 \text{ ft}^2/\text{day}$. The results for peat showed an average coefficient of consolidation of $0.0412 \text{ ft}^2/\text{day}$, with values ranging from $0.02 \text{ ft}^2/\text{day}$ to $0.14 \text{ ft}^2/\text{day}$. From the lab data it is evident that lime sludge consolidates much faster than peat and the field tests confirmed this. The square-root-of-time method was used to back-calculate the coefficient of consolidation from both spider magnet and settlement plate data. The coefficient of consolidation for the lime sludge and peat was determined to be $0.557 \text{ ft}^2/\text{day}$ and $0.017 \text{ ft}^2/\text{day}$, respectively.

Lab Test Results	Ave. C_v (ft ² /day) *	C_r	$C_{\alpha'}$
Lime Sludge	0.671	0.04 +	0.001 +
Peat	0.041	0.82 &	0.007 &

Field Test Results			
Lime Sludge	0.557	N/A	0 ^
Peat	0.017	N/A	0.083 ~

* Determined from Square-Root-of-Time Method

^ From Test Surcharge Area #1 Spider Magnets - Showed no appreciable secondary compression effects

~ From Test Surcharge Area #2 Settlement Plate Data

+ From Test Surcharge Area #1

& From Test Surcharge Area #1

Table 1 Comparison of Lab and Field Test Surcharge Data

The final surcharge areas in Phase 1A were instrumented with both settlement plates and horizontal inclinometers. The data were used to determine when the surcharge could be removed, as well as to draw a comparison between the predicted and the observed settlement. Using the time rate of consolidation relationships, the estimated duration of surcharge for both Phase 1A areas was 6 to 9 months. Maximum plate settlement in Surcharge Area #1 was approximately 9.5 inches, while the horizontal inclinometers showed approximately 10.5 inches in 327 days. In Surcharge Area #2, both the settlement plate and the horizontal inclinometer data showed a settlement of approximately 17 inches in 361 days. The surcharge duration for both areas was slightly longer than the estimated time of 6 to 9 months due to construction delays as well as the construction time to reach the full surcharge height. Data from the test surcharge areas and laboratory tests are presented in Table 1.

ROLLOVER SURCHARGE SCHEME

Higher rates of consolidation for lime sludge and organic clays, obtained from the concurrent lab and test surcharge program, were used to develop the rollover surcharge scheme for the intermodal yard. This rollover-preloading scheme is shown in Figure 6 and is being implemented currently. Early results indicate that the test surcharge results were representative of the whole compressible deposit. This rollover scheme will save the project approximately \$7 million, compared to a cost of \$13 million estimated for a single surcharge over the entire area.

To ensure speedy consolidation in the first phase of surcharge, wick drains were installed on 4 ft. centers. These areas are identified as Phase 1A in Figure 6. Based on a comparison of the settlement data from this first rollover phase in addition to the test area data, it was decided to construct the remaining surcharge areas without wick drains.

Another aspect of the surcharge program was the requirement to ensure sufficient

consolidation of all compressible soils. The RTG cranes, used to handle containers, have a tight differential settlement tolerance. Because of a potential for irregular geometry of lime sludge lagoons and variable thickness of organic clay and peat deposits, it was decided to place horizontal slope inclinometers along with settlement plates to monitor settlement.

Both the settlement plate data and horizontal inclinometer data from Phase 1A are shown in Figures 7 and 8, respectively. The Phase 1A horizontal inclinometer data show the pockets of largest settlement. The preloading of the area identified as Phase 1B North in Figure 6 was recently completed in February 2005. The data from settlement plate readings and horizontal inclinometer are presented in Figure 9 and 10 for comparison with the data from Phase 1A.

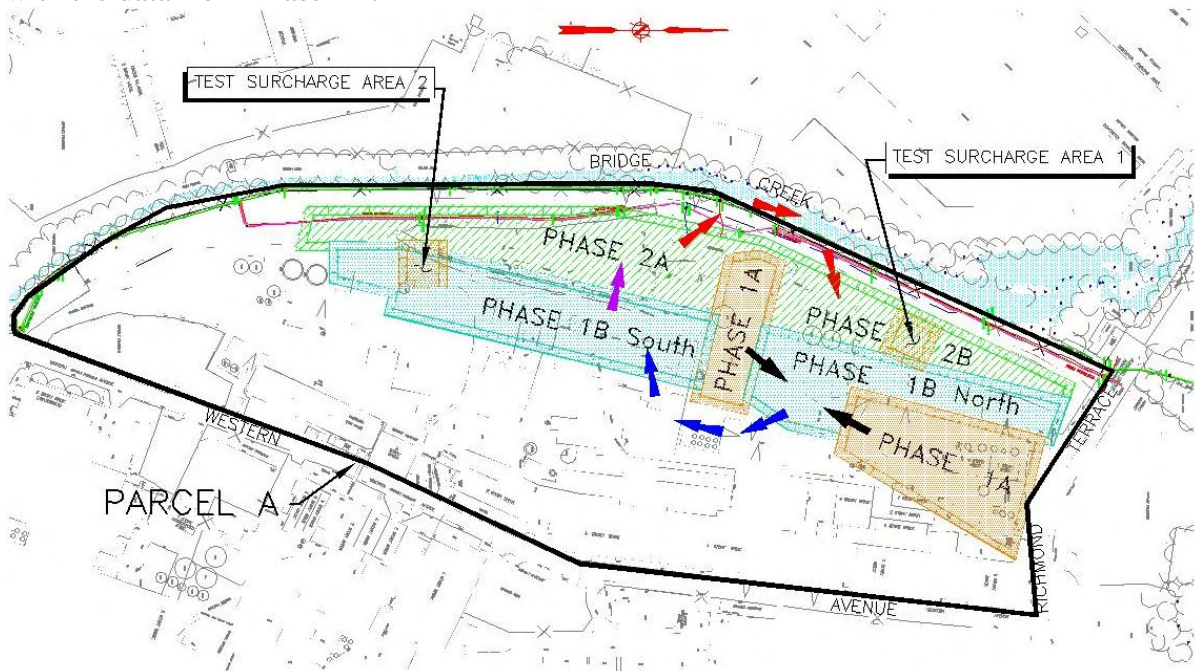


Figure 6 Surcharge Rollover Scheme

ESTIMATES OF LONG TERM AND DIFFERENTIAL SETTLEMENT

Once the selection of RTG crane type and preliminary design of RTG runway beams were completed, the following criteria were established.

- a. Eight wheeled Liebherr RTG with average wheel load of 60 kips
- b. Thickness of RTG runway beam of 39 in. (based on CBR of 5)
- c. Average soil pressure at the bottom of beam of 850 psf
- d. Allowable differential settlement of 2 in. between crane wheel bogies (30 ft. apart)

Long-term settlement and differential settlement due to secondary compression of

peat/organic clays on the operation of RTG cranes was estimated for a service life of 20 years. The stress imposed on compressible soils by the loaded RTG on the beams was estimated to be 850 psf, which was less than the 1200 psf surcharge stress.

For a service life of 20 years, a maximum settlement due to secondary compression was estimated to be 1.38 inches using a secondary compression index (C_{α}) of 0.007 obtained from laboratory test data. From Rollover Phase 1A area #2 field settlement plate records, a value of C_{α} of 0.083 to 0.087 was estimated, which is significantly higher than the lab test data. This value resulted in a secondary settlement of 2.5 to 3 inches for a 5 ft. thick layer of compressible soils. Using the settlement and horizontal slope inclinometer records from the Phase 1A surcharge area, the differential settlement due to RTG loading on a 30 ft. bogie span over runway beams was estimated to be 3/8- to 1/2-inch.

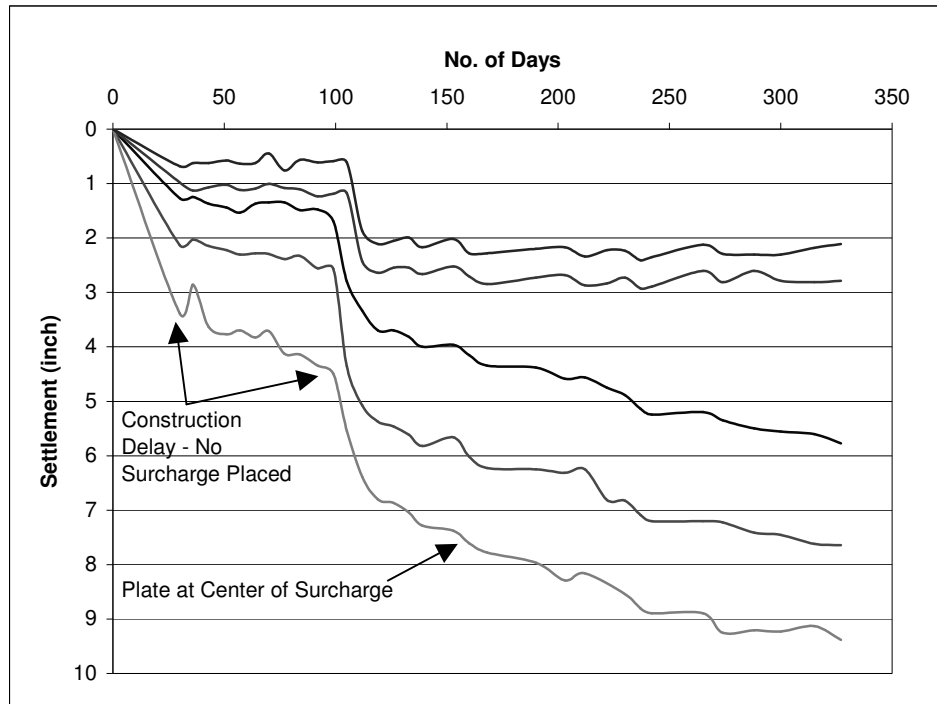


Figure 7 Phase 1A Settlement Plate Data

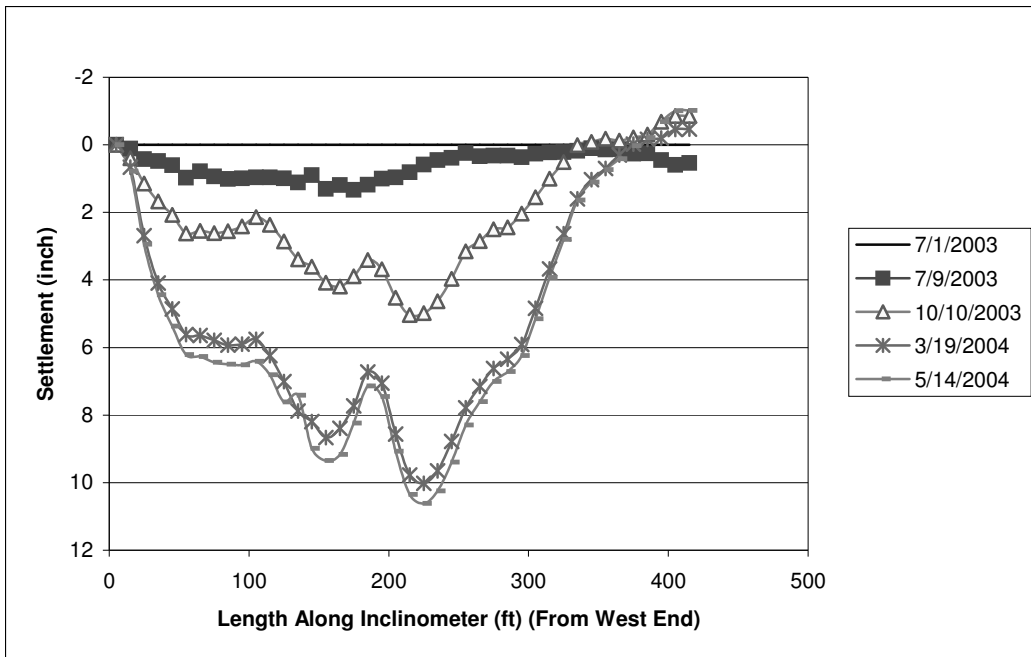


Figure 8 Phase 1A Horizontal Inclinerometer Data

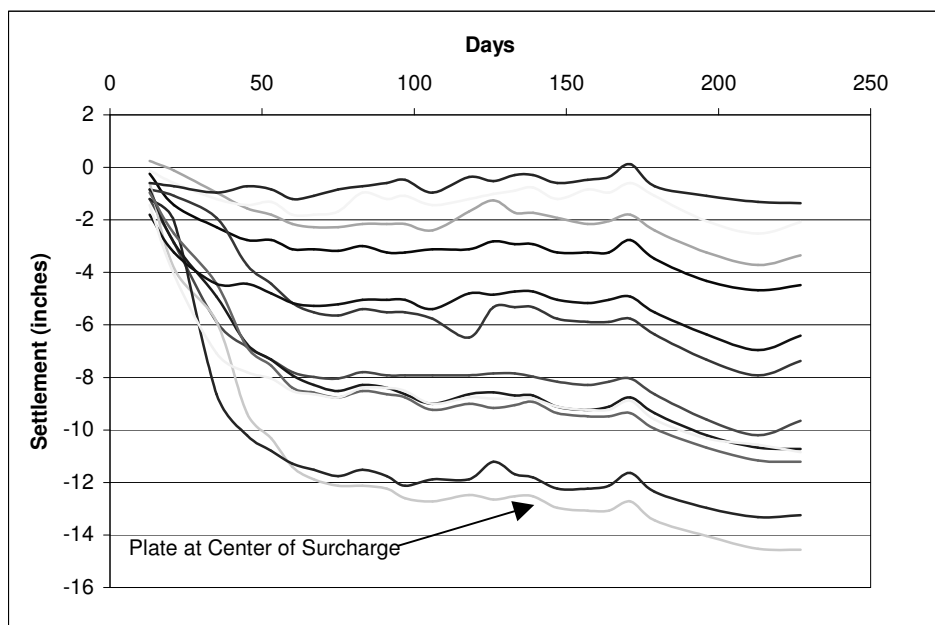


Figure 9 Phase 1B North Settlement Plate Data

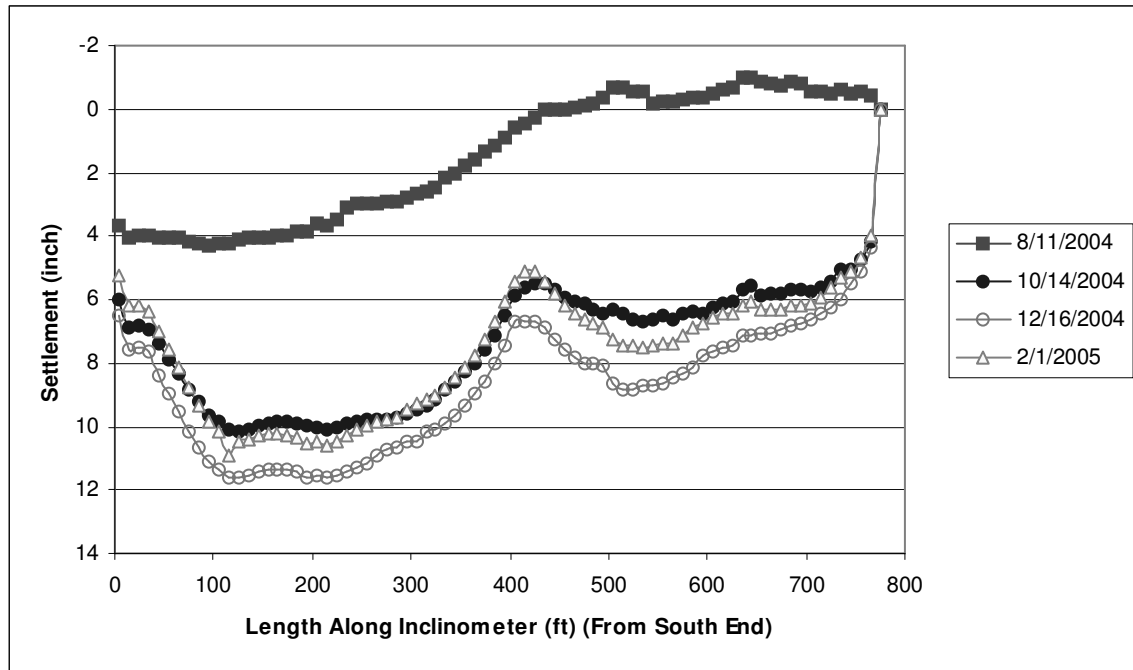


Figure 10 Phase 1B North Horizontal Inclinometer Data

Since the runway beams were designed as structural members using beam on an elastic foundation, it is judged that they will undergo differential movements less than 3/8- to 1/2-inch throughout the service life. This is within the gantry crane tolerance.

LOAD PAD AREAS ASPHALT PAVEMENT

The load pad areas between tracks are used for the movement of trucks, reach stackers, and other heavy equipment for handling containers. A flexible pavement design was selected for the load pad area. An initial conservative CBR value of five (5) was estimated prior to the surcharging. This low CBR value required a design with geogrids, a reinforced sub-base and thicker base and subbase layers. The total thickness for flexible pavement design in the load pad area was 39 inches.

As part of the project, a series of field CBR and Dynamic Cone Penetrometer (DCP) tests were performed to investigate the condition of the proposed grade both in areas with and without surcharge. As a result of the nature of the lime sludge and its proximity to the proposed ground surface, there was a concern about the bearing capacity under the pavement section. The proposed pavement section is 39 inches below the proposed grade, so CBR and DCP tests were performed at both the PGL and 39 inches below the PGL. These CBR values (covered in a later section) show that most are significantly higher than five. Based on these data, it was possible to eliminate the need for geogrid reinforcement resulting in savings of time, material and construction cost.

CBR TESTING

In addition to consolidation issues, the project required CBR data for load pad design. Therefore a number of DCP and CBR tests were performed. Results of the DCP testing in areas with no surcharging indicated CBR values ranging from 6 to 100, with an average of 26. Only five tests predicted a CBR below 10, and these were in areas where buildings had been demolished and the basements had been backfilled. The results for the field CBR tests in Surcharge Area #1 after the surcharge was removed indicated an average CBR value of 47 at the surface and 35 at a depth of 39 inches. In Surcharge Area #2, the average CBR value was 12 at the surface, and 36 at a depth of 39 inches. The results for the DCP tests in Surcharge Area #1 after the surcharge was removed indicated an average CBR value of 28 at the surface and 58 at a depth of 39 inches. In Surcharge Area #2, the average CBR value as determined by the DCP test was 19 at the surface, and 80 at a depth of 39 inches. These data show that the initial estimated design CBR value of 5 was very conservative and a value of 10 was easily achievable.

The use of DCP testing to predict CBR values provided a marked time savings compared to field CBR testing. From the results of the two types of tests, it appears that the DCP at PGL test gave a slightly different, but comparable CBR value to the field CBR test. However, at depths below the PGL, the DCP test gave a generally higher CBR value compared to the field CBR test. This is most likely due to the differing confining stress in each test. In the DCP test, the rod extends down with each drop of the hammer and is surrounded by the overburden. However, in the field CBR test, a hole is dug and a surcharge weight of 20 lbs. is placed within the hole. Although there was a difference in the values obtained by the two tests, both tests showed that a design CBR value of 10 was easily achievable.

SUMMARY

Significant portions of the Port Ivory Parcel 'A' site have 10 to 15 ft. of lime sludge deposits overlying approximately 4 to 6 ft. of compressible soils. The site was difficult to develop for use as an intermodal facility under an expedited project schedule, due to the undefined soil properties of the lime sludge.

The construction of test surcharge areas concurrently with subsurface investigations and laboratory testing was instrumental in developing a rollover preloading scheme. The implementation of this scheme resulted in significant cost savings (approximately \$7 million) while still meeting the expedited schedule.

The surcharging of poor soil areas resulted in an increased CBR, which was used for pavement designs. The use of a higher CBR value also resulted in elimination of the geogrid soil beam from the pavement design. This provided savings to the overall construction cost of the project.

In conclusion it can be stated that constructing and monitoring test surcharge areas concurrently with laboratory testing resulted in implementation of a rollover surcharge

scheme, which provided significant cost savings.

ACKNOWLEDGEMENTS

The authors are appreciative of the constructive comments provided by Raymond Sandiford, Chief Geotechnical Engineer and Samuel Leifer, Assistant Chief Geotechnical Engineer throughout this project and in the preparation of this paper.

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TAPERTUBE PILES IN SANDY SOILS

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Abstract

More than 1200 tapertube piles have been driven for New York City Transit Authority's Grand Avenue Bus Depot and Central Maintenance Facility. This project involved construction of a multistory bus depot structure on a site approximately 370 feet wide and 540 feet long. The general subsurface stratigraphy typically encountered in the borings consisted of fill overlying a natural sandy soil or clayey soil deposit. In general, a 5-foot thick layer of organic silt and peat was found directly beneath the fill throughout the project site. Boulders were also encountered in isolated locations. Pile capacities and pile lengths were estimated based on subsurface conditions. A field test program consisting of five static load tests and fifteen dynamic tests using a pile driving analyzer (PDA) was developed to verify the pile capacities and to establish a pile driving criteria. During pile driving, the pile capacities could not be achieved during initial driving due to build up of excess pore water pressure. Therefore, the piles were driven to the theoretical estimated depth and sufficient time was allowed for the pore water pressure to dissipate before restriking the piles. The pile capacities were then verified by static and dynamic load tests. During pile driving, a significant amount of piles were damaged due to the presence of boulders. Partially concrete filled or fully concreted piles were utilized to increase the stiffness of the piles and to avoid further damage during pile driving. The pile driving criteria was revised to accommodate the concrete filled piles with criteria verified by dynamic testing. Ground vibrations during pile driving were also monitored. Several adjacent existing buildings founded on spread footings within 5 feet from pile driving were vulnerable to damage during pile driving. Special measurements such as sheet pile installation and pre-augering to 25 feet were utilized to reduce the vibration. Vibration measurement records indicated that the peak particle velocities measured during the pile driving were within the limiting peak particle velocity. This paper presents the results of the pile test program correspond to predicted values and vibration monitoring and control due to pile driving.

Key words: Pile, PDA, CAPWAP, Set-up, Vibration Monitoring.

RECENT DEVELOPMENT IN PILE CAPACITY

After driving, pile capacity increases with time. This time-dependent capacity is known as pile set-up. Pile set-up can contribute significantly to long-term pile capacity. The exact mechanism by which this occurs is not fully understood, but the majority of set-up is likely related to the dissipation of excess pore pressure within soil that is displaced and disturbed as the pile is driven and subsequent remolding and reconsolidation of that soil. Additional set-up may occur due to aging. Set-up rate and magnitudes are functions of several variables such as pile type, soil type, pile size, etc. Set-up is recognized as occurring for virtually all driven pile types, in organic and inorganic saturated clay, and loose to medium dense silt, sandy silt, silty sand, and fine sand, and is related to both soil and pile properties. Depending on soil type, set-up has been demonstrated to account for increases of up to 12 times the initial end-of-drive pile capacity.

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Pile set-up is characterized by dynamic monitoring using a Pile Driving AnalyzerTM (PDA) during initial driving, and restrike testing several days to a few weeks after initial driving. Subsequent CAsE Pile Wave Analysis ProgramTM (CAPWAP) analysis of the PDA data allows assessing set-up distribution along the pile shaft.

Set-up is predominately associated with an increase in soil resistance acting on the sides (shaft) of a pile (Komurka 2004, Tan et al 2004, Stevens 2004, Fellenius and Altaee 2002, Chow et al 1998). However, decrease in tip resistance is observed in few occurrences (Fellenius 2002, Svinkin 2004). Komura and Wagner (2003) suggested that depending on soil permeability and amount of disturbance, dissipation of excess porewater pressures is non-uniform (non-linear) with respect to the log of time for some time after driving. Subsequently, excess porewater pressure dissipation becomes uniform (linear) with respect to the log of time. After excess porewater pressures have dissipated, and independent of effective stress, additional set-up may occur due to aging. A number of empirical relationships have been proposed to estimate or predict pile setup (Axelsson 2002, Tan et al 2004), and have demonstrated reasonable success (accuracy) in a number of studies.

PROJECT DESCRIPTION

The bus depot and central maintenance facility is located in the Maspeth section of Queens, New York at the intersection of Grand Avenue and 49th Street. It is near the upper reach of the Newton Creek, which forms part of the boundary between the Boroughs of Brooklyn and Queens. The site is relatively level with most of the land paved for parking lots.

The site topography gently slopes to the east. The ground surface on the site is between approximate Elevation 108 and 114 feet.

Geological Information

The project site is located in the Coastal Plain Province and appears to be within an area of a former glacial lakebed. Geologic mapping indicates that bedrock consists of igneous and metamorphic rocks varying from granites to schists, which are typically considered hard rocks. USGS maps indicate bedrock depth in the area varying from 200 to 250 feet below mean sea level. Aside from seismic considerations the influence of the bedrock on the behavior of the surface structures is considered minimal. The soils overlying bedrock consist of interbedded sands and clays overlain with interglacial and more recently deposited glacial outwash sands and tills. The shore areas have been filled in as harbor development occurred in the Creek vicinity.

Subsurface Conditions

A number of borings have been drilled throughout the site to investigate the subsurface conditions. Boring locations are shown in Figure 1. Fill material was encountered in all the borings performed for this project. The fill thickness varies from 13 to 23 feet. The fill predominantly consists of sand, gravel, brick, asphalt and wood. Standard Penetration Test Blow Counts (SPT N values) ranged from weight of hammer (WOH) to over 100 blows per foot (bpf). A Peat and Organic Silt layer was encountered in most of the borings beneath the fill strata. The

thickness of the layer ranged from 1 to 8 feet. SPT N values for the peat layer ranged from WOH to 22 bpf. In general the peat layer is interspersed with layers of sand, clay and silty clay. Underneath the peat/organic layer, a brown to gray, coarse to fine sand layer with varying amount of silt and clayey silt is present. The depth of the sand layer varies from approximately 25 to 75 feet below the ground surface. However, a few borings at the South corner of the building site encountered a clay and clayey silt layer from 38 to 75 feet below the ground surface. The SPT N-values for the sand layer ranged from 4 to over 100 bpf and the clayey silt and clay layer ranged from 16 to 45 bpf.

FOUNDATION RECOMMENDATIONS

Steel Tapertube piles were utilized to support the structure. The recommended Tapertube pile dimensions are as follows: a butt diameter of 18-inches, a tip diameter of 8-inches, and a 20-foot long tapered section. The piles were fabricated using Grade 50 steel with a wall thickness of 0.344-inch for the tapered section and a 0.375-inch for the remaining portion of the pile. Due to presence of peat the piles will experience negative skin friction above the elevation of 85 feet. The estimated negative skin friction is 20 kips.

Except for the South corner of the structure, an allowable axial capacity of 300 kips used for the design. Thus, the corresponding ultimate axial driving capacity is 640 kips. Using Nordlund (1963, 1979) Method the pile lengths were estimated for the required axial capacity of 640 kips. For the South corner of the structure, an allowable axial capacity of 200 kips were used for the design. Thus, the corresponding ultimate axial driving capacity is 440 kips. The estimated pile lengths vary depending on the fines content in the granular material and are shown in Figure 1. The actual pile tip elevations were determined later during the pile static and dynamic load testing.

PILE TEST PROGRAM

As the site soils are relatively uniform, the majority of the piles will be bearing within the sand layer except at the South corner of the structure where the piles will bear in clay. Dynamic and static load testing (ASTMD4945 and ASTM D1143) in accordance with the New York City Building Code and the New York State Building Code will be performed to verify design capacity of the piles. Two wave equation analyses were performed to determine the pile and hammer compatibility during driving. Based on wave equation results Junttan HHK-7 hammer (Hammer energy of 61.9 kips-ft. and Maximum Stroke of 48 inches) was selected to drive all the piles. In order to limit overstressing the piles the stroke lengths were reduce to 39 inches and 36 inches for piles bearing on sand stratum and clayey silt stratum, respectively.

A total of fifteen Tapertube test piles (TP-1 to TP-15) were driven across the site. All the test piles (TP-1 to TP-15) were subjected to dynamic testing using a pile driving analyzer (PDA). In addition five test piles were subjected to static load testing. It is recommended that three of the test piles (TP-4, TP-10 and TP-15) bear in the sand stratum and two of the test piles (TP-1 and TP-2) bear in the clay stratum. The locations of the test piles are shown in Figure 1. During pile driving, the pile capacities could not be achieved during initial driving due to build up of excess pore water pressure. Therefore, the piles were driven to the theoretical estimated depth and

sufficient time was allowed for the pore water pressure to dissipate before restriking the piles. The results are summarized in Table 1. The subsurface profiles, pile driving records, PDA results, CAPWAP results, and load test results are presented as Figures 2 and 3. There is a shift in shaft resistance distribution, obtained using CAPWAP, noted between initial drive and restrike. It could be due to signal mismatching.

Dynamic Tests

The two test piles, TP-1 and TP-2, in the clayey silt area were driven to a driving resistance of 19 blows per foot. These piles were driven to a depth of 52 and 59 feet below the existing ground surface. The PDA results show that the piles reached the required capacity of 440 kips during the initial drive. These piles were subjected to restrike. During the restrike of these piles the penetration resistances for the first two consecutive inches were 3 blows per inch and the CAPWAP capacity was 580 kips.

The remaining 13 test piles, TP-3 to TP-15, bearing in the sand stratum were driven to a depth of 37 to 63 feet below existing ground surface. Test Pile TP-12 was driven to 37 feet with a penetration resistance of 33 blows in the last foot of penetration. The PDA test showed that the pile had a capacity of 660 kips. Remaining piles did not reach the capacity during the initial drive. Depth of these piles varied from 44 to 59 feet below the existing ground surface and were driven to penetration resistance varying from 21 to 25 blows in the last foot of penetration. Test Piles with a penetration depth of 60 feet or greater were driven to a penetration resistance of 20 blows in the last foot. During the restrike of these piles the penetration resistances for the first two consecutive inches vary from 3 to 6 blows in the first inch and 2 to 4 blows in the second inch. The CAPWAP pile capacity during restrike varied from 670 kips to 780 kips.

Static Load Tests

After obtaining the capacity based on the PDA test, three test piles (TP-4, TP-10, and TP-12) bearing in the sand stratum and two test piles (TP-1 and TP-2) bearing in the clay stratum were subjected to static load testing. All the piles were initially loaded to 50% of the design load and then loaded up to 200% of the design load by incrementing with 25% of the design load. For the test piles bearing in the sand stratum (TP-4, TP-10, and TP-12) the ultimate load (640 kips) remained in place for a minimum of 48 hours. For the test pile bearing in the clay stratum (TP-1 and TP-2) the ultimate load (440 kips) remained in place for not less than 96 hours. After the ultimate loads remained in place for 48 hours or 96 hours the piles were unloaded with a decrement of 25% of the design load. The final dial gauge reading was taken after 24 hours. All these test piles were then reloaded to 150% of the design load and unloaded.

Production Pile Driving Criteria

Based on the PDA, CAPWAP, Static Load Tests, pile driving logs, and wave equation analysis the following production pile driving criteria was established:

Pile Bearing Stratum	Pile Penetration Depth (ft.)	Hammer Blows (Blows/ft.)
Sand	< 45	36
	45 to 59	24
	>60	20
Clay	>52	19

PILE SET-UP

All the test piles were subjected to restrike after one to four days from initial drive. Percent increase in pile capacity and increase in hammer blows with time are plotted as shown in Figure 4. The results show that the restrike capacity increased by at least 20 percent of the capacity of initial drive. Similarly the hammer blows also increased by at least 50 percent. All the piles subjected to restrike during production pile driving also included in Figure 4. Based on the Figure 4, one could conclude that the 50 percent increase in hammer blows can provide at least 20 percent more capacity than the initial drive. More data from various sites needed to formulate these figures. Such a database can be very helpful in estimating pile capacities for small projects where the dynamic tests cost significantly compared to the project cost.

DAMAGED PILES

During production pile driving, pile damage was observed at different locations across the site. In order to avoid pile damage the pile was driven to about 21 feet, cut the pile at about 25 feet, filled the driven pile with concrete, welded the extension pipe, waited to achieve a concrete strength of 4,000 psi, and drove the pile to reach the desired capacity. Three partially concreted piles were subjected to PDA testing and the driving criteria were revised based on PDA and CAPWAP results.

Some of the partially filled concrete piles exhibited localized buckling of the steel immediately above the top of the concrete. To prevent similar damage to additional piles, a total of fourteen replacement piles and production piles were completely filled with concrete prior to final driving. The piles were initially driven to a penetration resistance of 15 bpf prior to PDA testing.

VIBRATION MONITORING

There are a few structures adjacent to the project site, which have vulnerability to damage due to ground vibration, were monitored for vibration levels during the pile driving operations. Prior to the start of construction activities vibration-monitoring equipment was installed and ambient background level of vibrations were measured. Ground vibrations were recorded during test pile program by installing four sensors at distances 10, 15, 25 and 35 feet from the pile. Peak particle velocities were recorded and utilized to find out the limiting distance where vibrations approach alert level. Using the measured peak particle velocities an attenuation curve was generated as shown in Figure 5. The vibration monitoring further continued during production pile driving where the piles were close to the adjacent buildings. These field data were compared with Woods and Jedge's (1985) energy-attenuation curves and also shown in Figure 5.

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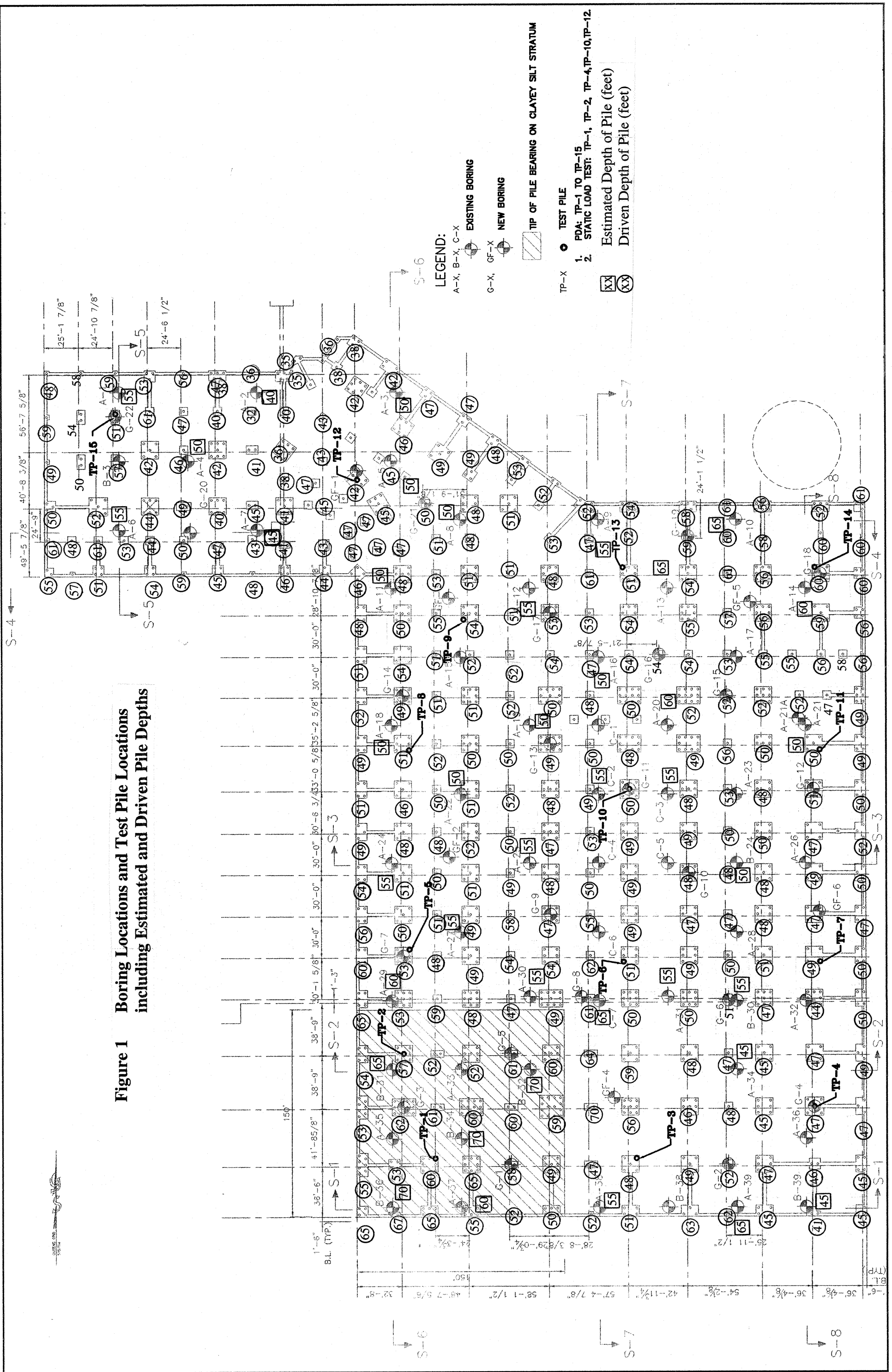
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TABLE 1 SUMMARY OF PILE DYNAMIC TESTS

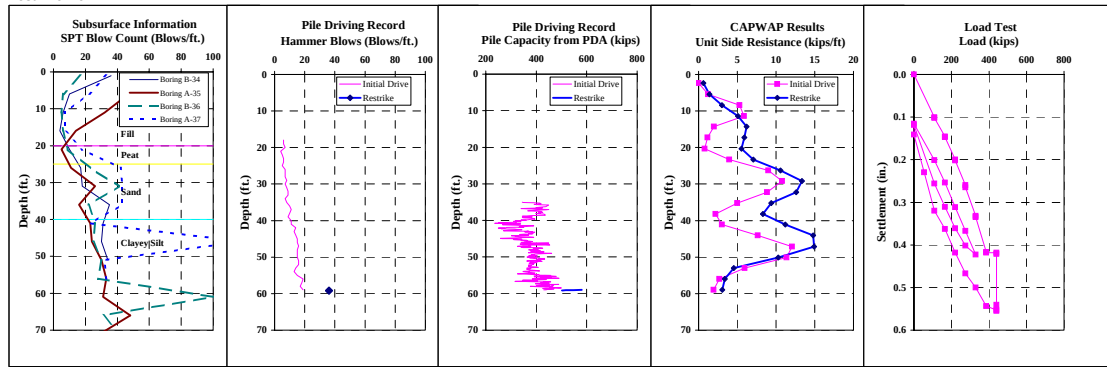
Pile No.	Penetration Depth (ft.)	Initial Drive			Hammer Blows (bpf)	Restrike			Pile Set-up (days)	Smith Damping Factor (s/ft.)				Quake Factor (in.)				Case Damping Factor (s/ft.)																									
		Hammer Blows (bpf)	Capacity (kips)			Hammer Blows (bpf)	Capacity (kips)			Toe	Total	Shaft		Toe		Shaft		Toe		Shaft		Toe																					
			Shaft	Toe			Total	Shaft				Toe	Initial	Restrike	Initial	Restrike	Initial	Restrike	Initial	Restrike	Initial	Restrike	Initial	Restrike																			
Test Piles																																											
TPP-1 (772)	55	19	300	131	431	36	450	130	581	2	0.09	0.08	0.05	0.05	0.1	0.1	0.5	0.1	0.728	0.969	0.175	0.175	0.175	0.175																			
TPP-2 (825)	48	19	280	170	450	36	411	170	581	2	0.08	0.1	0.05	0.05	0.1	0.1	0.6	0.1	0.603	1.099	0.229	0.232	0.232	0.232																			
TPP-3 (351)	59	20	431	160	591	30	530	160	690	3	0.09	0.08	0.05	0.05	0.1	0.1	0.2	0.1	1.042	1.142	0.216	0.216	0.216	0.216																			
TPP-4 (65)	40	23	350	180	531	54	530	180	710	4	0.11	0.08	0.07	0.05	0.1	0.1	0.4	0.1	1.035	1.140	0.356	0.244	0.244	0.244																			
TPP-5 (739)	51	22	290	210	501	36	570	210	780	2	0.05	0.08	0.05	0.05	0.1	0.1	0.7	0.1	0.391	1.228	0.272	0.283	0.283	0.283																			
TPP-6 (381)	53	23	371	200	571	36	500	201	701	3	0.07	0.07	0.05	0.07	0.1	0.1	0.4	0.1	0.697	0.944	0.269	0.378	0.378	0.378																			
TPP-7 (80)	47	23	330	210	540	36	500	210	710	4	0.08	0.08	0.10	0.05	0.1	0.1	0.1	0.1	0.711	1.081	0.566	0.280	0.280	0.280																			
TPP-8 (865)	50	25	340	210	551	42	570	210	781	4	0.08	0.08	0.05	0.05	0.1	0.1	0.1	0.1	0.733	1.227	0.283	0.283	0.283	0.283																			
TPP-9 (840)	51	22	340	220	560	48	520	220	740	1	0.00	0.08	0.05	0.05	0.1	0.1	0.5	0.1	0.641	1.121	0.296	0.297	0.297	0.297																			
TPP-10 (405)	42	21	340	211	551	36	520	210	731	3	0.06	0.08	0.05	0.05	0.1	0.1	0.3	0.1	0.542	1.116	0.289	0.286	0.286	0.286																			
TPP-11 (100)	47	25	281	250	531	54	530	250	780	4	0.10	0.08	0.10	0.05	0.1	0.1	0.3	0.1	0.762	1.133	0.667	0.343	0.343	0.343																			
TPP-12	33	33	200	431	631	30	270	430	700	1	0.08	0.05	0.05	0.03	0.1	0.1	0.3	0.3	0.432	0.364	0.579	0.347	0.347	0.347																			
TPP-13 (446)	50	24	340	191	531	54	481	190	671	3	0.06	0.07	0.05	0.07	0.1	0.1	0.4	0.1	0.549	0.906	0.256	0.358	0.358	0.358																			
TPP-14 (119)	57	21	280	240	521	54	530	240	770	3	0.09	0.08	0.05	0.05	0.1	0.1	0.2	0.1	0.667	1.142	0.324	0.323	0.323	0.323																			
TPP-15 (1121)	56	20	370	200	570	30	490	200	690	1	0.08	0.08	0.05	0.05	0.1	0.1	0.4	0.1	0.798	1.056	0.269	0.269	0.269	0.269																			
Partially Concrete Piles: Drove piles to about 20 feet, filled concrete up to about 25 feet, remove the piles.																																											
1101	53	12				30	470	180	650		0.07			0.05		0.1		0.4		0.887				0.242																			
1117	53	24				36	440	300	740		0.08			0.10		0.1		0.2		0.949				0.808																			
1133	45	19				36	380	300	680		0.10			0.10		0.1		0.2		0.969				0.863																			
1138	57	16				72	380	300	680		0.10			0.10		0.1		0.3		1.023				0.808																			
Fully Concrete Piles: Drove piles to about 20 feet, filled concrete up to the top, remove the piles.																																											
1101.1	44					36	600	300	900		0.30			0.05		0.1		0.3		1.149				0.096																			
1102	43					72	570	330	900		0.25			0.15		0.1		0.2		0.910				0.316																			
1103	44					120	520	350	870		0.14			0.15		0.1		0.2		0.473				0.341																			
1104	42					120	480	330	810		0.20			0.05		0.1		0.4		0.613				0.105																			
1105	40					24	280	400	680		0.20			0.05		0.1		0.4		0.357				0.128																			
1106	47					120	550	300	850		0.25			0.15		0.2		0.4		0.878				0.287																			
1107	46					22	350	350	700		0.13			0.06		0.1		0.5		0.291				0.134																			
1125.1	39					20	300	360	660		0.10			0.05		0.1		0.7		0.192				0.115																			
1128	50					20	500	180	680		0.20			0.10		0.1		0.4		0.638				0.115																			
1131	46					36	380	360	740		0.25			0.05		0.1		0.2		0.607				0.115																			
1132	50					54	580	160	740		0.15			0.05		0.1		0.4		0.564				0.048																			
1139.1	47					48	420	280	700		0.20			0.10		0.1		0.5		0.537				0.179																			
1143.1	42					36	480	180	660		0.15			0.50		0.1		0.5		0.459				0.172																			
1144.1	53					72	560	190	750		0.20			0.05		0.1		0.3		0.715				0.061																			



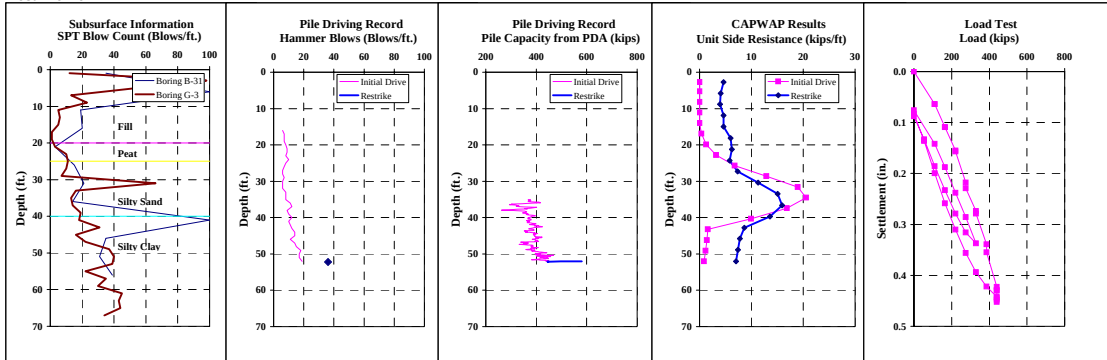
Figure 1 Boring Locations and Test Pile Locations
including Estimated and Driven Pile Depths



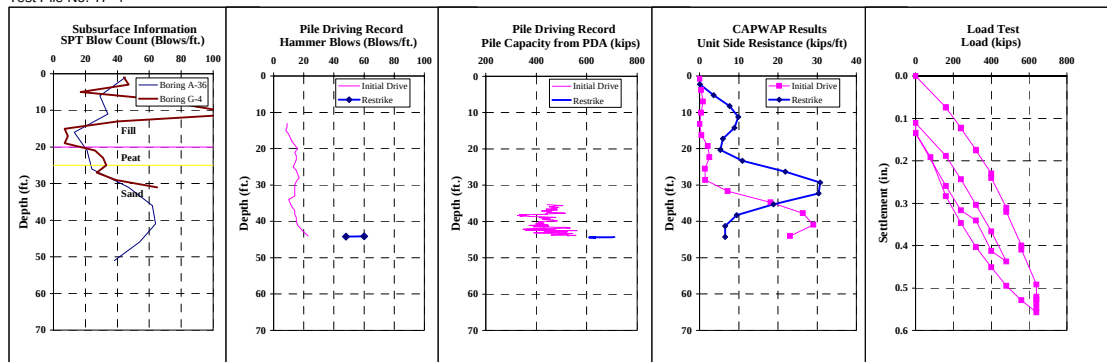
Test Pile No. TP-1



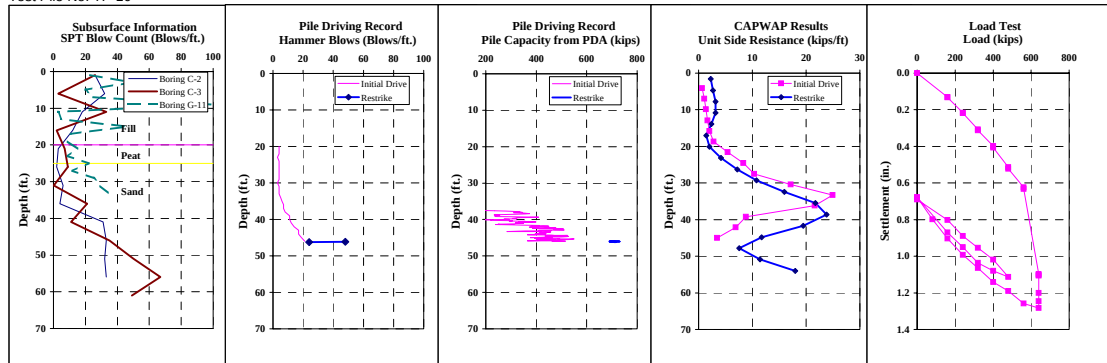
Test Pile No. TP-2



Test Pile No. TP-4



Test Pile No. TP-10



Test Pile No. TP-12

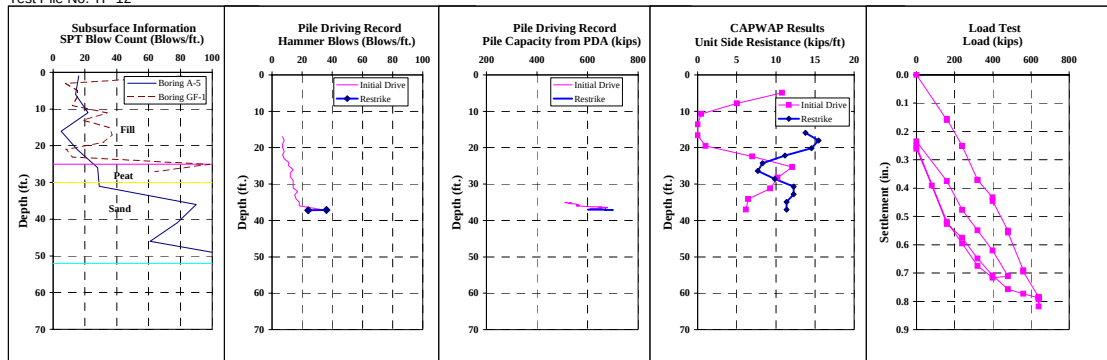


FIGURE 2 PILE TEST RESULTS

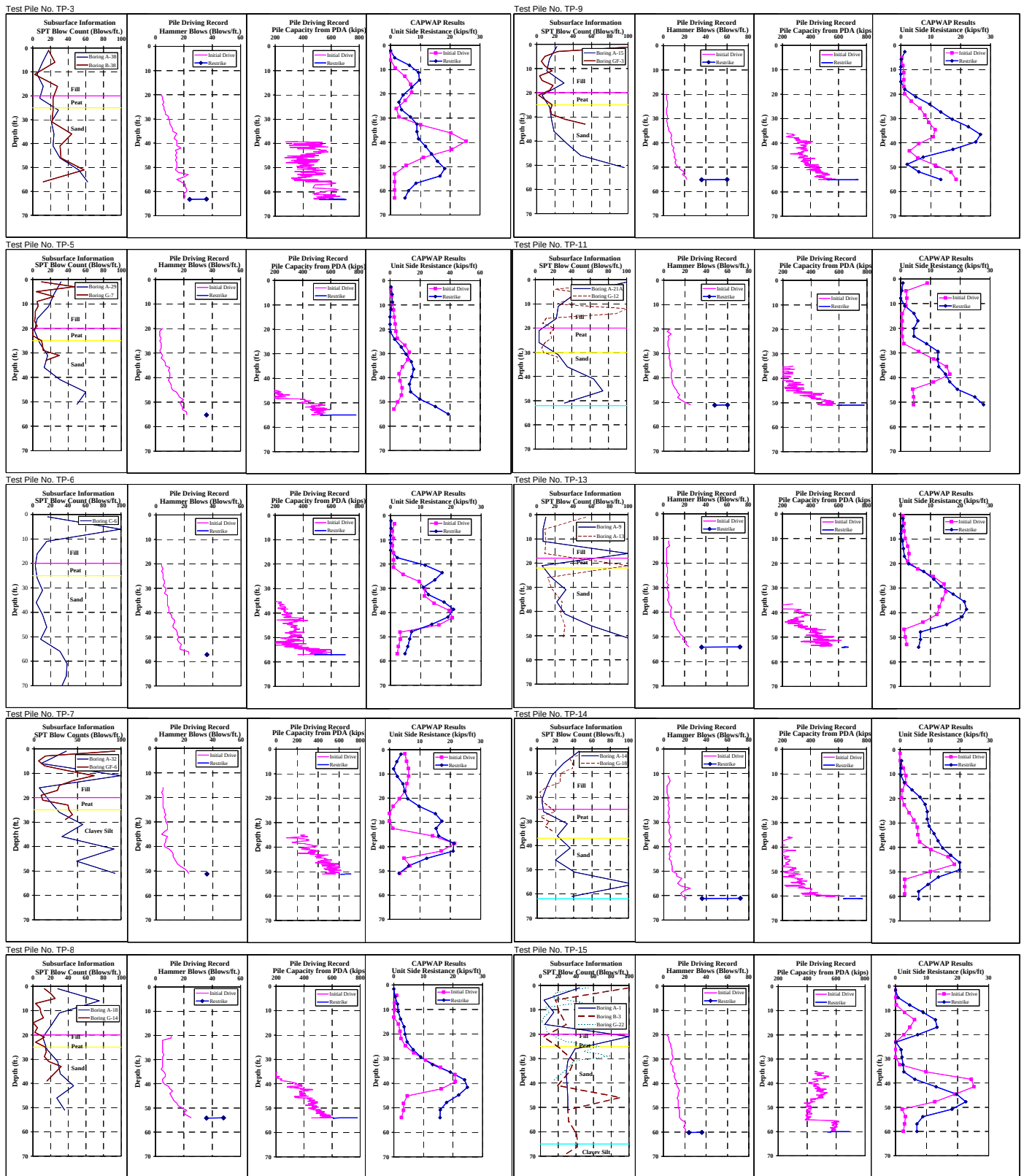


FIGURE 2 (Cont.) PILE TEST RESULTS

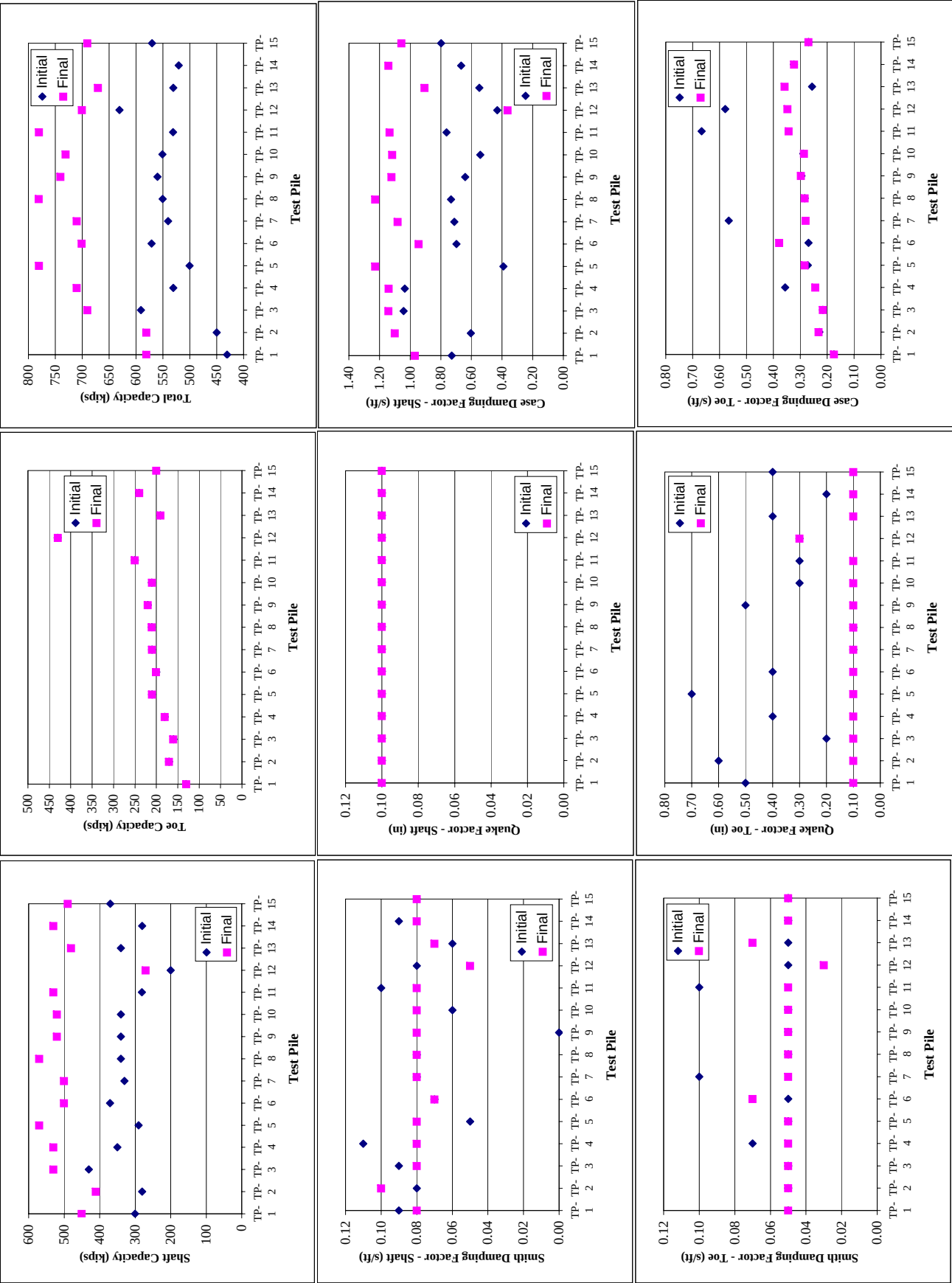


FIGURE 3 CAPWAP RESULTS

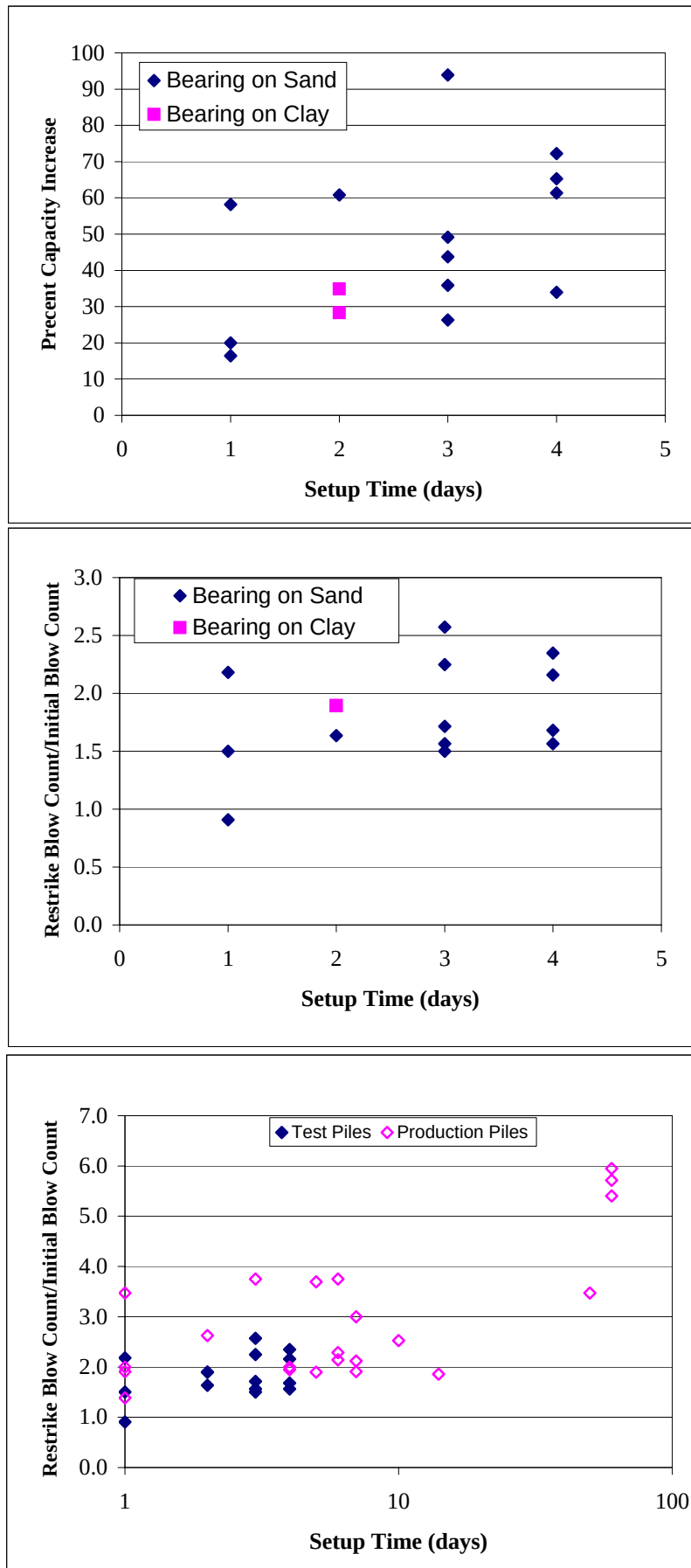


FIGURE 4 PILE SET-UP

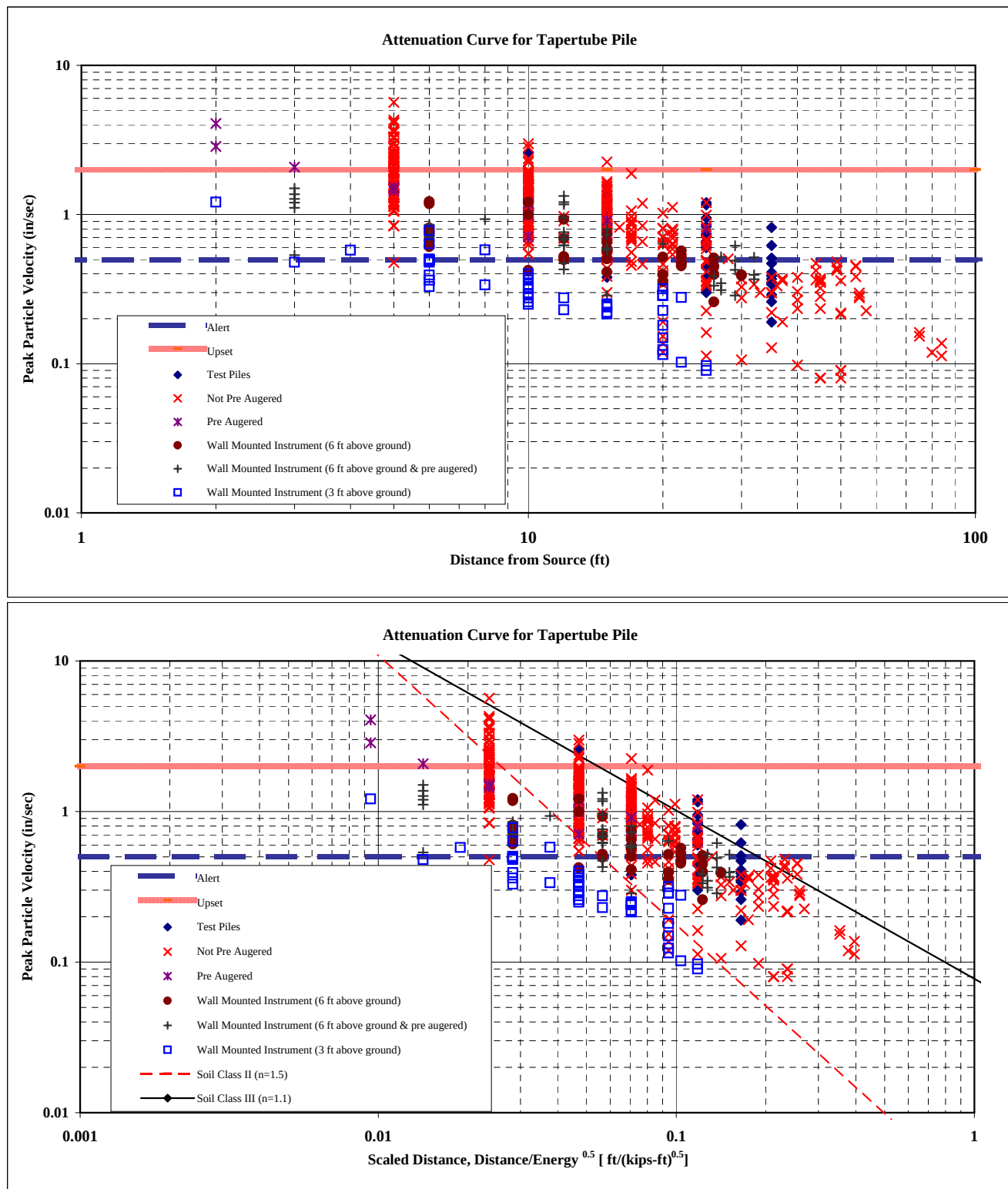


FIGURE 5 VIBRATION MONITORING RESULTS