MEASURED PERFORMANCE OF A LARGE EXCAVATION ON THE MIT CAMPUS

By

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ABSTRACT

This thesis evaluates the measured performance of a large excavation on the MIT Campus. The new Ray and Maria Stata Center, currently under construction, was designed with a 42ft deep basement carpark. The excavation plan area is approximately rectangular and measures 386 ft by 316 ft. Lateral earth support during excavation was provided by a 30 in thick, 70ft deep concrete diaphragm wall, that forms part of the permanent structure. The toe of the wall floats within a deep layer of Boston Blue Clay that underlies the site and ranges in thickness from 60 to 90 ft within the footprint of the new building. The wall was supported by a combination of a) three levels of prestressed tieback anchors on three sides, b) two levels of prestressed rakers braced against the foundation slab on the fourth side, and c) two levels of corner bracing in each of the four corners of the site. Control of ground movements was a critical aspect of the subsurface design, due to the close proximity of the excavation to the Alumni swimming pool (Building #57).

The thesis summarizes the site investigations and evolution of the design for the lateral earth support system. The project was monitored closely through measurements of lateral wall deflections, building settlements, pore water pressures and and soil heave beneath the base of the excavation. The author has compiled this information together with a very detailed record of the construction history to provide a very complete case history of the project that can be used to evaluate predictions using advanced non-linear finite element analyses. Links between the lateral wall deflections and construction activities are most clearly illustrated through a series of isometric drawings of successive construction phases. These highlight important three-dimensional aspects of wall support.

Measured wall deflections and surface settlements exceeded the allowable limits prescribed at the start of the project, but fortuantely did not damage the adjacent structures. Careful control of the excavation sequence and additional bracing of the wall using concrete grade beams, contributed to successful completion of the excavation. The performance of the Stata Center excavations were compared with data from other diaphragm-wall-supported excavations in the Boston area. The ground movements that occurred during this excavation are much larger than those obtained for most of these published case studies, but are very similar to one earlier project in the Back Bay area (500 Boylston St.) that also used a floating wall system and prestressed tieback anchors. Improved control of ground movements for these types of support system require careful geotechnical evaluation of tieback locations and careful site control of the excavation sequence.

Thesis Supervisor: Professor Andrew J. Whittle
Title: Professor of Civil and Environmental Engineering

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Chapter 1
Introduction

Ground movements play an important role in the design of deep excavations, especially in cohesive soils. Engineering Practice calls for the design of excavation support systems to resist failure using structural analysis procedures, but greater damages and consequent litigation expenses arise from associated soil movements than from overall system "failure" (Clough et al., 1989). The ability to make accurate and reliable predictions of ground movements offers many benefits. First, it provides a basis for defining performance criteria, or acceptable levels of movement for the protection of adjacent facilities, and therefore reduces the costs brought on by the damage and the litigation that follows. Second, it allows for identification of areas requiring special construction methods and assist in choosing between various construction options. Third, it facilitates the design and interpretation of a geotechnical field monitoring program (Whelan, 1995).

This thesis presents a case study of a well instrumented project where ground movements represent one of the more critical criteria in the design of the excavation support. The Stata Center is one of the centerpieces of MIT's most ambitious program of campus construction since the middle of the last century. The 713,000 SF facility will include the William H. Gates Building, the Alexander Dreyfoos, a below grade service facility, a child-care facility, a food services facility, a fitness center, outdoor gathering spaces, and 2 levels of below grade parking. It will house the Laboratory for Computer Science, the Artificial Intelligence Laboratory, the Laboratory for Information Decision Systems, and the Department of Linguistics and Philosophy. It is located near the intersection of Vassar Street and Main Street in the northeast section of campus (Figure 1.1). The figure shows that the project is in close proximity to several existing campus buildings, notably the Alumni swimming pool (Building 57) a structure deemed historically important.

The goal of this thesis is to describe accurately the project and to document the construction history and performance so that it can be used to develop better methods of ground movement prediction and of excavation support design. The thesis provides detailed data that can be used to reconstruct the three-dimensional and temporal reconstruction in finite element analysis, for future use in verifying soil models and analysis techniques.

The main focus of this thesis is to present the information in a clear and concise manner that allows the reader to understand the project sufficiently to use in further research. Chapter 2 reviews background information on diaphragm supported walls. The construction and support methods for diaphragm walls are discussed along with the predictions of soil movements associated with their installation and performance.

Chapter 3 describes the site information. Including geologic history, adjacent structures, site investigations, and soil properties. Chapter 4 describes the evolution of the project with a detailed outline of the support system chosen for the project, the proposed excavation sequencing, and the construction monitoring program. In Chapter 5, the construction history is presented with graphical representations of the excavation progress and associated wall movements. The chapter also summarizes the results from the construction monitoring program, including inclinometer, settlement point, extensometer, and piezometer data. Chapter 6 contains further results and analysis of the performance of the lateral earth support system using comparisons with other projects in the Boston area and similar projects in Chicago, and Chapter 7 contains the summary and conclusions.

The main text is followed by three appendices. The first of these contains all of the available soil properties obtained from the investigation and testing program. The second contains representations of the excavation progress obtained from Haley & Aldrich. The last appendix contains photos of the construction progress over the six months that it took to complete the excavation.

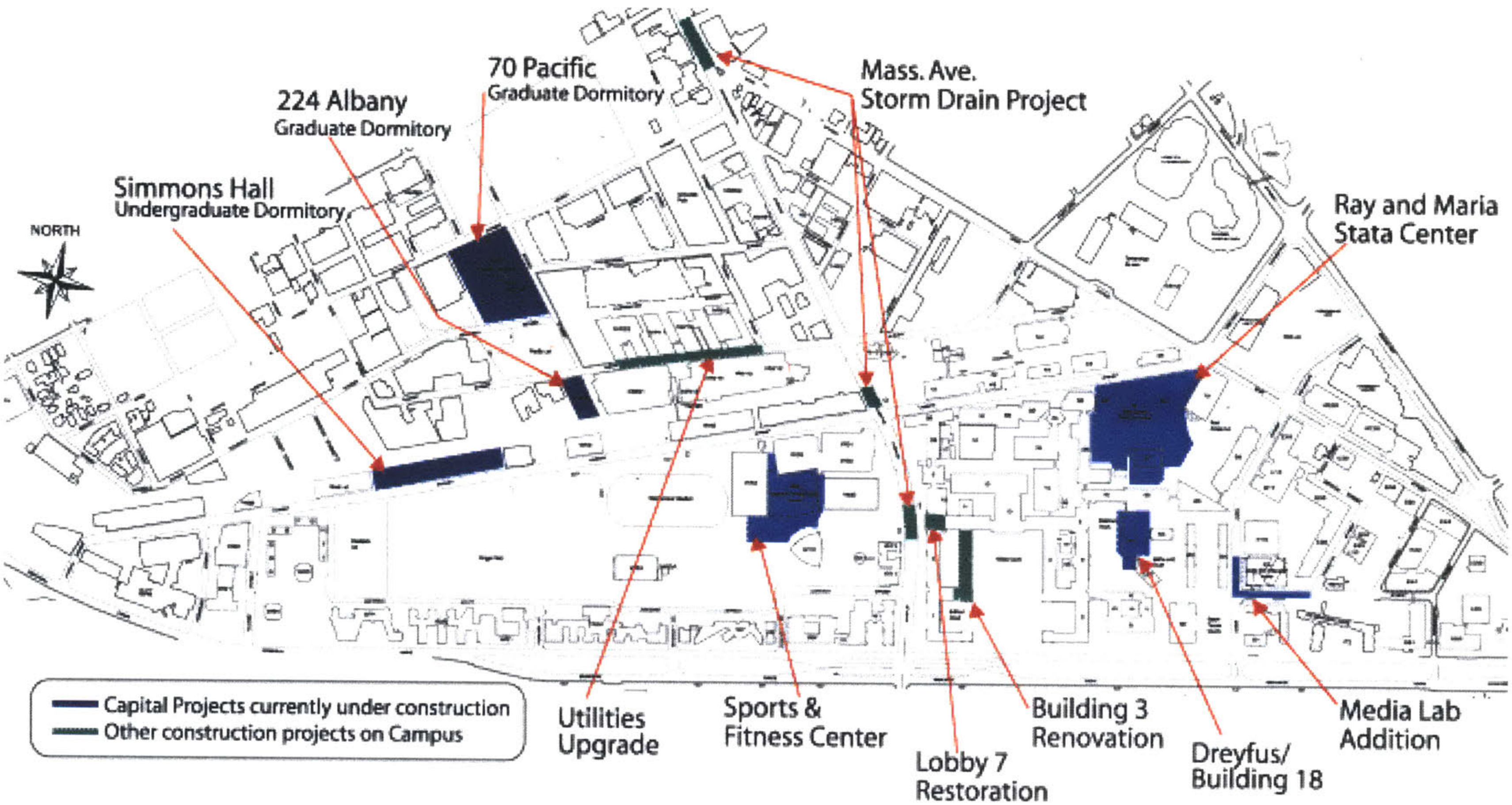

FIGURE 1.1: Map of MIT Campus showing major construction and renovation projects with the Stata Center in the northeast section of campus.

Chapter 2
Background

2.1 Introduction

The construction of deep excavations[1] in soft soils requires a support system comprising a perimeter wall and structural bracing elements and/or ground improvements. The support system must either resist lateral earth pressures, pore water pressures, and surcharges from adjacent buildings acting on the wall or transfer (bridge) these forces across the excavation pit. The wall achieves this primarily through the resisting force of the soil inside the excavation acting on the embedded length of the wall, the bending stiffness of the wall itself, and the axial load capacity of tieback anchors, rakers, or cross-lot bracing systems.

In general, excavations change the state of stress within the soil mass and induce both elastic and plastic deformations of the soil. These movements are typically characterized by the adjacent soil moving laterally inward towards the excavation[2] and associated settlement of the retained soil, shown schematically in Figure 2.1. Stress relief and upward flow soil within the excavation generate upward heave movements in the soil below the excavated grade.

The support system is intended to eliminate these movements and keep the excavation stable and safe. Some movement is inevitable due to the changing stress state, but other movements can be controlled through careful construction procedures such as cased drilling, careful construction of watertight seals, and careful planning of excavation

[1] The term "deep" refers to excavations with depths exceeding 10m, shallower excavations can be conventionally supported by cantilever bending action of embedded walls. More recently very high bending capacity, T-section diaphragm walls have been successfully used to support excavations in Boston.
[2] Prestressing operations can actively reverse this pattern.

sequencing. Further movements can be induced by activities such as dewatering (consolidation settlement), installation of the wall itself (by driving or slurry trenching), or losses due to drilling for tiebacks. The successful control of these ground movements is very important and the extent of these movements and their effects on surrounding structures is the primary method of assuring the performance of the support system.

2.2 Overview of Diaphragm Wall Supported Excavations

2.2.1 General

There are many types of walls used to support deep excavations. The selection of wall types is driven by costs and design aspects (eg. temporary vs. permanent application), top/down vs. bottom up construction, potential inflows (need for watertightness), and prevention of damage to adjacent structures. Of these, three of the most common are soldier piles and lagging, sheetpile walls, and diaphragm (slurry) walls. In general the wall is installed (or at least the primary supports) prior to any excavation work. A wall consisting of soldier piles and lagging is constructed by first driving a series of H or I beams (piles) into the soil at regular intervals surrounding the future excavation, and then as the excavation progresses lagging elements are placed horizontally between the piles. Sheetpile walls consist of a series of thin interlocking steel sections or 'sheets' that are driven into as continuously interlocking sections. Many different wall cross-sections are used, but in general they are all bent so that the wall is corrugated and thus resists bending better than plain sheets. The diaphragm wall is generally a cast in place reinforced concrete wall although a new technology consists of a 'soil-mix' made by mixing grout with the existing soil using augers and then inserting piles vertically as reinforcement.

2.2.2 Diaphragm Wall Construction

A concrete diaphragm wall is constructed by joining a series of panels constructed in a specific order so that they will form a continuous wall. The panels are first excavated to predetermined dimensions while a bentonite or polymer based slurry supports the trench, then the reinforcing cage is inserted, and then concrete is placed by tremie (ie. displace most of the slurry). Excavation only begins once the entire wall has had sufficient time to gain strength. This sequence for the construction of diaphragm walls is shown in Figure 2.2.

The first step in construction of wall panels is the removal of any possible obstructions near the surface and the construction of guide walls. The guide walls are small concrete walls on the surface that serve to stabilize the surface soils (so they do not collapse into the trench) and to guide the trenching equipment, keeping the wall vertical. Once the guide walls are in place, the trench is excavated under slurry using the equipment chosen for the job. Slurry wall contractors have developed a large variety of specialized excavation equipment. Mechanical or hydraulic clamshell excavators are commonly used in the United States. The clamshells come in varying sizes corresponding to different wall thicknesses and panel lengths. Panel sizes are usually chosen to correspond with the equipment being used so that only one pass is needed to excavate the panel, but in cases when larger panels are desired[3] the clamshell can accomplish this by making more than one pass as shown in Figure 2.3. The typical panel

[3] Larger panels are often desirable because the larger the panel the fewer joints between panels, the drawback is that the larger the panel the less stable the trench, and therefore more susceptible to collapse.

length is 20 ft, but lengths up to 30 ft in length have been excavated (Konstantakos, 2000). The wall is usually constructed in an alternating sequence of primary and secondary panels. This method improves trench stability by allowing the lateral soil stresses to arch across the secondary panels, but requires a special design of end stops between primary and secondary panels to ensure adequate shear connection and watertightness.

Once the panel has been excavated under slurry, an end stop is inserted to help form a clean edge for formation of joints and the steel reinforcing cage is lowered into the trench by lifting equipment. Depending on the support elements chosen for the excavation, the reinforcing cage may include steel panels to reinforce areas where bracing elements are to be connected or it may contain sleeves (or cutouts) for tieback installation through the wall.

Prior to pouring the concrete, the bottom of the trench is cleaned of any material that may have fallen in during excavation or installation of the cage. This is important because loose materials can become trapped in the concrete and can undermine the strength, stiffness, or water-tightness of the wall. Once this material is removed, the concrete is poured into the trench through pipes that extend to the bottom half of the trench and allow the slurry to be displaced from the bottom up and this technique usually avoids trapping slurry within the concrete causing weak spots in the wall.

2.2.3 Support Systems and Construction Sequence

There are four principal ways of supporting an excavation (Figure 2.4). The methods are 1) a cantilever wall (no support), 2) cross-lot bracing, 3) raker supports, and 4) tieback anchors. The cantilever wall system is generally not used in deep excavations because of the expense associated with wall needing to withstand the bending and keep the movements small would be too expensive and in soft soils the resistance needed could not be achieved. The other methods will be discussed below. Further work has been done recently on the use of ground improvement techniques (such as deep soil mixing, jet grouting, and ground freezing) to help reduce soil movements and minimize the need of other support systems.

2.2.3.1 Tieback Anchors

Tieback anchors function by transferring lateral loads acting on the wall back into the ground to anchorage points located either in the underlying bearing layer (usually bedrock) or to stable zones within the soil mass (areas unaffected by potential instabilities of the excavation). Tiebacks comprise high strength steel tendons (cable or strand wire) that are pretensioned and locked-off at the front face of the wall (ie. inside the excavation) (Figure 2.7). Ground support is achieved by grouting the tendon to form a strong bond within the anchorage ("fixed length") zone, while the intervening "free length" of the tendon is sheathed to prevent load transfer at points closer to the wall. Watertightness and corrosion protection are critical components of anchor design, especially where tiebacks are used as permanent anchorages. When used for temporary support, the tiebacks are detensioned upon completion of subsurface structures. Tiebacks are a popular solution for bottom-up construction sequences as they minimize obstructions within the work areas.

There are two principal drawbacks of tieback anchors: 1) they require skilled installation and each anchor must be proof tested, 2) the anchor points must often be located beneath foundations of adjacent structures[4] (hence special care must be taken to avoid damage during installation and operation), and 3) the tieback holes can act as flow conduits through otherwise impermeable walls and hence pressure seals must be designed to ensure watertightness.

Tieback prestress loads depend many factors. These include vertical and horizontal spacing of anchors, surcharge conditions, and the strength of the anchorage strata. Depending on these factors, typical prestress loads range from 40 to 250 kips in soil (Konstantakos, 2000). These loads can be greatly reduced if are spaced too closely because of interference between the zones on influence around each anchor.

Figure 2.5 illustrates the typical installation of tieback anchors. The first step is to over excavate the area in front of the wall to a depth at least two feet below the level of the proposed anchor, this provides room for the drill rig. The hole is then drilled using a percussive or rotary drill with casing for the granular materials so that the hole does not collapse. Once the hole is completed the steel tendon is placed together with a pipe for grouting. The tendon is then grouted and left in place for the grout to set-up. Once the anchor has gained sufficient strength, the anchor is prestressed and proof tested (to 120% to 150% of its design capacity), and locked off at the face of the wall using a jack such as the one shown in Figure 2.6. Soil anchors that do not pass the proof test are often re-grouted and re-tested. If this does not solve the problem, then the tieback may be redrilled and retested or additional anchors installed. Once sufficient anchor capacity has been established, the excavation can proceed to the next level. The vertical spacing between rows of tiebacks usually ranges from 5 to 15 ft depending on the situation

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[4] It is clear that tieback anchors may not be permissible in situations where they cross existing property lines. This frequently prohibits their applications as permanent wall supports.

2.2.3.2 Compressive Bracing Systems

Cross-Lot bracing is designed to transfer lateral earth and water pressures between the walls of an excavation through compressive struts. These struts are usually either pipe or I-beam sections that are able to withstand the high compressive loads needed to reduce movements. Cross-lot bracing is generally used in narrow excavations (less than 100 ft wide). It is generally used on these projects because it is often the simplest and most cost-effective option for narrow excavations but becomes inefficient in wider cuts (greater than 150-200 ft) where strut buckling and self weight bending can become serious problems. They are not used in wide excavations because they will bend under their own weight if they are not supported. In many early applications of cross-lot bracing (e.g. Chicago subway) the cross-lot bracing struts were not preloaded. However, most recent practice preloads the strut to half of their expected design load in order to increase the system stiffness and reduce/minimize movements. One advantage of cross-lot bracing is that higher capacity braces can be spaced farther apart than tiebacks, and do not require skilled installation or proof testing in the field. The other advantage is that there is no need for holes in the wall that could cause leakage. The disadvantages of this system are that the braces obstruct excavation equipment and impede the work. The struts are also subject to thermal expansion and contraction that can affect wall movements.

In order to install cross-lot bracing, the excavation progresses to just above the proposed strut elevation and then a channel is excavated for each strut (Figure 2.8). Often the area in the middle of the site will be excavated deeper than this while there is no obstruction since this soil is not usually needed to support the wall until the struts are in place. The struts are then installed and preloaded to minimize movements. Once this is completed, the excavation continues in the same manner to the next level.

Corner bracing can be considered a special type of cross-lot bracing that transfers the loads between intersecting walls that form corners of an excavation. In some excavations where the plan dimensions are small, corner braces are sufficient to support the entire wall, leaving the center of the site free of obstructions. Corner bracing is also easily used in conjunction with tieback anchors.

Raker supports are another form of compressive strut support (Figure 2.4), often used when the distance between opposite walls of the excavation is too large to enable cross-lot bracing, while tiebacks cannot be used due to obstructions in the soil mass or the lack of access from adjacent property owners. Raker systems transfer the load from the wall to either a large concrete block that is constructed within the excavation (kicker block) or to a section of the foundation slab that has already been poured. This system relies on a soil berm that is left in front of the wall to provide temporary lateral earth support, while the soil is excavated to final grade in the interior of the excavation. After the bracing is installed and preloaded the berm can be removed.

2.2.3.3 Top-Down Construction

Top-down construction offers an alternative approach for lateral earth support. This method used the permanent basement floor slabs as the lateral earth support system, and is accomplished by constructing the diaphragm wall and interior support columns (for the floor slabs) through slurry trenching or drilled pile methods. Once the support columns are in place, the ground floor (or uppermost basement slab) can be constructed to brace the top section of the wall while excavation progresses (in a mining operation) below this slab. The soil (and construction equipment) is usually removed through glory holes left in the overlying floor slab. Excavation progresses once the overlying concrete slab has gained sufficient strength. Top/down construction can be very efficient as a one pass method (ie. no temporary lateral earth support system is used) for subsurface construction and is one of the most effective methods of controlling movements of the superstructure. It can also be used to accelerate construction. The method relies on high quality construction of foundation, perimeter wall, and basement floor elements.

2.3 Prediction of Soil Movements

Attempts at estimating ground movements caused by deep excavations generally fall into two categories, 1) empirical and 2) numerical analysis. The first of these categories deals with design charts developed through case histories and the second uses mathematical models and computer simulations such as finite element analysis to make predictions of soil movements and support performance. Each of these methods has its advantages and disadvantages. As stated in Ladd and Whittle (1993),

> "... methods which are based on field data collected from case histories provide a useful guide for estimating a likely range of movements, but cannot be used reliably for site-specific, predictions. Finite element methods have the ability to analyze complex design problems, but have questionable accuracy due to one or more of the following factors: inadequate site investigation to properly define relevant soil properties; use of simplistic soil models that do not describe the actual behavior of

natural clays; and no consideration of changes in the groundwater conditions during excavation."

This statement points out that the work of estimating ground movements is not always easy and therefore requires engineers to rely on their engineering judgement to decide what is relevant and what is not. Much work has been and is being done to develop more reliable design methods that will apply to a wide range of sites. This work involves both empirical and numerical analyses. Below is a summary of some of the work that has been done and is commonly referenced when predictions of movements caused by deep excavations are needed.

2.3.1 Empirical Analyses

Empirical studies attempt to develop relationships between observed ground movements and construction activities based on a number of similar projects. In order to develop these relationships a large amount of work has to go into deciding which projects have the necessary reliable information and then extracting the specific relations required. This development of relations is not trivial and entails a detailed compilation of numerous cases including the unique nature of each.

The first and most often referenced compilation of field data was published by Peck (1969). The information used for his relationships came from projects in Chicago, San Francisco, Seattle, St. Louis, and Oslo (Norway) that had depths ranging from 20 to 63 ft. These excavations were all supported by either soldier piles and lagging or sheet pile walls and various wall supports, including cross-lot bracing, pre-stressed rakers, H-pile tiebacks, and anchors. Along with this wide variation in support systems, the excavations were constructed in a wide range of soils including soft to medium clays, stiff clay, cemented sand, and cohesionless sand. Using this database, a summary of the expected normalized limits and normalized magnitudes of the settlement troughs as a function of soil type, excavation depth, and 'workmanship' was developed. Both the limits and the magnitudes were normalized with respect to the total excavation depth, H.

Figure 2.9 shows the design chart that was developed from this data. It divides the movements into three zones according to the dominant soil type and conditions. Zone I is characterized by sands and soft of hard clays of average workmanship, while Zone II is characterized by very soft to soft clays to a limited depth below the excavation, and Zone III is characterized by very soft clays to a significant depth below the excavation. The predictions that are shown were based on relatively flexible support walls and would up to 4.8 in of settlement in Zone I and more than 9.6 in of settlement in Zone III for a 40 ft deep excavation, with the zone of disturbance extending 160 ft from the excavation. Although these numbers are excessive they are a good basis for the work that was to follow and is good starting point when predictions are made for excavation induced movements.

Mana and Clough (1981) presented a more recent analysis of observed movements for cuts in clay that were also braced using sheetpile walls or soldier piles (and were mostly braced using cross-lot bracing). Out of 130 case histories, 11 were chosen that had sufficiently 'simple' histories, in which soil movements were "generated primarily by excavation stress relief" and not by any secondary construction factors such as consolidation due to dewatering. They report a correlation between the measured normalized lateral wall deflection (δ_w/H) and the factor of safety against basal heave greater than 1.5 as defined by Terzaghi (1993) (Figure 2.10). For factors of safety greater than 1.5, the expected maximum wall movement (δ_w/H) ranged from 0.2% to 1.0%. However, much larger wall deflections can occur for deep excavations in soft clay where factors of safety against basal heave are usually less than 1.5. Uncertainties in the estimates of δ_w are generally much larger than are needed in design. Since damage to adjacent structures is related to surface settlements and horizontal displacements, Mana and Clough (1981) also attempted to relate maximum lateral wall movement to maximum settlement. Figure 2.11 shows that maximum settlement, δ_v, is generally between 0.5 and 1.0 times the maximum wall displacements, δ_w. Because settlements by Peck (1969) and Mana and Clough (1981) are based on observations from excavations supported by soldier piles or sheet pile walls they cannot be relatively extrapolated to other support conditions easily.

Clough et. al. (1989) updated the existing database by incorporating the performance of excavations supported by diaphragm walls and relating the deformations to the support conditions and soil profiles. They attempted to correlate the wall movements, δ_w/H, with the stiffness of the lateral earth support system through the stiffness parameter, $EI/(\gamma_w h^4_{ave})$. In this definition EI is the elastic bending stiffness of the wall, h_{ave} is the average vertical spacing between supports, and γ_w is the unit weight of water. The proposed design method (Figure 2.12) was guided by results of FE analyses and suggests that wall movements decrease significantly with system stiffness. Although the experimental data in Figure 2.12 from Terzaghi, Peck, and Mesri (1996) only minimally justify the proposed design chart, the results are widely quoted and have been used to justify the selection of stiff diaphragm wall sections in order to control ground movements.

2.3.2 Numerical Analyses

The most common methods of numerical analysis used in analyzing excavations are finite element, finite difference, and boundary element methods. No matter which technique is used, all methods have the goal of estimating the behavior of the excavation through models that meet all the theoretical requirements and boundary conditions that properly reflect the various components of an excavation. The Finite Element Method (FEM) involves recreating a construction activity and the affected soil mass in a mathematical simulation. Computers utilize pre-defined soil model parameters and algorithms in conjunction with a carefully defined spatial geometry and time history to iteratively calculate ground movements around the excavation. By varying the factor of interest and monitoring the resulting behavior, a prediction can be made of how the excavation will perform.

Initial use of FEM to analyze excavations began in the early 1970's with work by Clough and Duncan (1971), Morgenstern and Eisenstein (1970), Wong(1971), and Tsui (1974). Their work began with examinations of the effects of boundary conditions and changes in lateral pressure distributions within elastic medium and the performance of braced excavations and tieback walls. Over time advancements in computer hardware and software have allowed the use of FEM analyses to model excavations to become more sophisticated and more widely used. Despite advancements, FEM is still limited when it comes to its predictive capacity and it is mainly used for back-analyses.

The limitations of the Finite Element Method stems from the inability to develop and use constitutive models to describe soil behavior and field conditions. Most analyses have used simple constitutive models such as linearly elastic-perfectly plastic, hyperbolic and Modified Cam Clay models to describe the soil behavior. Recent efforts at MIT have led to the development of more accurate soil models. The MIT-E3 model (Whittle and Kavvadas, 1994) describes most rate-independent aspects of clay constitutive behavior, including anisotropy , small strain non-linearity, and hysteresis associated with load reversals. By using this model with case histories and parametric studies, design charts for the prediction of diaphragm wall deflections in Boston Blue Clay have been developed (Hashash and Whittle, 1994). The charts shown in Figure 2.13 were a result of the parametric study that concluded that for excavations in soft clays: 1) Wall length was found to have a strong influence on the potential for bottom instability, with short to medium length walls showing susceptibility to development of a basal heave failure mechanism, 2) That the initial unsupported excavation depth has only a "transient" effect on subsequent wall deflections and settlements, and 3) As OCR was increased, deformations reduced significantly and failure mechanisms were much less likely to develop.

As a continuation of this work, a detailed finite element study was carried out by Jen (1998). In this parametric study, the effects of geometry (wall length, excavation width, depth to bedrock), soil profile, and support system were studied. The study found that walls undergo three phases of deformation: 1) Unsupported cantilever deflections, 2) bulging (subgrade bending), and 3) toe kick out. It was also concluded that the deformation phase was determined by the embedment depth, that stiffness was more effective in reducing deformations for soft soils than in stiffer soils, and that the depth to bedrock had a significant impact on the surface settlement a distance from the excavation.

2.4 Boston Diaphragm Wall Experience

Recently, Konstantakos (2000) summarized and compared the performance of major slurry wall projects in the United States. In this thesis there is a comparison of roughly 30 projects in five cities including Boston, Chicago, New York, Washington D.C., and San Francisco. Not only are the projects in each city analyzed and compared, but also all the projects are divided into four categories (floating walls, keyed tieback walls, Top/Down construction, and Cross-lot/Internally braced walls) and compared.

There have been a large number of projects in the Boston area using diaphragm walls to support the excavation, often as part of the permanent structure. Diaphragm walls are a common choice in the area because they can be used a part of the permanent structure and because of the presence of a high water table which necessitates a system that is more watertight. Konstantakos (2000) summarized 13 archived projects for which extensive data was available and also pointed out that there are at least 8 other projects that were not included (6 of these projects were constructed within 4 years of the study)[5]. These projects represent a wide range of wall lengths, excavation depths, thicknesses, and support systems. Three of the projects were floating tieback wall, two were keyed tieback walls, six were Top/Down excavations, and two were cross-lot/internally braced excavations. A summary of the project types and the soil profiles can be found in Figures 2.14 and 2.15.

Diaphragm walls were first used in Boston in early 1969 with the construction of the South Cove cut-and-cover tunnel (Lambe et. al, 1972, D'Appolonia, 1973), and in 1975 with the construction of the 60 State Street office building (Johnson, 1976). Usage

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[5] These numbers do not include the large portion of the Central Artery/Tunnel Project for which the data is not yet available.

continued to increase as the expertise of the companies in the area increase, and today diaphragm wall are often used and they are being used on several ongoing projects.

For the 13 projects studied, an "overwhelming majority of the inclinometers deflected less than 1.0" with an average maximum horizontal deflection of 0.70". It is interesting to note that over 80% of all of the inclinometers from these projects showed 1.0" of movement or less no matter the support system or construction method. Also, if the deflections are normalized by the excavation depth almost 70% of the inclinometers had a deflection ratio of 0.2% or less with, an average of 0.17.

If the settlements for the database are graphed according to Peck (1969) all of the points with the exception of the 500 Boylston project (that had poor tieback performance) settlements fall within Zone I and settlements decrease with distance from the excavation.

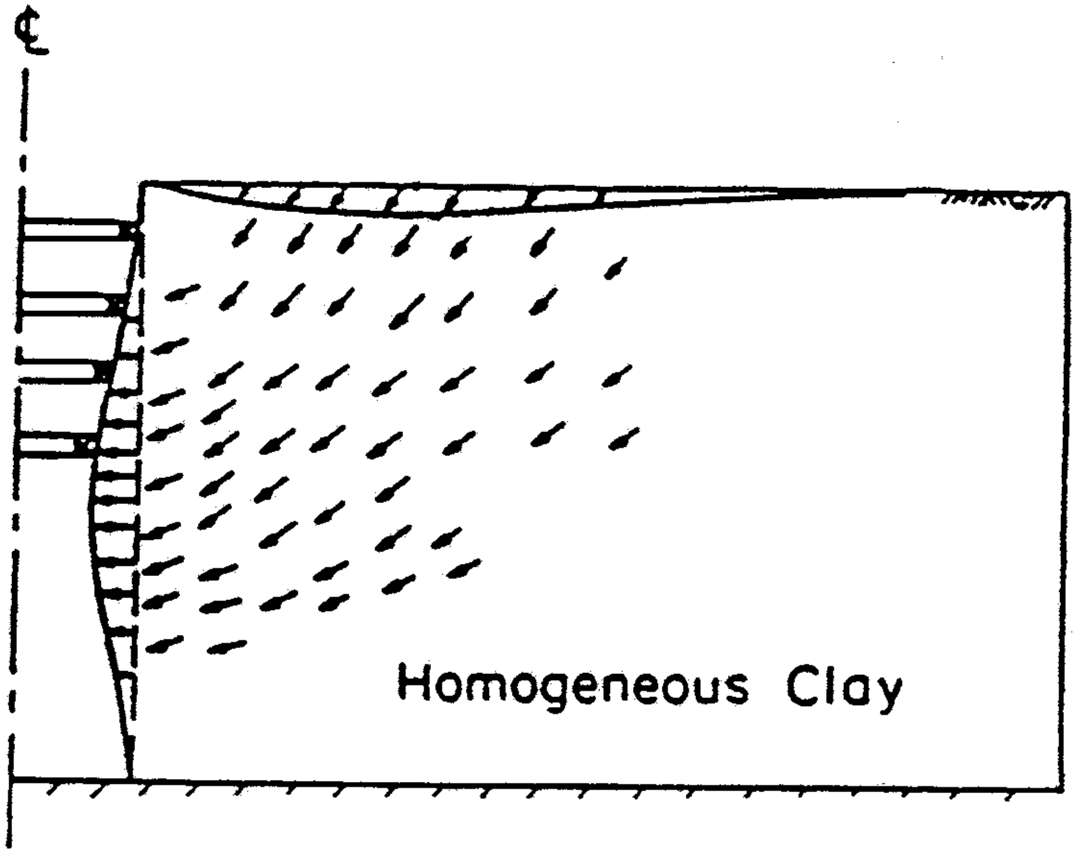


FIGURE 2.1: General movement trends around braced excavations. (Clough, 1985)

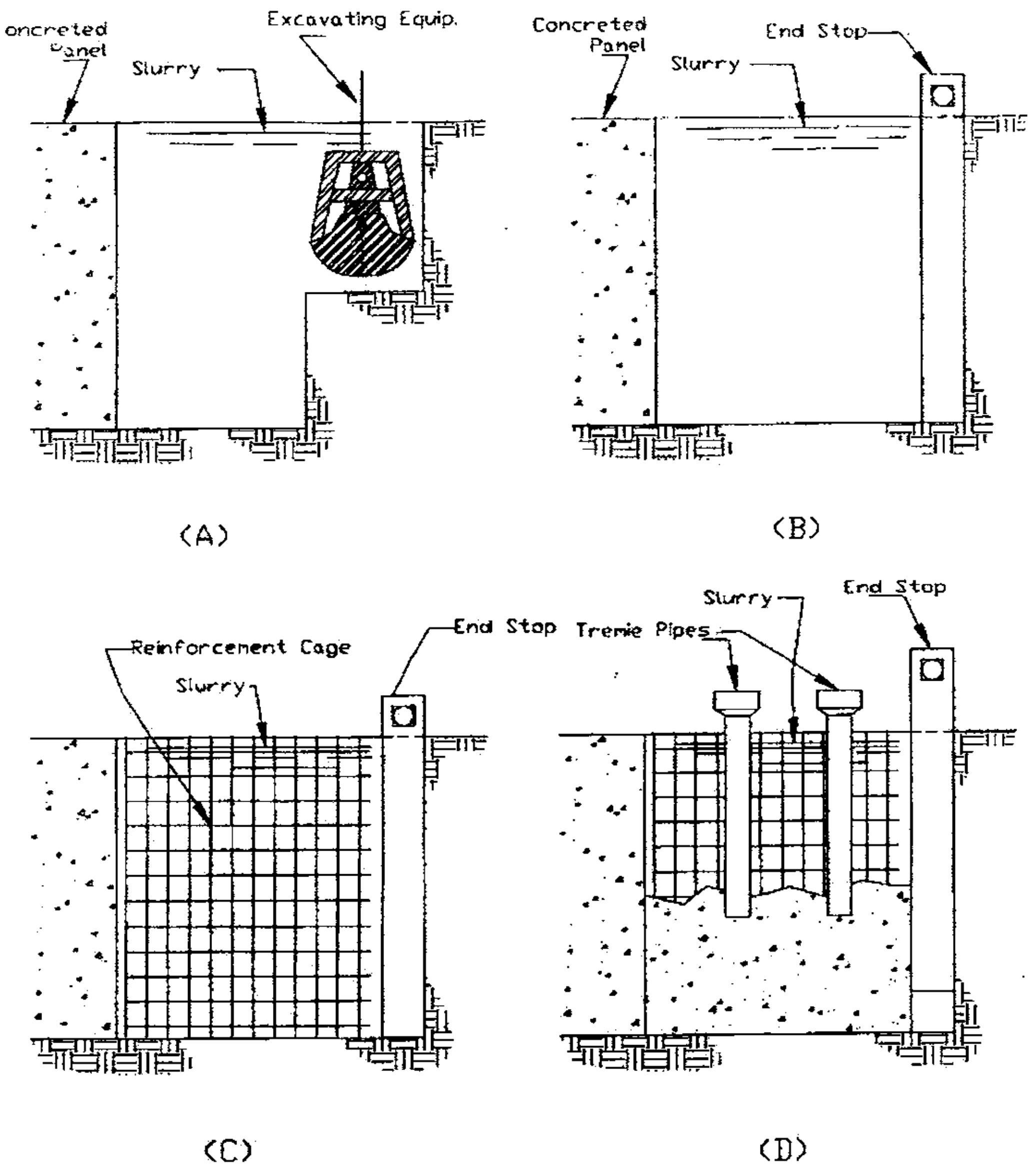


FIGURE 2.2: Typical construction sequence for diaphragm walls: A) Trenching under slurry, B) End stop inserted, C) Reinforcing cage inserted in slurry filled trench, D) Concreting by tremie pipes. (Konstantakos, 2000)

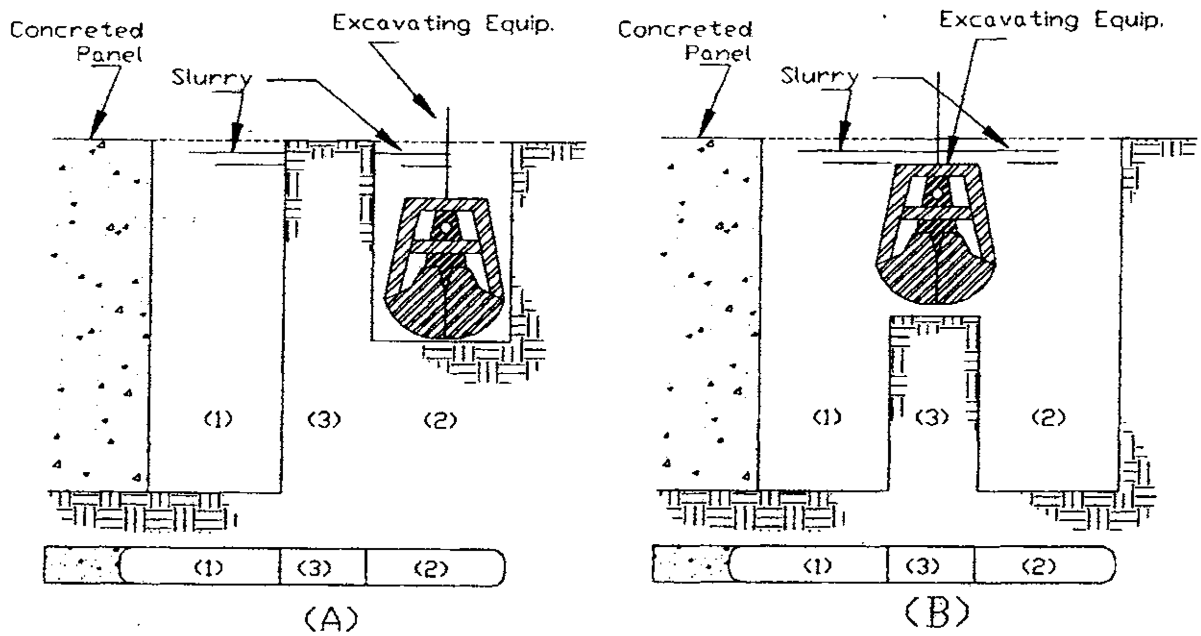
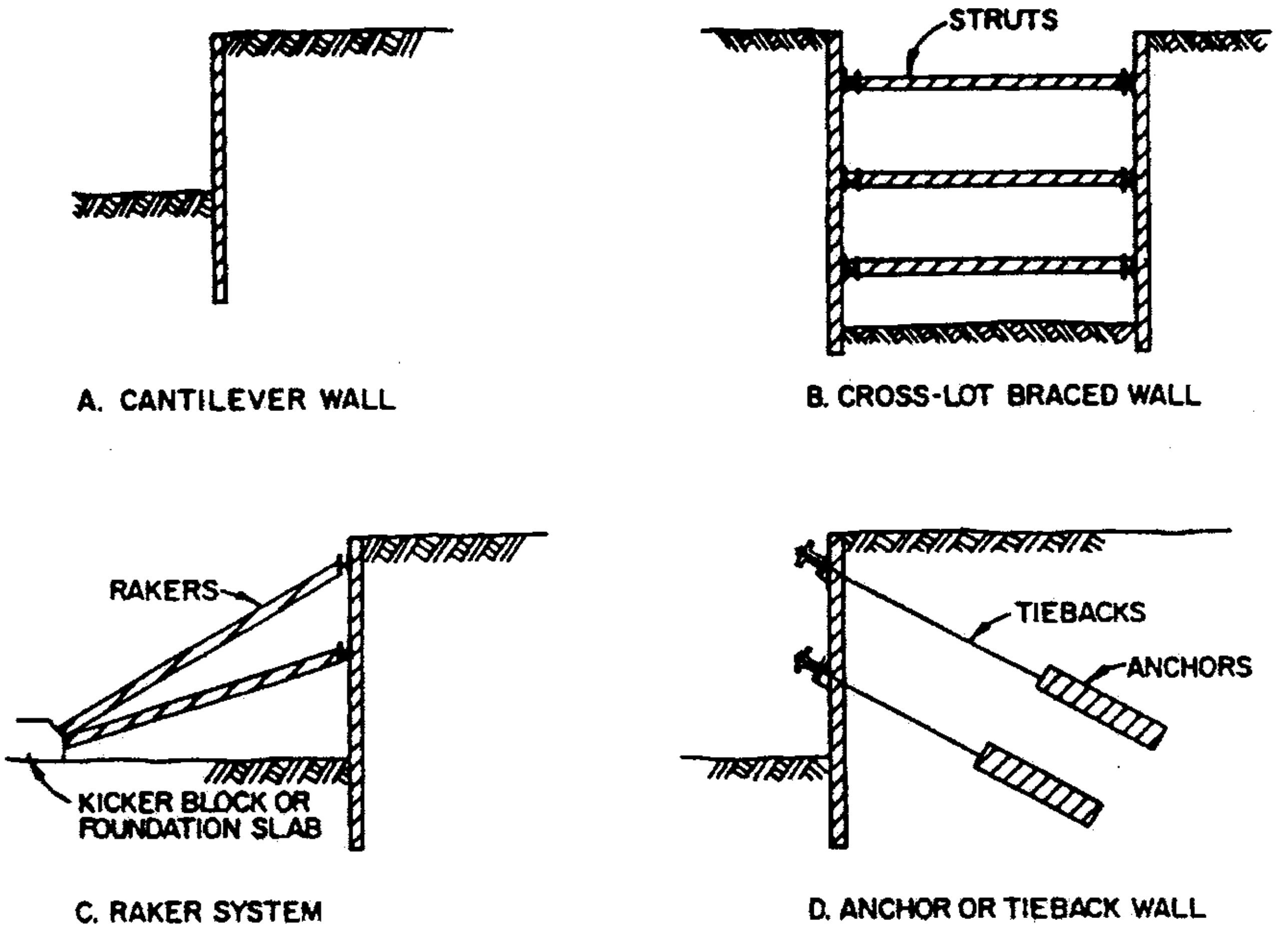


FIGURE 2.3: Typical Trenching Sequence: A) Outer bites excavated, middle bite left in place, B) middle bite excavated typically on the same day when concreting is scheduled. (Konstantakos, 2000)



<u>FIGURE 2.4:</u> Types of excavation support systems. (NAVFAC, 1982)

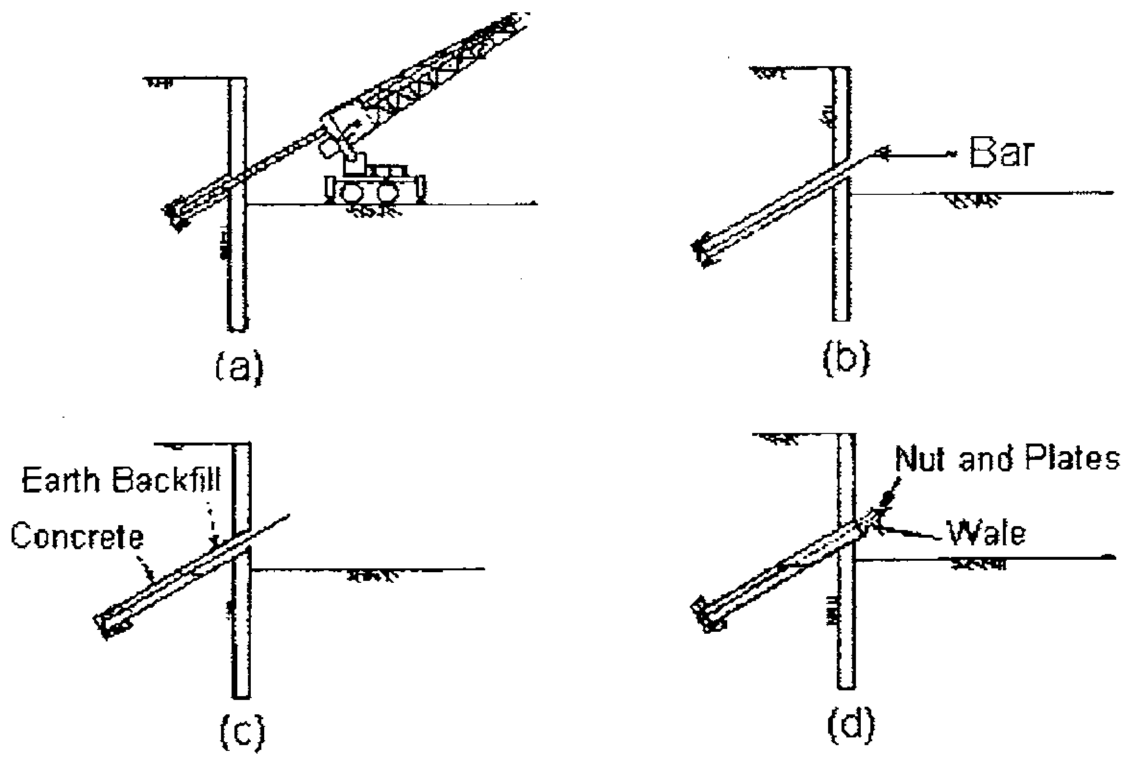


FIGURE 2.5: Tieback installation steps: A) Drill Hole, B) tendon or bar inserted, C) Cement Grout injected, D) Wall connection made and tieback prestressed. (Schnabel, 1982)

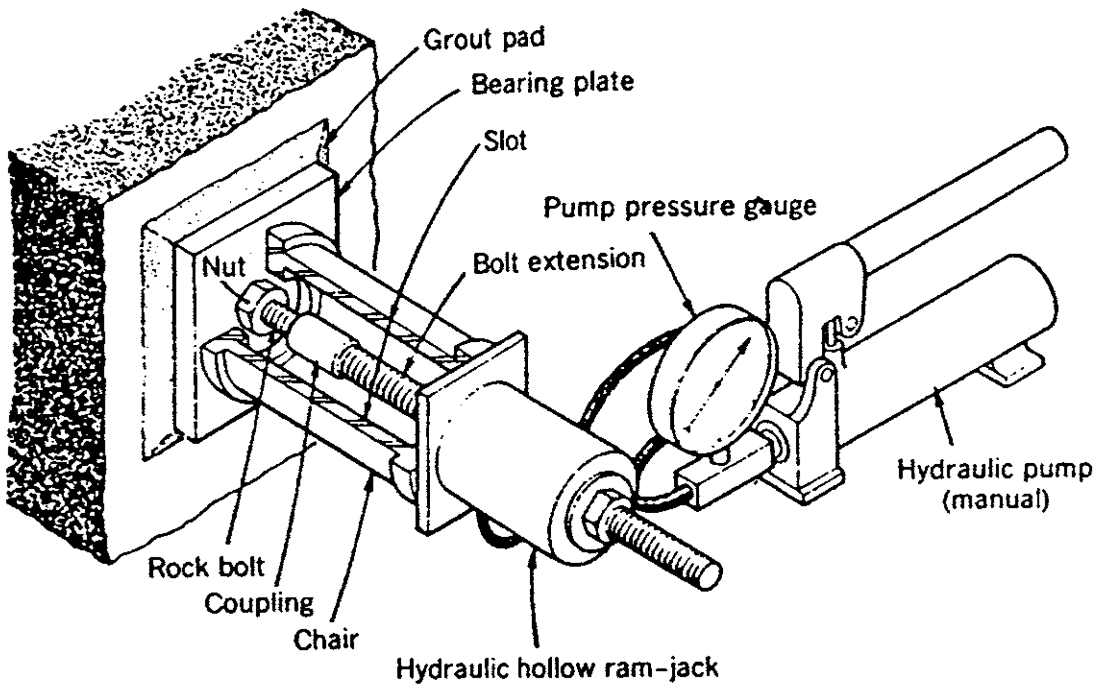


FIGURE 2.6: Equipment used for applying stresses to tiebacks and for measuring applied loads. (Xanthakos, 1991)

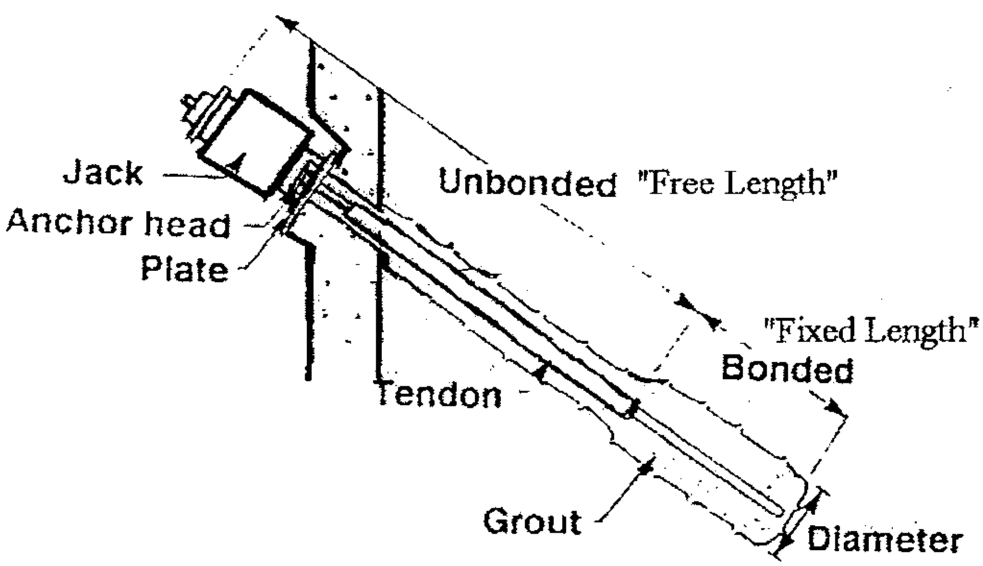


FIGURE 2.7: Tieback configuration, free and fixed lengths. (Schnabel, 1982)

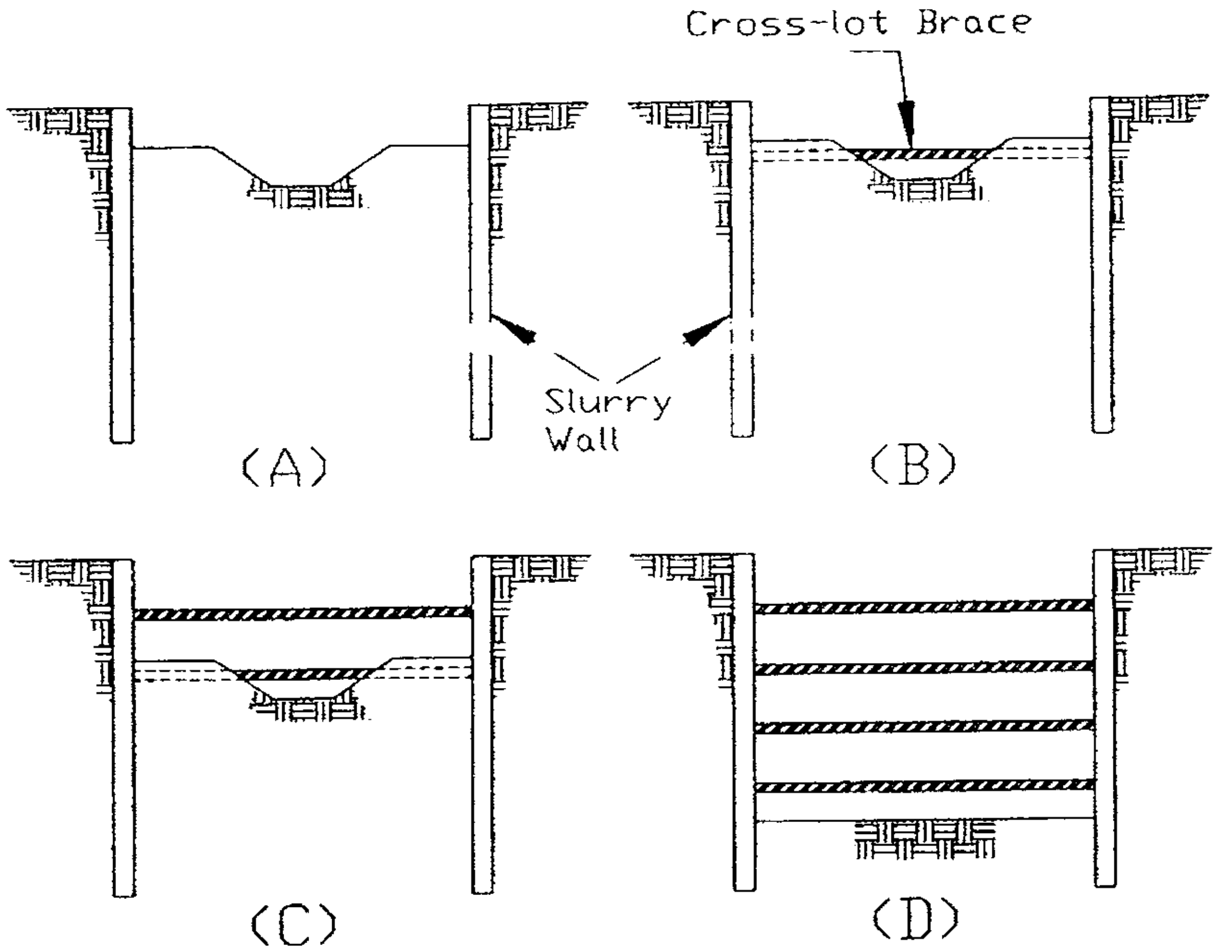
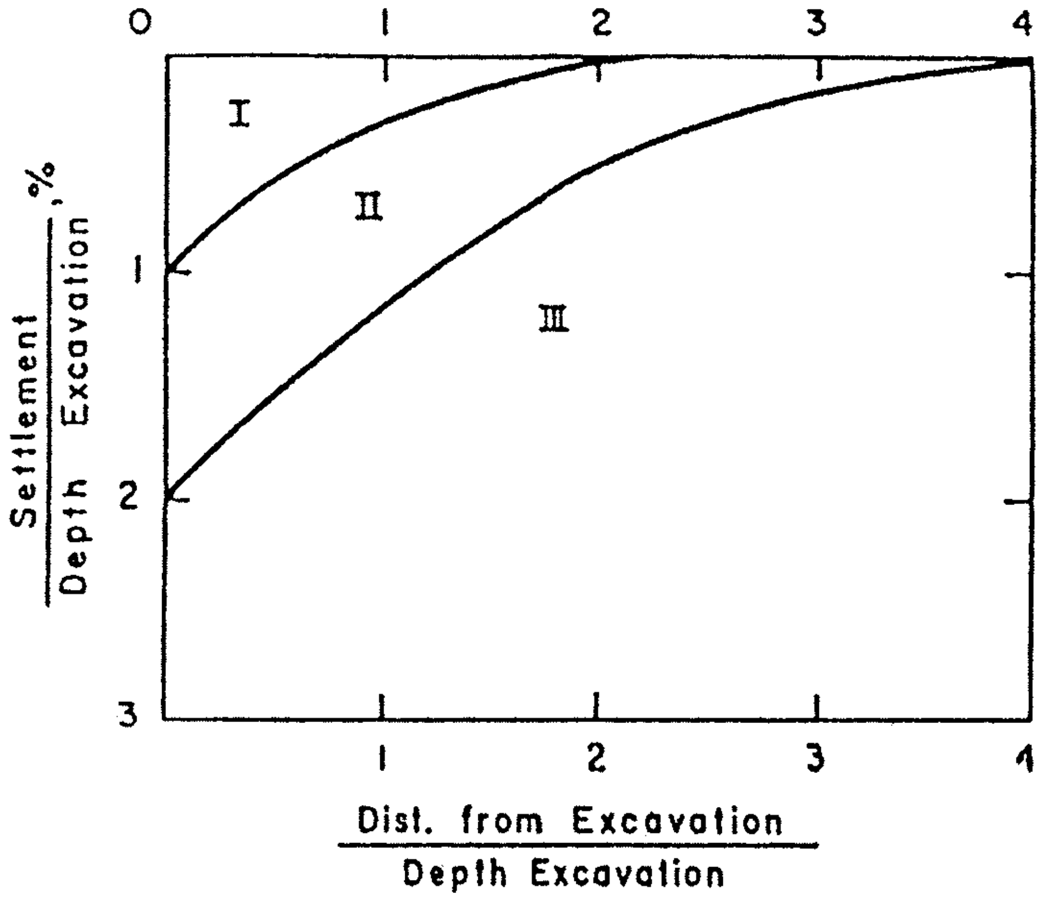


FIGURE 2.8: Typical excavation sequence for cross-lot bracing: A) V-cut initial cantilever excavation, B) Strut installation and pre-loading in small trenches in soil berm, C) V-cut excavation to the next level and strut installation, D) Final grade. (Konstantakos, 2000)



I - Sand and Soft to Hard Clay, Avg. Workmanship

II - Very Soft to Soft Clay
1. Limited Depth of Clay Below Bott. Exc.
2. Significant Depth of Clay Below Bott. Exc., But $N_b < N_{cb}$*

III - Very Soft to Soft Clay to a Significant Depth Below Exc. Bott. and $N_b > N_{cb}$

* N_b = Stability No. Using C "Below Base Level" = $\frac{\gamma H}{C_b}$
N_{cb} = Critical Stab No. for Basal Heave

FIGURE 2.9: Summary of observed settlements behind sheet pile and soldier pole walls. (Peck, 1969)

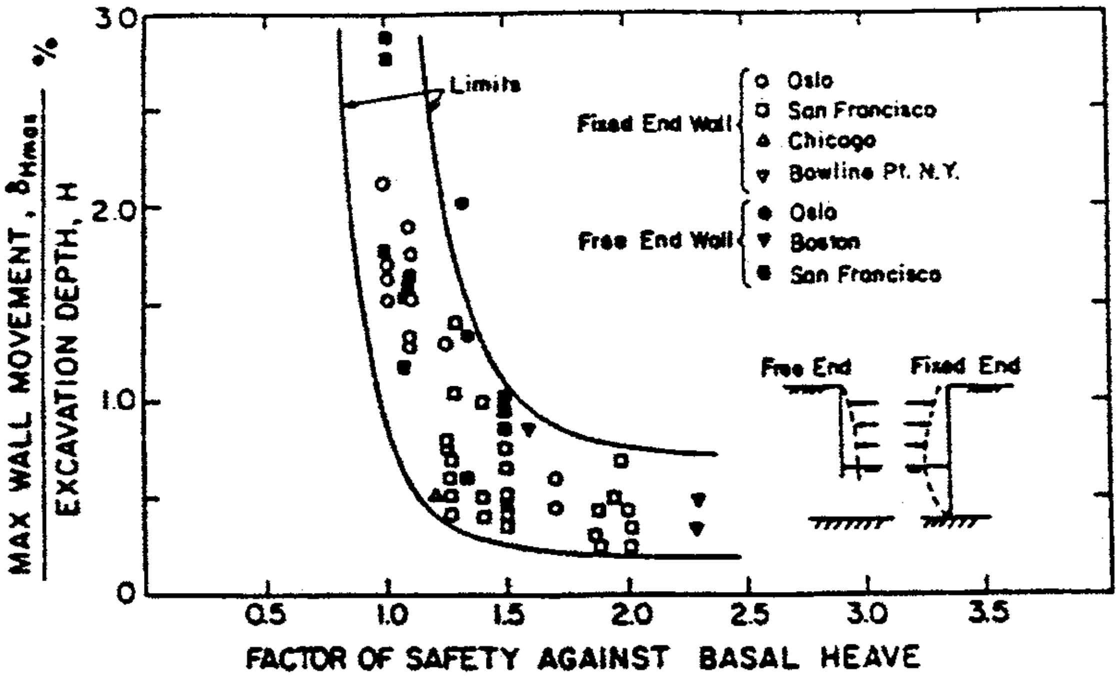


FIGURE 2.10: Correlation between basal heave stability and measured wall deflections. (Mana and Clough, 1981)

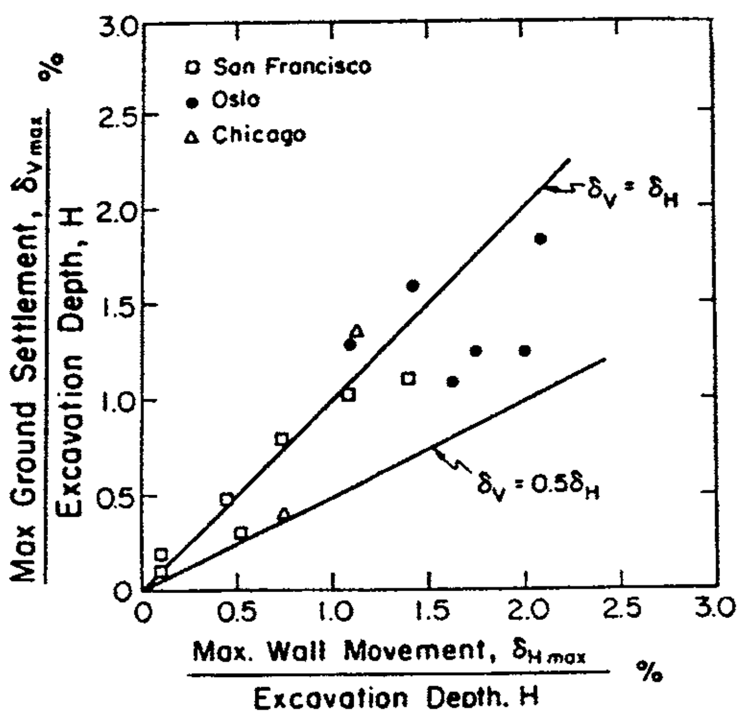


FIGURE 2.11: Correlation between maximum ground settlement and maximum lateral wall movement based on case history data. (Mana and Clough, 1981)

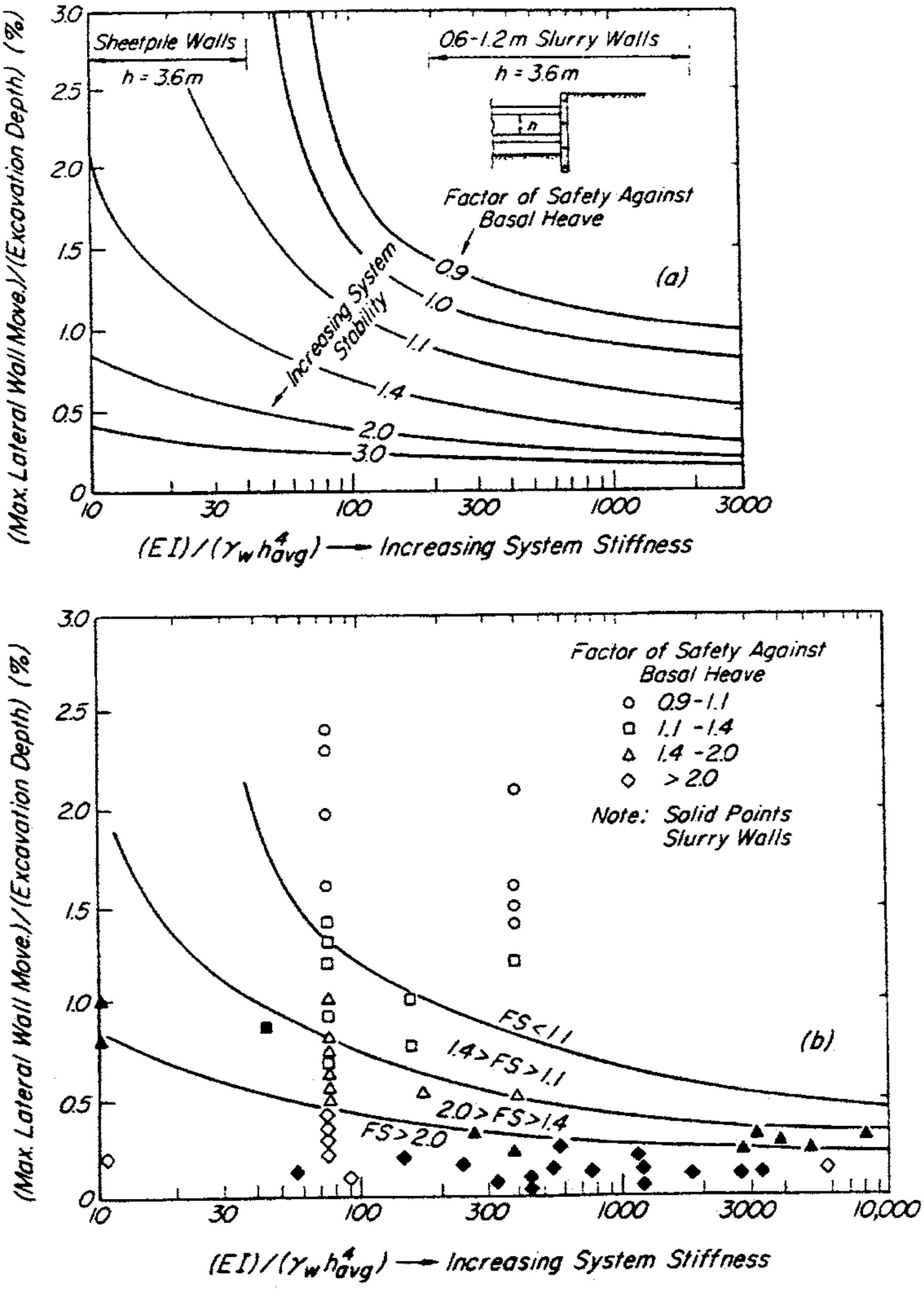


FIGURE 2.12:Correlation between maximum lateral wall movements, system stiffness, and factor of Safety against basal heave for cuts in plastic clay (Terzaghi, Peck, and Mesri, 1996): a) Calculated by finite element solutions, b) comparison with field measurements (after Clough et al., 1989)

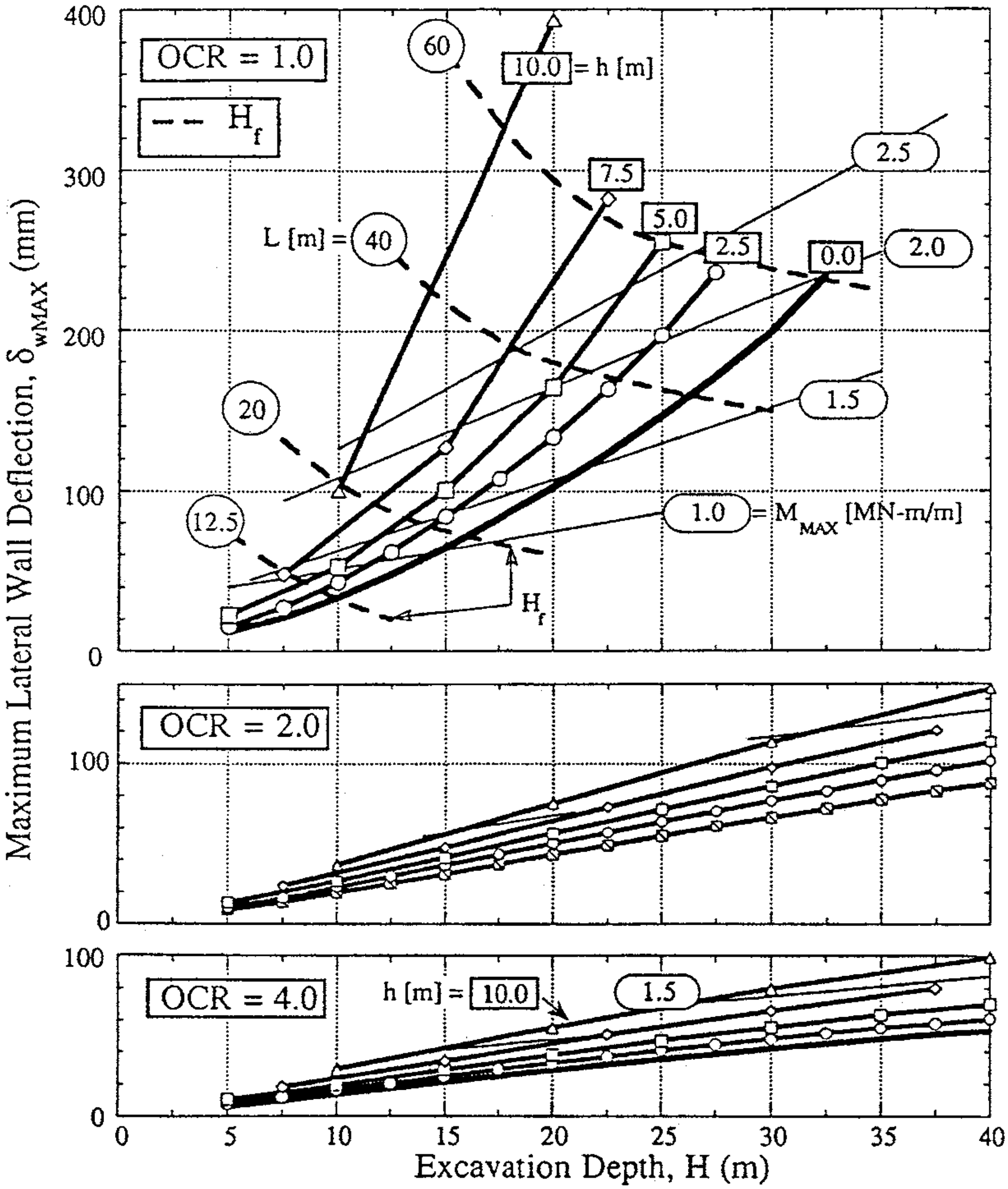


FIGURE 2.13: Estimation of maximum lateral wall deflections from numerical experiments for excavation in Boston Blue Clay. (Hashash and Whittle, 1996)

ID	Project Name	Year	Depth(ft) H	Depth(ft) D	Thick (inches)	Soil Type*	Bracing	Wall Type	Toe Fixity
B1*	MBTA South Cove	1969	50	30	36	A	3-Lev CLB	RCDW	√
B2	60-State Street	1975	35	27	30	A, B	3, 2 –Lev TB	RCDW	√
B3	State Transportati on Building	1982	27	19	24	B	2-Lev TB	RCDW	
B4 **	75 State Street	1984	65	35	30	A	6-Lev TD	RCDW	√
B5	Rowes Wharf	1984	55	15	30	B	5-Lev TD	RCDW	√
B6	One Memorial Drive	1985	30	11	24	A	2, 1-Lev. TB	RCDW	
B7	500 Boylston	1987	42	14	24	A	4 Lev. TB or 1-TB, 2 R	RCDW	
B8	Flagship Wharf	1989	47	13	30	C	3-Lev CLB	PT	√
B9 *	125 Summer Street	1990	60	?	30	B	5-Lev TD	RCDW	√
B10	Post Office Square Garage	1989	75	12	36	A	7-Lev TD	RCDW	√
B11	Beth Israel Deaconess	1994	65	24	36	B	5-Lev TD	RCDW	√
B12	Dana Farber Tower	1995	60 90	2	36	B	4-Lev TB 6-Lev TB	RCDW	√
B13	Millennium Place	1999 2000	55	41	36	B	5-Lev TD	RCDW	√

Note: * According to Johnson 1986, **H** – Excavation Depth, **D**- Embedment depth, **TB** - Tiebacks, **CLB** - Cross-Lot Bracing, **TD** - Top/Down, **R** – Rakers, **SB**- Soil Berms, **CB** – Corner Bracing, **IB**-Internal Bracing, **PC** – Precast, **PT** – Post Tensioned, **SP** – Soldier Piles, RCDW – Reinforced Concrete Diaphragm Wall, **SPTC** – Soldier Piles & Tremie Concrete.

FIGURE 2.14: List of studied slurry wall excavations in Boston. (Konstantakos, 2000)

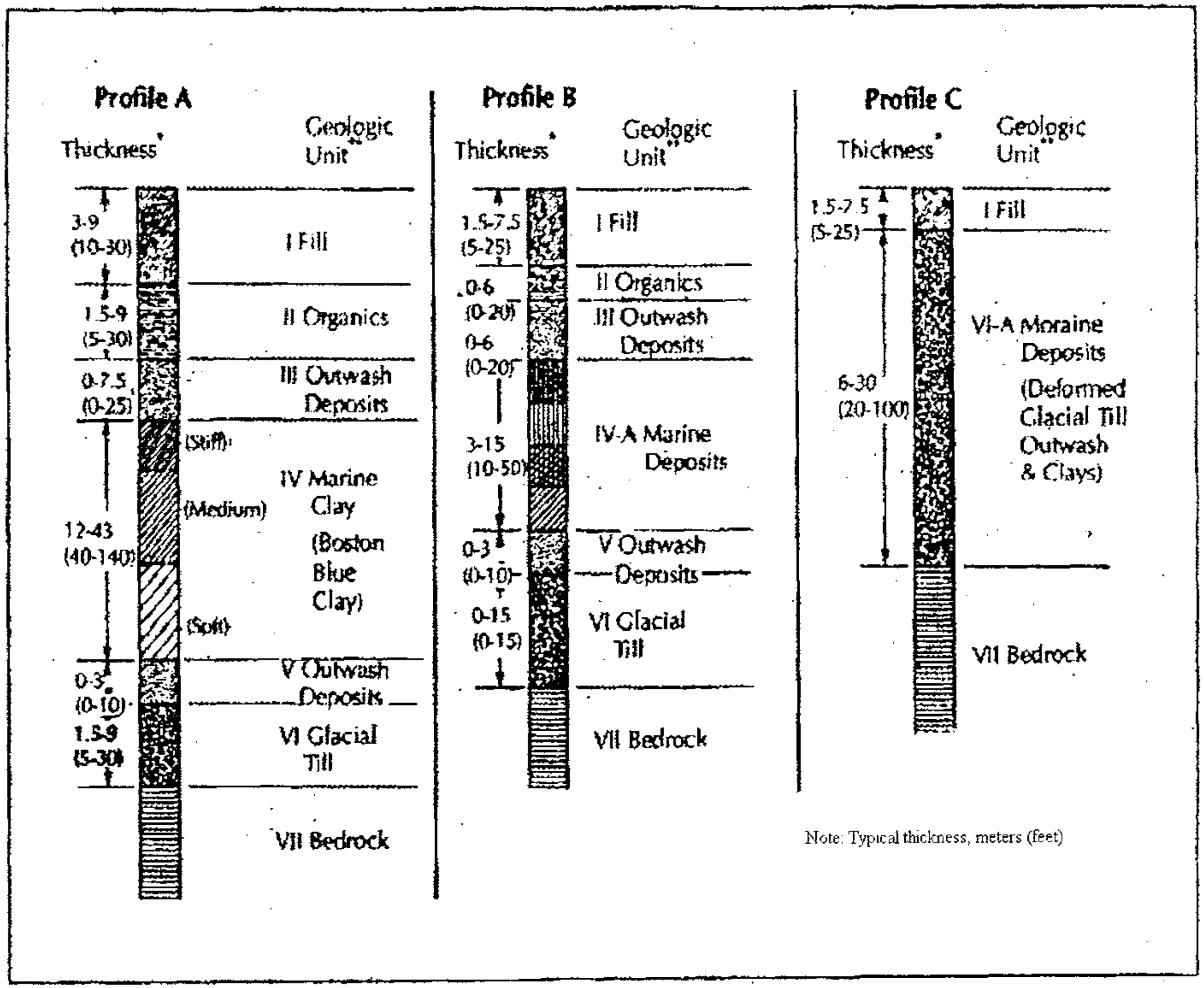


FIGURE 2.15: Geological units encountered in typical major foundations in Boston. (Johnson, 1989)

Chapter 3
Site Information

3.1 Location

The MIT campus is located on approximately 154 acres of predominately reclaimed land on the north bank of the Charles River in Cambridge, Massachusetts (Figure 3.1). Previous to 1889 much of the campus was tidal marsh of the Charles River, but in the period from 1889 to 1899 the sea wall was built and the land was hydraulically filled with material dredged from the river. Then, in 1903 the Charles River Dam was authorized and it was completed in 1910.

Construction of the "main" building complex at MIT was completed in 1915 and 1916, during which time the ground level was raised further to the current elevation of approximately +21 ft. Cambridge City Base(CCB). The MIT campus continued to grow over time. From 1920 to 1960, MIT built an average of five buildings per decade; there was a major expansion program in the period from 1960 to 1973 including the construction of more than twenty new buildings. Since that time, construction has been much slower. However, the Institute recently began another program of revitalizing the infrastructure that will transform the campus. The current plans include the Stata Center (May 2000-Fall 2003), a new sports and fitness center (Fall 2000-June 2002), new dorms (3 from September 2000-August 2002), and expansion of the Media Lab (Spring 2001-December 2003); and many existing building are, or will be, undergoing major renovations. Some of the projects (both ongoing and planned for the near future) are shown in Figure 3.2. Also, many new projects are on the drawing boards, not having not been announced, and most open spaces on campus are being looked at for new construction.

3.1.1 Site History

The Stata Center site is located in the Northeast section of campus as shown in Figure 3.2. Most of the site is part of the area of campus that was once a tidal marsh and was filled in the late 1800s. It is unclear what the area was used for prior to its acquisition by MIT, but sometime after the construction of the main campus buildings the site was turned into athletic fields. The site continued as athletic fields until the 1940s when, due to needs for innovations to help the allied efforts in World War II, Building 20 was constructed. It was a three story wooden building with no basement and six wings, and was only intended for use during the war and for a period of six months afterwards. It was initially home to the Radiation Laboratory that designed much of the radar used in the Second World War. Over the years the buildings was used by at least 50 different groups. In 1998, the building was finally demolished and the site was cleared in preparation for construction of the Stata Center.

3.1.2 Adjacent Structures

As can be seen in figure 3.3, the Stata Center site is surrounded on all sides by structures that impact both the performance and the design of the excavation support system. A summary of the building foundations for the surrounding area of the MIT campus information is given in Table 3.1 and those in close proximity are described below. Figure 3.4 shows the locations and foundation systems of the adjacent buildings.

- **Building 36** – Building 36 is a 9-story concrete building with two below-grade levels located to the west of the site near the northwest corner, 24-ft from the diaphragm wall. It is built on a mat foundation bearing on the marine sand at El. –2 ft.

- **Building 26** – Building 26 is a 5-story concrete building with a penthouse and one below-grade level located to the west of the site 54 ft from the diaphragm wall. The foundation system is footings with a strip footing in the center bearing on the marine sand at El. 1 ft.

- **Building 57** – Building 57, the Alumni Pool Building, is a 3-story building with a swimming pool that extends below grade. It is located to the south of the site at a distance of approximately 3.5 ft from the diaphragm wall. The building was built in 1940 and is supported by belled concrete caissons bearing primarily in the marine sand at El. –12 ft. The layout of the caissons can be seen in Figure 3.5. These caissons have a typical shaft diameter of 3-ft with the bells varying from 3 to 10 ft. The loads supported by the caissons vary from 80 to 320 kips. The bells were designed so that each caisson, no matter what the load, transmitted a bearing pressure of 3000 psf to the top of the marine clay. In the area below the pool and below the northeast portion of the building the weight of excavated soil slightly exceeded the building, therefore in these sections the net load is negative (Taylor, 1944).

- **Building 70** – Building 70 is a parking garage with 8 stories with 1 below-grade level located to the east of the site with the nearest point 28-ft from the diaphragm wall. The foundation system comprises pressure injected footings bearing in the marine sand and extending to El. –6.25 ft.

- **Vassar Street** – Vassar Street is a two lane road that is approximately 45 ft wide located to the north of the site and at its nearest point is 20 ft from the diaphragm wall. Numerous utilities are located beneath the street and have been carefully considered in the design of the excavation support system.

3.2 Geologic History of MIT

In recent years, the geologic history of the Boston and Cambridge area has been a controversial issue. Debate has generally been focused on the number of episodes of glaciation that reached the area and the correct classification of the layer overlying the bedrock. This material has traditionally been considered glacial till when it may actually be a glaciomarine deposit. The geological origin (glacial till or a glaciomarine deposit) is of little concern to this study, therefore, the following description of the geologic history of MIT is based on the original ideas outlined in Horn and Lambe (1964) and Aldrich (1981).

The soils that are present in this area are believed to have originated during the Pleistocene Era, which is a period that was characterized by successive advances and retreats of glaciers followed by extreme variations in climate and sea level. The geologic profile is presented in Figure 3.6. The bedrock that underlies the campus is part of the Cambridge Slate formation that underlies the Boston and Cambridge area, and is referred to as Cambridge Argillite. It consists of shale with some slate, which was formed during the Permian-Carboniferous periods. The depth to bedrock is quite erratic over the entire campus and the upper part of the rock is generally highly weathered, fractured, and quite soft.

The glacial till that overlies the argillite was deposited at the base of glaciers during the Boston Substage of the Wisconsin Glaciation. The thickness of the layer varies over the campus from nonexistent up to 17 feet. It is a very compact and heterogeneous mixture of very dense gravelly sand or sandy gravel with cobbles and boulders. It also contains roughly 10 to 30 percent fines by weight, and is generally gray in color.

A thick layer of inorganic clay, called Boston Blue Clay (BBC), overlies the glacial till and was also formed during the Boston Substage of the Wisconsin Glaciation. The clay was transported by proglacial streams to the waters of the Boston Basin and deposited in these quiet marine waters. The clay contains sand seams and the sand content may increase with depth in some locations. Occasional boulders in the lower portion of this layer are believed to be the result of ice rafting. The top 2 to 10 feet of the clay has a yellowish or buff color and is usually quite stiff. This color and stiffness is believed to have been caused by desiccation and oxidation which occurred when the sea level was below its present level in relation to the top of the clay. Although it is called Boston "Blue" Clay, the clay below this crust is gray or olive-green in color. The upper portion of this layer is overconsolidated and can be said to have a "medium" consistency while the lower portion is nearly normally consolidated and has a much softer consistency. The thickness of this layer varies from 45 feet to 160 feet and is generally thicker towards the western end of campus.

The next layer is a medium compact to compact well-graded gravelly sand that was deposited over the clay by fast moving streams. This occurred following a readvance of glacial ice during the Lexington substage of the Wisconsin Glaciation, 12,000 to

14,000 years ago. The thickness of this layer varies widely over the campus from a few feet to more then 20 feet. Within in a small area the thickness can vary erratically because of stream erosion that occurred on the stratum's surface.

As the glaciers receded and melted, the sea level in the Boston area rose and the glacial deposits, which comprised the campus area, were submerged. This area then became part of the Charles River Tidal Basin. The resulting river and tidal currents coupled with the slowly rising sea level deposited a layer of plastic organic silt. Due to migration of the shoreline over time, the thickness of this layer of salt marsh peat is quite erratic and its characteristics can vary considerably, even over short distances.

The surface layer, which overlies the organic silt, is a man-made fill that was placed to reclaim the land. The lower portion is comprised of fill that was dredged from the bottom of the Charles and pumped onto campus during the late nineteenth and early twentieth centuries in order to raise the grade a safe distance above the water level. After the original dredge spoils were dumped, miscellaneous fill was dumped at various time to further raise the site. This fill is comprised mainly of sand and gravel and city waste.

3.3 Site Investigations

In order to accurately model the stratigraphy for the Stata Center and its surroundings, a large amount of information needs to be compiled. Figure 3.7 shows the location of all borings used in determining the stratigraphy.

3.3.1 Past Investigations

Over the years, a large amount of information has accumulated concerning the subsurface conditions at MIT. Although much of the information has been published in technical journals, textbooks, and MIT research reports, most of it is not centrally organized. Beyond that, much of the information can only be found in reports issued by consulting firms. References for some of the professional papers, research reports, students' thesis, and consulting reports relating to the subsurface conditions at MIT can be found in Appendix A. A large number of borings from the site investigations of surrounding buildings and previously existing buildings were reviewed and used to extend the understanding of the stratigraphy as far as possible beyond the immediate construction site. These borings date back to the original construction of the institute buildings in 1914.

3.3.2 Recent Investigations

In preparation for the design and construction of the Stata Center four exploration programs were undertaken to get a clear understanding of the soils within the footprint of the proposed complex. The four exploration programs were, 1) the preliminary test boring program, 2) the soil pre-characterization program, 3) the test boring program, and 4) the cone penetrometer test program. The following descriptions are according to information obtained from Haley & Aldrich (Haley & Aldrich, June 2000).

The preliminary test boring program was undertaken in November of 1998 in order to obtain information on the subsurface soil and groundwater conditions for design and construction of the proposed structures with a one-level basement 15 to 18 feet deep. This would have meant a structure bearing in the marine sand; therefore the characteristics of the marine deposits were of primary interest. As a result none of the borings extended to the base of the clay. Other objectives of this program were to characterize chemical contaminants in the fill soils and groundwater, as preliminary information on the environmental conditions. This program consisted of seven test borings (HA-1 through HA-7, Table 3.2) that were drilled to depths ranging from 46 to 51 ft, and the installation of two groundwater observation wells (HA-1 and HA-5). The locations of these borings can be found in Figure 3.8, and the data is summarized in Table 3.2.

The soil pre-characterization program was under taken in July, November, and December of 1999 and consisted of 57 borings ranging in depth from 9 ft to 42 ft. The objectives of this second program were to characterize the soil conditions at the site relative to the presence of oil and hazardous materials, to classify the material to be excavated for on-site reuse or off-site disposal/recycling/treatment, to assess the approximate soil quantities for each disposal and treatment option, and to generate additional data on the depth to the marine sand and the marine clay for the geotechnical design of the mat foundation. The locations of these borings can be found in Figure 3.8, and the data is summarized in Table 3.2.

The test boring program was undertaken in November and December of 1999 and was comprised of two borings. These borings, designated B-101 and B-102, extended to the bedrock at depths of 134.5 ft and 126.0 ft, respectively. The purpose of these borings was to collect tube samples of the marine clay in order to determine its properties and to determine the depth to the till and the bedrock. The locations of these borings can be found in Figure 3.8, and the data is summarized in Table 3.3.

Piezocone penetrometer testing was carried out during the beginning of December 1999 and comprised 10 continuous pentration records to a depth varying from 92 ft to 130 ft. The objectives of this program were, 1) to identify interfaces between principal soil layers, 2) to investigate sand layers and lenses within the marine clay, 3) to provide shear wave velocity[1] data profiles, and 4) to identify relative changes in stress history and undrained shear strength of the marine clay. The cone penetrometer test logs can be found in Appendix A and the information on the strata determined from them can be found in Table 3.3 and the location is shown in Figure 3.8.

3.3.3 Subsurface Soil and Rock Conditions

The subsurface profile underlying the Stata Center is consistent with the typical profile discussed in the section on the geological history. Using all of the reliable boring data available, contour maps were generated showing the elevation for the surface of each of the five main layers (argillite, till, clay, sand, organics) with the ground surface at approximately level at El. +21. These contour maps can be found in Figures 3.9 through 3.13. Five subsurface profiles (marked out on Figure 3.14) showing soil and water conditions, adjacent structures, and the Stata Center excavation are shown in Figures 3.15 and 3.16.

The subsurface has been divided into six layers as follows:

1) Miscellaneous Granular Fill – This layer consists of coarse to fine sand or gravelly sand, containing varying quantities of cinders, glass, brick, rubble, and wood. It varies in density from loose to dense with an average SPT N-value of 15 and ranges in thickness from 8.0 to 19.0 ft.
2) Organic Silt and Peat – The organic silt is a soft, gray, compressible soil containing few fibers, and interbedded with peat that is soft, brown, and

[1] The peizocone was specially equipped with seismic accelerators in order to make this measurement possible.

fibrous. Some fine sand and shells are mixed in. The layer thickness varies from 4 to 22 ft.

3) Marine Sand – The marine sand is a dense, gray, fine to medium sand with some gravel. This stratum ranges in thickness from 0.5 to 16.5 ft. and the top of the layer is located at depths ranging from 15 to 30 ft. (El. +7 ft to –9 ft). It should be noted on the contour map for the surface of this layer (Figure 3.10) that there appear to be troughs in the northeast corner and running through the middle of the site from north to south that were probably caused by stream erosion after the sand was deposited. The SPT blow counts ranged from 19 to 32 blows/ft.

4) Marine Clay – The Boston Blue Clay is made up of medium stiff to stiff, silty clay with occasional fine sand seams. This stratum is typical of the Boston area with an upper layer consisting of a stiff crust, becoming softer and more compressible with depth. The thickness of the clay varies greatly from 60 to 90 ft within the footprint of the Stata Center and varies from 20 ft to 100 ft within the entire area studied. Figure 3.11 Shows that the surface elevation of the clay only varies by about 10 feet (from El. –4 ft to –14 ft) within the footprint of the building, and by looking at the contour map for the till (Figure 3.12), it can be seen that the clay layer tends to be thinnest in the northeast corner and getting thicker towards the southwest.

5) Glacial Till – This is a heterogeneous layer of dense sand, gravel, silt, clay and boulders. The trend mentioned above of the clay becoming thicker indicates that in this area the till appears to be a glacial in nature and when the

sea rose formed the bank and the sea bed, after which the clay was deposited above it. Figure 3.12 shows that the till is shallowest in the northeast and deepest toward the south and southwest. This layer appears to vary in thickness from 5 ft to possibly 60 ft in some places with it averaging roughly 12 ft in thickness.

6) Bedrock -- The bedrock is a part of the Cambridge Argillite formation, which is a relatively soft gray sedimentary rock made up of silt and clay size particles. The surface is highly weathered, but with increasing depth it becomes moderately hard but still fractured. The bedrock was encountered at elevations ranging from El. –100 ft to –110 ft. Figure 3.13 show that the surface of the bedrock has the same trend as the till, with it being shallowest in the northeast and deepest toward the south and southwest.

3.3.4 Groundwater Conditions

Two groundwater observation wells were installed in the completed test borings HA-1 and HA-5 on the northeast and southwest sides of the site, respectively. Groundwater was observed in completed test borings at elevations ranging from El. 9.4 ft to El. 14.9 ft. The water level in the wells was then checked again before excavation work began and was found to be at El. 14.0 ft and El. 14.2 ft. There are many factors affecting the groundwater table elevation, including below-grade structures, precipitation, surface runoff, local construction activity, pumping of dewatering systems, leakage from utilities, and seasonal variations; not to mention the Charles River that is maintained in the range of El. 12 to El. 13. Although previous investigations have found that the water pressures in the clay and the till are higher than hydrostatic by about 5 ft to 7 ft of head

(Berman, 1993), there was no evidence to support this claim during investigations or construction. Therefore, hydrostatic pressures were used in all calculations and design.

3.4 Engineering Properties of Soils

A geotechnical laboratory testing program was undertaken primarily to provide data on the engineering properties of the Boston Blue Clay. Laboratory testing was performed in accordance with ASTM standards or equipment manufacturers' recommended procedures by Haley & Aldrich and the MIT Geotechnical Laboratory. The following information comes from the description in Haley & Aldrich(Haley & Aldrich, June 2000).

Five undisturbed tube samples were obtained from test borings B-101 and B-102 during drilling using a heavy drilling mud to limit disturbance of the clay. Brass thin-walled Shelby tubes (3 in diameter by 24 in long) were used in collection the soil and the tubes were then stored in a humidity-controlled room. The tubes were radiographed to assess disturbance, macro-fabric, the presence of stones, and to determine the best location to find representative specimens for Constant Rate of Strain Consolidation (CRS) testing.

Index tests, including water content, total density, torvane strength, and Atterberg Limits, were run on the sections of the tube samples, on the trimmings from the consolidation sample preparation, and on the consolidation test specimens themselves. A total of twenty one-dimensional consolidation test were conducted on clay specimens from the tubes using the CRS test procedure at the MIT Geotechnical Laboratory. A summary of this test data can be found in Table 3.4 and in Figures A-1 through A-19 in Appendix A.

The preconsolidation pressure of the clay specimens was estimated using the Strain Energy technique (Becker et al, 1986) and the overconsolidation ratio (OCR) was calculated using the effective stress at the location of sample using total unit weights obtained from Berman et al (1993). The lab data was then used to correlate the results of the cone penetrometer test data to the preconsolidation pressure and OCR. Correlations were then developed using lab data and cone data from locations P-4 and P-5 (adjacent to B-101 and B-102) using the procedure described in Berman et al (1993). The correlations used were mostly empirical and would take up a large portion of this chapter to explain them and are beyond the scope of this thesis. For more information the reader is advised to look at Berman, 1993. Undrained shear strength profiles were developed for the lab and cone data using the SHANSEP equation (Ladd el al. 1997):

$$\frac{S_u}{\sigma'_{vo}} = S(OCR)^m$$

The SHANSEP parameters, m=0.77 and S=0.205, were used to calculate the shear strength; and using the regression line through the lab preconsolidation pressure data points, undrained shear strength profiles were developed for test borings B-101 and B-102 and the correlations were used to calculate the shear strength for each of the cone locations. A large number of plots were developed to represent the data. The graphs that represent the results of lab analyses including the determination of the preconsolidation pressures from the Constant Rate of Strain Consolidation (CRS) test were provided by the MIT Geotechnical Laboratory and Dr. Germaine. A list of these plots can be found in

Table 3.4 and the plots themselves can be found in Figures A-1 through A-19 and A-40 through A-39 in Appendix A.

A typical cross-section of the soil in the middle of the site would consist of 9 ft of fill (El 21 ft to El 12 ft), 10 ft of organics (El 12 ft to El 2 ft), 7 ft of sand (El 2 ft to El –5 ft), 85 ft of clay (El –5 ft to El –90 ft), 10 ft of glacial till (El –90 ft to El –100 ft), and below that bedrock. Estimated unit weights for this part of campus are summarized in Table 3.6. Key characteristics and properties of the clay deposit can be found in Figures 3.17 through 3.19. These figures include profiles of the in-situ effective stress, maximum past pressure, undrained shear strength, and the atterberg limits and natural water contents versus elevation.

TABLE 3.1 SUMMARY OF AVAILABLE INFORMATION ON SURROUNDING BUILDINGS

Building Number	Building Use	Year of Construction	Number of Floors	Number of Basements	Elevation of Lowest Floor (ft)	Elevation Range of Fdn. Support (ft)	Type of Foundation	Fondation Bearing Stratum	Foundation Design Loads
1 to 11 (except 9)	Main Buildings	1915 to 1916	3 to 5	1	17.1	-33.0	Timber Piles both end bearing and friction	End bearing-Sand/Gravel: Friction 18' into BBC	6.7 to 11 tons/pile
12 & 12A		1941	1	1	17.1	NA	Timber Piles	Sand/Gravel	12 tons/pile
16	Dorrance Laboratory	1951	9	1	10.28	3.0 to -7.2	Spread ftgs/ Belled Caissons	Sand/Gravel	4 tons/ft^2
24	Synchrotron	1941	6	0	20.1	0.0	Belled Caissons	Sand/Gravel	NA
26	Compton Laboratory	1955	5 + Penthouse	1	10.42	1.0	Spread ftgs/Strip ftgs in center	Sand/Gravel	NA
36	Electrical Engineering	1971	9 + roof	2	1.93	-2.0	Mat	Sand/Gravel	NA
44	Cyclotron	1962	2	0	21.8	7.0	Belled Caissons	Sand/Gravel	NA
45	Department of Facilities	1977	1	0	22.0	NA	Caissons	NA	4 tons/ft^2

TABLE 3.1(cont.) SUMMARY OF AVAILABLE INFORMATION ON SURROUNDING BUILDINGS

Building Number	Building Use	Year of Construction	Number of Floors	Number of Basements	Elevation of Lowest Floor (ft)	Elevation Range of Fdn. Support (ft)	Type of Foundation	Fondation Bearing Stratum	Foundation Design Loads
48	Hydrodynamics Lab	1949	4	1	12.8	7.6	Mat	Sand/Gravel	NA
54	Green Bldg	1962-64	20	1	10.6	NA	Thin shelled concrete filled piles	glacial till	50 tons/pile
56	Whitacker Bldg	1964	9	2	-5.17	-11	Mat	Sand/Gravel	NA
57	Alumni Swimming Pool	1940	3	1	7.75	1.25 to -12.0	Gow Caissons	Sand/Gravel	4 tons/ft^2
62 & 64	Alumni House	1923-27	5	1	14.6	NA	Timber Piles	Sand/Gravel	NA
66	Chemical Engineering	NA	5	1	NA	NA	NA	NA	NA
68	Biology Bldg	1991	6 + penthouse	2	-14.1	-22.5	Mat	Clay	NA
70	Parking Garage	1960	8 levels	1	18.45	1.25 to -6.25	Pressure Injected Footings (PIF)	Sand/Gravel	100 tons/PIF
N4	Parking Garage	NA	3	1	NA	NA	NA	NA	NA

TABLE 3.2 SUMMARY OF SHALLOW SUBSURFACE EXPLORATIONS (from Haley & Aldrich,June 2000)

BORING NO.	GROUND SURFACE EL.	DEPTH OF BORING (ft)	THICKNESS (ft)				TOP OF SAND		TOP OF CLAY		GROUNDWATER	
			MISC. FILL	ORGANIC DEPOSITS	MARINE SAND	MARINE CLAY	DEPTH (ft)	ELEV. (ft)	DEPTH (ft)	ELEV. (ft)	DEPTH (ft)	ELEV. (ft)
HA-1(OW)	21.3	46.0	9.0	10.0	10.0	17.0	19.0	2.3	29.0	-7.7	8.0	13.3
HA-2	21.2	51.0	19.0	10.5	3.5	18.0	29.5	-8.3	33.0	-11.8	8.3	12.9
HA-3	22.9	56.0	13.0	6.5	9.0	27.5	19.5	3.4	28.5	-5.6	8.0	14.9
HA-4	21.3	51.0	11.0	17.0	5.0	18.0	28.0	-6.7	33.0	-11.7	8.5	12.8
HA-5 (OW)	20.0	51.0	13.0	13.0	7.0	18.0	26.0	-6.0	33.0	-13.0	6.0	14.0
HA-6	20.7	46.0	14.0	8.5	3.0	20.5	22.5	-1.8	25.5	-4.8	7.7	13.0
HA-7	20.9	46.0	9.0	5.0	16.5	15.5	14.0	6.9	30.5	-9.6	11.5	9.4
A-1	20.7	15.0	8.5	4.0	2.5*	-	12.5	8.2	-	-	NR	-
A-2	20.7	17.0	10.0	5.0	2.0*	-	15.0	5.7	-	-	NR	-
A-3	20.2	21.0	9.0	9.6	2.4*	-	18.6	1.6	-	-	NR	-
A-4	21.5	23.0	12.0	10.0	1.0*	-	22.0	-0.5	-	-	NR	-
A-5	20.3	28.0	10.0	15.8	2.2*	-	25.8	-5.5	-	-	NR	-
A-6	21.9	35.0	14.0	19.0	2.0*	-	33.0	-11.1	-	-	NR	-
A-6A	22.1	42.0	11.0	14.7	2.3	14.0	25.7	-3.6	28.0	-5.9	10.0	12.1
A-6B	22.1	9.0	9.0	-	-	-	-	-	-	-	NR	-
A-6C	21.1	9.0	9.0	-	-	-	-	-	-	-	NR	-
A-6D	20.9	9.0	9.0	-	-	-	-	-	-	-	NR	-
A-7	21.5	32.0	9.5	19.7	2.8*	-	29.2	-7.7	-	-	NR	-
A-8	21.1	29.0	8.9	17.1	3.0*	-	26.0	-4.9	-	-	NR	-
A-9	20.0	26.0	9.0	9.7	7.3*	-	18.7	1.3	-	-	NR	-
A-10	20.7	42.0	6.5	9.5	16.0	10.0*	16.0	4.7	32.0	-11.3	NR	-
B-1	20.9	17.0	8.0	6.5	2.5*	-	14.5	6.4	-	-	NR	-
B-2	21.7	17.0	8.9	7.1	1.0*	-	16.0	5.7	-	-	NR	-
B-3	22.4	21.0	9.0	9.5	2.5*	-	18.5	3.9	-	-	NR	-
B-4	23.1	23.0	10.0	10.5	2.5*	-	20.5	2.6	-	-	NR	-
B-5	22.9	29.0	12.0	14.0	0.5	2.5*	26.0	-3.1	26.5	-3.6	NR	-
B-6	20.9	33.0	9.5	18.5	2.0	3*	28.0	-7.1	30.0	-9.1	NR	-
B-7	20.9	29.0	9.5	19.0	0.*	-	28.5	-7.6	-	-	NR	-
B-8	20.1	29.0	9.5	18.5	1.0*	-	28.0	-7.9	-	-	NR	-
C-1	21.1	19.0	8.0	7.0	4.0*	-	15.0	6.1	-	-	NR	-
C-2	21.9	21.0	9.0	9.0	2.0*	-	18.0	3.9	-	-	NR	-
C-3	22.6	21.0	10.8	7.7	2.5*	-	18.5	4.1	-	-	NR	-
C-4	22.8	21.0	12.0	7.0	2.0*	-	19.0	3.8	-	-	NR	-
C-5	22.5	24.0	9.5	11.5	3.0*	-	21.0	1.5	-	-	NR	-
C-6	21.9	26.0	9.5	14.2	2.3*	-	23.7	-1.8	-	-	NR	-

TABLE 3.2 (cont.) SUMMARY OF SHALLOW SUBSURFACE EXPLORATIONS (from Haley & Aldrich,June 2000)

BORING NO.	GROUND SURFACE EL.	DEPTH OF BORING (ft)	THICKNESS (ft)				TOP OF SAND		TOP OF CLAY		GROUNDWATER	
			MISC FILL	ORGANIC DEPOSITS	MARINE SAND	MARINE CLAY	DEPTH (ft)	ELEV. (ft)	DEPTH (ft)	ELEV. (ft)	DEPTH (ft)	ELEV. (ft)
C-7	21.0	29.0	8.9	19.1	1.0*	-	28.0	-7.0	-	-	NR	-
C-8	20.4	31.0	8.5	16.0	6.5*	-	24.5	-4.1	-	-	NR	-
D-1	21.0	19.0	8.0	8.0	3.0*	-	16.0	5.0	-	-	NR	-
D-2	20.8	19.0	10.0	6.0	3.0*	-	16.0	4.8	-	-	NR	-
D-3	21.1	19.0	8.8	7.2	3.0*	-	16.0	5.1	-	-	NR	-
D-3A	21.4	42.0	10.0	6.5	7.5	18.0*	16.5	4.9	24.0	-2.6	NR	-
D-3B	20.6	9.0	9.0*	-	-	-	-	-	-	-	NR	-
D-3C	21.2	9.0	8.0	1.0*	-	-	-	-	-	-	NR	-
D-3D	21.8	9.0	7.1	1.9*	-	-	-	-	-	-	NR	-
D-4	21.1	42.0	7.5	9.5	9.5	15.5*	17.0	4.1	26.5	-5.4	8.0	13.1
D-5	21.9	22.0	12.0	6.0	4.0*	-	18.0	3.9	-	-	NR	-
D-6	20.9	42.0	8.5	16.5	6.0	11.0*	25.0	-4.1	31.0	-10.1	NR	-
D-7	20.6	30.0	4.5	25.5	-	-	-	-	-	-	NR	-
D-8	22.0	42.0	8.0	16.0	10.0	8.0*	24.0	-2.0	34.0	-12.0	NR	-
E-1	20.8	19.0	9.0	8.0	2.0*	-	17.0	3.8	-	-	NR	-
E-2	20.8	19.0	8.0	8.5	2.5*	-	16.5	4.3	-	-	NR	-
E-3	21.0	19.0	9.0	9.0	1.0*	-	18.0	3.0	-	-	NR	-
E-4	21.2	19.0	7.0	9.0	3.0*	-	16.0	5.2	-	-	NR	-
E-5	20.6	42.0	7.7	9.3	14	11.0*	17.0	3.6	31.0	-10.4	NR	-
E-7	20.2	42.0	8.5	25.0	NE	8.5*	-	-	33.5	-13.3	NR	-
F-1	20.7	28.0	7.0	18.0	3.0*	-	25.0	-4.3	-	-	NR	-
F-1A	21.0	42.0	6.0	8.0	11.5	14.0*	16.5	4.5	28.0	-7.0	NR	-
F-2	21.3	20.0	6.0	8.0	3.5*	-	16.5	4.8	-	-	NR	-
F-3	21.2	42.0	6.0	13.0	5.5	17.0*	19.5	1.7	25.0	-3.8	5.0	16.2
F-7	20.8	42.0	7.4	22.2	3	9.0*	30.0	-9.2	33.0	-12.2	NR	-
G-1	21.4	26.0	7.0	15.0	4.0*	-	22.0	-0.6	-	-	NR	-
G-2	21.4	16.0	12.0	4.0	-	-	-	-	-	-	NR	-
G-3	21.8	32.0	7.0	22.0	3.0*	-	29.0	-7.2	-	-	NR	-
G-4	21.2	16.0	10.8	5.2	-	-	-	-	-	-	NR	-

NOTES:

NR=Not Recorded

NE=Not Encountered

(*)=Test boring terminated in this stratum

Elevations refer to Cambridge City Base (CCB)as measured by Cullinan Engineering Co,Inc. and Haley & Aldrich, Inc.

TABLE 3.3 SUMMARY OF DEEP SUBSURFACE EXPLORATIONS (from Haley & Aldrich, June 2000)

BORING NO.	GROUND SURFACE EL.	DEPTH OF BORING (ft)	THICKNESS (ft)				TOP OF SAND		TOP OF CLAY		TOP OF TILL		TOP OF ROCK	
			MISC. FILL	ORGANIC DEPOSITS	MARINE SAND	MARINE CLAY	DEPTH (ft)	ELEV. (ft)	DEPTH (ft)	ELEV. (ft)	DEPTH (ft)	ELEV. (ft)	DEPTH (ft)	ELEV. (ft)
Test Borings														
B-101	22.3	134.5	12.0	6.0	10.0	99.0	18.0	4.3	28.0	-5.7	127.0	-104.7	132.0	-109.7
B-102	20.4	126.0	8.5	21.5	2.0	77.0	30.0	-9.6	32.0	-11.6	109.0	-88.6	120.5	-100.1
Piezocones														
P-1	20.6	119.6	10.1	5.5	13.0	89.0	15.6	5.0	28.6	-8.0	117.6	-97.0	119.6*	-99.0
P-2	20.9	120.9	11.9	14.0	3.0	85.0	25.9	-5.0	28.9	-8.0	113.9	-93.0	120.9*	-100.0
P-3	20.6	92.4	6.6	8.0	17.0	60.0	14.6	6.0	31.6	-11.0	91.6	-71.0*	-	-
P-4	22.1	129.7	11.6	5.5	10.0	89.0	17.1	5.0	27.1	-5.0	116.1	-94.0	129.7*	-107.6
P-5	20.4	123.3	8.5	20.9	2.0	76.0	29.4	-9.0	31.4	-11.0	107.4	-87.0	121.4*	-101.0
P-6	21.4	126.1	11.4	5.0	9.0	80.0	16.4	5.0	25.4	-4.0	105.4	-84.0	125.4*	-104.0
P-7	20.6	125.0	6.6	16.8	9.2	79.0	23.4	-2.8	32.6	-12.0	111.6	-91.0	120.6*	-100.0
P-8	21.0	94.3	8.0	8.0	12.0	63.0	16.0	5.0	28.0	-7.0	91.0	-70*	-	-
P-9	20.6	127.6	7.0	9.6	12.0	72.0	16.6	4.0	28.6	-8.0	100.6	-80.0	127.6*	-107.0
P-10	20.8	124.1	10.8	19.0	3.0	68.0	29.8	-9.0	32.8	-12.0	100.8	-80.0	122.8*	-102.0

Notes:

(*) signifies that the exploration was terminated in this stratum

Elevations are based on the Cambridge City Base (CCB), as surveyed by Haley & Aldrich, Inc.

TABLE 3.4 SUMMARY OF CONSTANT RATE OF STRAIN TEST RESULTS

Boring	Sample Designation	Test Number	Elevation (ft)	Water Content (%)	Density (gm/cm3)	Saturation (Gs=2.78)	Calculated Eff. Stress (psf)	Preconsolidation Pressue (psf)	OCR
B101	U1	CRS316	-21.16	37.58	1.878	1.008	2,858	11,981	4.19
B101	U1	CRS344	-20.03	39.98	1.841	0.998	2,797	8,806	3.15
B101	U2	CRS317	-31.07	39.55	1.845	0.997	3,394	8,602	2.53
B101	U2	CRS335	-30.03	41.39	1.833	1.006	3,338	9,830	2.94
B101	U3	CRS318	-40.99	43.22	1.814	1.006	3,931	9,421	2.4
B101	U3	CRS338	-39.95	40.65	1.826	0.991	3,875	6,861	1.77
B101	U4	CRS319	-51.45	39.88	1.845	1.001	4,506	6,861	1.52
B101	U4	CRS341	-49.95	39.42	1.841	0.998	4,416	7,885	1.79
B101	U5	CRS320	-60.35	42.39	1.817	1.000	5,041	8,397	1.67
B101	U5	CRS342	-59.31	36.86	1.879	1.000	4,978	8,806	1.77
B102	U1	CRS325	-27.72	33.92	1.912	0.996	2,562	7,680	3.00
B102	U1	CRS333	-26.76	38.91	1.858	1.003	2,510	11,776	4.69
B102	U2	CRS323	-38.06	36.13	1.888	1.000	3,121	6,349	2.03
B102	U2	CRS336	-36.85	39.73	1.849	1.003	3,056	9,011	2.95
B102	U3	CRS326	-48.06	41.55	1.829	1.003	3,662	7,373	2.01
B102	U3	CRS345	-47.39	35.77	1.886	0.994	3,626	7,680	2.12
B102	U4	CRS330	-58.79	37.86	1.866	0.999	4,307	6,144	1.43
B102	U4	CRS339	-57.84	42.93	1.817	1.006	4,239	5,734	1.35
B102	U5	CRS331	-73.06	24.98	2.045	0.993	5,154	8,192	1.59
B102	U5	CRS343	-71.93	24.51	2.037	0.975	5,086	9,011	1.77

TABLE 3.5 LAB TEST DATA SUMMARY PLOTS

Plot	Figure
Water Content vs. Elevation	A-1
Torvane Strength vs. Elevation	A-2
Total Density vs. Elevation	A-3
Water Content and Atterberg Limits vs. Elevation	A-4
Preconsolidation Pressure vs.Elevation	A-5
Overconsolidation Ratio vs. Elevation	A-6
Compression Ratio vs. Elevation	A-7
Recompression Ratio vs. Elevation	A-8
Swell Ratio @ OCR=2 vs. Elevation	A-9
Plasticity Index vs. Elevation	A-10
Swelling Strain @ OCR=3 vs. Elevation	A-11
Initial Hydraulic Conductivity vs. Elevation	A-12
Slope of Void Ratio vs. log(hyd. Conductivity) vs. Elevation	A-13
Coefficient of Consolidation vs. Elevation	A-14
Water Content vs. Torvane Strength	A-15
Torvane Strength vs. Preconsolidation Pressure	A-16
Water Content vs. Preconsolidation Pressure	A-17
Overconsolidation Ratio vs. Normalized Torvane Strength	A-18
Hydraulic Conductivity vs. Initial Void Ratio	A-19
Vertical Strain vs. Vertical Consolidation Stress	A-20 to A-39

Piezocone Plots	
Maximum Past Pressure vs. Elevation	A-40 to A-42
Overconsolidation Ratio vs. Elevation	A-43 to A-45
Undrained Shear Strength vs. Elevation	A-46 to A-49

TABLE 3.6: Total Unit Weights (Berman, 1993)

Soil Layer	γ_t (pcf)	γ_t (g/cc)
Fill above the water table	110	1.76
Fill below the water table	120	1.92
Organic silt and peat	100	1.6
Sand and Gravel	130	2.08
BBC above -45 ft	116.7	1.87
BBC below -45 ft	122.9	1.97

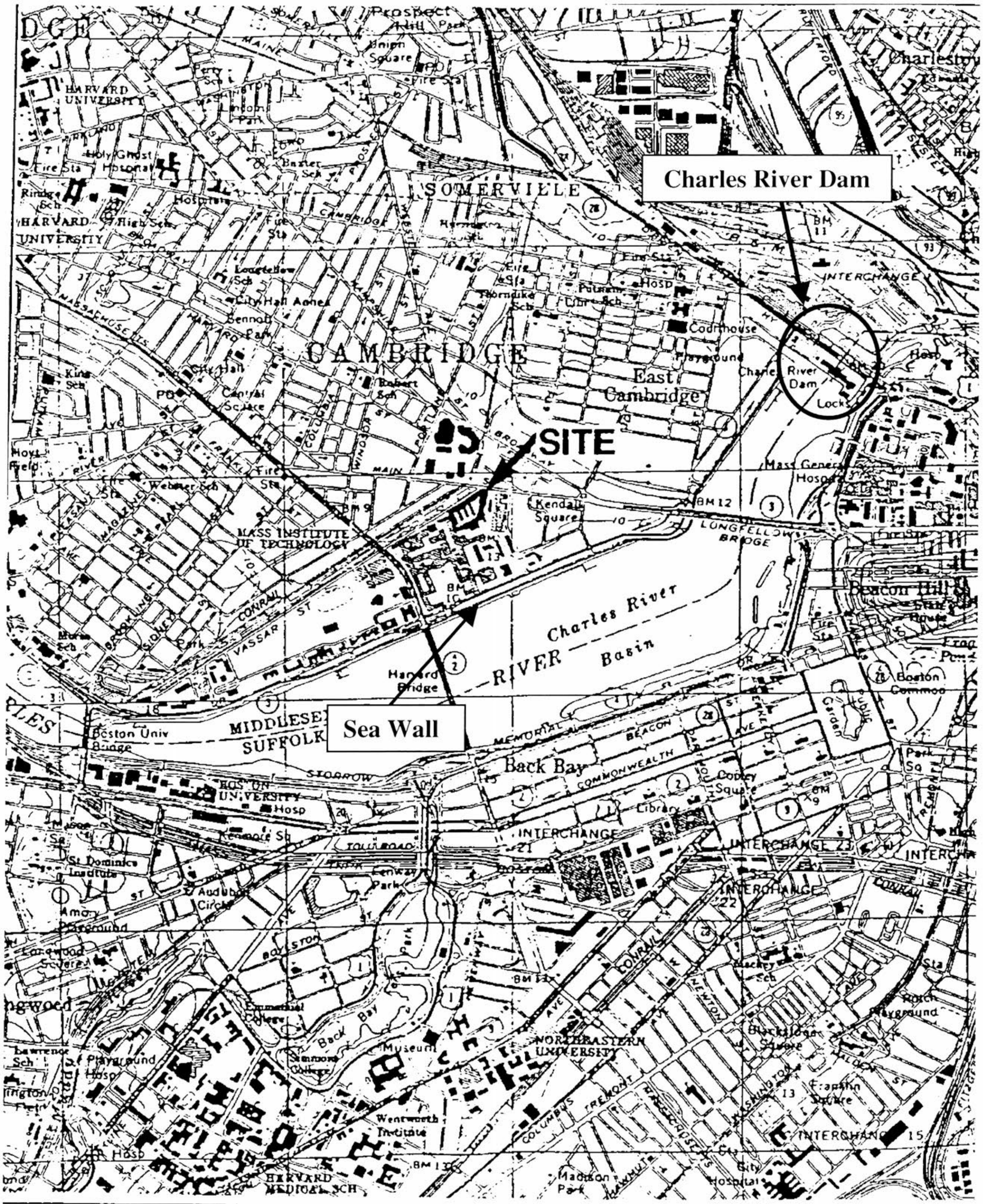


Figure 3.1: Project Location. MIT Campus, Cambridge, MA

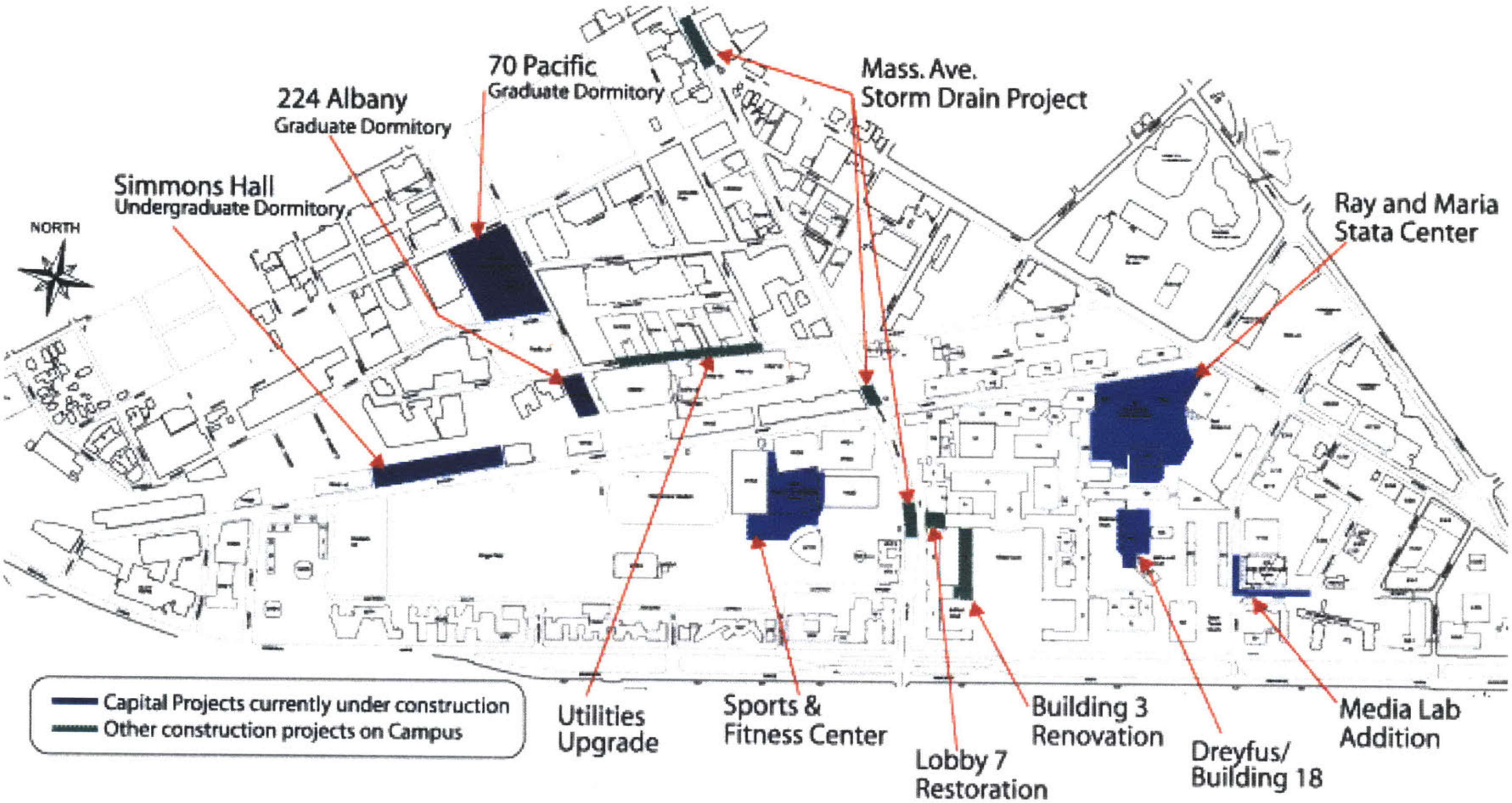


FIGURE 3.2: Map of MIT Campus showing major construction and renovation projects

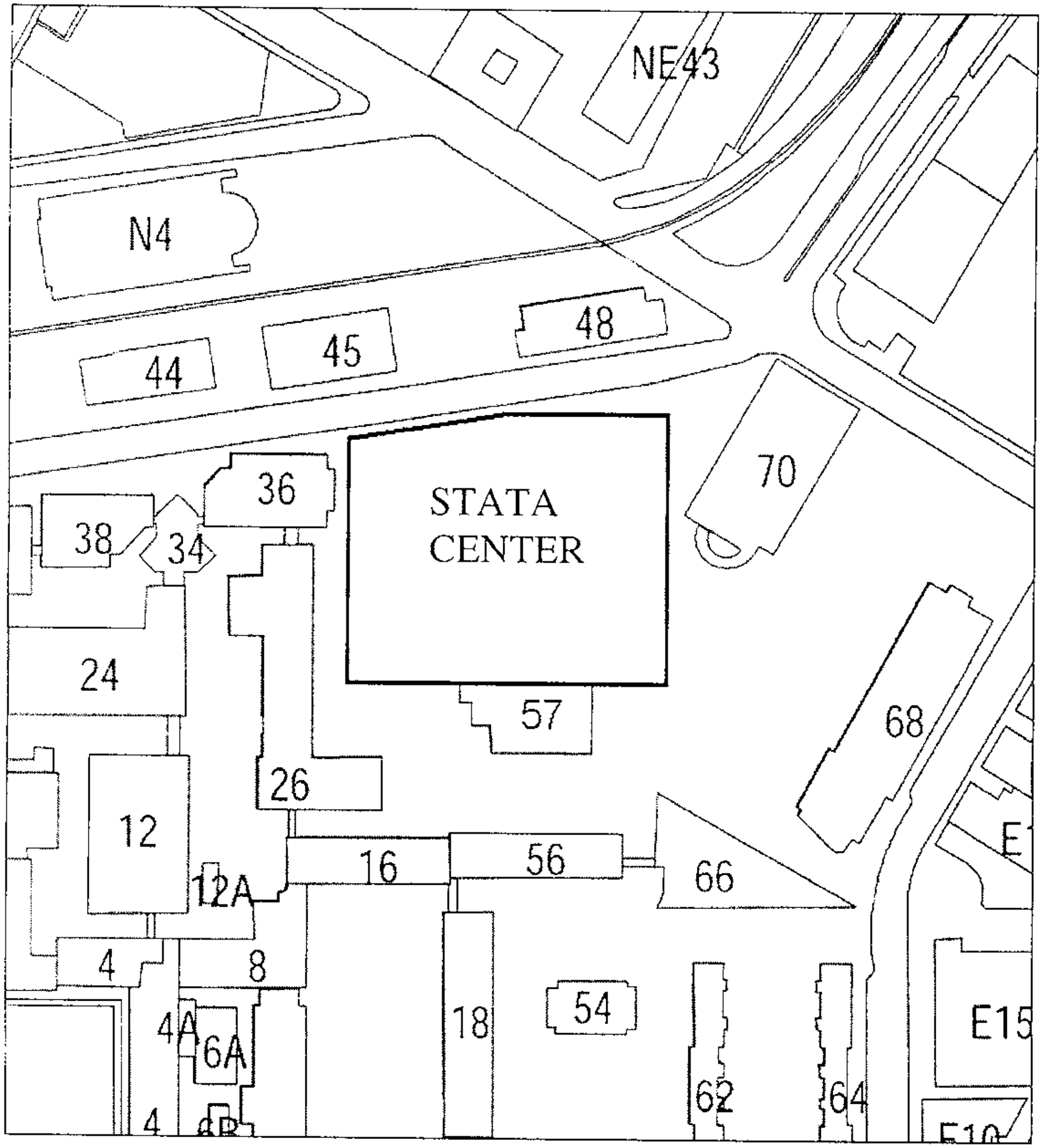


FIGURE 3.3 MAP OF SITE AND SURROUNDING BUILDINGS (not to scale)

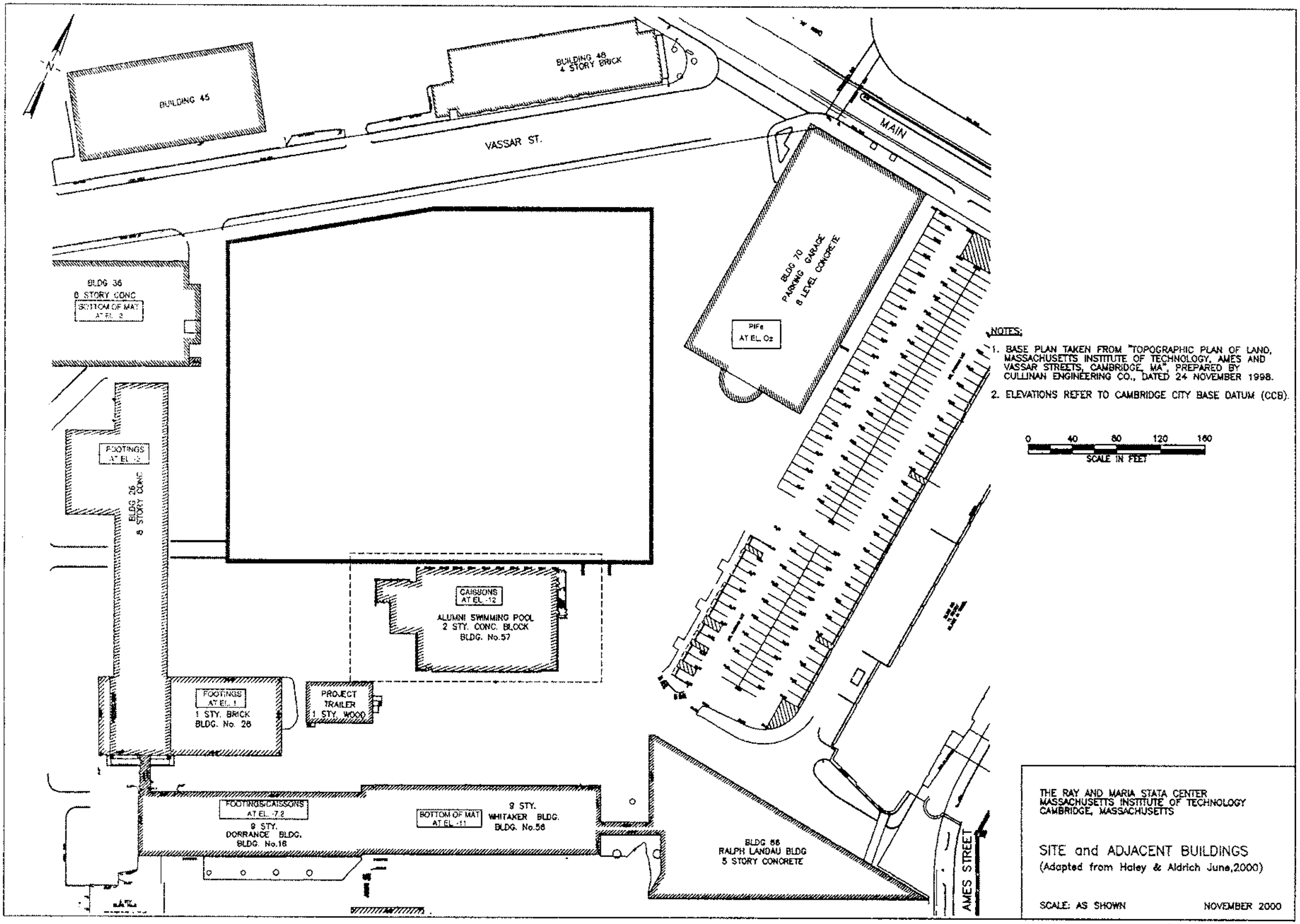

N
BUILDING 45
BUILDING 48
4 STORY BRICK
VASSAR ST.
MAIN
BLDG 36
8 STORY CONC
BOTTOM OF MAT
AT EL. 2
BLDG 70
PARKING GARAGE
8 LEVEL CONCRETE
PIFs
AT EL. 0±
FOOTINGS
AT EL. -2
BLDG 26
8 STORY CONC
CAISSONS
AT EL. -12
ALUMNI SWIMMING POOL
2 STY. CONC. BLOCK
BLDG. No.57
FOOTINGS
AT EL. 1
1 STY. BRICK
BLDG. No. 28
PROJECT
TRAILER
1 STY. WOOD
FOOTINGS/CAISSONS
AT EL. -7.2
9 STY.
DORRANCE BLDG.
BLDG. No.16
BOTTOM OF MAT
AT EL. -11
9 STY.
WHITAKER BLDG.
BLDG. No.56
BLDG 66
RALPH LANDAU BLDG
5 STORY CONCRETE
AMES STREET
NOTES:
1. BASE PLAN TAKEN FROM "TOPOGRAPHIC PLAN OF LAND, MASSACHUSETTS INSTITUTE OF TECHNOLOGY, AMES AND VASSAR STREETS, CAMBRIDGE, MA", PREPARED BY CULLINAN ENGINEERING CO., DATED 24 NOVEMBER 1998.
2. ELEVATIONS REFER TO CAMBRIDGE CITY BASE DATUM (CCB).
0 40 80 120 160
SCALE IN FEET
THE RAY AND MARIA STATA CENTER
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CAMBRIDGE, MASSACHUSETTS
SITE and ADJACENT BUILDINGS
(Adapted from Haley & Aldrich June,2000)
SCALE: AS SHOWN
NOVEMBER 2000

FIGURE 3.4

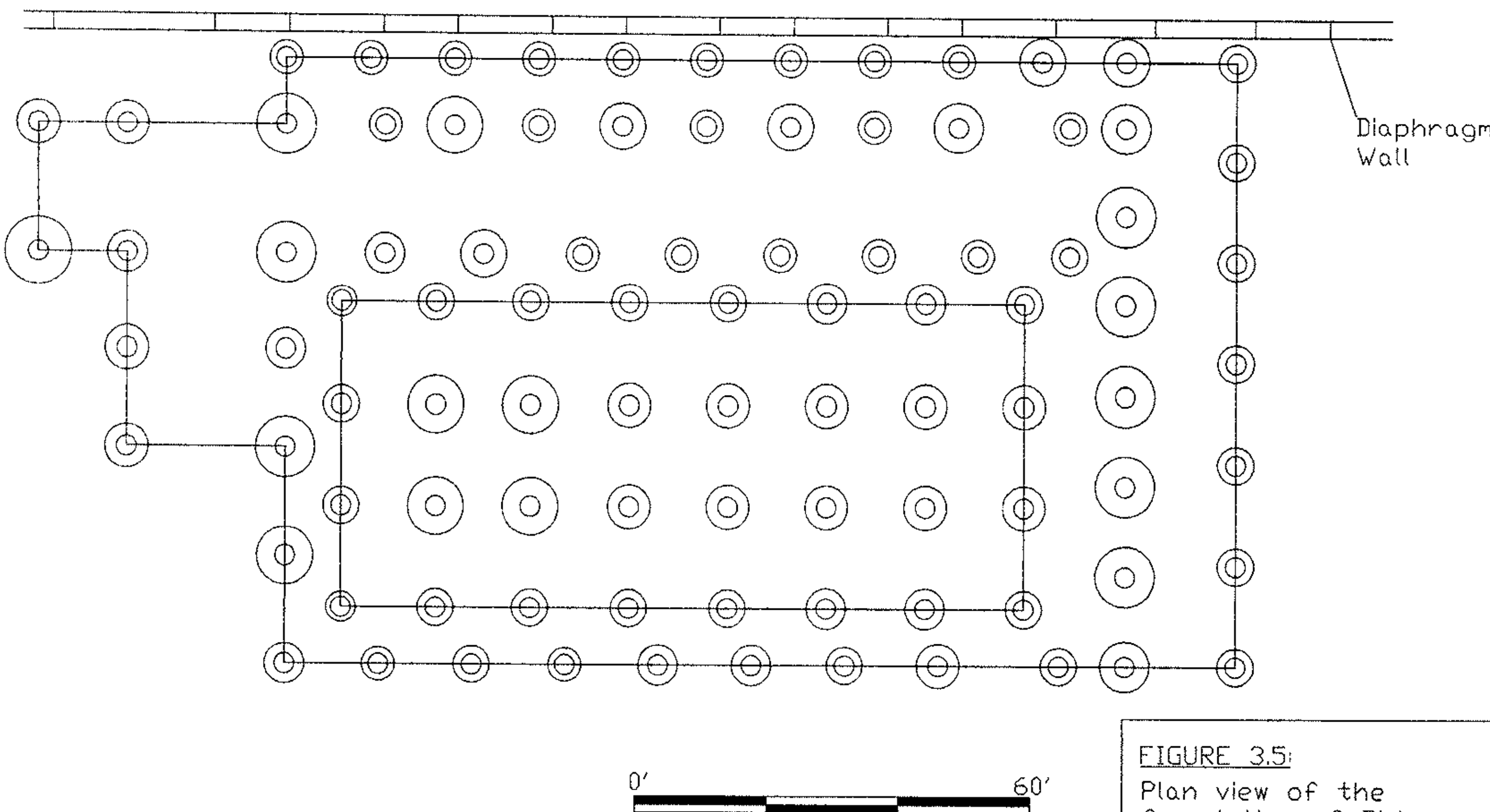


FIGURE 3.5:
Plan view of the foundation of Bldg. 57 (The Alumni Pool).

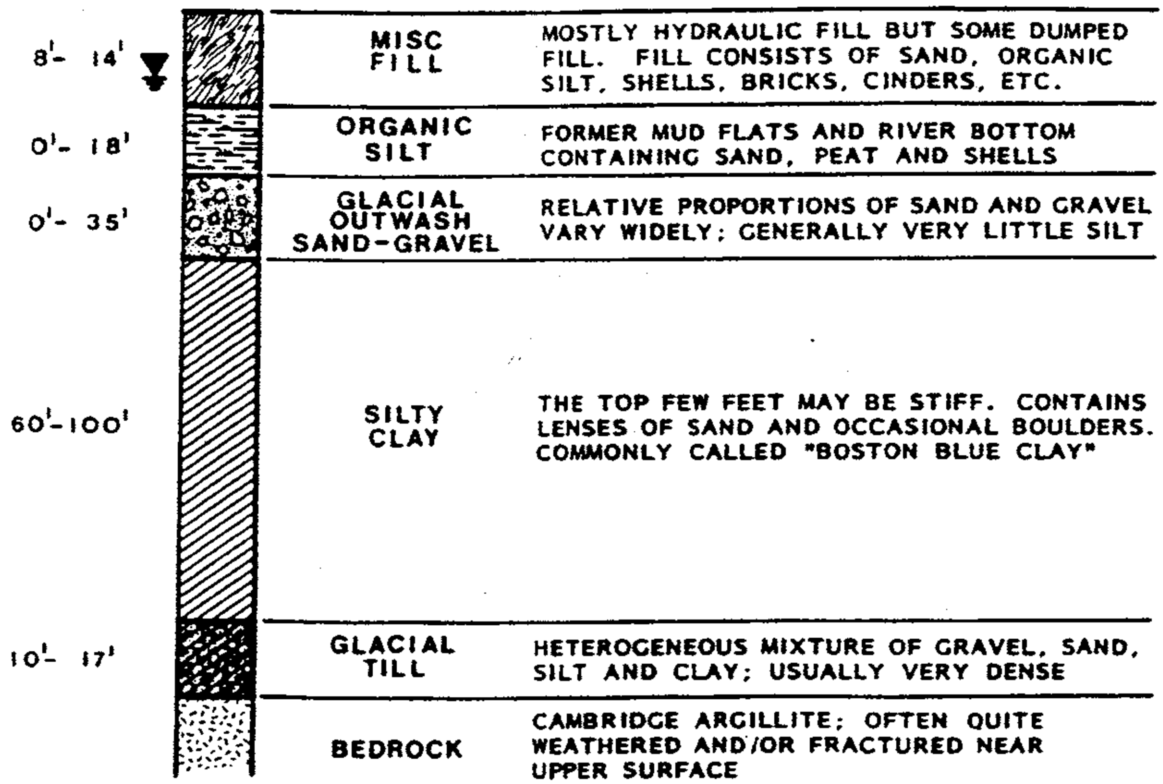


Figure 3.6 Geologic Profile for the MIT Campus (from Aldrich 1981)

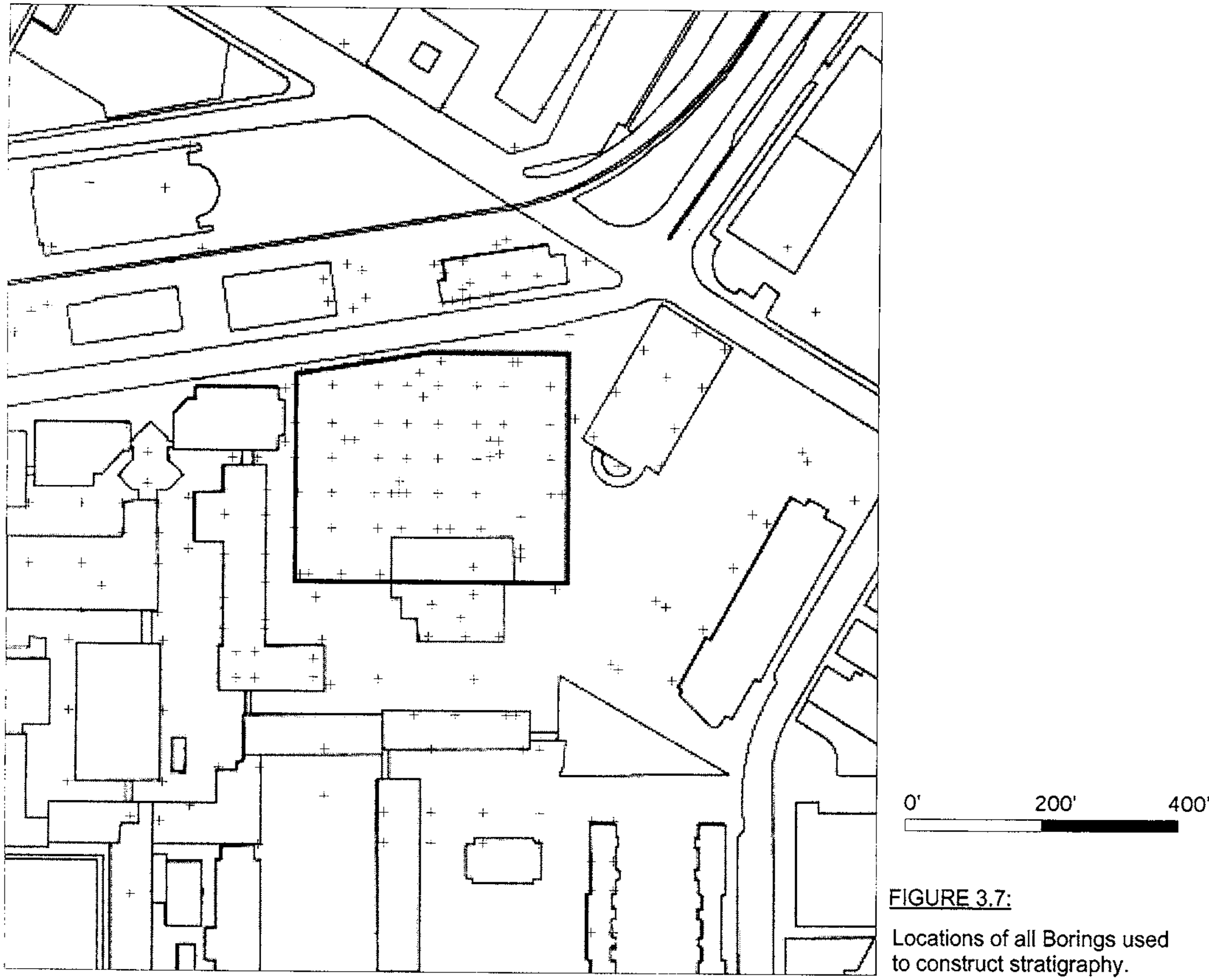


FIGURE 3.7:

Locations of all Borings used to construct stratigraphy.

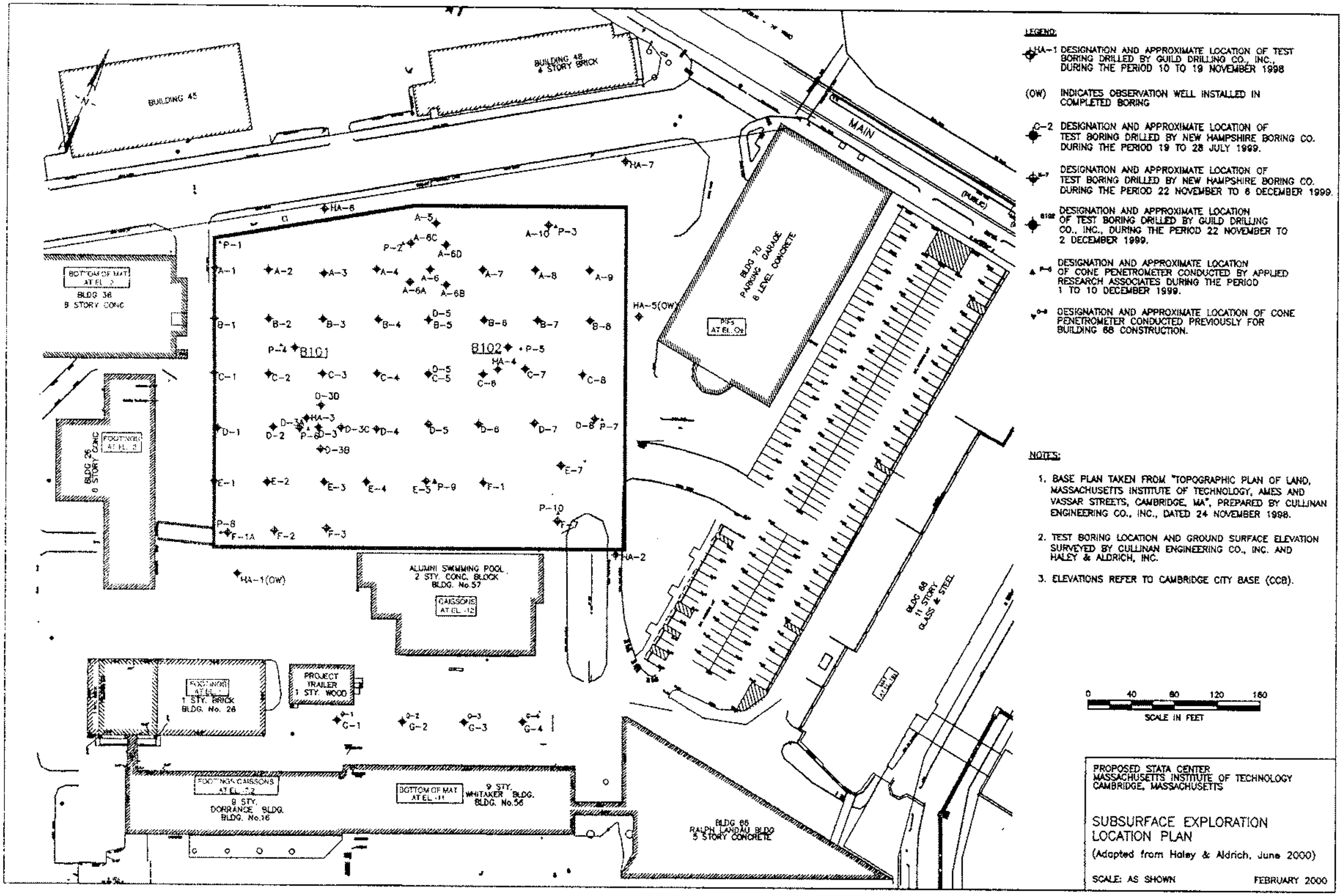

LEGEND:
HA-1 DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY GUILD DRILLING CO., INC., DURING THE PERIOD 10 TO 19 NOVEMBER 1998
(OW) INDICATES OBSERVATION WELL INSTALLED IN COMPLETED BORING
C-2 DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY NEW HAMPSHIRE BORING CO. DURING THE PERIOD 19 TO 28 JULY 1999.
DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY NEW HAMPSHIRE BORING CO. DURING THE PERIOD 22 NOVEMBER TO 6 DECEMBER 1999.
B102 DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY GUILD DRILLING CO., INC., DURING THE PERIOD 22 NOVEMBER TO 2 DECEMBER 1999.
DESIGNATION AND APPROXIMATE LOCATION OF CONE PENETROMETER CONDUCTED BY APPLIED RESEARCH ASSOCIATES DURING THE PERIOD 1 TO 10 DECEMBER 1999.
DESIGNATION AND APPROXIMATE LOCATION OF CONE PENETROMETER CONDUCTED PREVIOUSLY FOR BUILDING 68 CONSTRUCTION.
NOTES:
1. BASE PLAN TAKEN FROM "TOPOGRAPHIC PLAN OF LAND, MASSACHUSETTS INSTITUTE OF TECHNOLOGY, AMES AND VASSAR STREETS, CAMBRIDGE, MA", PREPARED BY CULLINAN ENGINEERING CO., INC., DATED 24 NOVEMBER 1998.
2. TEST BORING LOCATION AND GROUND SURFACE ELEVATION SURVEYED BY CULLINAN ENGINEERING CO., INC. AND HALEY & ALDRICH, INC.
3. ELEVATIONS REFER TO CAMBRIDGE CITY BASE (CCB).
0 40 80 120 160
SCALE IN FEET
BUILDING 45
BUILDING 48 4 STORY BRICK
MAIN
(PUBLIC)
BLDG 70 PARKING GARAGE 6 LEVEL CONCRETE
BLDG 36 9 STORY CONC
BLDG 26 6 STORY CONC
ALUMNI SWIMMING POOL 2 STY. CONC. BLOCK BLDG. No.57
CAISSONS AT EL. -12
PROJECT TRAILER 1 STY. WOOD
1 STY. BRICK BLDG. No. 26
9 STY. DORRANCE BLDG. BLDG. No.16
BOTTOM OF MAT AT EL -11
9 STY. WHITAKER BLDG. BLDG. No.56
BLDG 66 RALPH LANDAU BLDG 5 STORY CONCRETE
BLDG 68 11 STORY GLASS & STEEL
PROPOSED STATA CENTER
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CAMBRIDGE, MASSACHUSETTS
SUBSURFACE EXPLORATION LOCATION PLAN
(Adapted from Haley & Aldrich, June 2000)
SCALE: AS SHOWN
FEBRUARY 2000

FIGURE 3.8

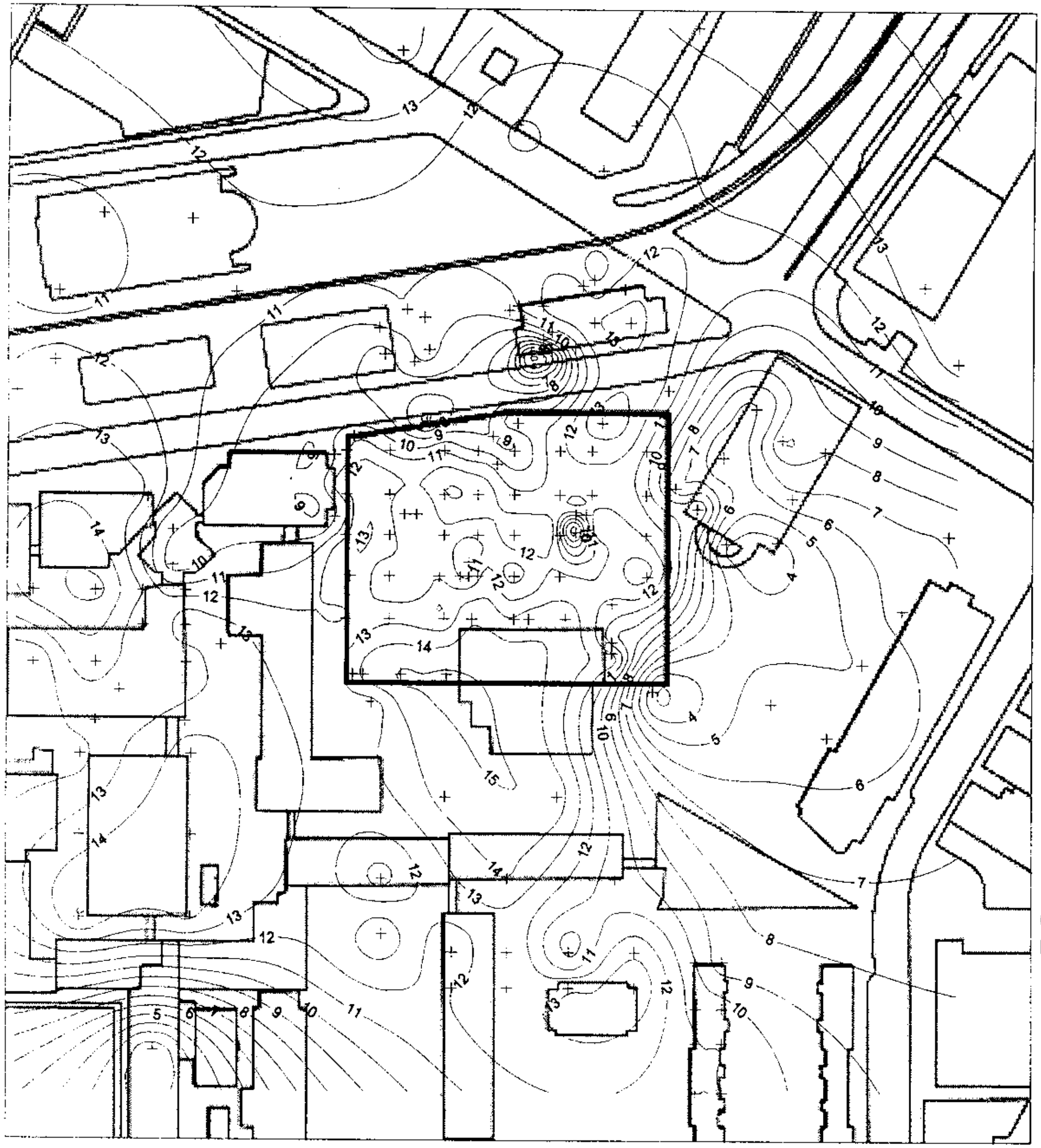


FIGURE 3.9:
Top of Organics Contour Map

Surface Elevation is
Approx. El. 21

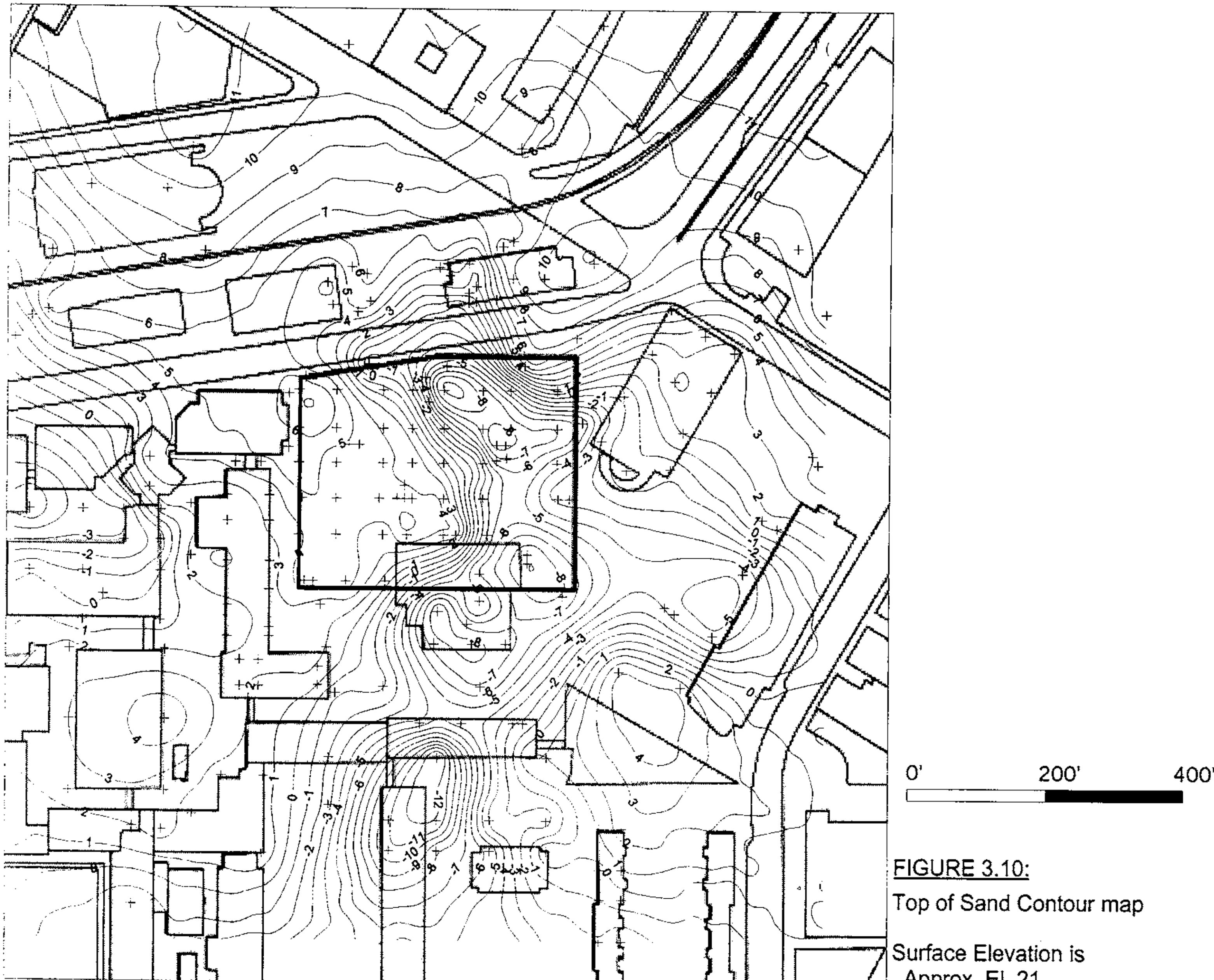


FIGURE 3.10:

Top of Sand Contour map

Surface Elevation is
Approx. El. 21

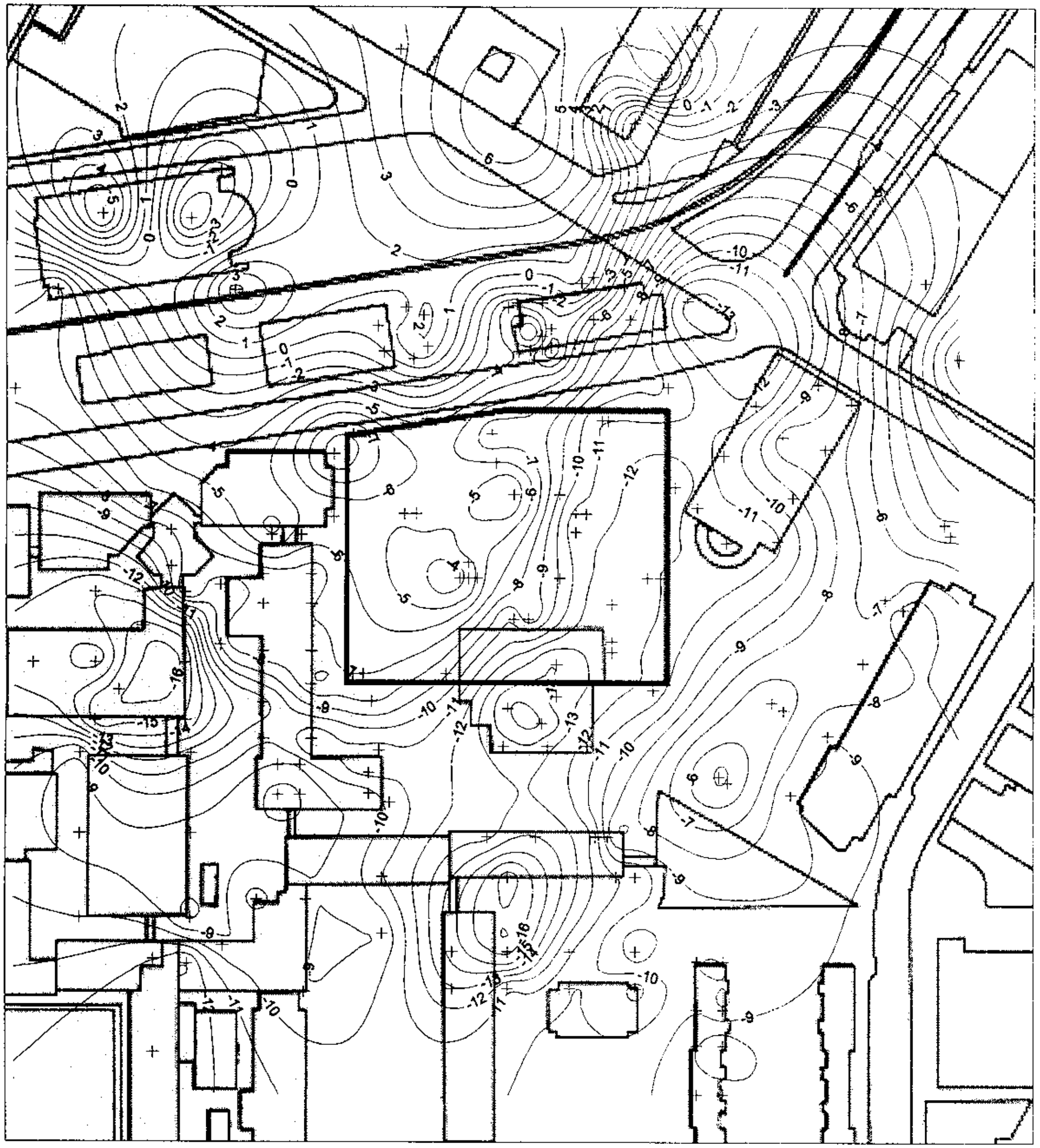


FIGURE 3.11:

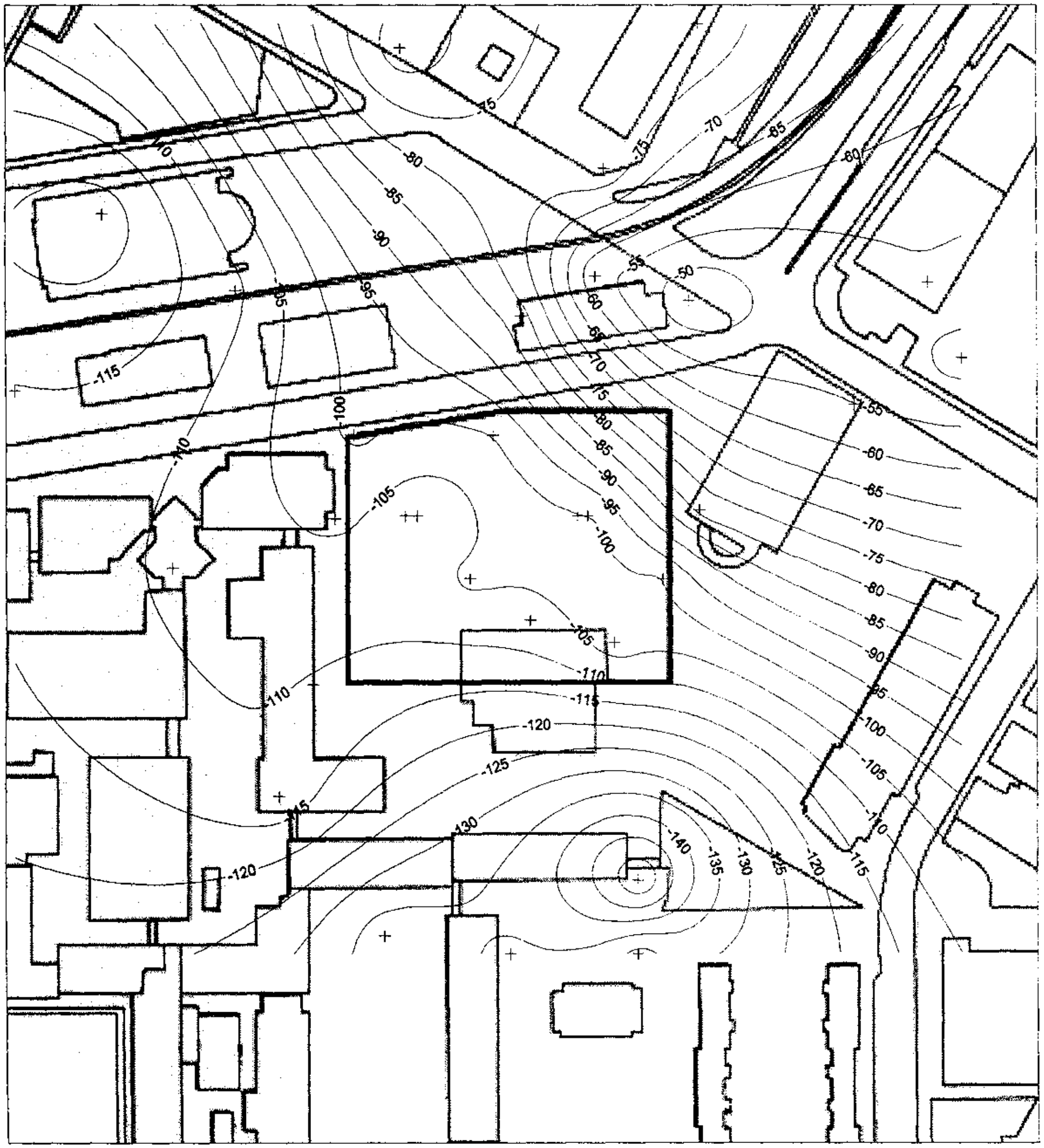
Top of BBC Contour Map

Surface Elevation is
Approx. El. 21

FIGURE 3.12:

Top of Glacial Till Contour Map

Surface Elevation is
Approx. El. 21



0' 200' 400'

FIGURE 3.13:
Top of Bedrock Contour Map

Surface Elevation is
Approx. El. 21

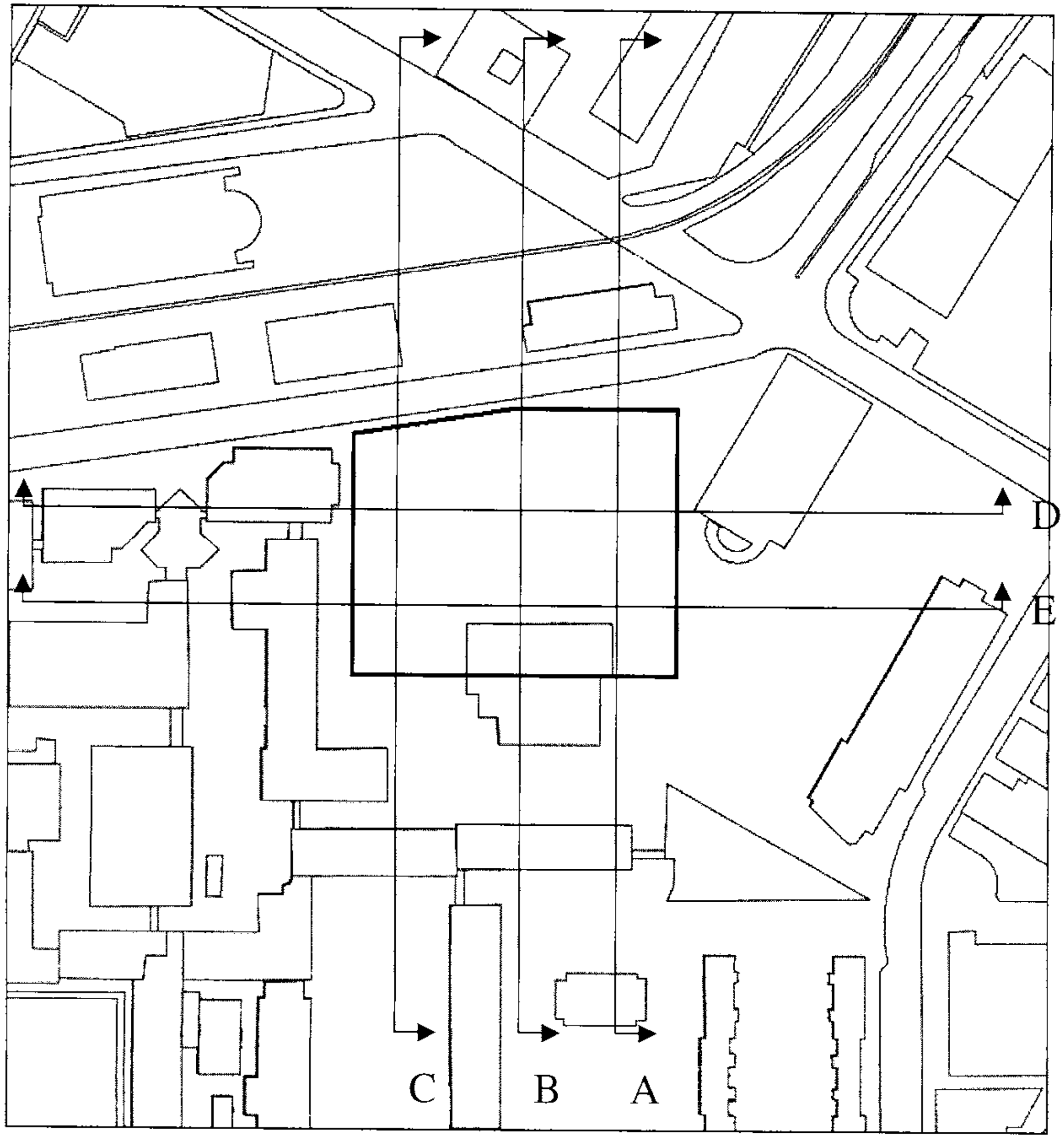


FIGURE 3.14 APPROX. LOCATION OF SUBSURFACE PROFILES

0 200 400

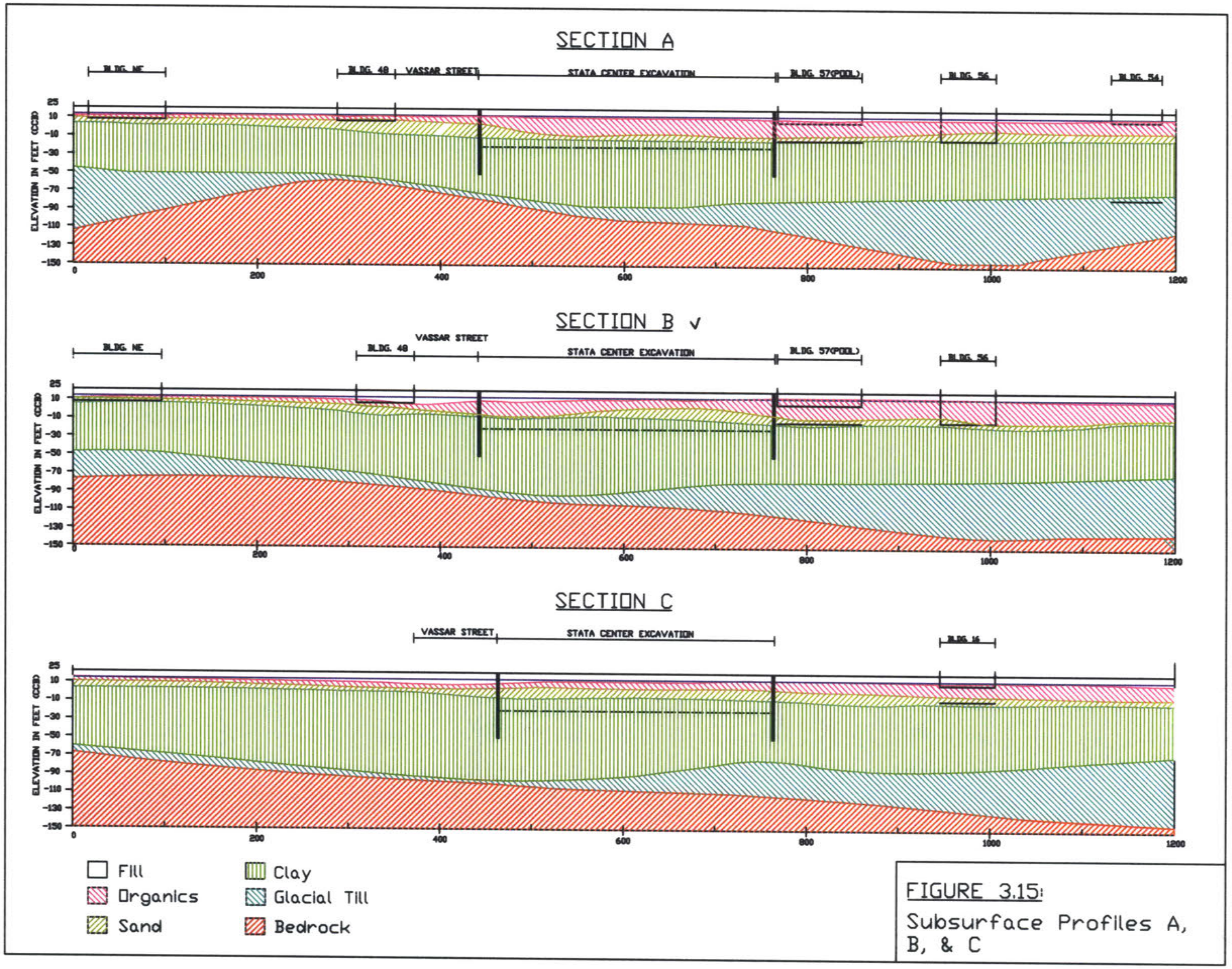


FIGURE 3.15: Subsurface Profiles A, B, & C

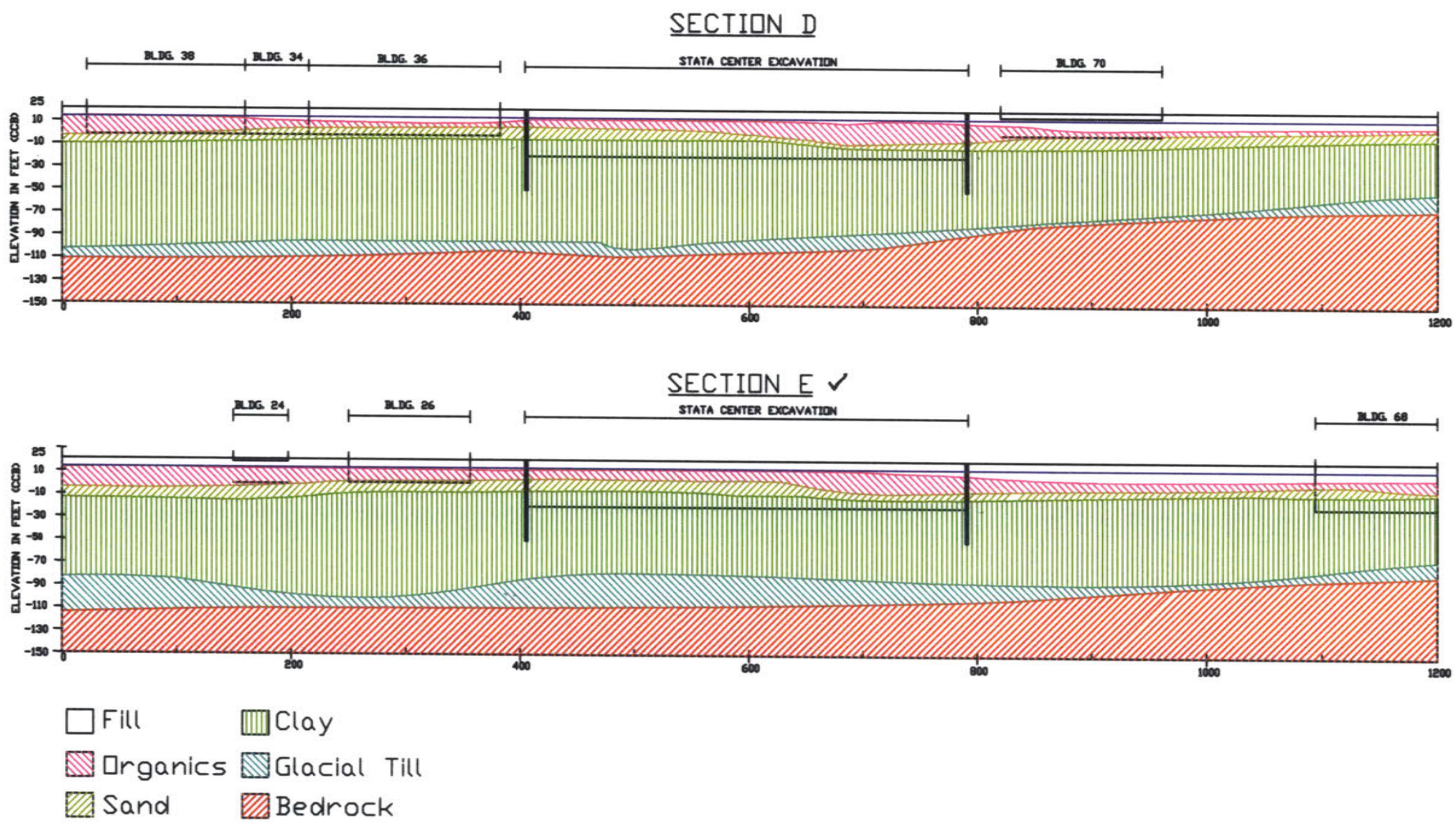


FIGURE 3.16: Subsurface Profiles D & E

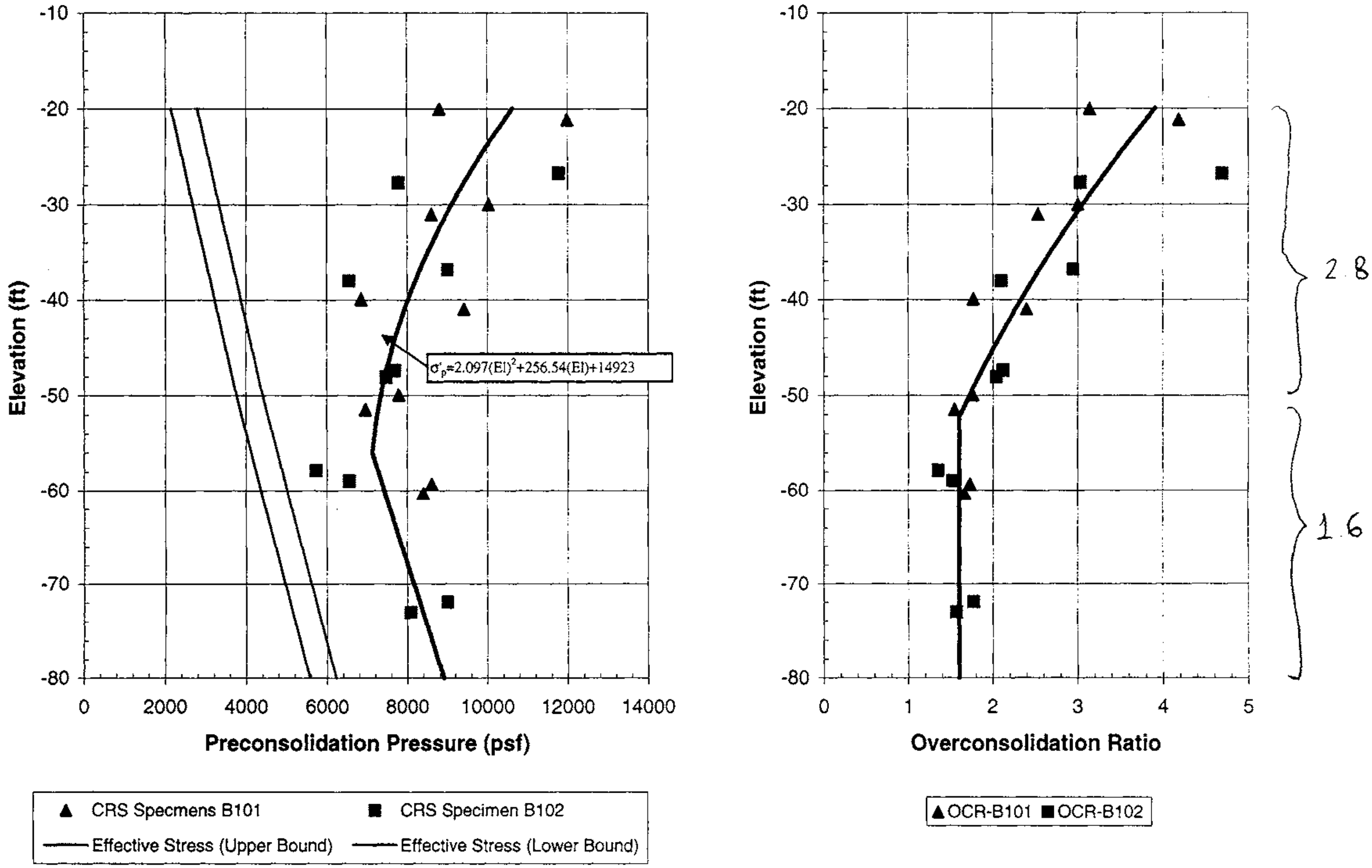


FIGURE 3.17: Preconsolidation Pressure and OCR vs Elevation for the clay. (from H & A, 2000)

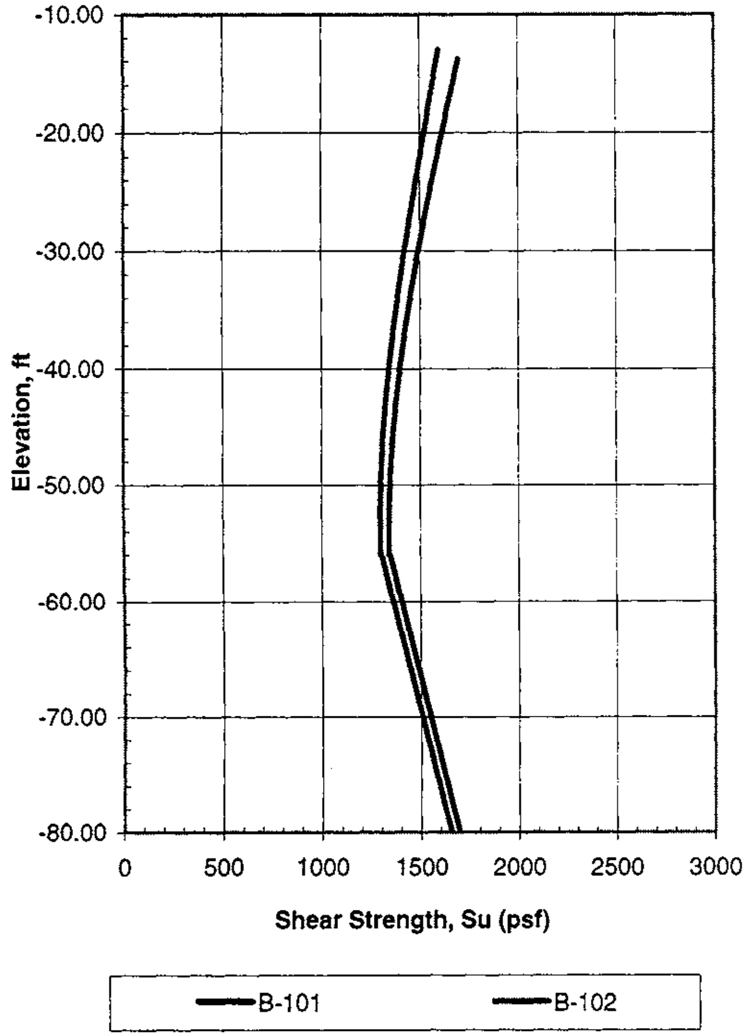


FIGURE 3.18: Undrained shear strength of clay computed using SHANSEP. (from H & A, 2000)

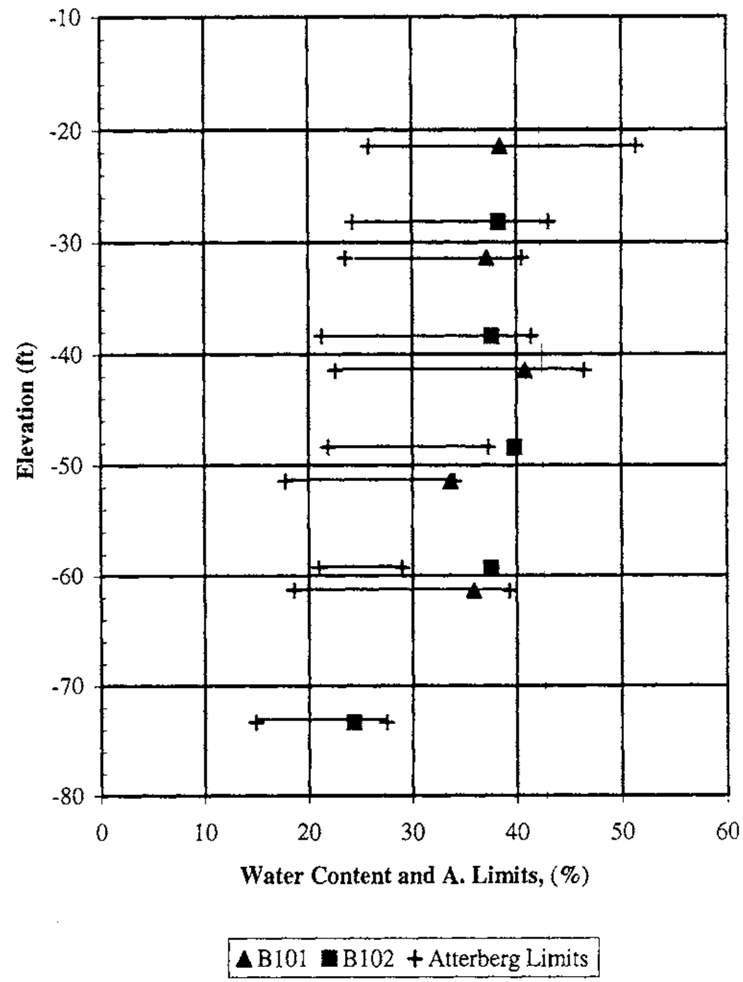


FIGURE 3.19: Atterberg Limits and Natural Water Content of the clay. (from H & A, 2000)

Chapter 4
Excavation Support Design and Sequence

4.1 Background

It has been found that buildings that impose a net stress increase on the deep clay deposit, such as those originally constructed on the MIT campus, will suffer large long term settlements due to the high compressibility of the lower clay units (that are nearly normally consolidated). Taylor (1944) in his paper on the design of the foundation for the Alumni Pool Building, showed that within the first two years after the construction of the main buildings (1-11), settlements of three inches were observed. This led to speculation as to the amount of long-term settlement that could be expected. Subsequently, the consolidation settlements increased to 7 in. after ten years (buildings 2 and 10, Figure 3.2) and to 8 in after twenty-two years. Although the long-term settlement was much smaller than some of the initial predictions, the magnitude of measured settlements led to changes in construction practices at MIT. Since that time, virtually all new buildings have been supported either on deep piles to bedrock or on floating (mat) foundations. A floating foundation is one were the weight of the excavated soil approximately balances the entire weight of the building and therefore causes no change in stress at the foundation grade. In the case of the Stata Center the gross bearing pressure is 4.0 ksf which means that the building weight approximately balances with the 40 feet of soil that was removed.

In the 1960s a research project called "Foundation Engineering Research MIT (FERMIT)" (FERMIT, 1963 and 1967) was undertaken that included extensive study and monitoring of the foundations of many of the major buildings on campus. This project concluded "that floating foundations can provide a very cost effective alternative to piled foundations with proper design and construction." (Berman, 1993) Some examples of buildings with floating foundations are the Student Center (W20), CAES (9), and Life Sciences Building (56). The Stata Center was also designed to act as a floating (mat) foundation, with an excavation approximately 40 feet below existing ground surface. The below ground space is to be used for underground parking, utilities and storage. For an excavation of this depth and plan area, a complex combination of support methods is needed to reduce movements of adjacent ground and mitigate any possible damage to structures, while maximizing space available for (bottom up) construction of the new Stata Center building.

4.2 Lateral Excavation Support

The very close proximity of the Alumni Pool (building 57) to the Stata Center excavation played a very significant role in the design of the lateral earth support system. One of the first key decisions was to use a reinforced concrete diaphragm wall instead of a sheet pile wall (as had been used in some of the earlier buildings built on campus, e.g., Building 9). The main advantages of the diaphragm wall that apply in this case are:

1) Minimize excavation-induced deformations due to the high wall bending stiffness (compared to conventional sheet pile sections).
2) Provides a relatively impermeable wall and hence, limits groundwater flow into the excavation.
3) Use as part of the permanent structure.
4) Their installation produces less noise than conventional pile and sheet pile driving.

The principal methods used for the temporary (length of construction) support of a diaphragm wall with an excavation depth on the order of 40 ft are the use of cross-lot bracing, corner bracing, rakers, tiebacks, and top/down construction. Cross-lot bracing involves spanning the site with compressive steel struts that transfer the load (lateral earth and water pressures) across the excavation. Corner bracing operates on the same principle, but the compressive struts are oblique to the walls at the corner of an excavation. Rakers are inclined compressive steel supports that transfer loads from the wall to a slab or kicker block built within the excavation. Pre-stressed tieback anchors are high tensile steel tendons that are anchored in the retained soil and are tensioned at the front face of the wall. Top/down construction uses cast in-situ floor slabs as support while excavation is performed by mining beneath the existing floor slab. The Stata Center excavation has a final a depth of 42 feet (to El –21 ft) and plan dimensions of 386.0 ft by 316.33 ft (Figure 4.1). As a result, it is not feasible to use cross-lot bracing due to problems of strut buckling, while the advantages of top/down construction (primarily reduced wall and soil movements) would not be achieved. Therefore, available bracing systems involve combinations of corner bracing, rakers, and tieback anchors.

4.2.1 Support Options

There are many possible designs of temporary lateral earth support systems for the Stata Center excavation. All of the combinations that were considered use two levels of corner bracing at each of the four corners, as this is the easiest and most effective support method. Three possible combinations of tiebacks and/or were also considered.

The first option was to use three levels of tiebacks on all four sides of the excavation between the corner bracing (Figure 4.2). This is a very appealing option because the tiebacks cause only a minor loss of space within the interior of the excavation. This maximizes the amount of space available for movement of excavation equipment and construction of the below grade slabs and mat foundation. As a result, the below ground excavation work and slab construction can proceed quickly. However, the efficiency is then contingent on installing and proof testing of each anchor. The installation of tieback anchors can also interfere with existing utilities (e.g., beneath Vassar Street on the north side of the excavation) and existing deep foundations (e.g., Caissons beneath Building 57). There is probably very little interference from the other buildings since they are a little farther from the excavation and are supported on shallow foundations.

A second option was the use of two levels of raker supports on the north and south sides between the corner bracing and the use of three levels of tiebacks on the east and west sides between the corner bracing (Figure 4.3). Using rakers on the north and south walls eliminates possible interference with foundations of the alumni pool (building 57) and utilities beneath Vassar Street. The primary disadvantage of this option is that in order to implement this plan, the excavation would have to proceed at a slope from each of the sides into the center so that the center portion of the slab could be used to support the rakers. Further, the rakers on either side would need to be preloaded equally to minimize racking of the two walls. The other disadvantage of this lateral support system is that it impedes excavation activities between the rakers affecting construction of the foundation and basement structures.

A third option uses two levels of rakers on the north side between the corner bracing and to three levels of tiebacks on the other three sides between the corner bracing (Figure 4.4). This option could be achieved by excavating the south, east and west sides in order to install the tiebacks and corner bracing. Meanwhile, a berm is left in front of the north side until the mat can be poured from the south wall. This approach transfers the load from the north wall through the mat to the south side (Figure 4.5). This option has the advantage of minimizing interference with the utilities beneath Vassar Street, while providing a much larger open area for excavation and construction compared to option two. There is still the problem of less space and interference in construction and excavation, but avoidance of the utilities could outweigh this problem. The positioning of tieback anchors on the south wall must be carefully selected to avoid the caisson foundation for building 57.

4.2.2 Support Design

In deciding among the lateral earth support options, the principal issues were available open workspace inside the excavation, ease of implementation, cost, and expected performance. The first option, no rakers, was eliminated when the City of Cambridge refused to give permission for the installation of tiebacks beneath Vassar Street and utilities on the north side of the site. This left options two and three. After considering the advantages and disadvantages of each plan, option 3 was chosen. Ultimately, this decision was made based on the belief that it would be easier to implement, it provided more space for excavation and construction, and could achieve better performance (less wall movements and ground settlements). Each of the components of the excavation support system will be discussed below.

4.2.2.1 Diaphragm Wall System and Mat Foundation

The wall system consists of a 30 in reinforced concrete diaphragm wall as the permanent exterior basement wall. The slurry wall was laid out in panels as shown in plan view (Figures 4.1) and elevation views (Figures 4.7 to 4.9). A total of sixty-five panels were used with numbering starting in the southwest corner and continuing counter-clockwise around the site. The wall was designed to extend to El. –50 ft (approximately 28 ft into the marine clay deposits) and the elevation of the top of the wall varied from El. 15.4 ft to El. 19.0 ft with the surface at El. 21.0 ft. The wall was designed to support loads through skin friction and not end-bearing. The depth of the wall was initially set at El. –70 ft but it was decided that the cost savings on installation greatly outweighed any decrease in movements that would be expected with a larger embedment depth. The diaphragm wall was cast in-situ with sleeves pre-installed to simplify the installation of tieback anchors and with bearing plates to strengthen contact areas with corner braces and rakers.

The mat foundation consists of a reinforced concrete slab that is 4.0 ft thick. After the excavation reaches El. –20.9 ft, a 4 in thick concrete mud mat was placed to stabilize the surface of the clay and to make it easier to construct the reinforcing cage for the slab and then to pour it. The as-built top of the mat foundation is at El. –16.6 ft.

4.2.2.2 Tiebacks

The support system was designed with a total of 289 temporary tiebacks spread out on the south, east, and west sides of the site to provide lateral support. They were designed to be installed roughly 5 ft on center in the middle portions of the walls at El. 10.0 ft, -1.0 ft, and –12.0 ft and inclined at an angle of 20° from the horizontal. The design loads are 112 kips for the top level and 128 kips for the second and third levels. The ties were designed to have total lengths of 90 ft, 48 ft, and 38 ft, with bond lengths of 40 ft, 28 ft, and 28 ft for top, middle, and bottom layers respectively. A typical cross section showing the ties and the wall and the excavation is in Figure 4.10. The location of each of the tiebacks can be seen in Figures 4.7 to 4.9 and the design loads and lengths can be found in Table 4.1.

The tiebacks each consist of 4, 0.6 in. diameter high strength steel tendons (270 ksi). The free lengths are sheathed with a polyethylene tube material to preclude bonding in that portion. The grouted anchors consist of Type I/II Portland Cement and potable water. The water to cement ratio of 0.49 by weight and a minimum 28 day compressive strength of 4,000 psi. In the event that post-grouting was required, the secondary grout injection was specified with a cement-water ratio of 0.58 by weight.

The tiebacks are drilled with a casing through the fill, organics, and sand using internal flush methods, to anchorage locations in the overconsolidated clay crust. After each hole is drilled, it is filled with grout and the tendon and post-grouting pipes are inserted and the casing is removed. The post-grouting pipe consists of a ¾ in PVC pipe with holes drilled three feet on center within the bond length in order that post-grouting can be undertaken (instead of redrilling) if the anchor fails to achieve its proof load. If post-grouting is necessary, the grout can be applied at pressures up to 800 psi. The tiebacks will then be locked-off at 100% of their design load once the grout has had time to set and they have been proof tested to 130% of their design load.

4.2.2.3 Cornerbracing

The cornerbracing consists 36 in diameter steel pipe struts[1] that will be installed at El. 10 ft and El. –10 ft. These were pre-loaded by jacking them in place at 50% of their design load. The location of each of the struts can be found in Figures 4.6 to 4.9. The details of each of the struts can be seen in Table 4.2, including design loads, jacking loads, strut sizes, and angle formed between the braces and the wall. The numbering system used to identify each brace, e.g. NW1-, identifies the corner location, the bracing level (e.g., 1 for El 10 ft and 2 for El –10ft), and then its position from the corner (e.g., 1 for the brace closest to the corner). In all but the southwest corner the two outer braces were designed to be supported by pin piles at their midpoints in order to reduce their unbraced lengths. Also, the cornerbracing in the southeast corner is attached to a waler on the south wall in order to bridge a gap in the wall at panel number 20 that is used for an exhaust structure.

4.2.2.4 Rakers

The rakers were designed to support the north wall by bracing it against the mat foundation which in turn transfers the load through basal shear resistance with the underlying clay across to the south wall. The construction sequence assumes that a large portion of the site will be excavated to final grade leaving a berm in front of the north wall. The foundation mat is then poured from the south wall and the rakers are then set in place and pre-loaded, and the finally the berms can be removed and the foundation mat completed to the north wall. Each of the raker supports comprise a 36 in diameter steel

[1] The inner brace in each of the corners is a wide flanged beam instead of a pipe strut and is not be pre-loaded by jacking.

pipe strut that supports the wall (at El 10 ft and El –10 ft) and is inclined downward to a kicker block cast into the mat.

Figure 4.11 shows the top level raker is inclined at 16.61° from the horizontal and the lower level at 3.6° from the horizontal.. All rakers were pre-loaded to 50% of their design load. The details of each of the rakers can be found in Table 4.3 and the locations for their installation can be found in Figures 4.6 to 4.9.

4.3 Excavation Sequence

The excavation sequence can significantly affect the performance of the support system and therefore the movement of the surrounding structures. In general the practice for tiebacks and corner bracing is to over-excavate a trench a few feet below the required wall elevation for ease of access. After installation, each tieback must be proof tested prior to further excavation. No specific excavation sequence was provided prior to the actual construction. However, a simplified sequence of construction steps for the option 3 lateral earth support system is as follows:

1. Grouting under Building 57 in order to prevent settlement during installation of the diaphragm wall due to the marine sand running into the bentonite supported slurry trench.
2. Installation of the diaphragm wall.
3. Excavate to El. +8.0 ft
4. Install the first level of tiebacks at El. +10.0 ft and installation of the cornerbracing at El. +10.0 ft.
5. Excavate to El. –3.0 ft maintaining a 25 ft wide (at the top) berm sloping from El. 8 ft at a slope of 2H:1V away from the north wall.
6. Install the second level of tiebacks at El. –1.0 ft.
7. Excavate to El. –12.0 ft.

8. Install cornerbracing at El. –10.0 ft.
9. Excavate to El. –14.0 ft maintaining the berm on the north side.
10. Install the third level of tiebacks at El. –12.0 ft
11. Excavate to El. –20.9 ft (bottom of excavation) maintaining the berm.
12. Construct the mat in sections starting at the south wall and proceeding to the bottom of the berm.
13. Install the first level of rakers at El. 10.0 ft and pre-load the rakers.
14. Excavate the berm to El. –12.0 ft.
15. Install the second level of rakers at El. –10.0 ft and pre-load the rakers.
16. Excavate to El. –20.9 ft (bottom of Excavation).
17. Install the last section of the mat.

4.4 Excavation Monitoring

The performance of he excavation support is to be monitored throughout the excavation in order to ensure that movements are within acceptable limits to mitigate possible damage to the surrounding structures. The main cause of damage to surrounding structures is the movement of the diaphragm wall into the excavation. Any movement toward the excavation will generate settlements and lateral deformations in the retained soil. Building damage is primarily related to the differential settlement and lateral strain between structural supports (Boscardin and Cording, 1989). In order to monitor the movements and provide advance warning of any potential failures, a system of instruments including vibrating wire piezometers, magnetic extensometers, vertical inclinometers, groundwater observation wells, and settlement points was implemented. The instrumentation plan can be seen in Figure 4.12 and the elevations of where the piezometers and magnetic extensometers were placed are detailed in Figures 4.13 to 4.17.

The vibrating wire piezometers are used to determine changes in pore pressure within the clay layer. These measurements of pore pressures can reflect changes in groundwater pressure due to overburden release, pumping, or inflows into the excavation. The information is essential for calculating changes in the effective stress within the clay (and hence, estimating soil settlement). The magnetic extensometers consist of magnets positioned at various depths in a boring in soft soil and a pipe that runs inside of them. When a probe is passed through the pipe it detects the magnets when it gets close and the location is recorded. By recording the locations over time, vertical deformation distributions can be measured within the soil column. The vertical inclinometers consist of tubes that are placed vertically within the diaphragm wall and move with it horizontally. The bottom of the tube is fixed in the bedrock and therefore will not move. By sending a specially designed probe through the pipe and measuring the movement at intervals, with the bottom being zero[2], a profile of the horizontal movement of the wall can be determined. The settlement points consist primarily of metal screws that are fixed to buildings or sidewalks and are surveyed to measure whether a building or the ground surface is settling or heaving. On this project, a total of eleven inclinometers, twelve piezometers, five borehole extensometers, and between 100 and 150 settlement points were installed and monitored along with two groundwater observation wells.

[2] The bottom is considered to be zero because it is grouted into the bedrock and therefore it is assumed that it does not move.

Table 4.1: Design details for the tiebacks at the MIT Stata Center

Wall	Tiebacks	Number of Tiebacks	Design Load (kips)	Prestressing Load (kips)	Installed Angle (deg)	Free Length (ft)	Bond Length (ft)
South	S1-1 to S1-45	45	112	112	20	50	40
	S2-1 to S2-45	45	128	128	20	20	28
	S3-1 to S3-45	45	128	128	20	10	28
East	E1-1 to E1-25	25	112	112	20	50	40
	E2-1 to E2-25	25	128	128	20	20	28
	E3-1 to E3-25	25	128	128	20	10	28
West	W1-1 to W1-26	26	112	112	20	50	40
	W2-1 to W2-26	26	128	128	20	20	28
	W3-1 to w3-27	27	128	128	20	10	28

TABLE 4.2: Design Details for Corner Bracing at the MIT Stata Center

Strut	Elevation (ft-CCB)	Length (ft)	Unbraced Length (ft)	Design Load (normal to the wall) (kips/ft)	Tributary Length (ft)	Angle B/N Strut and Wall (degrees)	Strut Size Diameter (in)	Strut Size Wall Thickness (in)	Design Load (kips)	Jacking Load (kips)
NW1-1	10	8.6	8.6	26	16.25	40.39	W14X90		555	0
NW1-2	10	40.8	40.8	26	24.25	40.4	36	0.38	828	414
NW1-3	10	76.6	38.3	26	24.25	40.4	36	0.38	828	414
NW1-4	10	114.7	57.4	26	25	40.4	36	0.38	853	427
NW2-1	-10	8.6	8.6	44	16.25	40.39	W14X120		939	0
NW2-2	-10	40.8	40.8	44	24.25	40.4	36	0.519	1401	700
NW2-3	-10	76.6	38.3	44	24.25	40.4	36	0.519	1401	700
NW2-4	-10	114.7	57.4	44	25	40.4	36	0.63	1444	722
SW1-1	10	7.8	7.8	26	16.25	45	W14X90		598	0
SW1-2	10	37.8	37.8	26	23.25	45	36	0.38	855	427
SW1-3	10	73.2	73.2	26	14.25	45	36	0.38	524	262
SW2-1	-10	7.8	7.8	44	16.25	45	W14X132		1011	0
SW2-2	-10	37.8	37.8	44	23.25	45	36	0.519	1447	723
SW2-3	-10	73.2	73.2	44	14.25	45	36	0.519	887	443
NE1-1	10	7.8	7.8	26	16.25	45	W14X90		598	0
NE1-2	10	37.8	37.8	26	25	45	36	0.38	919	460
NE1-3	10	73.2	36.6	26	25	45	36	0.38	919	460
NE1-4	10	108.5	54.3	26	25	45	36	0.38	919	460
NE2-1	-10	7.8	7.8	44	16.25	45	W14X132		1011	0
NE2-2	-10	37.8	37.8	44	25	45	36	0.519	1556	778
NE2-3	-10	73.2	36.6	44	25	45	36	0.519	1556	778
NE2-4	-10	108.5	54.3	44	25	45	36	0.63	1556	778
SE1-1	10	7.8	7.8	26	10.84	45	W14X90		398	0
SE1-2	10	20.2	20.2	26	19.59	48.09	36	0.38	720	360
SE1-3	10	55.5	55.5	26	25	46.32	36	0.38	919	460
SE1-4	10	90.9	45.5	26	23.25	45.84	36	0.38	855	427
SE1-5	10	121.3	60.7	26	14.25	45.95	36	0.38	524	262
SE2-1	-10	7.8	7.8	44	10.84	45	W14X90		674	0
SE2-2	-10	20.2	20.2	44	19.59	48.09	36	0.393	1219	609
SE2-3	-10	55.5	55.5	44	25	46.32	36	0.63	1556	778
SE2-4	-10	90.9	45.5	44	23.25	45.84	36	0.519	1447	723
SE2-5	-10	121.3	60.7	44	14.25	45.95	36	0.393	887	443

*NOTE: This is the location of the strut described, with NW1-1 meaning Northwest corner level one nearest brace to the corner.

TABLE 4.3: Design Details for Raker Supports at the MIT Stata Center

Raker*	Elevation (@north wall) (ft-CCB)	Overall Length (ft)	Unbraced Length (ft)	Design Load (normal to the wall) (kips/ft)	Tributary Length (ft)	Angle B/N Raker and Wall (vertical) (degrees)	Angle B/N Raker and Wall (horizontal) (degrees)	Strut Size		Design Load (kips)	Jacking Load (kips)
								Diameter (in)	Wall Thickness (in)		
N1-1	10	88.6	88.6	26	25	16.61	91.53	36	0.519	678	339
N1-2	10	88.7	88.7	26	23.5	16.61	87.18	36	0.519	638	319
N1-3	10	89.5	89.5	26	23.5	16.61	81.61	36	0.519	644	322
N1-4	10	89	89	26	24	16.61	84.54	36	0.519	652	326
N1-5	10	88.6	88.6	26	24	16.61	91.53	36	0.519	650	325
N1-6	10	88.6	88.6	26	25	16.61	91.53	36	0.519	678	339
N1-7	10	88.6	88.6	26	25	16.61	91.53	36	0.519	678	339
N1-8	10	89.3	89.3	26	25	16.61	97.14	36	0.519	683	341
N2-1	-10	85.2	85.2	44	25	3.6	91.53	36	0.803	1103	551
N2-2	-10	85.6	85.6	44	23.5	3.6	84.18	36	0.803	1041	521
N2-3	-10	86.9	86.9	44	23.5	3.6	78.65	36	0.803	1057	528
N2-4	-10	86.2	86.2	44	24	3.6	81.06	36	0.803	1070	535
N2-5	-10	85.2	85.2	44	24	3.6	91.53	36	0.803	4057	529
N2-6	-10	85.2	85.2	44	25	3.6	91.53	36	0.803	1103	551
N2-7	-10	85.2	85.2	44	25	3.6	91.53	36	0.803	1103	551
N2-8	-10	85.4	85.4	44	25	3.6	94.14	36	0.803	1105	553

*NOTE: Raker Name is given as first: location, second: level, third: number from west to east
(ie. N1-1 means North raker level one farthest west)

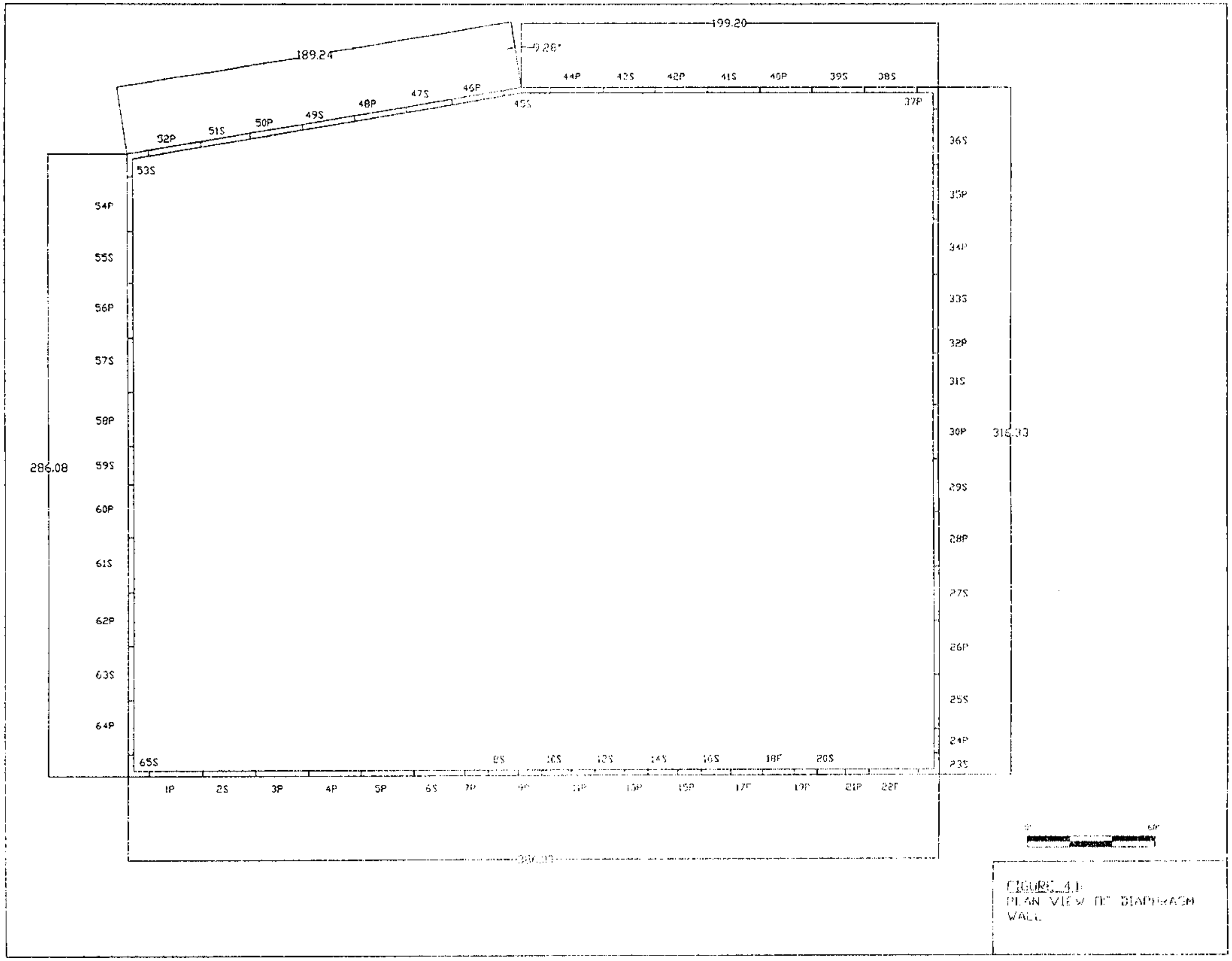


FIGURE 4.1
PLAN VIEW OF DIAPHRAGM WALL

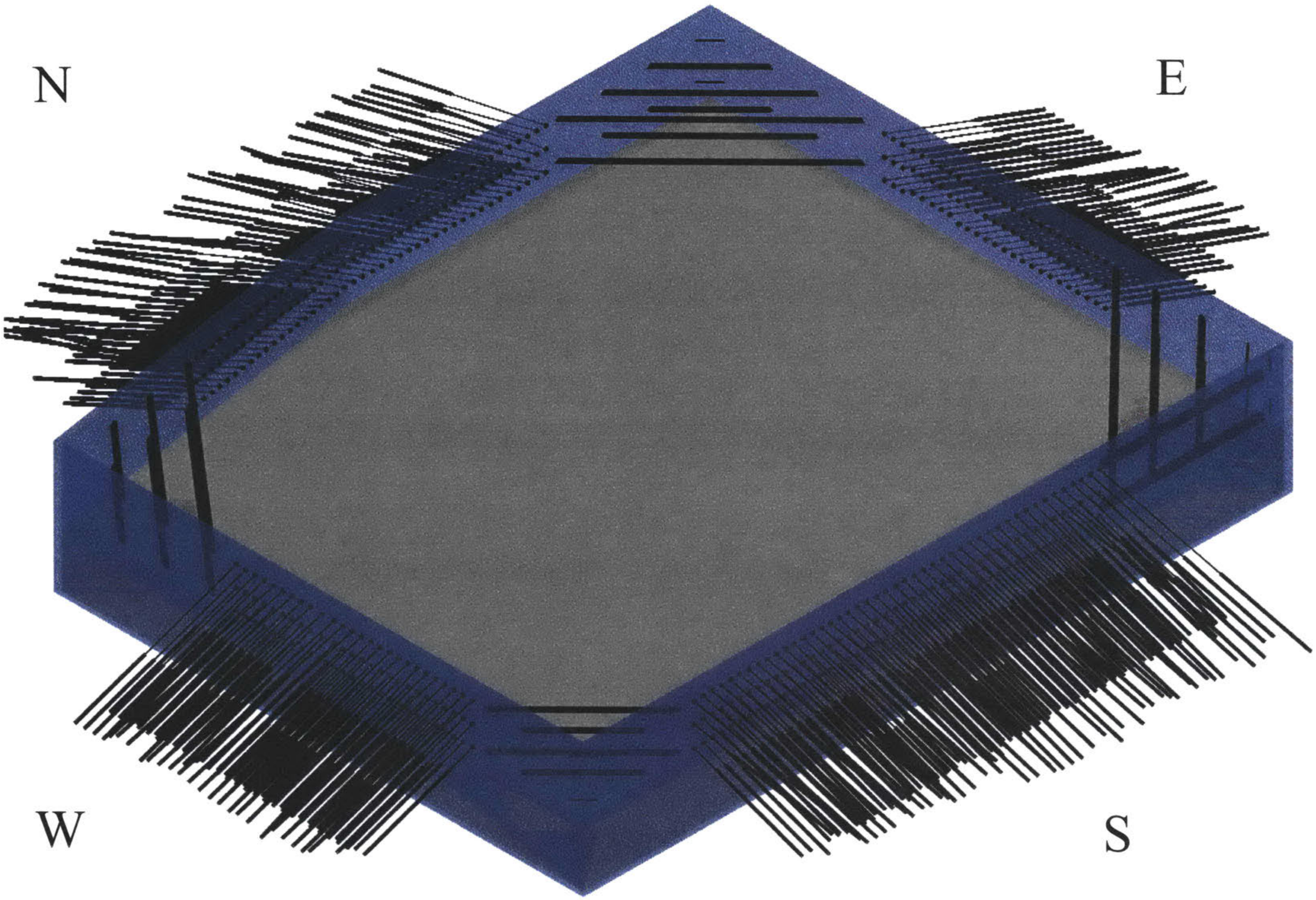


FIGURE 4.2: Schematic of the excavation with option 1: four sets of tiebacks. Isometric view from the southwest.

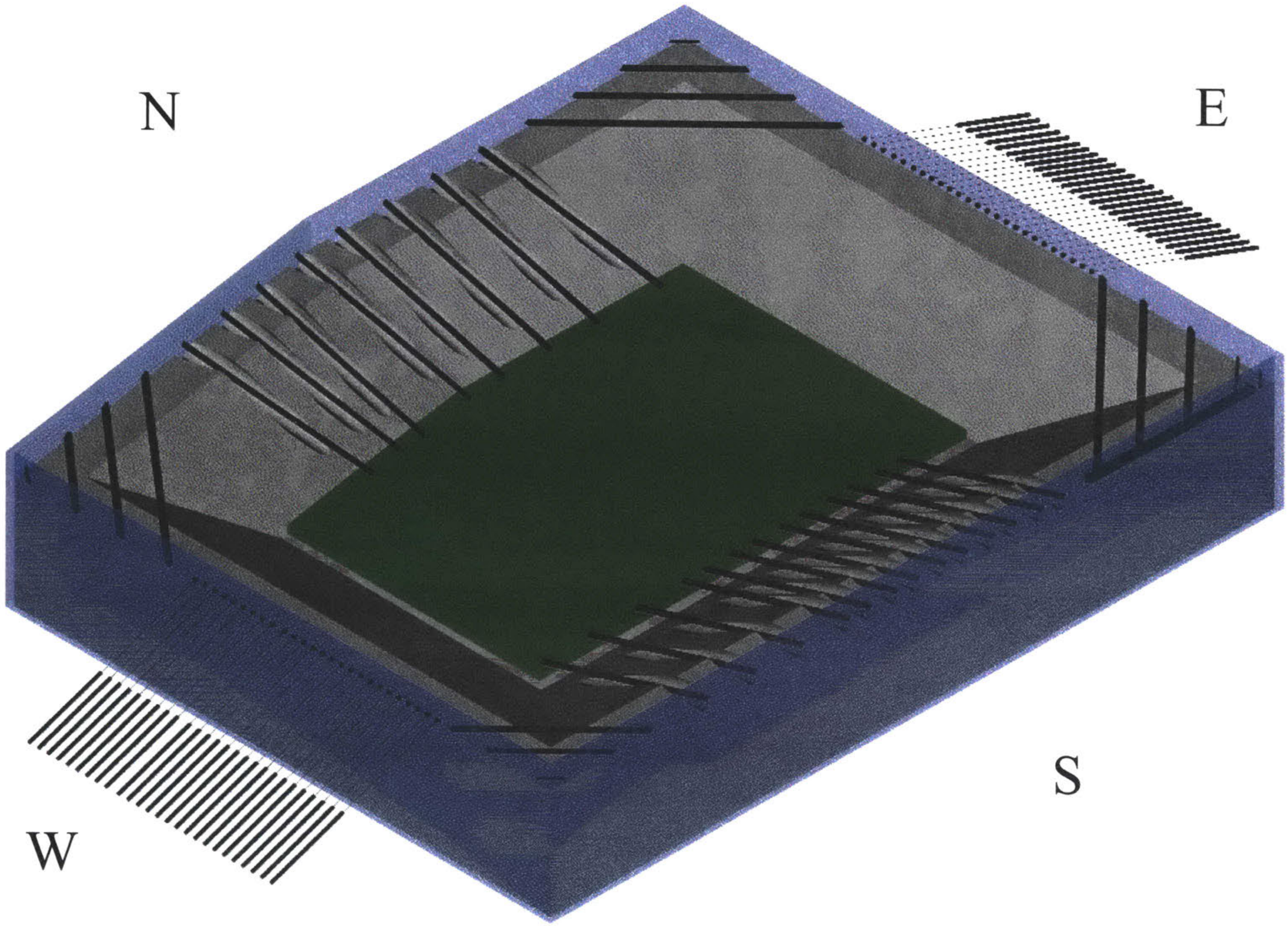


FIGURE 4.3: Schematic of option 2: two sets of tiebacks and two sets of rakers. Excavation progressing to the point of installation of the first set of bracing and tiebacks. Isometric view from the southwest.

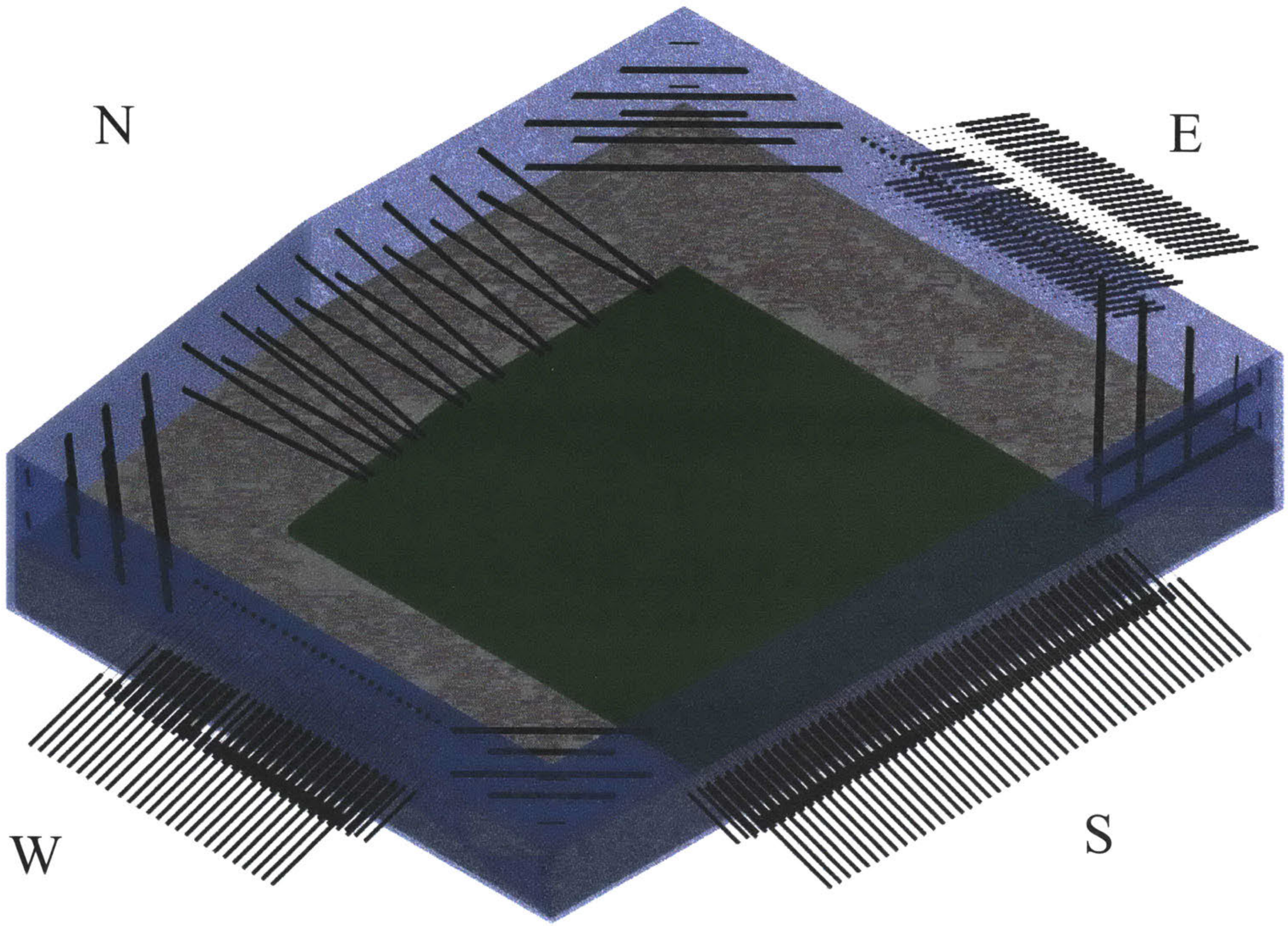


FIGURE 4.4: Schematic of excavation with tiebacks on the three sides and rakers on the other. (option 3) Isometric view from the southwest.

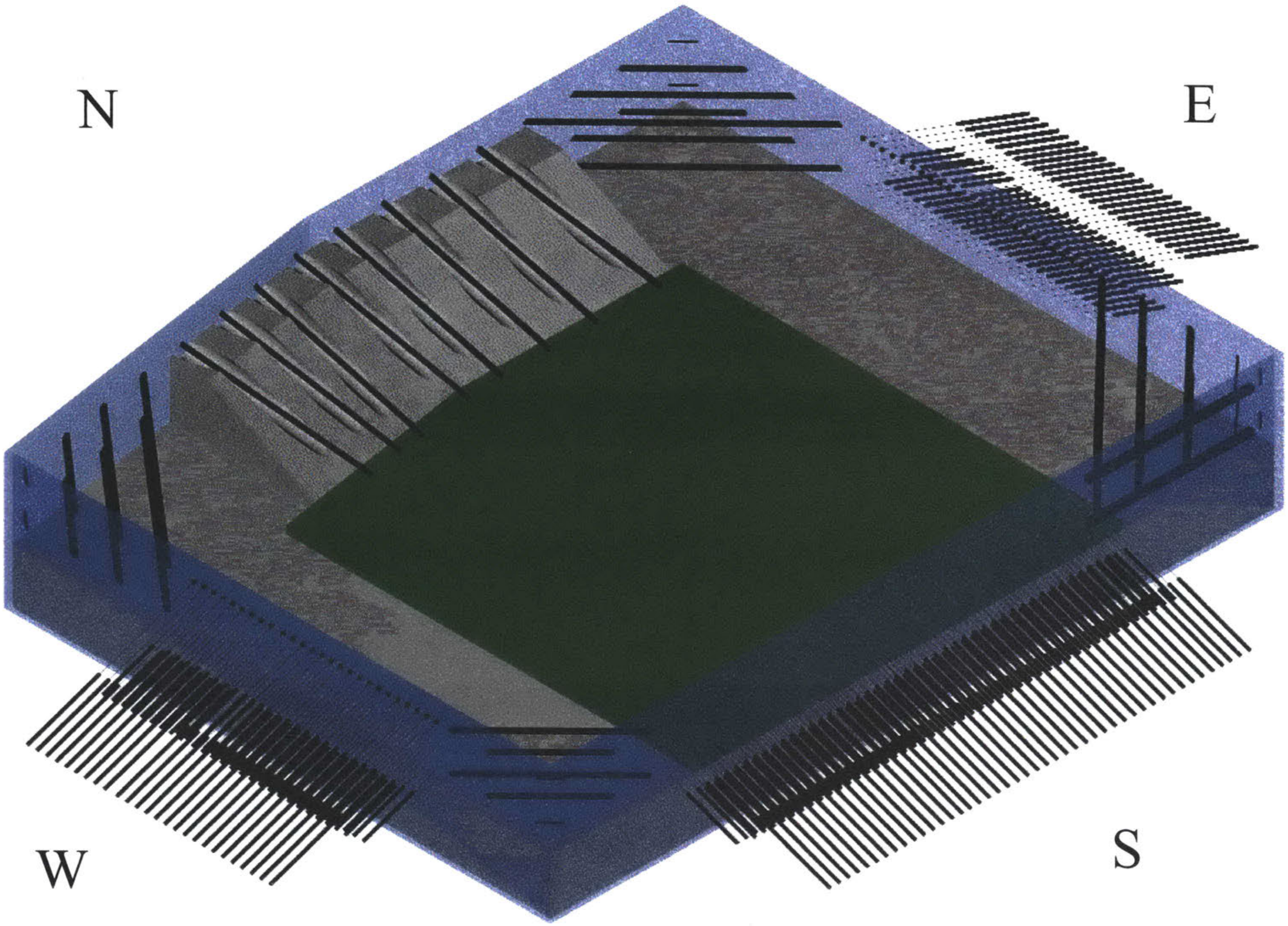


FIGURE 4.5: Schematic of excavation for option 3 showing berm and first level of rakers. Isometric view from the southwest.

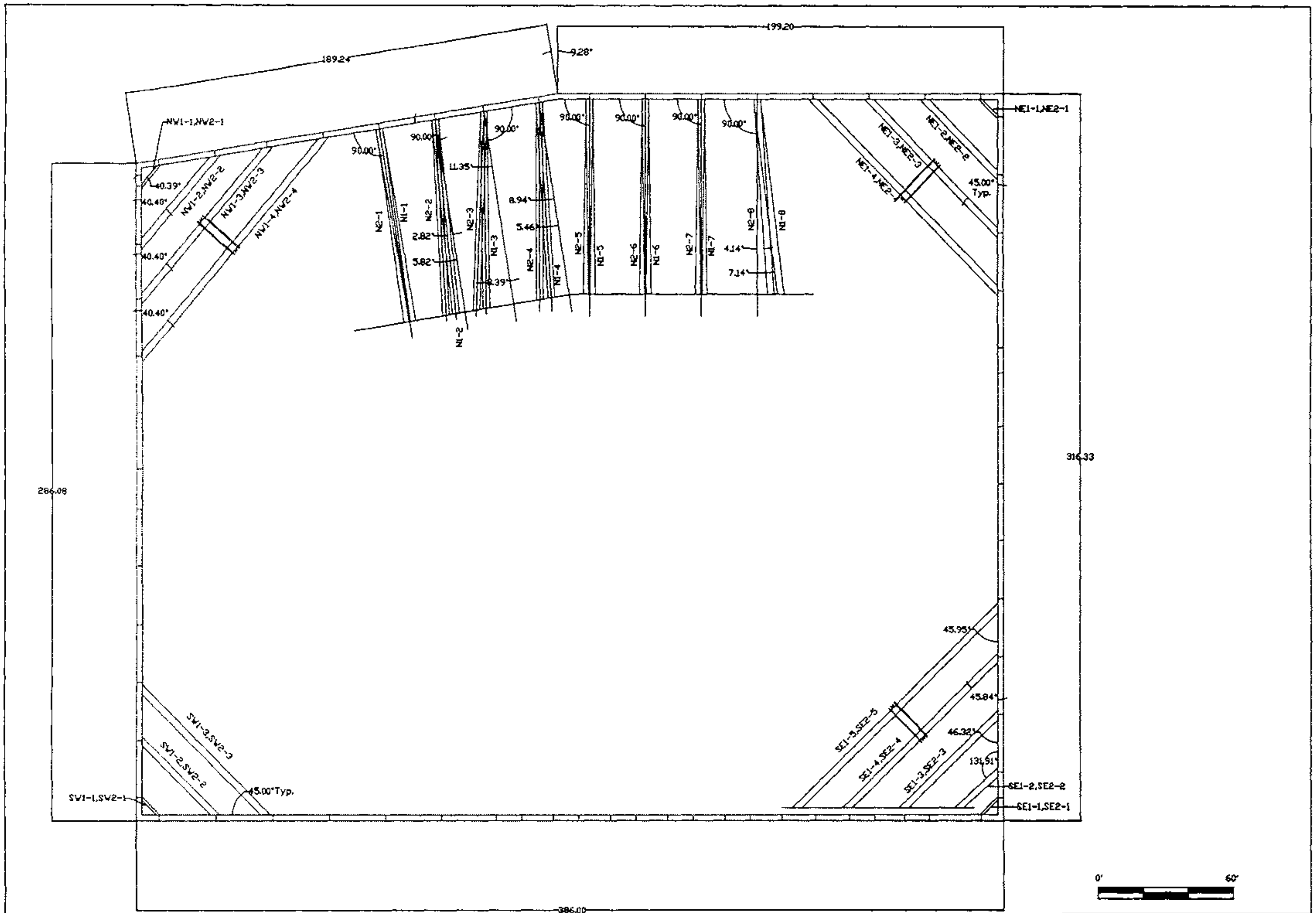


FIGURE 4.6:
PLAN VIEW OF DIAPHRAGM WALL AND SUPPORT

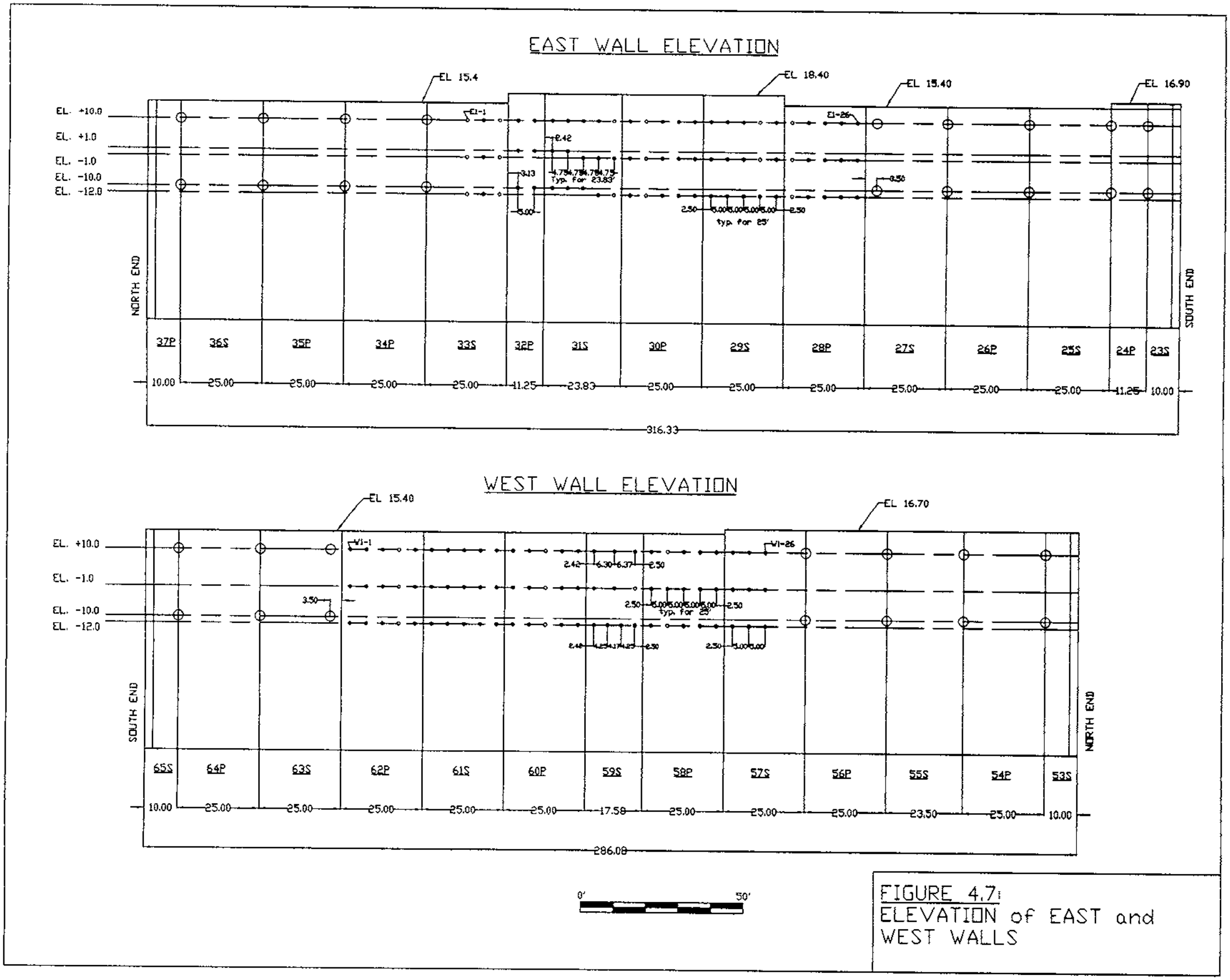
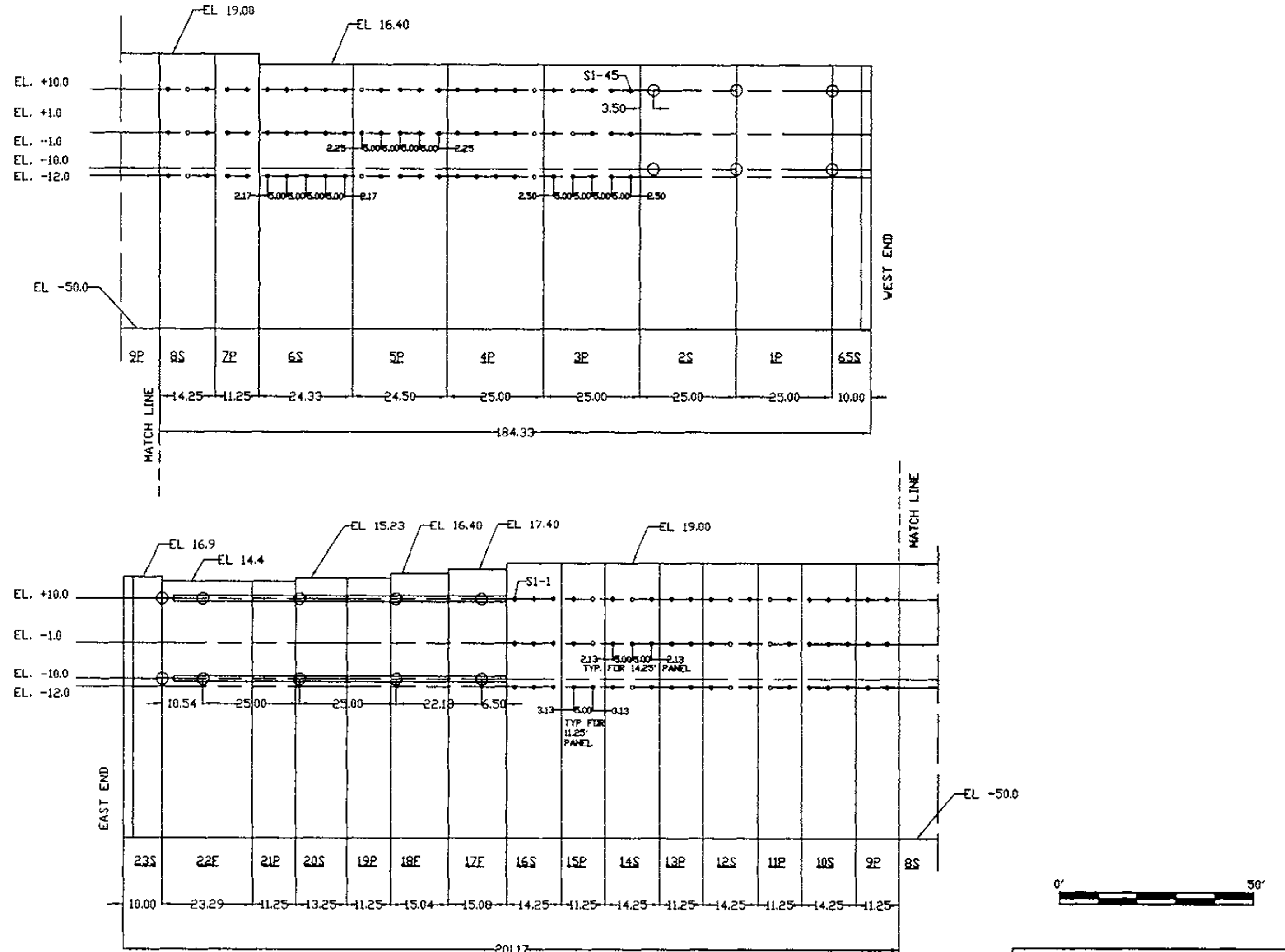
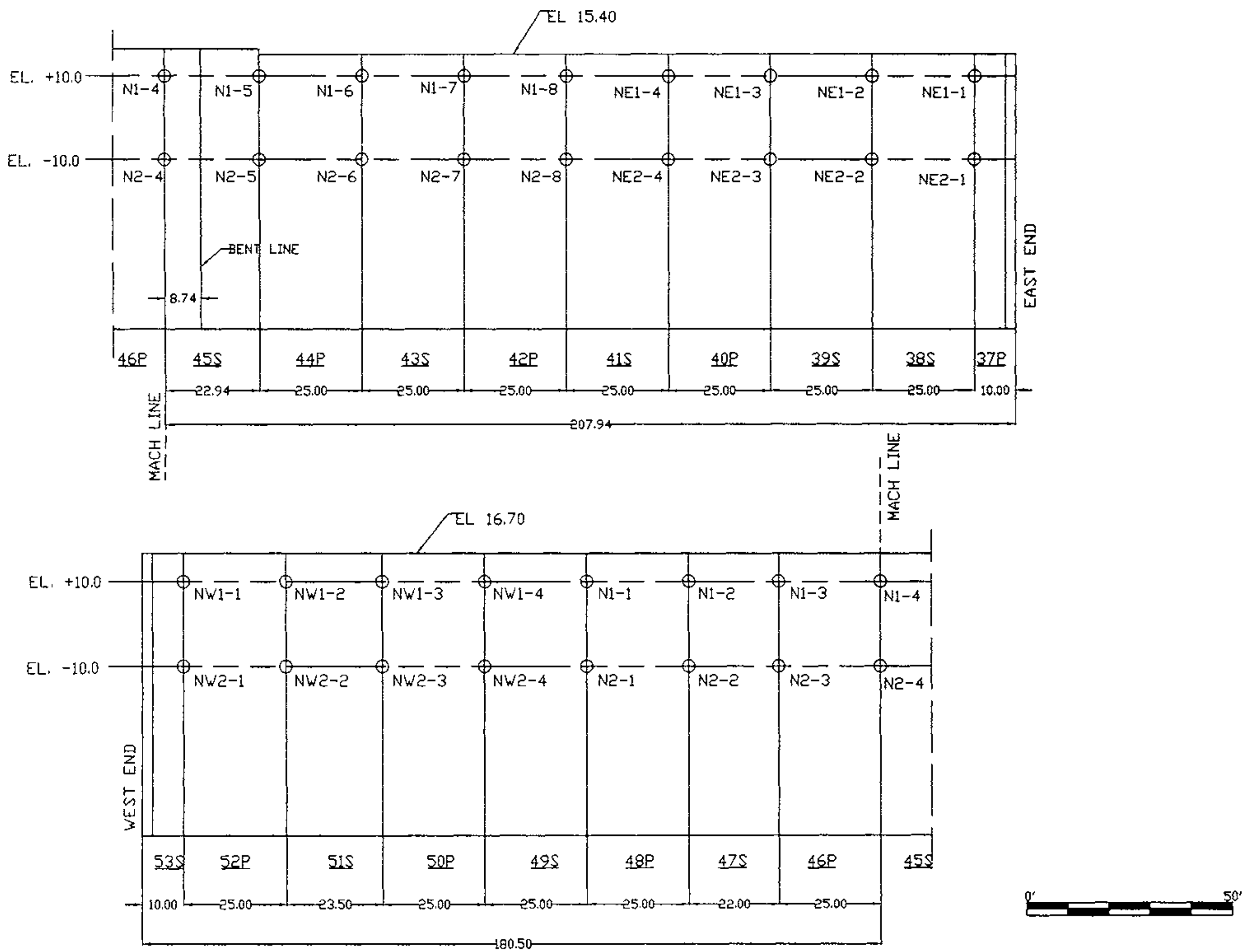


FIGURE 4.7: ELEVATION of EAST and WEST WALLS



SOUTH WALL ELEVATION

FIGURE 4.8
SOUTH WALL ELEVATION



NORTH WALL ELEVATION

FIGURE 4.9:
NORTH WALL
ELEVATION

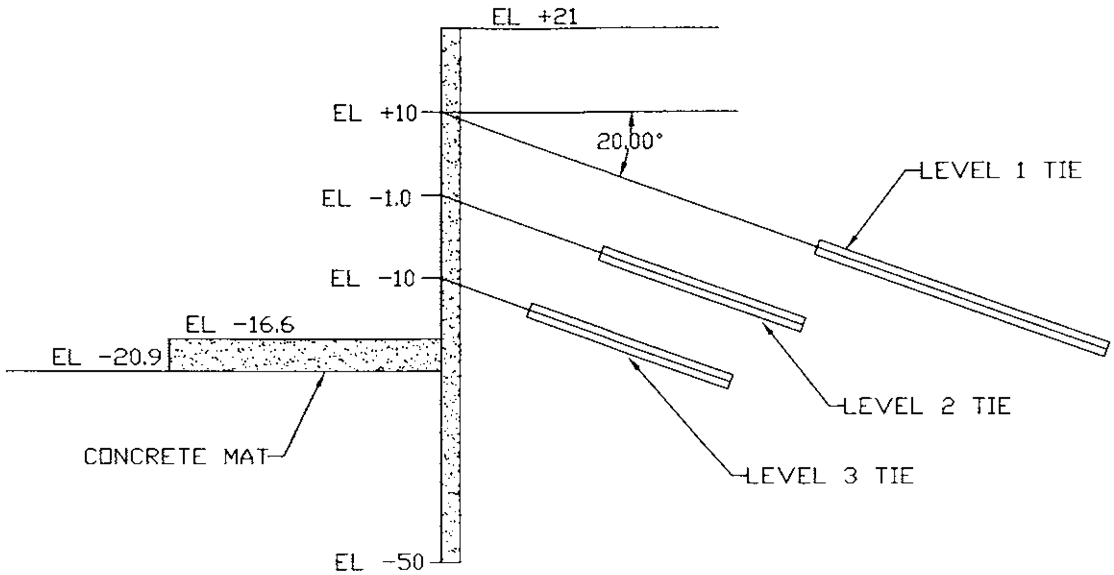


FIGURE 4.10: Typical cross-section of tiebacks and excavation.

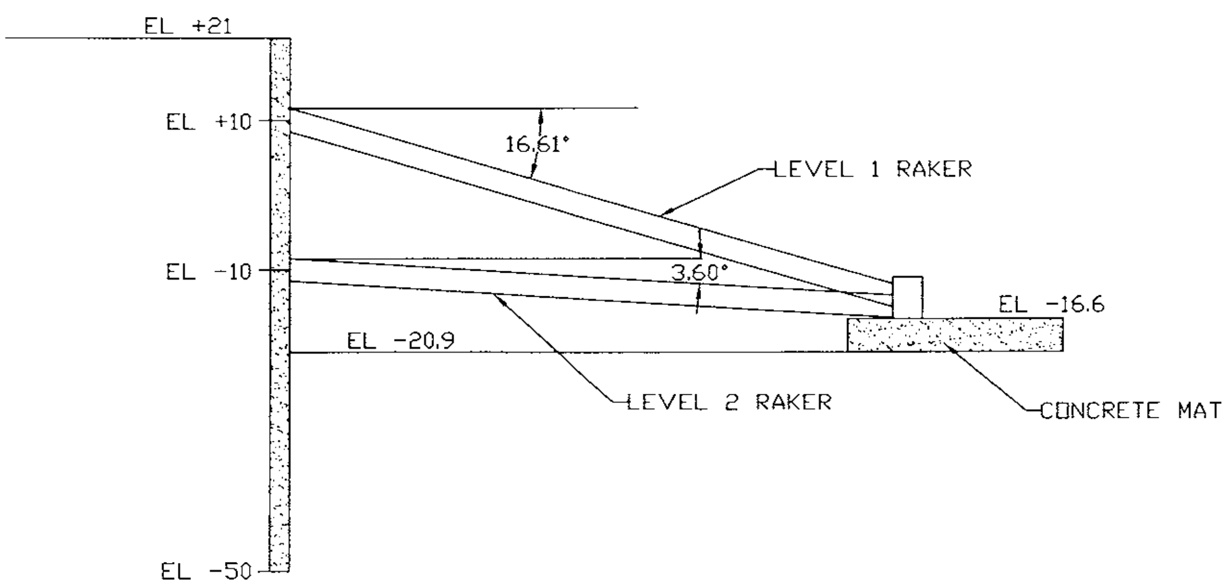


FIGURE 4.11: Typical cross-section of rakers and excavation.

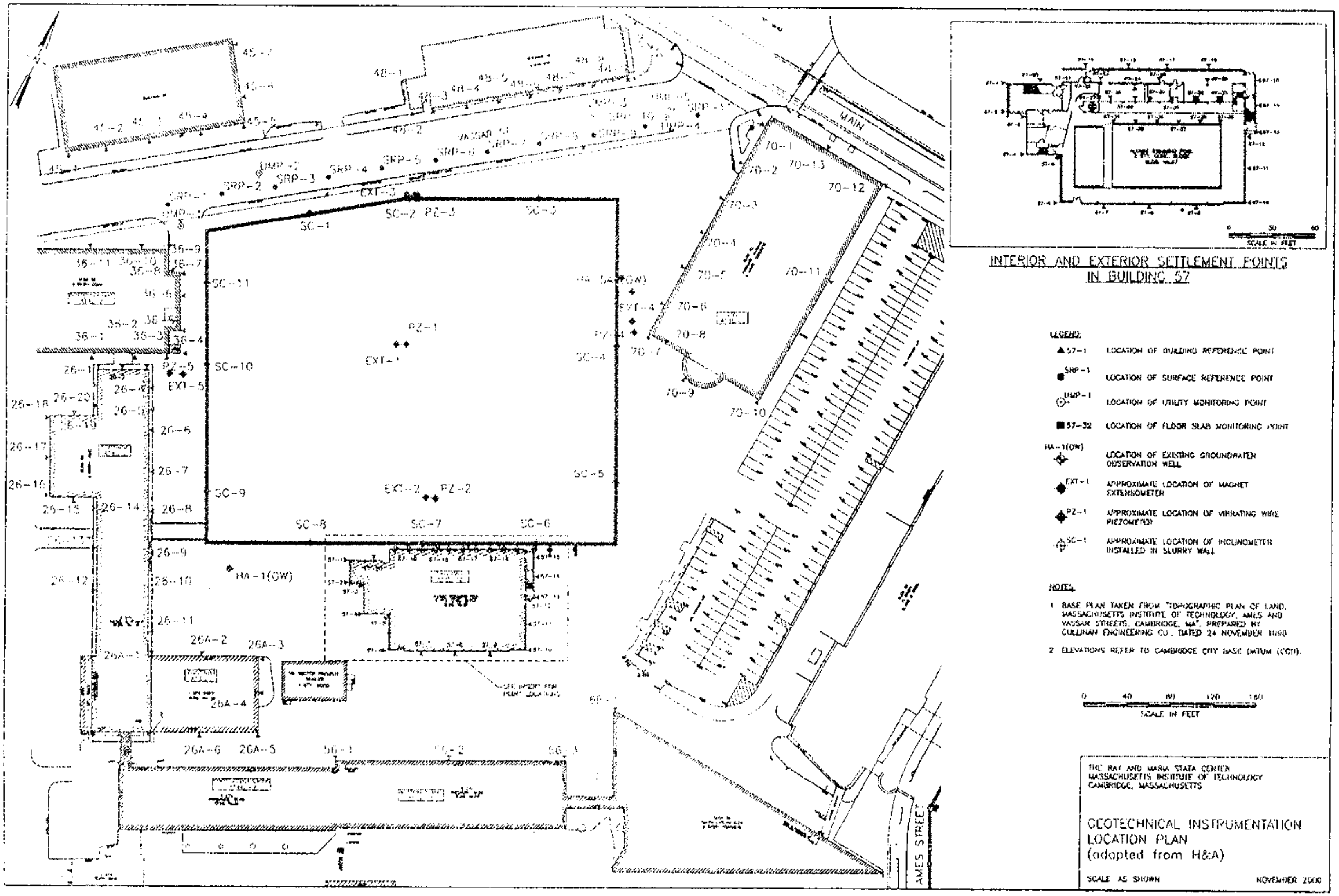
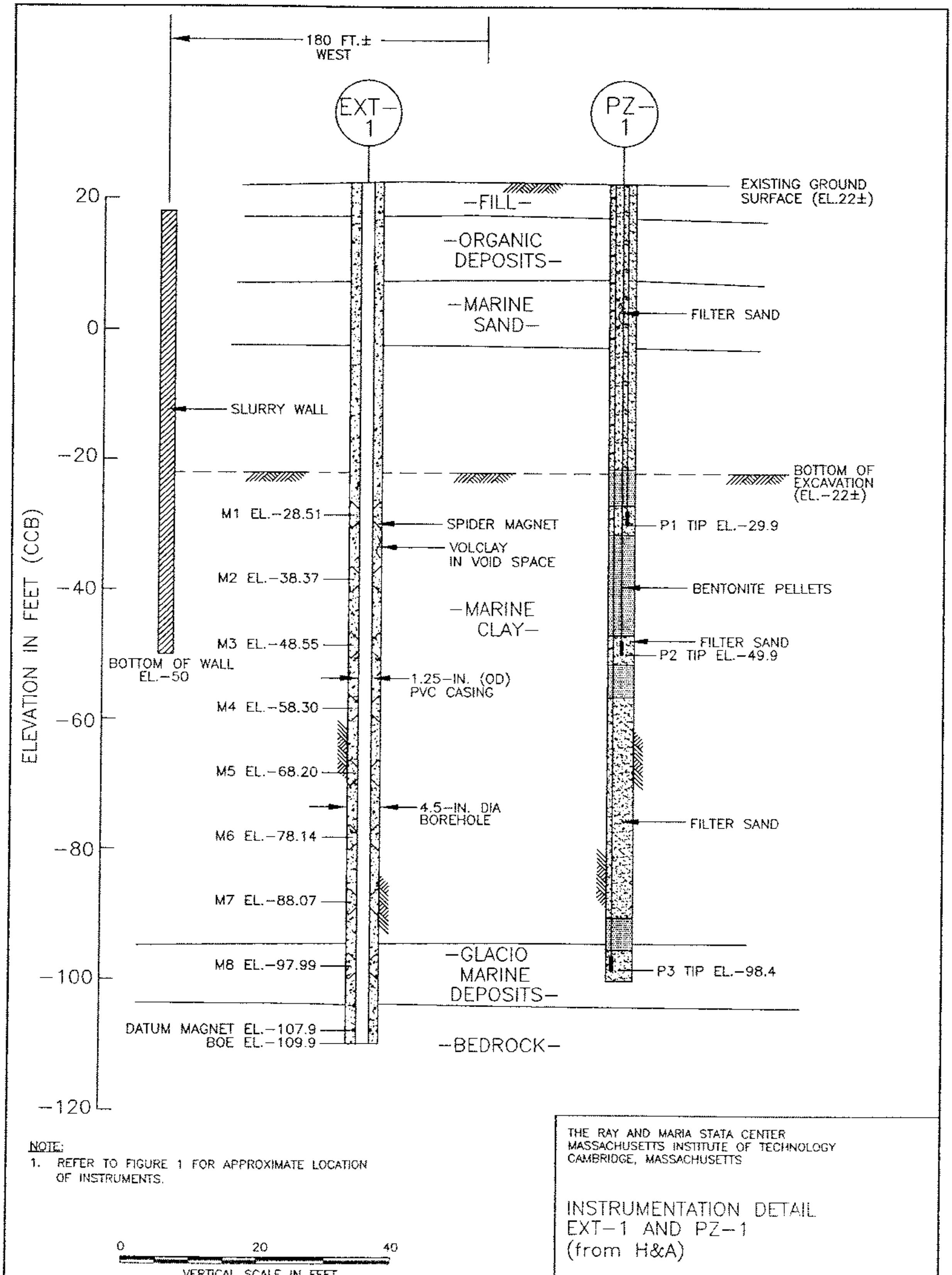

INTERIOR AND EXTERIOR SETTLEMENT POINTS
IN BUILDING 57
LEGEND:
57-1 LOCATION OF BUILDING REFERENCE POINT
SRP-1 LOCATION OF SURFACE REFERENCE POINT
UMP-1 LOCATION OF UTILITY MONITORING POINT
57-32 LOCATION OF FLOOR SLAB MONITORING POINT
HA-1(OW) LOCATION OF EXISTING GROUNDWATER OBSERVATION WELL
EXT-1 APPROXIMATE LOCATION OF MAGNET EXTENSOMETER
PZ-1 APPROXIMATE LOCATION OF VIBRATING WIRE PIEZOMETER
SC-1 APPROXIMATE LOCATION OF INCLINOMETER INSTALLED IN SLURRY WALL
NOTES:
1 BASE PLAN TAKEN FROM "TOPOGRAPHIC PLAN OF LAND, MASSACHUSETTS INSTITUTE OF TECHNOLOGY, AMES AND VASSAR STREETS, CAMBRIDGE, MA", PREPARED BY CULLINAN ENGINEERING CO., DATED 24 NOVEMBER 1990
2 ELEVATIONS REFER TO CAMBRIDGE CITY BASE DATUM (CCB).
SCALE IN FEET
THE RAY AND MARIA STATA CENTER
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CAMBRIDGE, MASSACHUSETTS
GEOTECHNICAL INSTRUMENTATION LOCATION PLAN
(adopted from H&A)
SCALE AS SHOWN
NOVEMBER 2000
MAIN
VASSAR ST
AMES STREET
SC-1
SC-2
SC-3
SC-4
SC-5
SC-6
SC-7
SC-8
SC-9
SC-10
SC-11
PZ-1
PZ-2
PZ-3
PZ-5
EXT-1
EXT-2
EXT-3
EXT-4
EXT-5
HA-1(OW)

FIGURE 4.12



NOTE:
1. REFER TO FIGURE 1 FOR APPROXIMATE LOCATION OF INSTRUMENTS.

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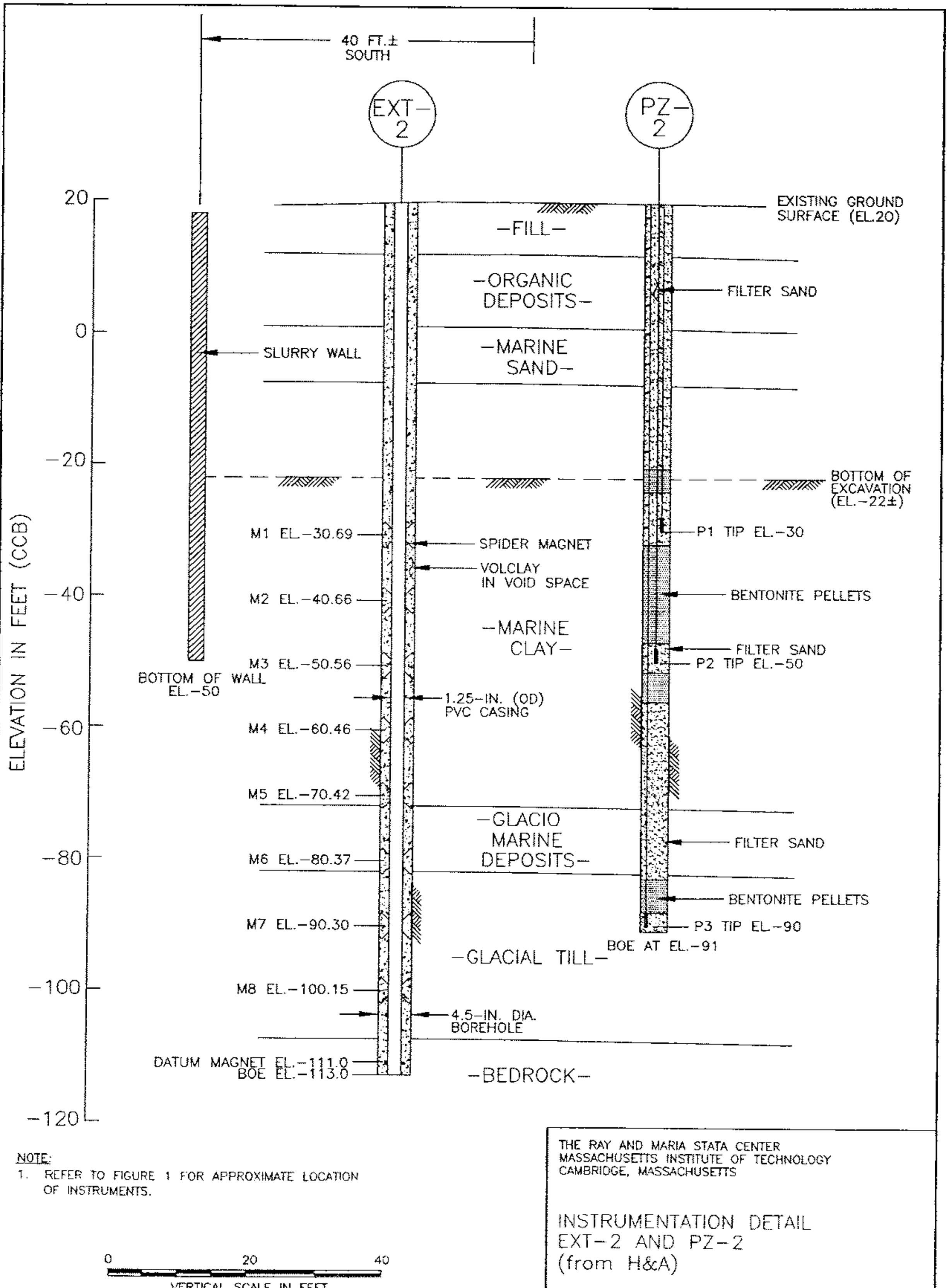
CAMBRIDGE, MASSACHUSETTS

INSTRUMENTATION DETAIL
EXT-1 AND PZ-1
(from H&A)

SCALE: AS SHOWN

FIGURE 4.13



NOTE:
1. REFER TO FIGURE 1 FOR APPROXIMATE LOCATION OF INSTRUMENTS.

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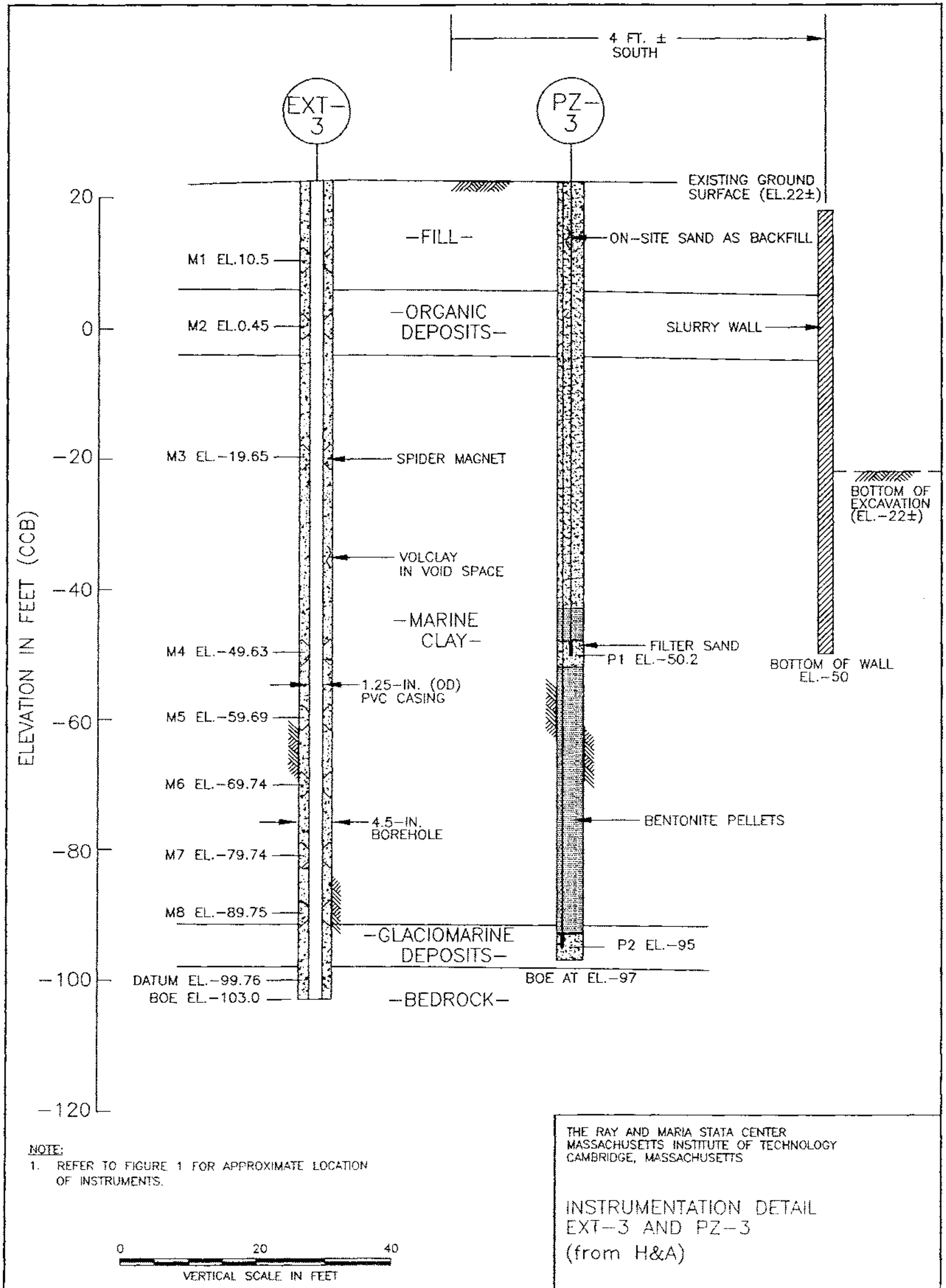
CAMBRIDGE, MASSACHUSETTS

INSTRUMENTATION DETAIL
EXT-2 AND PZ-2
(from H&A)

SCALE: AS SHOWN

FIGURE 4.14



NOTE:
1. REFER TO FIGURE 1 FOR APPROXIMATE LOCATION OF INSTRUMENTS.

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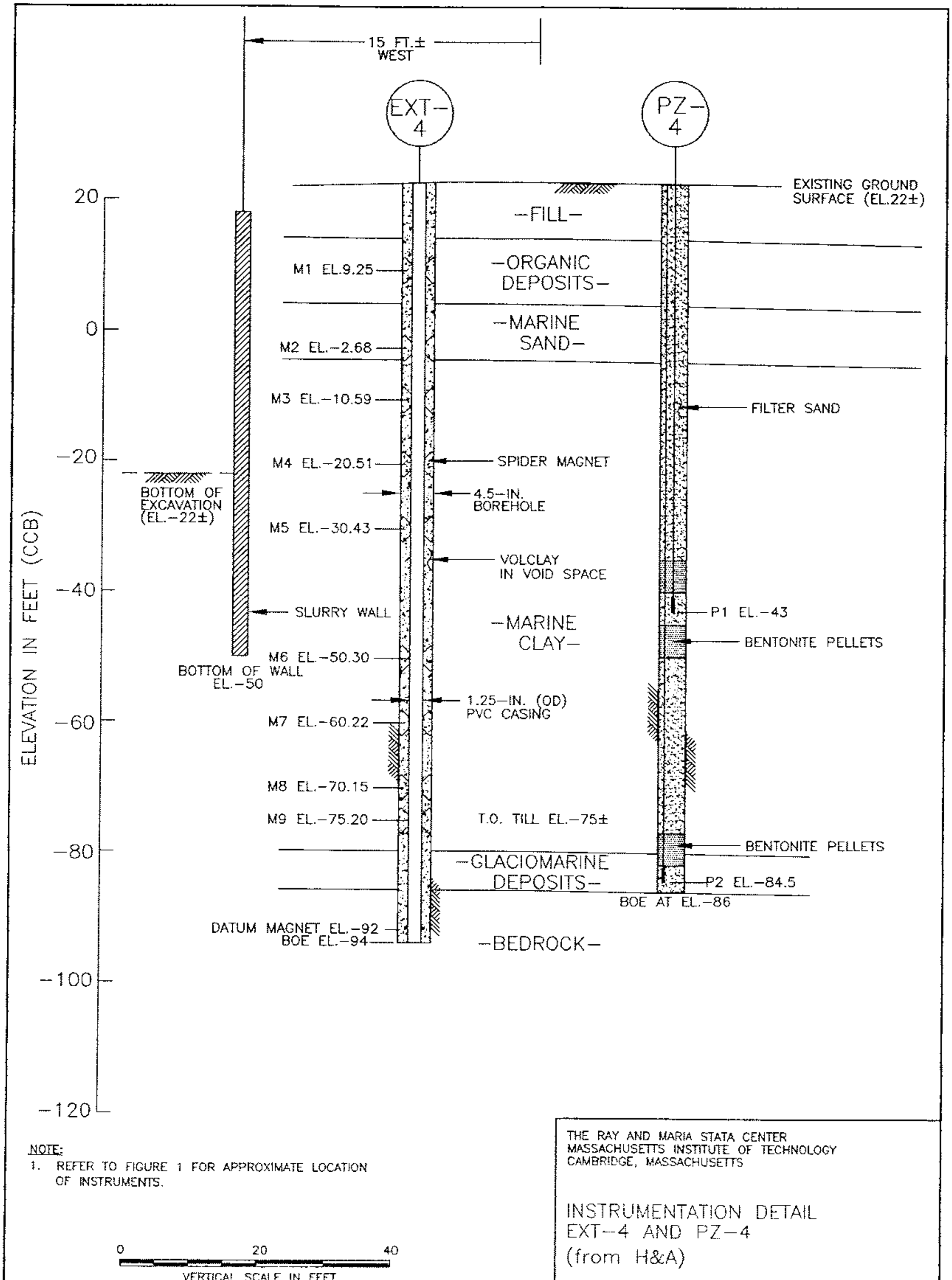
INSTRUMENTATION DETAIL
EXT-3 AND PZ-3
(from H&A)

SCALE: AS SHOWN

FIGURE 4.15



NOTE:
1. REFER TO FIGURE 1 FOR APPROXIMATE LOCATION OF INSTRUMENTS.

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CAMBRIDGE, MASSACHUSETTS

INSTRUMENTATION DETAIL
EXT-4 AND PZ-4
(from H&A)

SCALE: AS SHOWN

FIGURE 4.16

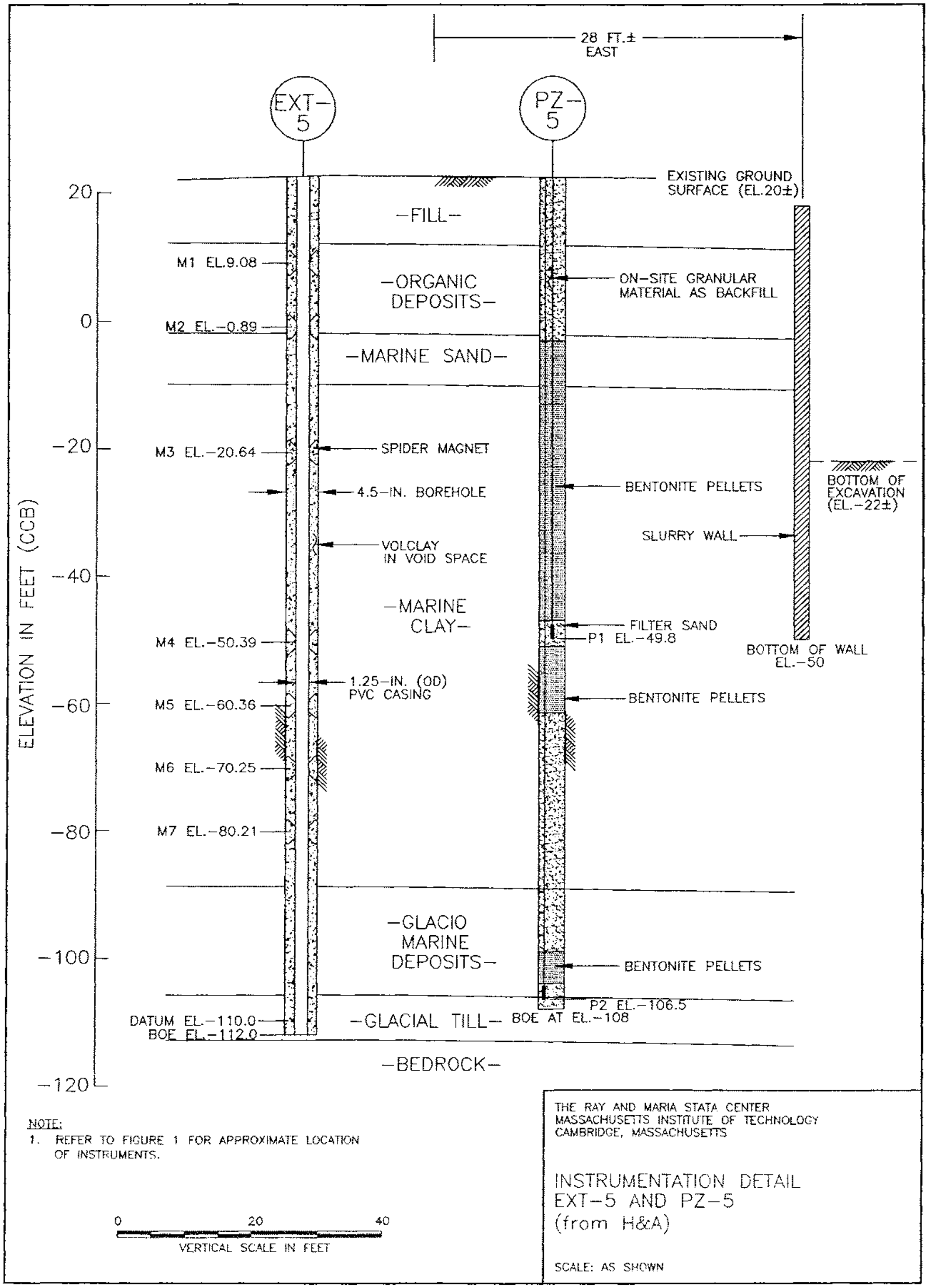
28 FT.± EAST
EXT-5
PZ-5
EXISTING GROUND SURFACE (EL.20±)
-FILL-
-ORGANIC DEPOSITS-
-MARINE SAND-
-MARINE CLAY-
-GLACIO MARINE DEPOSITS-
-GLACIAL TILL-
-BEDROCK-
ELEVATION IN FEET (CCB)
20
0
-20
-40
-60
-80
-100
-120
M1 EL.9.08
M2 EL.-0.89
M3 EL.-20.64
M4 EL.-50.39
M5 EL.-60.36
M6 EL.-70.25
M7 EL.-80.21
DATUM EL.-110.0
BOE EL.-112.0
SPIDER MAGNET
4.5-IN. BOREHOLE
VOLCLAY IN VOID SPACE
1.25-IN. (OD) PVC CASING
ON-SITE GRANULAR MATERIAL AS BACKFILL
BENTONITE PELLETS
FILTER SAND
P1 EL.-49.8
BENTONITE PELLETS
BENTONITE PELLETS
P2 EL.-106.5
BOE AT EL.-108
BOTTOM OF EXCAVATION (EL.-22±)
SLURRY WALL
BOTTOM OF WALL EL.-50
NOTE:
1. REFER TO FIGURE 1 FOR APPROXIMATE LOCATION OF INSTRUMENTS.
0
20
40
VERTICAL SCALE IN FEET
THE RAY AND MARIA STATA CENTER
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CAMBRIDGE, MASSACHUSETTS
INSTRUMENTATION DETAIL
EXT-5 AND PZ-5
(from H&A)
SCALE: AS SHOWN

FIGURE 4.17

Chapter 5
Construction History and Performance

5.1 Introduction

There are three principal factors influencing the soil movements associated with deep excavations: 1) the properties of the soil profile, 2) the characteristics of the excavation support system, and 3) the sequence in which the excavation progresses. In the design phase of a project the soil characteristics are determined through site investigation, and are then used to select and dimension the support system and the construction schedule. For a large complicated project, such as the Stata Center, numerical analyses (such as the Finite Element Method) are now widely used for estimating the design forces in the wall and bracing system and can also be used the estimate/predict the soil movements that can be expected with each proposed phase of construction. Once the project is completed, a back analysis using the actual sequence of construction can be used to evaluate the assumptions used in the analysis (selection of soil parameters, models, etc.) and the predictive reliability. Such case studies provide very important data for improving the methods of analysis and hence, refine the design of excavation support systems for future projects.

The purpose of this chapter is to summarize the construction history and monitoring data for the Stata Center basement in such a way that relationships between the construction history and soil movements can be identified. The presentation of this information is also intended to allow others to use this project for verification of analysis techniques, especially three-dimensional finite element analysis, through modeling of the construction history as closely as possible.

All of the information presented herein was obtained through analysis of data from field instrumentation, study of daily field reports and drawings, conversations with field personnel, and through personal observations. This chapter attempts to organize the large amount of available information into a clear description of what was actually done and how it performed. This text includes a discussion of general trends in the construction, as well as specific problems encountered and the methods adopted to resolve them. The construction history has been subdivided chronologically in order to facilitate the discussion and enable one to address performance issues for the excavation support system. Table 5.1 summarizes the complete calendar of events from the start of excavation (November 14, 2000) through completion of excavation work (June 19, 2001). This table shows columns for each of the important parts (tiebacks, corner bracing, rakers, concrete grade beams[1], and the mat foundation), the corresponding dates, and the figure that represents periods in the construction history. The weather information for the duration of the project is presented in Figure 5.47 (NOAA).

5.2 Construction History

The excavation support system was described in chapter 4. It comprises a reinforced concrete diaphragm wall 30-inches thick extending to El. –50 ft that is supported through the use of: a) three levels of tiebacks on the west, south, and east sides, b) two levels of corner bracing, and c) two levels of raker supports on the north side,

[1] The grade beams were added to the design as construction progressed and will be discussed in a later section.

braced against the central mat foundation. A summary of each of these members and their installation information can be found in Tables 5.2 through 5.6[2].

5.2.1 General Trends

The project started with the installation of the reinforced concrete diaphragm wll using the slurry trench method. This was accomplished over a period of 5 months from July to November 2000. Prior to installation of the south wall a grouting program was undertaken to strengthen the sand layer below building 57 and therefore minimize settlements due to loss of ground (sand flow into the bentonite supported slurry trench) for the wall directly adjacent to its foundation. The grouting program and the wall installation caused minimal movements in building 57.

After complete installation of the perimeter slurry wall, mass excavation began around November 14, 2000. In order to improve the work area, dewatering wells were installed within the excavation to maintain the water table below the excavation grade. Two groundwater observation wells were installed outside of the diaphragm wall (Figure 4.12) and registered no appreciable lowering of the water table surrounding the site during this process. The excavation and support installation did not follow a strict (i.e. prescribed) sequence. This was due in part to areas of contaminated soils that had to be sent to different disposal sites and the speed at which the tiebacks could be installed. No detailed plan of the excavation sequence was available prior to the beginning of mass excavation. This plan is important as it maps out a strategy that will minimize the support costs and construction timeline and maximize the efficiency of the resources

[2] The notation for the support elements is as follows: the beginning letters indicate location in the excavation (ie. E=east side, SW=southwest corner), the first number indicates the level counting down from the surface, and the number after the dash indicates the individual number of the member (ie. For tiebacks

available (manpower and machinery), but it must be flexible enough to adapt to changing ground and weather conditions and other unexpected construction events.

Figure 5.1 shows schematically the strategy for the excavation work. At each grade elevation, the excavation progresses from the southwest corner northwards along the west wall to enable installation of tieback anchors (marked 1 in Figure 5.1). The next phase included excavation along the south wall (for installation of tiebacks) and then north along the east wall (marked 2 and 3 in Figure 5.1). The soil was not removed immediately from the middle of the site, but instead kept as a central surcharge to reduce expected wall movements and improve the stability of the lateral earth support system. Gradually the excavation was then pulled back northward, as shown by the number 4 on the drawing. This same pattern was repeated for each tieback elevation. The north side was left to the end in order to install the raker supports. Also, as will be seen below, the work later developed a general trend of progressing from the southwest toward the northeast. Support installation progressed as quickly as the ties could be installed by the drill rigs (as many as three at one time) and as quickly as the corner bracing could be fabricated on site and put in place.

5.2.2 Construction Problems

Construction problems are inevitable on any project of this size and complexity. Many of the problems that were encountered were routine and rather easy to fix, while others led to modifications in the design of the lateral earth support system. The more routine problems included the collapse of tieback holes during drilling, grout communication between adjacent tiebacks, bond failure in the tiebacks, broken tendons

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and rakers they are numbered from the left when looking at the wall as if standing inside the excavation, and fore corner bracing the numbers start with the brace closest to the corner.)

during prestressing, and water flow through tieback sleeves. None of these problems were wide spread and only affected less than 20% of the tiebacks. The more serious problems included unexpected interference of caissons below building 57 with tieback anchor installation (more that twenty locations were affected), ground loss outside of the excavation through the holes being drilled for the tiebacks, and lateral movements of the wall and building settlements that exceeded the limiting values of 1.5 in.

The more routine problems led to some delays in construction but were resolved through the use of casing to prevent the holes from collapsing, regrouting and redrilling of some tiebacks, and placement of PVC pipe sleeves to reduce water flow. The more serious problems were not always as easily fixed. The ground loss problem was important because excessive ground loss leads to excessive settlement of adjacent buildings and possibly damage. In order to solve this problem casing was used for the entire hole and in some cases left after the tieback was installed. Fortunately most of the locations that experienced this problem were recognized early and the problem solved. Ground loss only became a major concern at one time when one small sinkhole appeared at the surface near the wall in the southeast corner. Fortunately it was not near an adjacent structure and the problem was resolved quickly.

At the start of the project, it was not expected that drilling of tiebacks along the south wall would encounter the caisson foundations of building 57. Early in the project tieback S1-28 (Table 5.4) hit one of the caissons and drilling was stopped on that hole while a solution was discussed. In the meanwhile, a second tieback (S1-23) hit another caisson. This was a problem because it potentially invalidated the use of tieback anchors along much of the south wall (totally altering the lateral earth support system). After some discussions the engineers decided to experiment with drilling through the caissons. They drilled one tieback slowly through the caisson and measured closely the movements it caused. This proved to be a practical solution and twenty-one anchors were installed through the caissons. In those situations, where caissons were located close to the bond zone, the free length of the anchors was extended so that the bond zone was at least 5-feet beyond the nearest caisson.

The most worrisome problem encountered during excavation was that limiting values for the horizontal movement of the diaphragm wall and for settlement of building 57 were exceeded before the final grade was reached. Observations showed that the toe of the wall was moving into the excavation more than expected, suggesting that further excavation could provoke deep-seated ground movements and potentially a basal failure mechanism. Deep-seated wall movements could also be associated with surface settlements, causing damage to the adjacent structure (buildings 57 and 26 in particular). In order to brace the toe of the wall, the project team decided to install unreinforced grade beams (3 ft wide by 5 ft deep by 100 ft long), as buttresses, below the final grade. These grade beams were installed when the excavation depth reached approximately 32-34 ft (i.e., 8-10 feet above final grade elevation). The location of these beams is shown in Figure 5.2. Subsequent data shows that although the grade beams did reduce the rate of wall displacement, the toe of the wall continued to move inwards.

5.2.3 Steps in Construction

Most excavations can be sub-divided into discrete phases (excavation of a lift, installation of supports, and etc.). However, the staging of the Stata basement involved many concurrent activities occurring at different parts of the site. At any given time, it was possible for excavation work, tieback installation, corner bracing installation, and concrete pours to be happening on the same day or on consecutive days. Throughout the project, the geotechnical engineers (Haley & Aldrich, Inc.) provided reports on instrumentation and summaries of the excavation progress (Appendix B), also photographs were taken periodically during excavation work (Appendix C). This section presents a more detailed interpretation based on three-dimensional representations of the construction over selected time intervals, and attempts to correlate these with the local measurements of performance. Figures 5.3 through 5.19 that correspond with each of the steps show as nearly as possible the state of the excavation and lateral earth support at the end of the selected time period along with inclinometer graphs that show the movements since the previous step. In these figures, tiebacks are only shown when the entire level is completed. As noted before, the same notation will be used for support members (ie. SW1-1 is southwest corner brace, top level, first support from the center and W2-5 is west side tieback on the second level, fifth from the left, or south end, when looking at the plans as if from within the excavation).

5.2.3.1 November 14th to December 5th

During this three-week period the excavation work on began in earnest down to the depth for the first level of supports along with construction dewatering (previous to this time, there was some work done on treating and removing surficial contaminated soils and removing the guidewalls used in slurry wall construction). Excavation to the first level of tiebacks and corner bracing began in the southwest corner and initially progressed northward along the west wall a distance of 190 ft, then it progressed east along the south wall, and then finally progressed north along the east wall a distance of

200 ft. All of these areas were excavated to a distance 40 ft from the wall with the southwest corner being excavated slightly more while the tiebacks were installed (Figure 5.3). All areas were excavated two feet below the level of the first supports to El. 8 ft (13 ft deep). During this time drilling and installation of tiebacks on the west side (W1) commenced with the first tiebacks being stressed and locked-off on December 5, as is noted in Tables 5.1 and 5.2.

The excavation progress can be seen in Figure 5.3 along with the initial movements registered in the inclinometers (SC-01 to SC-11). Those inclinometers that were adjacent to areas excavated (5-10) clearly show cantilever deflection of the wall around the site with the maximum movement of 0.9 in occurring at SC-09. This would be expected seeing as there was not yet any support to brace the movements at the top of the wall and also the southwest corner was excavated first leaving more time for the wall to deflect.

5.2.3.2 December 6th to December 19th

In the two weeks from December 6 to 19 the excavation work consisted primarily of the removal of underground structures, removal of soil northward away from the south wall so that the area excavated to El 8 ft was now 120 ft from the south wall, and extending the excavation along the east wall approximately 20 ft northward (Figure 5.4). Work continued on locking off the first level of tiebacks (W1) on the west side and by the end of this period, all but four of them had been completed. Also during this time, drilling was commenced for the first level of tiebacks on the south side (S1) with the first contact on December 14. Changes made to the bond and free lengths can be seen in

Table 5.3. On December 13 the first level of corner bracing in the southwest corner (SW1) as loaded and locked in place with the loads and positions as shown in Table 5.5.

This progress can be seen in Figure 5.4, except for the west tiebacks due to the fact that not all of the tiebacks have been locked off. The effect of the loads from the tiebacks can be seen in the graphs for SC-09 and SC-10 with a large reduction in the deflections seen in the corner, and a slight reduction at the top of SC-10 with continued movement below the depth of excavation. Both of these inclinometers show that with the installation of supports the mode of the wall deflections changes from a cantilever mode to a mode in which there is bending both above and below the wall. Meanwhile, the other inclinometers show minimal incremental movements.

5.2.3.3 December 20th to December 27th

Figure 5.5 summarizes the work completed during this week, with excavation to El. –4 ft for the next level of tiebacks. This work started in the southwest corner and moved north and then east. During this period the tieback row W1 was completed, while lock-off of the S1 tiebacks continued, and the drilling was initiated for both E1 and W2 tiebacks.

The effects of this work can be seen most clearly in SC-08 where the first Sl tiebacks that have been locked off have been able to move the wall back to where the deflections are almost negligible, although some deflection can be seen below the level of excavation. Unfortunately no readings were taken of the deflections experienced in the wall near SC-09 and SC-10 during this period. It would be expected that the excavation work would cause increased deflections around the depth of excavation with smaller movements at the top where movements are restricted by the tiebacks.

5.2.3.4 December 28th to January 11th

During this period, excavation continued to El. –4 ft along the south wall with the work reaching the southeast corner bracing (Figure 5.6). Also, the northwest corner was excavated to El. 7 ft in preparation for the installation of the first level of corner bracing (NW1). As for the support installation, there were many things done during this period. The W2 tiebacks began to be locked off, while the E1 and S1 rows were completed, and drilling commenced on the S2 tiebacks. Also, the first level of corner bracing in the southeast corner (SE1) was installed and loaded.

The effects of some of this work can be seen in the inclinometer data. Once again some of the dates of the readings were not at the end of the period and do not show all of the effects, specifically SC-05 and SC-04. Inclinometers SC-06 through SC-11 all show movements since the last readings. On the south side the effects are greatest on the west and decreasing to the east showing that the work progressed in that direction leaving the west side more time to react to the work.

5.2.3.5 January 12th to January 17th

During this short period, excavation to El. –4 ft was completed on the east side and work began in the southwest corner to excavate to El. –14 ft for the third level of tiebacks (W3) and the second level of corner bracing (SW2). As for the tiebacks, work on locking off W2 was continued and almost completed, work was started on locking off S2, and drilling work was progressing on E2. Then at the end of these five days the first level of corner bracing was installed in the northwest corner (NW1).

Some of the most notable results of this work that can be seen in Figure 5.7 are for SC-8, 9, and 10. They are all showing the effects of the loads added to the wall by the completion of local tiebacks and the recent progress of the excavation. First, they all show decreased movements at the top of the wall that can be attributed to preload forces, and second, while increased movements at depth can be attributed mostly to the new excavation work. Some of this movement and the movement in the other tiebacks can probably be attributed to creep of the tiebacks and of the soil. Also, the effects of the completion of E2 from the previous period can be seen in SC-04.

5.2.3.6 January 18th to January 29th

During this period the excavation involved a complex combination of levels and ramps to facilitate the work and to remove the mass of soil in the middle of the site. The most notable areas of excavation are the near completion of excavation to El. –14 ft near the walls for the west, south, and east sides and the added excavation near the middle of the site. It should also be noted that no work has been done in the northeast up until this point, as the entrance to the site is located in this corner and the trucks used to remove the soil entered there and needed that space as a temporary road to nearer to the excavation for filling. On the west side of the site, the last tiebacks of row W2 were completed, lock-off also began on the W3 tiebacks, and the second level of corner bracing was installed and loaded in the southwest corner (SW2). On the south side work was finished on locking off the S2 tiebacks and on the east work began on locking off the E2 tiebacks.

During this twelve-day period, the increment of movements in the wall on all three excavated sides was very noticeable with the largest increase coming at SC-10 (Figure 5.8). All of the inclinometers near areas of deep excavation also show the same shape of bending with the maximum wall displacement occurring close to the excavated grade elevation. At this time the maximum movements in the affected inclinometers ranged from about 1 inch at SC-04 and SC-06 to about 1.8 inches at SC-10. This last result was of particular concern because at this stage there remained 7 or 8 feet of excavation and the maximum expected deflection of 1.5 inches had been exceeded. This problem was addressed at a later date through installation of grade beams (mentioned in section 5.2.2).

5.2.3.7 January 30th to February 6th

During this week the excavation was concentrated on clearing the areas around the center of the site with further work in the northwest corner to El. –1 ft. The support work included the continuation of locking off the W3 row of tiebacks, drilling and locking off of S3, the completion of E2 , and the drilling for E3 tiebacks.

As Figure 5.9 shows, the movements during this time were less than during the previous period for all but SC-07 that showed about the same increase in its maximum deflection over a period that was four days shorter. The maximum wall deflection was slightly more than 2 inches at SC-07. This is of note since the maximum expected wall deflection was exceeded directly in front of Building 57, and a relatively large inherent movement (0.75 in) had occurred during a week in which no excavation work occurred adjacent to the south wall.

5.2.3.8 February 7th to February 13th

During these six days excavation progressed to El. 7 ft along the north wall from the west. The excavation was extended to El. –1 ft in the northwest corner, and to final grade (El. –21 ft) with installation of a 4 inch thick concrete mudmat in the southwest corner and a portion extending north along the west wall. On February 12th a project meeting was held to discuss the excessive movements that had been observed up to that date. The meeting produced a consensus that action was required to minimize further wall movements and to reduce risks of a possible failure mechanism. Two choices were suggested: 1) construction of large unreinforced concrete grade beams perpendicular to the wall and below the final grade (installed by excavating a trench and filling it with concrete), and 2) construction of steel struts above the final grade that would be braced against a deadman[3]. The first option was selected with a plan to install the grade beams at 25 feet intervals between the corner braces on the west and south sides. It was also decided that thesebeams would be 100 feet long, 3 feet wide, and 5 feet deep. The first three of these beams were installed on February 13 and are shown in Figure 5.10 and the location of all of the beams can be found in Figure 5.2. Also during this time, the final tiebacks of row W3 were completed, work continued on S3 lock off, and drilling for E3 tiebacks.

During this period inclinometerSC-06 showed the greatest increase in movement, but the largest maximum movement remained at SC-07 with a value of about 2.25 inches. Some movement would be expected in SC-09 and SC-10 due to excavation to final grade but this was only completed on the day the reading was made and the effects can be seen in the next figure. Finally it should be noted that cantilever movements of SC-01 are due to the initiation of excavation along Vassar Street. Maximum deflection of SC-01 is not at the top of the wall due to the effects of the corner bracing.

5.2.3.9 February 14th to February 23rd

During this time the excavation work progressed along the western portion of the north wall leaving berm (Figure 5.11) that was designed to support the north wall until raker installation. The northeast corner was also excavated to El. 7 ft in preparation for

[3] A mass of concrete poured below grade that would act as an anchor to brace the strut.

the first level of corner bracing (NE1). Excavation was completed to the final grade elevation over much of the southwest quadrant of the site (170 ft along the south and west walls). Further construction of five grade beams was completed along the south wall (Figure 5.11). The last of the S3 and E3 tiebacks were completed with the exception of E3-1, 2, 3, & 4 that were covered at that time. This means that with the exception of these four tiebacks, the tieback installation was completed. The other support structure completed during this week was the second level of corner bracing in the southeast corner (SE2).

Figure 5.11 shows the effects of excavation to final grade on wall movements measured by SC-10 and SC-09. The incremental movements of SC-10 (approximately 0.5 in.) are still significant and suggest that either the grade beams are only marginally effective, or movements were induced by grade beam construction. Inclinometer SC-04 also shows a large incremental movement prior to completion of the final level 3 tiebacks. Movements of SC-02 and SC-03 are associated with the nearby excavation work, although there is a possibility that this could be caused by the extra surcharge load outside the wall from the loading of trucks on that side of the site that began when the northeast corner was excavated.

5.2.3.10 February 24th to March 2nd

During this time the excavation was concentrated in the northwest quadrant of the site where the excavation to final grade was extended north to the base of the berm in front of the north wall and the berm was extended slightly farther east. This was the extent of work completed with the exception of mudmat placement on the remainder of the area that had been excavated to grade. Work was ongoing to prepare the reinforcement for the first pour for the mat foundation in the southwest corner.

The only significant movement recorded during this period was for SC-08. These movements are indicative of deep seated movements caused by the unloading of the toe of the wall related to the excavation at a distance from the wall but directly in front of it.

5.2.3.11 March 3rd to March 14th

Excavation work during this time comprised excavation to final grade in front of the south wall (extending approximately another 110 feet eastwards), the lowering of the grade to El. –18 ft north of that area, and the continued excavation of the material in the middle of the site (progressing northwards) as seen in Figure 5.13. The supports that were installed during this time are the first level of corner bracing in the northeast corner (NE1) and the last four grade beams along the eastern portion of the south wall. The first two portions (#1 & #2) of the mat foundation were also completed along the west side of the site during this period. The locations of each of the pours for the mat foundation can be seen in Figure 5.20.

As would be expected, most of the inclinometers show very little movement compared to the previous movements. The exceptions are SC-01 that continues to move with only the little support contributed by the berm and the corner bracing, SC-02 that continues to move in a cantilever mode probably due to some excavation immediately in front of it and SC-03 where the effects of the newly installed corner bracing can be seen. The inclinometer that would be expected to show change is SC-07. This is due to the excavation to final grade.

5.2.3.12 March 15th to April 3rd

During these three weeks, a large central area of the site was excavated to final grade and the mudmat was poured as seen in Figure 5.14. By the end of this period the excavation was completed over half of the site with the berms in front of the north wall. The final four tiebacks (E3-1,2,3, and 4)were uncovered and installed and the two more parts of the mat foundation were poured (#2A and #2B in Figure 5.20). Along with the completion of mat pour #2B, the kicker blocks were installed in the slab in preparation for the installation of the rakers N1-1 and 2 and Rakers N2-1 and 2.

Inclinometers SC-04 through SC-07 show significant movements associated with the construction activities, with the maximum of almost 3 inches occurring below the grade elevation at SC-07. It is clear from a comparison of incremental movements of SC-04 and SC-07 that the grade beam construction achieved only modest reduction in wall deflection. Maximum wall movements are twice the value anticipated at the state of the project. InclinometersSC-02 and SC-03 show the influence of the large amount of excavation that was done in their vicinity.

5.2.3.13 April 4th to April 25th

During this three week period the excavation work progressed slowly due to the preparatory work necessary for installing the north wall raker supports and the difficulty of removing the soil from the excavation[4]. Work continued on the mat foundation and three more sections were poured (#3, #4, and #5 in Figure 5.20). The excavation work included excavation of the northwest corner to El. –13 ft in preparation for the second level of bracing, cutting notches in the berm for the installation of the first three rakers on the north side (N1-1, 2, & 3), and excavation of the northeast corner to El. –1 ft. Details of the raker supports are shown in Table 5.6. The supports installed during this time are rakers N1-1, N1-2, and N1-3.

Movements would be expected principally in the northern corner of the site since that is where most of the excavation work took place. This did happen, with both of the inclinometers (SC-01 and SC-11) showing about 1 inch of incremental movement below the bracing. All other inclinometers show little or no movement.

5.2.3.14 April 26th to May 8th

The excavation work for this period was restricted to excavation to final grade in the northwest corner, excavation to El. –10 ft east along the north wall below rakers N1-1 and N1-2, notches in the berm for rakers N1-4, 5, &6, and the beginning of excavation to El. -13 ft for the second level of corner bracing in the northeast corner. The second level of corner bracing in the northwest and the rakers N1-4, 5, 6 were installed during this period. Also, section 9A of the mat foundation was poured (Figure 5.20) along with the formation of the first section of the first floor of the structure (P1, Figure 5.21).

As expected, the only significant wall movements occurred in the northwest corner (SC-01 and SC-11) associated with the excavations in that area. Inclinometer SC-02 also moved slightly due to jacking of the adjacent raker support. It is the load from the jacking that caused the movement.

5.2.3.15 May 9th to May 19th

During this period of the time the northwest corner was excavated to final grade and remainder of the site was excavated to El. –13 ft in preparation for the installation of the second level of rakers and the corner bracing in the northeast corner. Prior to removal of the berm the last two level one rakers (N1-7 and 8) were installed. Also, the first two

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[4] The soil removed from the northwest corner had to be carried across the site in order to be removed

level two rakers (N2-1 and N2-2) were installed. After the berm was removed preparations were begun for the installation of the last of the support members. Three concrete pours were completed during this time, they were mat pour # 6 in the northwest corner and level one pours number 2 and 3 (Figures 5.20 and 5.21, respectively).

The movements recorded by the inclinometers were restricted to SC-01, SC-02, and SC-03. This makes sense because all of the excavation work was restricted to the north side of the excavation. They show movements that range from 0.5 inch and 1 inch.

5.2.3.16 May 20th to June 7th

During this time the area beneath the rakers was excavated to final grade. The final excavation after the last six second level rakers (N2-3, 4, 5, 6, 7, and 8, Table 5.6) were installed. Near the end of this period the final support elements were installed, the second level of corner bracing in the northeast corner (NE2). The construction of the slab and the first parking level then continued with the completion of the mat pour #7A under the first four rakers and the first level pour #4 (Figures 5.20 and 5.21, respectively).

5.2.3.17 June 8th to June 19th

During this final period of excavation, mat pour #7B was completed leaving only two more to complete, and basement level pour #1 whose location can be found in Figure 5.22.

The only inclinometer that showed any appreciable movement is SC-03 due to the excavation directly in front of it. All other inclinometers show little or no movement. This would be expected since all supports were in place and almost all of the mat foundation was completed.

5.3 Measured Performance

The construction of a deep basement is always monitored closely to forewarn of problems associated with potential damage to adjacent structures, caused by excavation induced ground movements, and to update the design of the lateral earth support system ("observational method", Peck 1969). The principal interests in monitoring a deep excavation are measurements of: 1) lateral wall and soil deformations, 2) stresses acting on structural elements, and 3) groundwater pressures. In this project, the instrumentation program was restricted to the use of vertical inclinometers to measure wall deflections, settlement points to mainly measure the settlement of the surrounding buildings but also the vertical deformations both at the surface and within the soil mass (using magnet extensometers), and piezometers to measure the pore pressure changes in the clay layer caused by excavation work. The following sections will present the results of this performance monitoring program.

5.3.1 Inclinometers

Vertical inclinometers are widely used to measure horizontal deformations within soil masses and lateral support wall. The inclinometers consist of a casing that is installed in a borehole and a probe that can be lowered within the casing to measure the movements. As the probe is moved within the casing the angle of the probe from the vertical axis is measured in both directions with the use of a sensitive gravity pendulum, tiltmeters, or a servo accelerometer. The deflections are then calculated automatically using this angular measurement and the known distance between the guidewheels. Since the casing may not be installed vertically, the measured data are referenced to the original (zero) readings taken immediately after installation. In this project, eleven inclinometers were installed within the diaphragm wall as discussed in Chapter 4. These inclinometers were intended to monitor only the horizontal wall movements (there was no data on lateral soil movements outside of the excavation).

A summary of the wall deflections measured throughout the excavation work is presented in Figures 5.23 through 5.25. Each inclinometer is represented with a graph showing the measured movements (with positive deflections being into the excavation), the elevation of support members and excavation levels, and a description of what each of these members or excavation levels are and at what date they were installed or reached. The notation found in these figures either refers to an elevation of excavation[5] or the support members that were installed in the area near the instrument[6].

Prior to construction the maximum expected movement of any point in the diaphragm wall during excavation work was 1.5 inches. This value was first exceeded in inclinometer SC-10 during the middle of January. At that time there was still at least eight more feet of soil to excavate and the area would not be braced below the third level of tiebacks until the mat foundation was installed, which did not happen until March 10th. The use of 1.5 inches as the critical value for movement of wall was chosen because of the amount of settlement that would be expected from such a movement according to empirical relationships. A diaphragm wall such as the wall used here can support much larger movements without a structural failure, but it was believed that a movement greater than 1.5 inches would lead to damage to the surrounding structures caused by the corresponding ground movements. Fortunately, the experience showed that the

[5] ie. El.7 means that the excavation was at elevation 7 during the time frame shown
[6] ie. N1 means the installation of the first level of rakers on the north, NW1 means the installation of the first level of corner bracing in the northwest corner, and S1 means the installation of the first level of tiebacks on the south side of the excavation

surrounding structures could withstand larger settlements without sustaining damage. The final maximum movements measured in the inclinometers range from about 2 inches at SC-02, SC-09, and SC-11 to about 3.2 inches at SC-07.

The difference in each of the inclinometers can be attributed to many things. The primary differences are that each location has a different combination of surcharge loads outside of the excavation, the stratigraphy at each location is slightly different, the support which influence each point may be different, and the excavation sequence and lateral earth vary significantly around the site. Looking at these differences, it is clear that SC-07 would be expected to have the greater movements due to the lateral surcharge load from Building 57 (estimated at 600 psf at El –11 ft to 200 psf at the bottom of the wall, Haley & Aldrich) immediately adjacent to the wall and due to the fact that the support system, the tiebacks are less rigid than the corner supports. In general the greatest movements are expected with in the middle portion of a wall with the smallest movements occurring in the corners. This can be attributed to the increased rigidity of the corner braces combined with corner effects.

5.3.2 Settlements

Often the measured settlements of adjacent structures are the most important aspects of the performance monitoring program. This is because differential settlements are closely related to the degree of damage caused to adjacent structures. On this project settlement measurements were mostly concentrated on buildings (with the exception to this being the surface settlement points along Vassar Street, where movements might damage the road surface or the utilities that are below it).

Data from 125 settlement points were optically surveyed during construction. These data have been compiled into contour maps of the surface settlement for each of the four sides of the excavation at five one month intervals in order to show the progression of settlements (Figures 5.26 through 5.35). These figures use a positive sign convention for heave and negative for settlement. As expected, the largest movements on each side occur near the middle of each wall, where the largest wall deflections were measured. All of the surrounding buildings settled, with the largest vertical movements occurring on the south side beneath building 57. The western end of building 57 settled almost 2.3 inches (Figure 5.34) while the southeastern corner of the building moved only 0.3 inches. Hence, the differential settlement across the entire building exceeded 2 inches. Fortunately, this did not cause any noticeable damage to the structure and therefore was considered acceptable performance even though it exceeds the allowable values set before the start of construction.

In order give a comparison between the settlements measured on each side of the excavation, the settlements points showing the largest movements from each side can be seen in Figure 5.36. In this figure, 57-1 represents the south side, 26-6 and 36-4 represent the west side, 70-8 represents the east side, and SRP-6 represents the north side. The figure clarifies the sequence effect. The south wall settles first (as the tiebacks are installed) and the north wall last (as the rakers are installed). However, final settlements are similar on both the north and south walls which shows that the movements had little to do with the surcharge from adjacent structures. It is also interesting to note that the movements on the east and west are very similar although the grade beams were constructed on the west side but not on the east.

5.3.3 Extensometers

A system of five multipoint magnetic borehole extensometers were used on this project to measure the movements within the soil layers below and outside the excavation. The results from these extensometers are presented in Figures 5.37 through 5.41. Each of these figures shows the vertical movement versus time along with a summary of the level of excavation versus time[7] and a schematic of the location of the instrument. The two extensometers that were below the excavation (EXT-1 and EXT-2) measured heave associated with the relief of overburden as would be expected while the extensometers outside the excavation (EXT-3, EXT-4, and EXT-5) measured settlements that correspond well with the settlements measured on the surface. The largest amount of heave experienced was almost 1.5 inches in EXT-2 (Figure 5.38) and the largest settlements were approximately 1.5 inches in EXT-4 and EXT-5 (Figures 5.40 and 5.41). There is a very good correlation between deep soil movements and the level of excavation that can be seen for all of the extensometers. It should be pointed out that more than half of the heave is from the lower clay (below El. –58 ft) in EXT-1 while in next to the wall (EXT-2) the heave is much shallower. This could be an effect of the tieback installation.

5.3.4 Piezometers

A system of twelve vibrating wire piezometers was used to monitor pore pressure changes within the clay both below and outside the excavation. The measured fluctuations are presented in Figures 5.42 through 5.46. The six piezometers outside of the excavation (PZ-3, PZ-4, and PZ-5) showed minimal change in the pore pressures

[7] For EXT-3, EXT-4, and EXT-5 the level of excavation shown is for the nearest point within the excavation.

outside the excavation with the exception of PZ-3 P1 (Figure 5.44). This could be explained by the fact that the piezometer is located at the very bottom of the wall and could be registering the effects of the excavation work where the piezometers at higher and lower elevations would not. Large changes in pore pressure could indicate either a lowering of the groundwater through leakage in the diaphragm wall or through groundwater flow into the excavation below the wall. In either case the lowering of the groundwater level could lead to greater settlements. The six piezometers that were placed below the excavation performed very well. They show a very clear correlation with the level of excavation, the most dramatic being the comparison of PZ-1 P1 and P2 with the excavation level (Figure 5.42). This indicates that the lower pore pressures were a result of the removal of surcharge by way of construction dewatering and excavation.

TABLE 5.1: Calendar of Key Events in the Construction History

	Excavation	West Ties	South Ties	East Ties	Corners	Rakers	Mat and Slabs	Conc. Beams
Nov-00								
14	begin							
Dec-00								
5	Fig 5.3	begin W1						
13					SW1			
19	Fig 5.4							
21			begin S1					
26		end W1						
27	Fig 5.5							
Jan-01								
5		begin W2	end S1	begin E1	SE1			
9				end E1				
11	Fig 5.6							
12			begin S2					
17	Fig 5.7				NW1			
18		end W2						
23			end S2	begin E2				
29	Fig 5.8	begin W3			SW2			
Feb-01								
1				end E2				
2			begin S3					
6	Fig 5.9							
9		end W3						
13	Fig 5.10							Beams 1, 2, 3
15					SE2			Beams 4, 5
20			end S3					
21				begin E3				
22								Beams 6, 7, 8
23	Fig 5.11			end E3,5-26				

TABLE 5.1 (cont): Calendar of Key Events in the Construction History

	Excavation	West Ties	South Ties	East Ties	Corners	Rakers	Mat and Slabs	Conc. Beams
Mar-01								
2	Fig 5.12							
3							Mat #1	
9								Beams 9, 10
10							Mat #2	
12								Beams 11, 12
13					NE1			
14	Fig 5.13							
17							Mat #2A	
26				E3, 1-4				
28							Mat #2B	
Apr-01								
3	Fig 5.14							
6						N1-1,2,3		
7							Mat #3	
14							Mat #'s 4,5	
25	Fig 5.15							
27						N1-4,5,6		
28							Level 1, #1	
May-01								
1					NW2		Mat #9A	
8	Fig 5.16							
9						N2-1,2	Level 1, #2	
15						N1-7,8		
18							Level 1 #3	
19	Fig 5.17						Mat #6	
22						begin N2-3,4,5,6		
24						end N2-3,4,5,6		
Jun-01								
1						N2-7,8		
5							Mat #7A	
6					NE2			
7	Fig 5.18						Level 1 #4	
15							Level 2 #1	
16	Fig 5.19						Mat #7B	

TABLE 5.2a: Installation details for tiebacks on the West Side, Level 1.

Tieback No.	Elevation (ft-CCB)	Angle (deg)	Free Length(ft)	Bond Length(ft)	Installation		Design Load (kips)	Lock-off	
					Start Date	Finish Date		Load (kips)	Date
W1-1	10	*17*	50	40	11-30-00	12-1-00	112	*124*	12-11-00
W1-2	10	*19*	50	40	12-1-00	12-1-00	112	*124*	12-11-00
W1-3	10	*19*	50	40	11-30-00	11-30-00	112	112	12-18-00
W1-4	10	*18*	50	40	12-1-00	12-1-00	112	*110*	12-11-00
W1-5	10	*17*	50	40	11-30-00	11-30-00	112	*113*	12-11-00
W1-6	10	20	50	40	12-1-00	12-1-00	112	112	12-12-00
W1-7	10	*24*	50	40	11-30-00	11-30-00	112	112	12-12-00
W1-8	10	20	50	40	11-28-00	11-28-00	112	*106*	12-11-00
W1-9	10	*19*	50	40	11-29-00	11-29-00	112	*118*	12-11-00
W1-10	10	20	50	40	11-27-00	11-28-00	112	*118*	12-11-00
W1-11	10	*19*	50	40	11-29-00	12-29-00	112	112	12-11-00
W1-12	10	*19*	50	40	11-27-00	11-27-00	112	*109*	12-19-00
W1-13	10	*18*	50	40	11-28-00	11-28-00	112	*110*	12-5-00
W1-14	10	*17*	50	40	11-27-00	11-27-00	112	*115*	12-11-00
W1-15	10	*16*	50	40	11-28-00	11-29-00	112	*118*	12-11-00
W1-16	10	*16*	50	40	11-27-00	11-27-00	112	112	12-5-00
W1-17	10	*17*	50	40	12-4-00	12-4-00	112	*118*	12-11-00
W1-18	10	*18*	50	40	12-4-00	12-4-00	112	112	12-11-00
W1-19	10	20	50	40	12-4-00	12-4-00	112	*108*	12-12-00
W1-20	10	*16*	50	40	12-5-00	12-5-00	112	112	12-12-00
W1-21	10	*15*	50	40	12-5-00	12-5-00	112	*111*	12-12-00
W1-22	10	*17*	50	40	12-13-00	12-13-00	112	112	12-26-00
W1-23	10	20	50	40	12-13-00	12-18-00	112	*111*	12-26-00
W1-24	10	20	50	40	12-18-00	12-18-00	112	112	12-26-00
W1-25	10	20	50	40	12-19-00	12-19-00	112	*111*	12-26-00
W1-26	10	20	50	40	12-19-00	12-19-00	112	112	12-26-00

TABLE 5.2b: Installation details for tiebacks on the West Side, Level 2.

Tieback No.	Elevation (ft-CCB)	Angle (deg)	Free Length(ft)	Bond Length(ft)	Installation		Design Load (kips)	Lock-off	
					Start Date	Finish Date		Load (kips)	Date
W2-1	-1	20	20	50	12-26-00	12-27-00	128	124	01-05-01
W2-2	-1	20	20	50	12-29-00	12-30-00	128	132.7	01-13-01
W2-3	-1	20	20	50	12-30-00	12-30-00	128	131.3	01-13-01
W2-4	-1	20	20	50	12-27-00	12-27-00	128	129.8	01-13-01
W2-5	-1	20	20	50	12-28-00	12-28-00	128	129.8	01-13-01
W2-6	-1	20	20	50	12-30-00	01-02-01	128	126.8	01-13-01
W2-7	-1	20	20	50	12-27-00	12-27-00	128	128.3	01-13-01
W2-8	-1	20	20	50	12-28-00	12-28-00	128	132.7	01-13-01
W2-9	-1	20	20	50	12-27-00	12-27-00	128	129.8	01-13-01
W2-10	-1	20	20	50	01-02-01	01-03-01	128	126.8	01-13-01
W2-11	-1	20	20	50	01-03-01	01-03-01	128	131.3	01-13-01
W2-12	-1	20	20	50	01-03-01	01-03-01	128	129.8	01-13-01
W2-13	-1	20	20	50	01-03-01	01-04-01	128	131.3	01-13-01
W2-14	-1	20	20	50	01-04-01	01-04-01	128	129.8	01-18-01
W2-15	-1	20	20	50	01-04-01	01-04-01	128	132.7	01-15-01
W2-16	-1	20	20	50	01-05-01	01-05-01	128	128.3	01-13-01
W2-17	-1	20	20	50	01-05-01	01-05-01	128	131.3	01-18-01
W2-18	-1	20	20	50	01-05-01	01-05-01	128	129.8	01-15-01
W2-19	-1	20	20	50	01-05-01	01-05-01	128	129.8	01-15-01
W2-20	-1	20	20	50	01-06-01	01-06-01	128	131.3	01-15-01
W2-21	-1	20	20	50	01-06-01	01-06-01	128	129.8	01-15-01
W2-22	-1	20	20	50	01-06-01	01-08-01	128	131.3	01-15-01
W2-23	-1	20	20	50	12-29-00	01-02-01	128	141.6	01-15-01
W2-24	-1	20	20	50	01-02-01	01-03-01	128	128.3	01-15-01
W2-25	-1	20	20	50	01-03-01	01-04-01	128	129.8	01-15-01
W2-26	-1	20	20	50	01-04-01	01-04-01	128	141.6	01-15-01

TABLE 5.2c: Installation details for tiebacks on the West Side, Level 3.

Tieback No.	Elevation (ft-CCB)	Angle (deg)	Free Length(ft)	Bond Length(ft)	Installation		Design Load (kips)	Lock-off	
					Start Date	Finish Date		Load (kips)	Date
W3-1	-12	20	10	*50*	01-20-01	01-20-01	128	*131.3*	01-29-01
W3-2	-12	20	10	*50*	01-20-01	01-20-01	128	*128.3*	01-31-01
W3-3	-12	20	10	*50*	01-20-01	01-20-01	128	Re-drill location, see below	
W3-4	-12	20	10	*50*	01-22-01	01-22-01	128	*131.3*	01-31-01
W3-5	-12	20	10	*50*	01-22-01	01-22-01	128	*128.3*	01-29-01
W3-6	-12	20	10	*50*	01-22-01	01-22-01	128	*128.3*	01-31-01
W3-7	-12	20	10	*50*	01-22-01	01-22-01	128	*128.3*	01-31-01
W3-8	-12	20	10	*50*	01-22-01	01-22-01	128	*131.3*	01-31-01
W3-9	-12	20	10	*50*	01-22-01	01-23-01	128	*126.8*	01-31-01
W3-10	-12	20	10	*50*	01-23-01	01-23-01	128	*132.7*	01-31-01
W3-11	-12	20	10	*50*	01-23-01	01-23-01	128	*132.7*	01-31-01
W3-12	-12	20	10	*50*	01-23-01	01-23-01	128		01-31-01
W3-13	-12	20	10	*50*	01-23-01	01-23-01	128		01-30-01
W3-14	-12	20	10	*50*	01-23-01	01-23-01	128	*113.6*	02-03-01
W3-15	-12	20	10	*50*	01-23-01	01-23-01	128	*131.3*	01-30-01
W3-16	-12	20	10	*50*	01-23-01	01-23-01	128	*128.3*	01-30-01
W3-17	-12	20	10	*50*	01-23-01	01-23-01	128	*131.3*	01-30-01
W3-18	-12	20	10	*50*	01-23-01	01-23-01	128	*131.3*	01-30-01
W3-19	-12	20	10	*50*	01-23-01	01-23-01	128	*131.3*	01-30-01
W3-20	-12	20	10	*50*	01-22-01	01-22-01	128	Re-drill location, see below	
W3-21	-12	20	10	*50*	01-22-01	01-22-01	128	*126.8*	01-30-01
W3-22	-12	20	10	*50*	01-22-01	01-22-01	128	*131.3*	01-30-01
W3-23	-12	20	10	*50*	01-22-01	01-22-01	128	*126.8*	01-30-01
W3-24	-12	20	10	*50*	01-20-01	01-22-01	128	*128.3*	01-30-01
W3-25	-12	20	10	*50*	01-20-01	01-20-01	128	*131.3*	02-03-01
W3-26	-12	20	10	*50*	01-20-01	01-20-01	128	*131.3*	01-29-01
W3-27	-12	20	10	*50*	01-20-01	01-20-01	128	*128.3*	01-29-01
W3-3R	-12	20	*15*	*50*	02-05-01	02-05-01	128	*137.2*	02-9-01
W3-20R	-12	20	*15*	*50*	02-05-01	02-05-01	128	*138.7*	02-9-01

TABLE 5.3a: Installation details for tiebacks on the South Side, Level 1.

Tieback No.	Elevation (ft-CCB)	Angle (deg)	Free Length(ft)	Bond Length(ft)	Installation Start Date	Installation Finish Date	Design Load (kips)	Lock-off Load (kips)	Lock-off Date
S1-1	10	14.0	65.0	50.0	12-18-00	12-19-00	112	112	01-03-01
S1-2	10	15.0	50	50.0	12-18-00	12-18-00	112	112	01-03-01
S1-3	10	15.0	50	50.0	12-16-00	12-18-00	112	112	01-03-01
S1-4	10	16.0	55.0	50.0	12-16-00	12-16-00	112	112	01-03-01
S1-5	10	17.0	55.0	50.0	12-16-00	12-16-00	112	110.0	01-03-01
S1-6	10	16.0	50	50.0	12-19-00	12-19-00	112	112	01-03-01
S1-7	10	20	50	50.0	12-14-00	12-15-00	112	112	01-03-01
S1-8	10	19.0	50	50.0	12-20-00	12-20-00	112	112	01-03-01
S1-9	10	17.0	50	50.0	12-15-00	12-15-00	112	112	01-03-01
S1-10	10	18.0	50	50.0	12-20-00	12-21-00	112	112	01-03-01
S1-11	10	15.0	50	50.0	12-22-00	12-22-00	112	112	01-03-01
S1-12	10	18.0	50	50.0	12-22-00	12-23-00	112	112	01-05-01
S1-13	10	16.0	50	50.0	12-26-00	12-27-00	112	110.0	01-05-01
S1-14	10	14.0	50	50.0	12-27-00	12-28-00	112	112	01-05-01
S1-15	10	15.0	50	50.0	12-28-00	12-29-00	112	112	01-05-01
S1-16	10	14.0	50	50.0	12-16-00	12-19-00	112	112	01-02-01
S1-17	10	15.0	55.0	50.0	12-16-00	12-16-00	112	112	01-02-01
S1-18	10	17.0	50	50.0	12-19-00	12-19-00	112	112	01-02-01
S1-19	10	17.0	55.0	50.0	12-16-00	12-16-00	112	112	01-02-01
S1-20	10	16.0	50	50.0	12-15-00	12-15-00	112	112	12-29-00
S1-21	10	18.0	50	50.0	12-15-00	12-15-00	112	112	12-29-00
S1-22	10	20	50	50.0	12-14-00	12-14-00	112	112	12-29-00
S1-23	10	20	50	50.0	12-20-00	12-20-00	112	112	12-29-00
S1-24	10	20	50	50.0	12-14-00	12-14-00	112	112	12-29-00
S1-25	10	19.0	50	50.0	12-13-00	12-13-00	112	112	12-29-00
S1-26	10	15.0	50	50.0	12-12-00	12-13-00	112	112	12-29-00
S1-27	10	17.0	50	50.0	12-12-00	12-12-00	112	110.0	12-29-00
S1-28	10	17.0	50	50.0	12-21-00	12-21-00	112	112	12-29-00
S1-29	10	18.0	50	40	12-12-00	12-12-00	112	110.0	12-27-00
S1-30	10	12.0	50	40	12-11-00	12-11-00	112	112	12-27-00
S1-31	10	15.0	50	40	12-11-00	12-11-00	112	112	12-27-00
S1-32	10	18.0	50	40	12-6-00	12-6-00	112	110.0	12-27-00
S1-33	10	19.0	50	40	12-6-00	12-7-00	112	112	12-27-00
S1-34	10	19.0	50	40	12-7-00	12-7-00	112	112	12-27-00
S1-35	10	16.0	50	40	12-7-00	12-7-00	112	112	12-26-00
S1-36	10	14.0	50	40	12-8-00	12-8-00	112	112	12-26-00
S1-37	10	15.0	50	40	12-8-00	12-8-00	112	109.0	12-26-00
S1-38	10	14.0	50	40	12-7-00	12-7-00	112	111.0	12-21-00
S1-39	10	15.0	50	40	12-8-00	12-8-00	112	112	12-21-00
S1-40	10	12.0	50	40	12-11-00	12-11-00	112	111.0	12-21-00
S1-41	10	15.0	50	40	12-12-00	12-12-00	112	112	12-21-00
S1-42	10	15.0	50	40	12-12-00	12-13-00	112	114.0	12-21-00
S1-43	10	15.0	50	40	12-13-00	12-13-00	112	111.0	12-21-00
S1-44	10	15.0	50	40	12-14-00	12-14-00	112	112	12-21-00
S1-45	10	12.0	50	40	12-5-00	12-12-00	112	115.0	12-21-00

TABLE 5.3b: Installation details for tiebacks on the South Side, Level 2.

Tieback No.	Elevation (ft-CCB)	Angle (deg)	Free Length(ft)	Bond Length(ft)	Installation		Design Load (kips)	Lock-off	
					Start Date	Finish Date		Load (kips)	Date
S2-1	-1	20	20	*50.0*	01-09-01	01-09-01	128	*131.3*	01-23-01
S2-2	-1	20	20	*50.0*	01-09-01	01-10-01	128	*129.8*	01-22-01
S2-3	-1	20	*25.0*	*50.0*	01-10-01	01-10-01	128	*125.4*	01-22-01
S2-4	-1	*8.0*	*30.0*	*50.0*	01-10-01	01-11-01	128	*138.6*	01-22-01
S2-5	-1	*9.0*	*30.0*	*50.0*	01-11-01	01-11-01	128	*141.6*	01-23-01
S2-6	-1	20	20	*50.0*	01-12-01	01-12-01	128	*132.7*	01-23-01
S2-7	-1	20	20	*50.0*	01-15-01	01-15-01	128	*128.3*	01-19-01
S2-8	-1	20	*25.0*	*50.0*	01-12-01	01-15-01	128	*128.3*	01-19-01
S2-9	-1	20	20	*50.0*	01-12-01	01-12-01	128	*132.7*	01-19-01
S2-10	-1	20	20	*50.0*	01-12-01	01-12-01	128	*129.8*	01-19-01
S2-11	-1	20	20	*50.0*	01-10-01	01-10-01	128	*131.3*	01-19-01
S2-12	-1	20	20	*50.0*	01-10-01	01-12-01	128	*126.8*	01-19-01
S2-13	-1	20	20	*50.0*	01-11-01	01-11-01	128	*126.8*	01-18-01
S2-14	-1	20	20	*50.0*	01-11-01	01-11-01	128		01-18-01
S2-15	-1	20	20	*50.0*	01-10-01	01-10-01	128	*126.8*	01-18-01
S2-16	-1	20	20	*50.0*	01-08-01	01-08-01	128	*131.3*	01-18-01
S2-17	-1	20	20	*50.0*	01-08-01	01-09-01	128	*132.7*	01-18-01
S2-18	-1	20	20	*50.0*	01-09-01	01-09-01	128	*129.8*	01-18-01
S2-19	-1	20	20	*50.0*	01-10-01	01-10-01	128	*128.3*	01-18-01
S2-20	-1	20	20	*50.0*	01-10-01	01-10-01	128	*132.7*	01-17-01
S2-21	-1	20	20	*50.0*	01-09-01	01-09-01	128	*131.3*	01-17-01
S2-22	-1	20	20	*50.0*	01-09-01	01-09-01	128	*129.8*	01-17-01
S2-23	-1	20	20	*50.0*	01-09-01	01-09-01	128	*128.3*	01-17-01
S2-24	-1	20	20	*50.0*	01-09-01	01-09-01	128	*135.7*	01-17-01
S2-25	-1	20	20	*50.0*	01-08-01	01-08-01	128	*129.8*	01-17-01
S2-26	-1	20	20	*50.0*	01-05-01	01-08-01	128	*128.3*	01-17-01
S2-27	-1	20	20	*50.0*	01-08-01	01-08-01	128	*128.3*	01-17-01
S2-28	-1	*18.0*	20	*50.0*	01-08-01	01-08-01	128	*132.7*	01-17-01
S2-29	-1	*18.0*	20	*50.0*	01-08-01	01-08-01	128	*128.3*	01-17-01
S2-30	-1	*15.0*	20	*50.0*	01-05-01	01-05-01	128	*129.8*	01-17-01
S2-31	-1	*17.0*	20	*50.0*	01-05-01	01-05-01	128	*131.3*	01-17-01
S2-32	-1	20	20	*50.0*	01-06-01	01-06-01	128	*131.3*	01-17-01
S2-33	-1	20	20	*50.0*	01-06-01	01-06-01	128	*131.3*	01-16-01
S2-34	-1	21	20	*50.0*	01-05-01	01-06-01	128	*134.2*	01-16-01
S2-35	-1	21	20	*50.0*	01-05-01	01-05-01	128	*132.7*	01-16-01
S2-36	-1	*18.0*	20	*50.0*	01-05-01	01-05-01	128	*126.8*	01-16-01
S2-37	-1	20	20	*50.0*	01-05-01	01-05-01	128	*126.8*	01-16-01
S2-38	-1	*18.0*	20	*50.0*	01-05-01	01-05-01	128	*141.6*	01-16-01
S2-39	-1	*20.5*	20	*50.0*	01-04-01	01-04-01	128	*129.8*	01-16-01
S2-40	-1	20	20	*50.0*	01-04-01	01-04-01	128	*132.7*	01-12-01
S2-41	-1	*18.0*	20	*50.0*	01-03-01	01-03-01	128	*128.3*	01-12-01
S2-42	-1	*19.0*	20	*50.0*	01-04-01	01-04-01	128	*123.9*	01-12-01
S2-43	-1	*17.0*	20	*50.0*	01-04-01	01-04-01	128	*123.9*	01-12-01
S2-44	-1	*15.0*	20	*50.0*	01-04-01	01-04-01	128	*119.5*	01-12-01
S2-45	-1	*10.0*	20	*50.0*	01-03-01	01-03-01	128		01-12-01

TABLE 5.3c: Installation details for tiebacks on the South Side, Level 3.

Tieback No.	Elevation (ft-CCB)	Angle (deg)	Free Length(ft)	Bond Length(ft)	Installation Start Date	Installation Finish Date	Design Load (kips)	Lock-off Load (kips)	Lock-off Date
S3-1	-12	20	10	*50*	01-29-01	01-29-01	128	*134.3*	02-14-01
S3-2	-12	20	10	*50*	01-29-01	01-29-01	128	*132.8*	02-14-01
S3-3	-12	20	10	*50*	01-29-01	01-29-01	128	*129.8*	02-14-01
S3-4	-12	20	10	*50*	01-29-01	01-29-01	128	*144.6*	02-14-01
S3-5	-12	20	10	*50*	01-29-01	01-29-01	128	*126.9*	02-14-01
S3-6	-12	20	10	*50*	01-29-01	01-29-01	128	*134.3*	02-13-01
S3-7	-12	20	10	*50*	01-29-01	01-30-01	128	*135.7*	02-13-01
S3-8	-12	20	10	*50*	01-30-01	01-30-01	128		02-09-01
S3-9	-12	20	10	*50*	01-30-01	01-30-01	128	*134.3*	02-20-01
S3-10	-12	20	10	*50*	01-30-01	01-30-01	128	*129.8*	02-13-01
S3-11	-12	20	10	*50*	01-30-01	01-30-01	128	*134.3*	02-13-01
S3-12	-12	20	10	*50*	01-30-01	01-30-01	128	*131.3*	02-08-01
S3-13	-12	20	10	*50*	01-30-01	01-30-01	128	*131.3*	02-08-01
S3-14	-12	20	10	*50*	01-30-01	01-31-01	128	*132.7*	02-08-01
S3-15	-12	20	10	*50*	01-31-01	01-31-01	128	*129.8*	02-08-01
S3-16	-12	20	10	*50*	01-31-01	01-31-01	128	*131.3*	02-08-01
S3-17	-12	20	10	*50*	01-25-01	01-25-01	128	*129.8*	02-06-01
S3-18	-12	20	10	*50*	01-25-01	01-25-01	128	*129.8*	02-06-01
S3-19	-12	20	10	*50*	01-25-01	01-25-01	128		02-06-01
S3-20	-12	20	10	*50*	01-25-01	01-25-01	128	*131.3*	02-06-01
S3-21	-12	20	10	*50*	01-25-01	01-25-01	128	*131.3*	02-06-01
S3-22	-12	20	10	*50*	01-26-01	01-26-01	128	*131.3*	02-06-01
S3-23	-12	20	10	*50*	01-26-01	01-26-01	128		02-06-01
S3-24	-12	20	10	*50*	01-24-01	01-24-01	128	Redrilled:see below	
S3-25	-12	20	10	*50*	01-24-01	01-24-01	128	Redrilled:see below	
S3-26	-12	20	10	*50*	01-24-01	01-25-01	128	*129.8*	02-07-01
S3-27	-12	20	10	*50*	01-25-01	01-25-01	128	*128.4*	02-07-01
S3-28	-12	20	10	*50*	01-25-01	01-25-01	128	*132.8*	02-07-01
S3-29	-12	20	10	*50*	01-26-01	01-26-01	128	*129.8*	02-07-01
S3-30	-12	20	10	*50*	01-26-01	01-26-01	128	*132.8*	02-07-01
S3-31	-12	20	10	*50*	01-26-01	01-26-01	128	*131.3*	02-07-01
S3-32	-12	20	10	*50*	01-26-01	01-26-01	128	*131.3*	02-06-01
S3-33	-12	20	10	*50*	01-26-01	01-26-01	128	*128.4*	02-05-01
S3-34	-12	20	10	*50*	01-25-01	01-27-01	128	*132.8*	02-05-01
S3-35	-12	20	10	*50*	01-25-01	01-25-01	128	*131.3*	02-05-01
S3-36	-12	20	10	*50*	01-25-01	01-25-01	128	*131.3*	02-05-01
S3-37	-12	20	10	*50*	01-25-01	01-25-01	128	*128.4*	02-05-01
S3-38	-12	20	10	*50*	01-25-01	01-25-01	128	*135.7*	02-05-01
S3-39	-12	20	10	*50*	01-24-01	01-25-01	128	*132.7*	02-02-01
S3-40	-12	20	10	*50*	01-24-01	01-24-01	128	*128.3*	02-02-01
S3-41	-12	20	10	*50*	01-24-01	01-24-01	128	*126.8*	02-02-01
S3-42	-12	20	10	*50*	01-24-01	01-24-01	128	*134.3*	02-06-01
S3-43	-12	20	10	*50*	01-24-01	01-24-01	128	Redrilled:see below	
S3-44	-12	20	10	*50*	01-24-01	01-24-01	128	*131.3*	02-02-01
S3-45	-12	20	10	*50*	01-24-01	01-24-01	128		02-02-01
S3-43R	-12	20	15	*50*	02-06-01	02-06-01	128	*132.8*	02-16-01
S3-25R	-12	20	15	*50*	02-08-01	02-08-01	128	*140.2*	02-16-01
S3-24R	-12	20	15	*50*	02-08-01	02-08-01	128	*132.8*	02-20-01

TABLE 5.4a: Installation details for tiebacks on the East Side, Level 1.

Tieback No.	Elevation (ft-CCB)	Angle (deg)	Free Length(ft)	Bond Length(ft)	Installation		Design Load (kips)	Lock-off	
					Start Date	Finish Date		Load (kips)	Date
E1-1	10	20	50	40	12-29-00	12-29-00	112	*115*	01-08-01
E1-2	10	20	50	40	12-28-00	12-29-00	112	*113.5*	01-08-01
E1-3	10	20	50	40	12-28-00	12-28-00	112	*112.1*	01-08-01
E1-4	10	20	50	40	12-28-00	12-28-00	112	*113.6*	01-08-01
E1-5	10	20	50	40	12-27-00	12-28-00	112	*115*	01-08-01
E1-6	10	20	50	40	12-27-00	12-27-00	112	*115*	01-08-01
E1-7	10	20	50	40	12-22-00	12-26-00	112	*113.6*	01-08-01
E1-8	10	20	50	40	12-22-00	12-22-00	112	*112.1*	01-08-01
E1-9	10	20	50	40	12-26-00	12-26-00	112	*113.6*	01-08-01
E1-10	10	*19*	50	40	12-27-00	12-27-00	112	*112.1*	01-08-01
E1-11	10	*17*	50	40	12-30-00	12-30-00	112	*113.6*	01-08-01
E1-12	10	20	50	40	12-29-00	12-29-00	112	*113.6*	01-09-01
E1-13	10	*19*	50	40	12-30-00	12-30-00	112	*115*	01-09-01
E1-14	10	*17*	50	40	12-30-00	01-02-01	112	*113.6*	01-09-01
E1-15	10	*16*	50	40	01-02-01	01-02-01	112	*115*	01-09-01
E1-16	10	*16*	50	40	01-02-01	01-02-01	112	*115*	01-09-01
E1-17	10	*18*	50	40	01-02-01	01-02-01	112	*116.5*	01-09-01
E1-18	10	*15*	50	40	01-03-01	01-03-01	112	*113.6*	01-09-01
E1-19	10	*15*	50	40	12-22-00	12-22-00	112	*116.5*	01-09-01
E1-20	10	*18*	50	40	12-22-00	12-22-00	112	*115*	01-09-01
E1-21	10	*21*	50	40	12-22-00	12-22-00	112	*115*	01-09-01
E1-22	10	*16*	50	40	12-21-00	12-21-00	112	112	01-05-01
E1-23	10	20	50	40	12-21-00	12-21-00	112	112	01-05-01
E1-24	10	*19*	50	40	12-20-00	12-21-00	112	112	01-05-01
E1-25	10	*19*	50	40	12-20-00	12-20-00	112	112	01-05-01

TABLE 5.4b: Installation details for tiebacks on the East Side, Level 2.

Tieback No.	Elevation (ft-CCB)	Angle (deg)	Free Length(ft)	Bond Length(ft)	Installation Start Date	Installation Finish Date	Design Load (kips)	Lock-off Load (kips)	Lock-off Date
E2-1	-1	20	20	*50*	01-15-01	01-15-01	128	*141.6*	02-01-01
E2-2	-1	20	20	*50*	01-15-01	01-15-01	128	*126.8*	01-27-01
E2-3	-1	20	20	*50*	01-15-01	01-16-01	128	*134.2*	01-27-01
E2-4	1	20	20	*50*	01-18-01	01-18-01	128	*134.2*	01-27-01
E2-5	1	20	20	*50*	01-18-01	01-18-01	128		01-26-01
E2-6	1	20	20	*50*	01-18-01	01-18-01	128	*128.3*	01-26-01
E2-7	1	20	20	*50*	01-18-01	01-19-01	128		01-26-01
E2-8	-1	20	20	*50*	01-16-01	01-16-01	128	*129.8*	01-26-01
E2-9	-1	20	20	*50*	01-16-01	01-17-01	128	*128.3*	01-26-01
E2-10	-1	20	20	*50*	01-17-01	01-17-01	128	*126.8*	01-26-01
E2-11	-1	20	20	*50*	01-17-01	01-17-01	128	*131.3*	01-26-01
E2-12	-1	20	20	*50*	01-18-01	01-18-01	128	*128.3*	01-26-01
E2-13	-1	20	20	*50*	01-18-01	01-18-01	128	*128.3*	01-26-01
E2-14	-1	20	20	*50*	01-18-01	01-18-01	128	*132.7*	01-26-01
E2-15	-1	20	20	*50*	01-18-01	01-18-01	128	*132.7*	01-26-01
E2-16	-1	20	20	*50*	01-16-01	01-16-01	128	*132.7*	01-26-01
E2-17	-1	20	20	*50*	01-16-01	01-16-01	128	*134.2*	01-26-01
E2-18	-1	20	20	*50*	01-16-01	01-16-01	128	*132.7*	01-23-01
E2-19	-1	20	20	*50*	01-17-01	01-17-01	128	*138.6*	01-24-01
E2-20	-1	20	20	*50*	01-17-01	01-17-01	128	*128.3*	01-24-01
E2-21	-1	20	20	*50*	01-17-01	01-17-01	128	*129.8*	01-24-01
E2-22	-1	20	20	*50*	01-17-01	01-17-01	128	*129.8*	01-24-01
E2-23	-1	20	20	*50*	01-17-01	01-17-01	128	*131.3*	01-24-01
E2-24	-1	20	20	*50*	01-16-01	01-17-01	128	*131.3*	01-24-01
E2-25	-1	20	20	*50*	01-16-01	01-16-01	128	*132.7*	01-23-01

TABLE 5.4c: Installation details for tiebacks on the East Side, Level 3.

Tieback No.	Elevation (ft-CCB)	Angle (deg)	Free Length(ft)	Bond Length(ft)	Installation		Design Load (kips)	Lock-off	
					Start Date	Finish Date		Load (kips)	Date
E3-1	-12	20	15	50	03-14-01	03-14-01	128		03-26-01
E3-2	-12	20	15	50	03-15-01	03-15-01	128		03-26-01
E3-3	-12	20	15	50	03-15-01	03-15-01	128		03-26-01
E3-4	-10	20	15	50	03-15-01	03-15-01	128		03-26-01
E3-5	-10	20	15	50	02-09-01	02-09-01	128	132.7	02-23-01
E3-6	-10	20	15	50	02-12-01	02-12-01	128	128.4	02-23-01
E3-7	-10	20	15	50	02-12-01	02-12-01	128	129.8	02-23-01
E3-8	-10	20	15	50	02-13-01	02-13-01	128	131.3	02-23-01
E3-9	-12	20	15	50	02-13-01	02-13-01	128	132.7	02-23-01
E3-10	-12	20	15	50	02-09-01	02-09-01	128	132.7	02-23-01
E3-11	-12	20	15	50	02-09-01	02-09-01	128	132.7	02-22-01
E3-12	-12	20	15	50	02-09-01	02-09-01	128	135.7	02-22-01
E3-13	-12	20	15	50	02-07-01	02-07-01	128	129.8	02-22-01
E3-14	-12	20	15	50	02-07-01	02-07-01	128	131.3	02-22-01
E3-15	-12	20	15	50	02-07-01	02-07-01	128		02-22-01
E3-16	-12	20	15	50	02-05-01	02-07-01	128	129.8	02-22-01
E3-17	-12	20	10	50	02-02-01	02-02-01	128	131.3	02-22-01
E3-18	-12	20	10	50	02-02-01	02-02-01	128	128.4	02-22-01
E3-19	-12	20	10	50	02-01-01	02-02-01	128	132.7	02-22-01
E3-20	-12	20	10	50	02-01-01	02-01-01	128	131.3	02-22-01
E3-21	-12	20	10	50	02-01-01	02-01-01	128	134.3	02-21-01
E3-22	-12	20	10	50	02-01-01	02-01-01	128	134.3	02-21-01
E3-23	-12	20	10	50	02-01-01	02-01-01	128	131.3	02-21-01
E3-24	-12	20	10	50	02-01-01	02-01-01	128	128.4	02-21-01
E3-25	-12	20	10	50	02-01-01	02-01-01	128	131.3	02-21-01

TABLE 5.5: Summary of Corner Bracing Struts and date of installation

Strut	Elevation (ft-CCB)	Length (ft)	Unbraced Length (ft)	Design Load (normal to the wall) (kips/ft)	Tributary Length (ft)	Angle B/N Strut and Wall (degrees)	Strut Size		Design Load (kips)	Jacking Load (kips)	Date of Preloading
							Diameter (in)	Wall Thickness (in)			
NW1-1	10	8.6	8.6	26	16.25	40.39	W14X90		555	0	1-17-01
NW1-2	10	40.8	40.8	26	24.25	40.4	36	0.38	828	414	1-17-01
NW1-3	10	76.6	38.3	26	24.25	40.4	36	0.38	828	414	1-17-01
NW1-4	10	114.7	57.4	26	25	40.4	36	0.38	853	427	1-17-01
NW2-1	-10	8.6	8.6	44	16.25	40.39	W14X120		939	0	4-30-01
NW2-2	-10	40.8	40.8	44	24.25	40.4	36	0.519	1401	700	4-30-01
NW2-3	-10	76.6	38.3	44	24.25	40.4	36	0.519	1401	700	4-30-01
NW2-4	-10	114.7	57.4	44	25	40.4	36	0.63	1444	722	5-1-01
SW1-1	10	7.8	7.8	26	16.25	45	W14X90		598	0	12-13-01
SW1-2	10	37.8	37.8	26	23.25	45	36	0.38	855	427	12-13-01
SW1-3	10	73.2	73.2	26	14.25	45	36	0.38	524	262	12-13-01
SW2-1	-10	7.8	7.8	44	16.25	45	W14X132		1011	0	1-26-01
SW2-2	-10	37.8	37.8	44	23.25	45	36	0.519	1447	723	1-26-01
SW2-3	-10	73.2	73.2	44	14.25	45	36	0.519	887	443	1-29-01
NE1-1	10	7.8	7.8	26	16.25	45	W14X90		598	0	3-12-01
NE1-2	10	37.8	37.8	26	25	45	36	0.38	919	460	3-12-01
NE1-3	10	73.2	36.6	26	25	45	36	0.38	919	460	3-12-01
NE1-4	10	108.5	54.3	26	25	45	36	0.38	919	460	3-13-01
NE2-1	-10	7.8	7.8	44	16.25	45	W14X132		1011	0	6-5-01
NE2-2	-10	37.8	37.8	44	25	45	36	0.519	1556	778	6-5-01
NE2-3	-10	73.2	36.6	44	25	45	36	0.519	1556	778	6-6-01
NE2-4	-10	108.5	54.3	44	25	45	36	0.63	1556	778	6-6-01

Table 5.5 (cont): Summary of Corner Bracing Struts and date of installation

			Design Load			Angle B/N	Strut Size				
Strut	Elevation (ft-CCB)	Length (ft)	Unbraced Length (ft)	(normal to the wall) (kips/ft)	Tributary Length (ft)	Strut and Wall (degrees)	Diameter (in)	Wall Thickness (in)	Design Load (kips)	Jacking Load (kips)	Date of Preloading
SE1-1	10	7.8	7.8	26	10.84	45	W14X90		398	0	1-5-01
SE1-2	10	20.2	20.2	26	19.59	48.09	36	0.38	720	360	1-5-01
SE1-3	10	55.5	55.5	26	25	46.32	36	0.38	919	460	1-5-01
SE1-4	10	90.9	45.5	26	23.25	45.84	36	0.38	855	427	1-5-01
SE1-5	10	121.3	60.7	26	14.25	45.95	36	0.38	524	262	1-5-01
SE2-1	-10	7.8	7.8	44	10.84	45	W14X90		674	0	2-14-01
SE2-2	-10	20.2	20.2	44	19.59	48.09	36	0.393	1219	609	2-14-01
SE2-3	-10	55.5	55.5	44	25	46.32	36	0.63	1556	778	2-14-01
SE2-4	-10	90.9	45.5	44	23.25	45.84	36	0.519	1447	723	2-15-01
SE2-5	-10	121.3	60.7	44	14.25	45.95	36	0.393	887	443	2-15-01

TABLE 5.6: Summary of Raker Supports and date of installation.

Raker	Elevation (@north wall) (ft-CCB)	Overall Length (ft)	Unbraced Length (ft)	Design Load (normal to the wall) (kips/ft)	Tributary Length (ft)	Angle B/N Raker and the horizontal (vertical) (degrees)	Angle B/N Raker and Wall (horizontal) (degrees)	Strut Size		Design Load (kips)	Jacking Load (kips)	Date of Jacking
								Diameter (in)	Wall Thickness (in)			
N1-1	10	88.6	88.6	26	25	16.61	91.53	36	0.519	678	339	04-06-01
N1-2	10	88.7	88.7	26	23.5	16.61	87.18	36	0.519	638	319	04-06-01
N1-3	10	89.5	89.5	26	23.5	16.61	81.61	36	0.519	644	322	04-06-01
N1-4	10	89	89	26	24	16.61	84.54	36	0.519	652	326	04-26-01
N1-5	10	88.6	88.6	26	24	16.61	91.53	36	0.519	650	325	04-27-01
N1-6	10	88.6	88.6	26	25	16.61	91.53	36	0.519	678	339	04-27-01
N1-7	10	88.6	88.6	26	25	16.61	91.53	36	0.519	678	339	05-14-01
N1-8	10	89.3	89.3	26	25	16.61	97.14	36	0.519	683	341	05-15-01
N2-1	-10	85.2	85.2	44	25	3.6	91.53	36	0.803	1103	551	05-07-01
N2-2	-10	85.6	85.6	44	23.5	3.6	84.18	36	0.803	1041	521	05-09-01
N2-3	-10	86.9	86.9	44	23.5	3.6	78.65	36	0.803	1057	528	05-22-01
N2-4	-10	86.2	86.2	44	24	3.6	81.06	36	0.803	1070	535	05-23-01
N2-5	-10	85.2	85.2	44	24	3.6	91.53	36	0.803	4057	529	05-24-01
N2-6	-10	85.2	85.2	44	25	3.6	91.53	36	0.803	1103	551	05-24-01
N2-7	-10	85.2	85.2	44	25	3.6	91.53	36	0.803	1103	551	06-01-01
N2-8	-10	85.4	85.4	44	25	3.6	94.14	36	0.803	1105	553	06-01-01

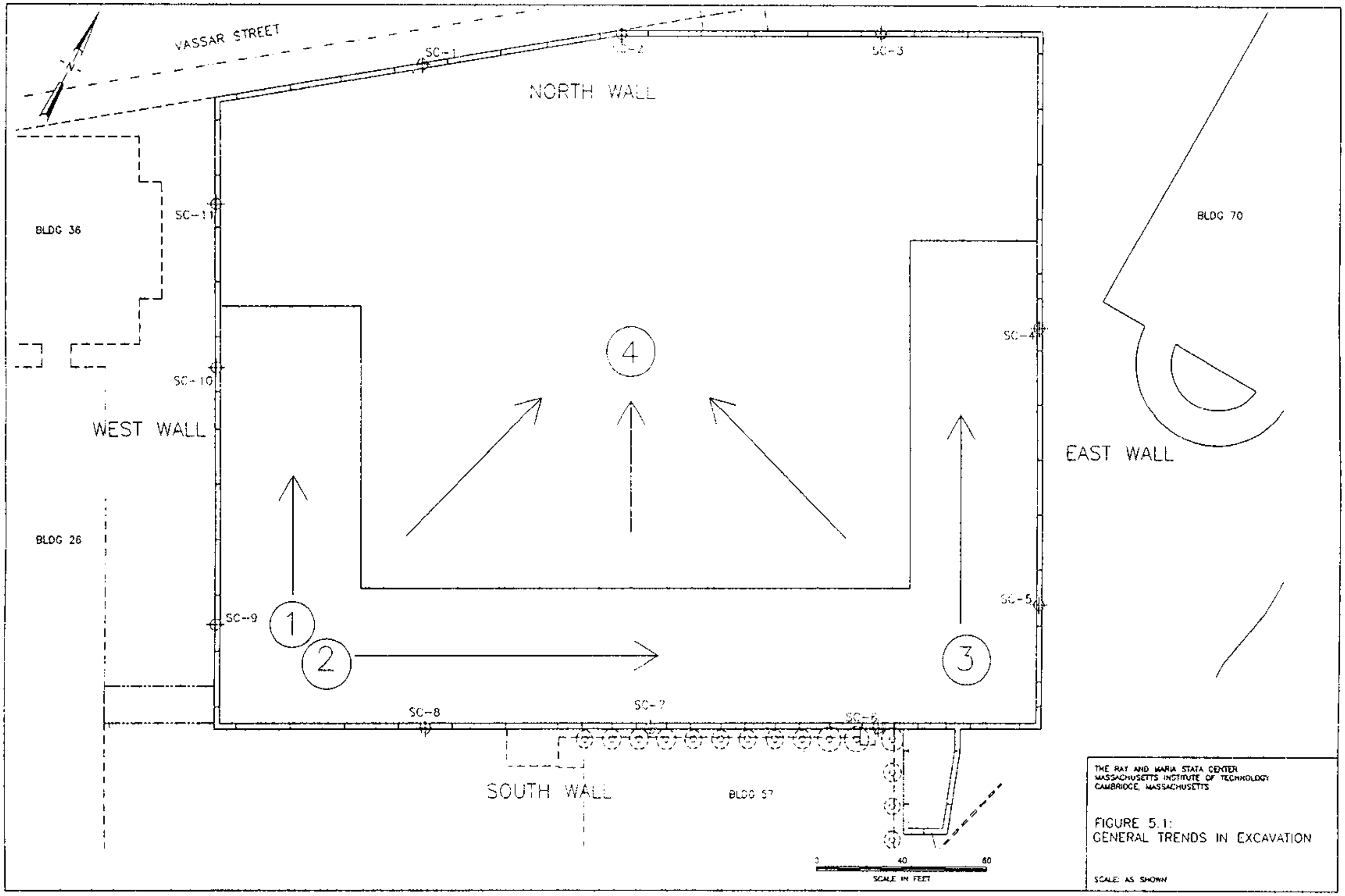


FIGURE 5.1:
GENERAL TRENDS IN EXCAVATION

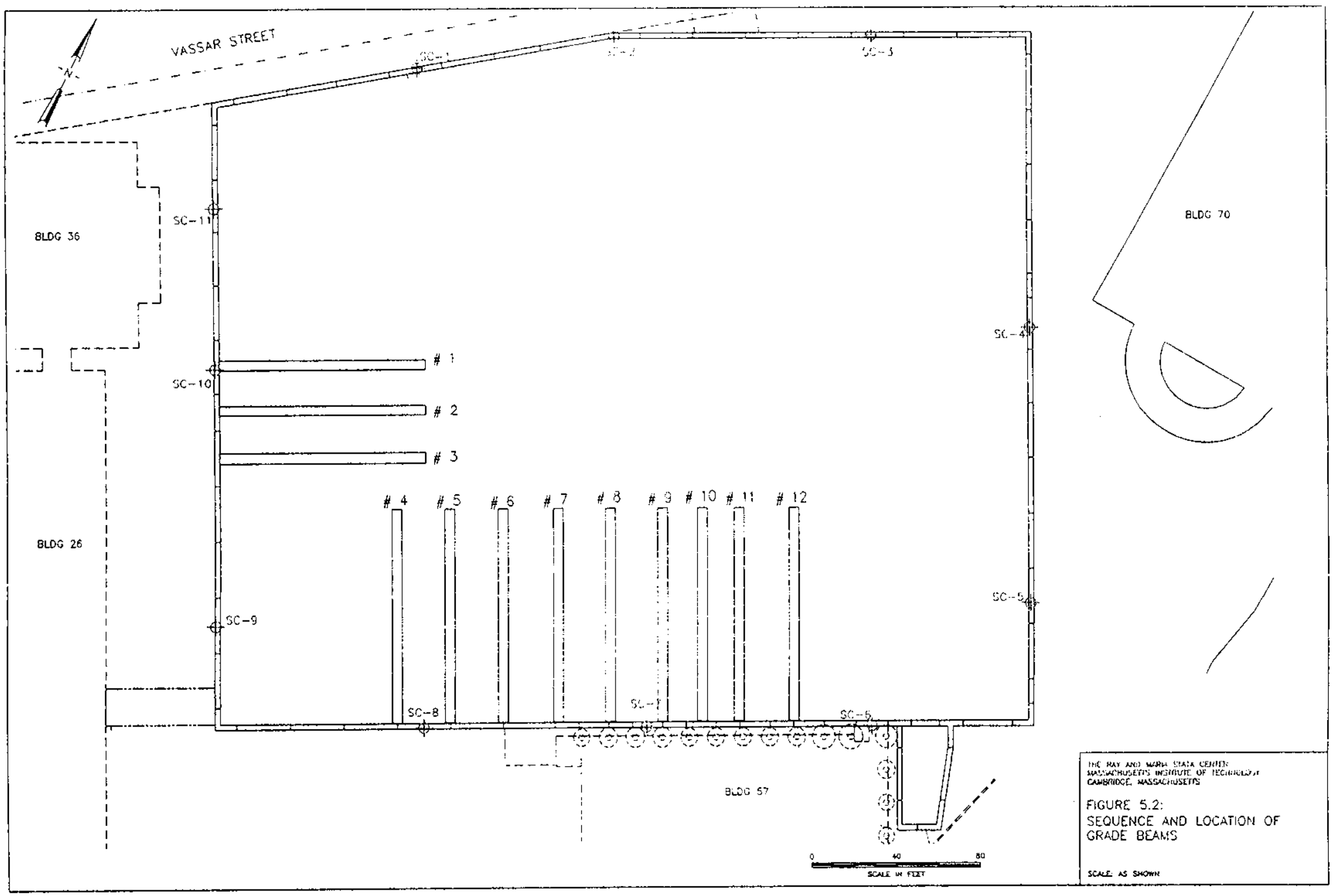


FIGURE 5.2:
SEQUENCE AND LOCATION OF GRADE BEAMS

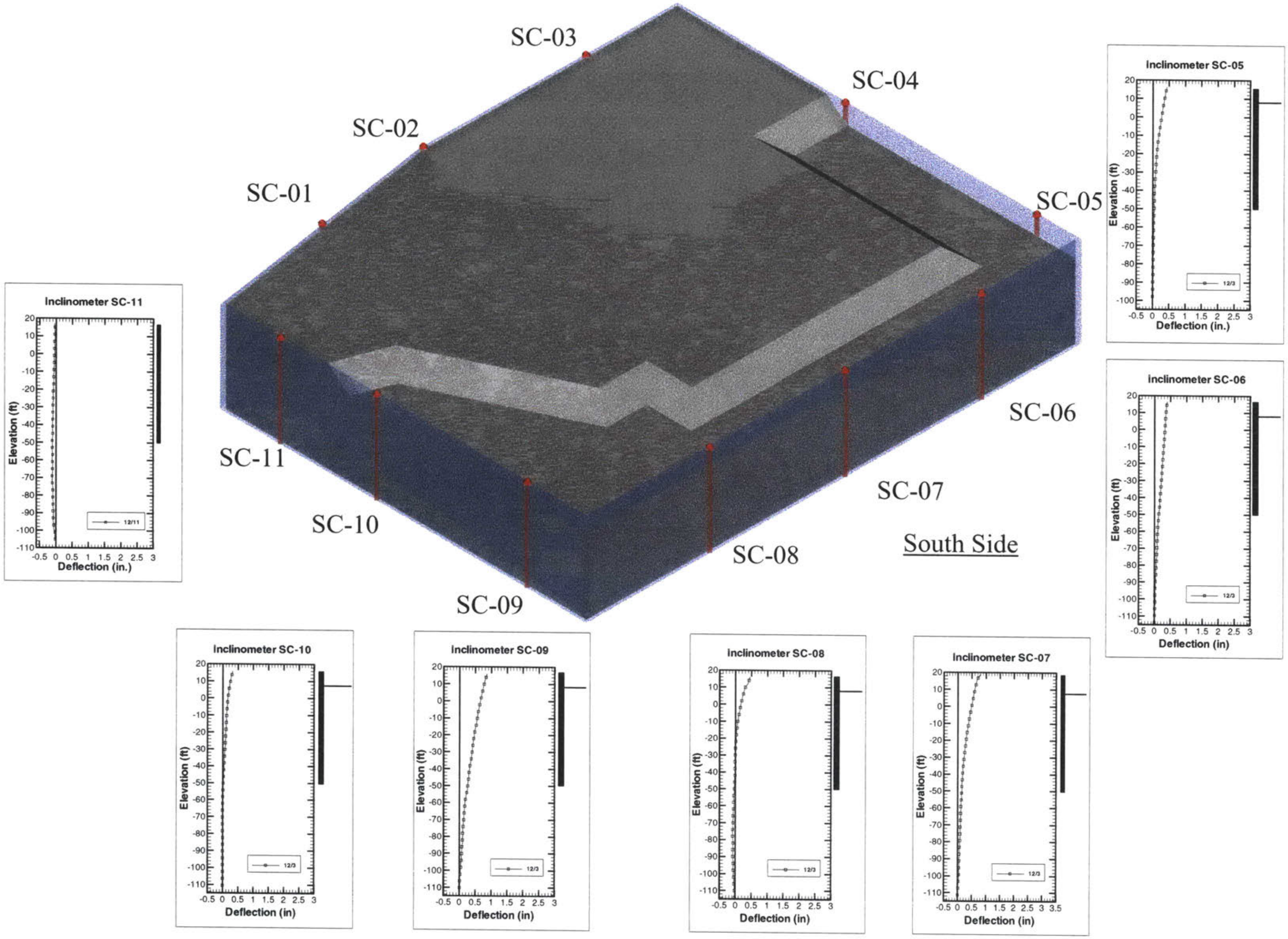


Figure 5.3: Excavation progress and wall deflections on December 5, 2001

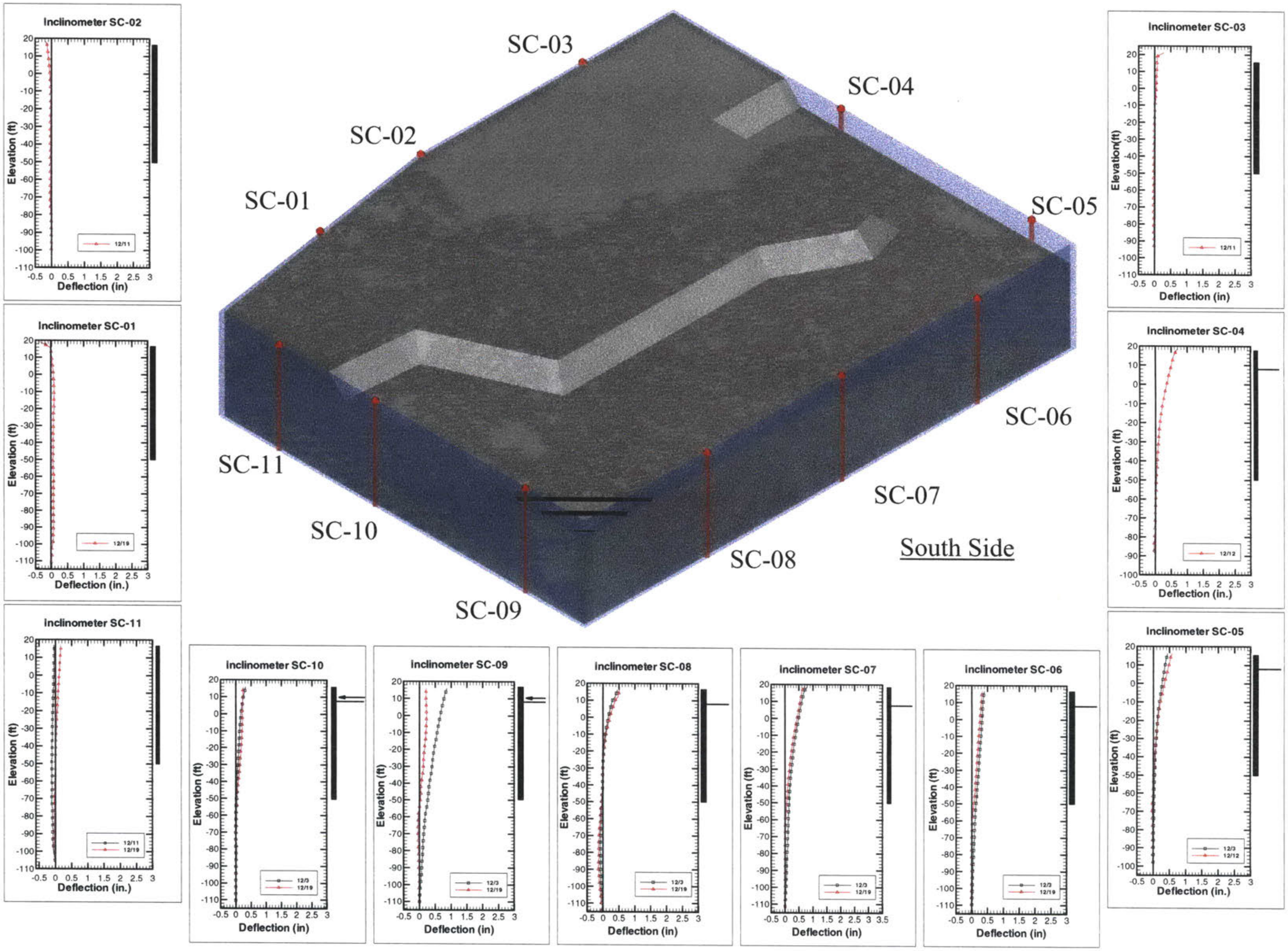


Figure 5.4: Excavation progress and wall deflections on December 19, 2001

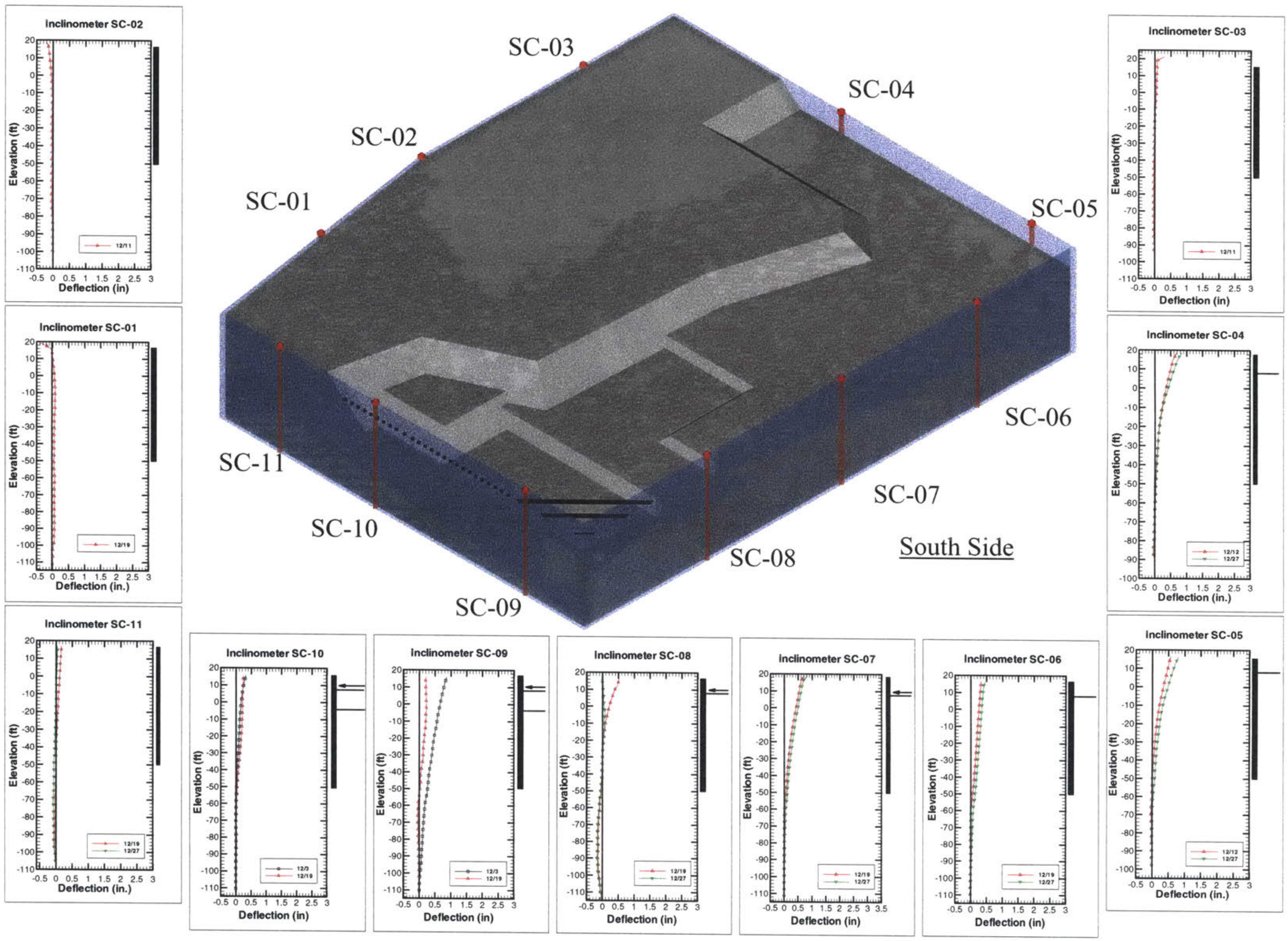


Figure 5.5: Excavation progress and wall deflections on December 27, 2001

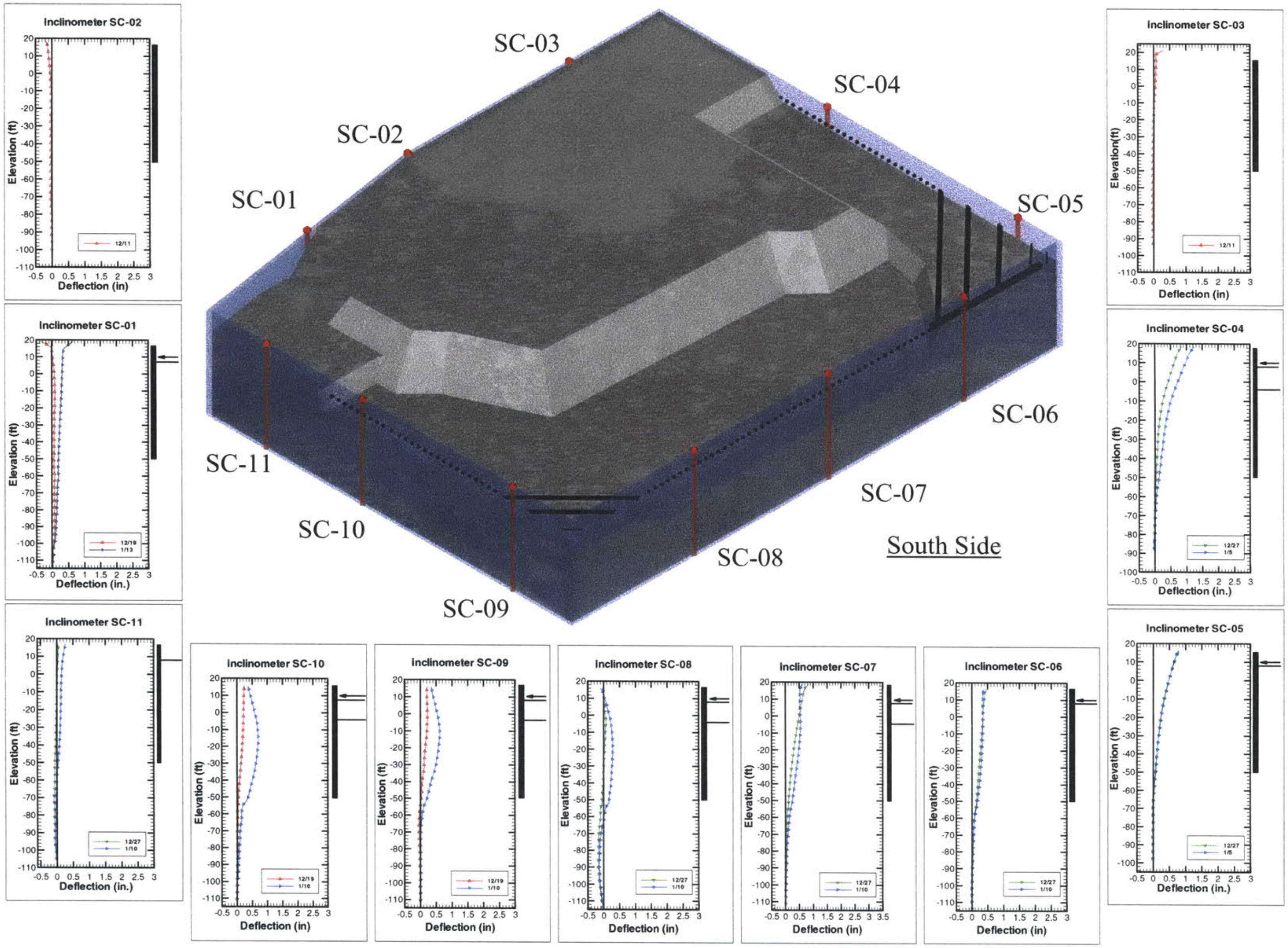


Figure 5.6: Excavation progress and wall deflections on January 11, 2001

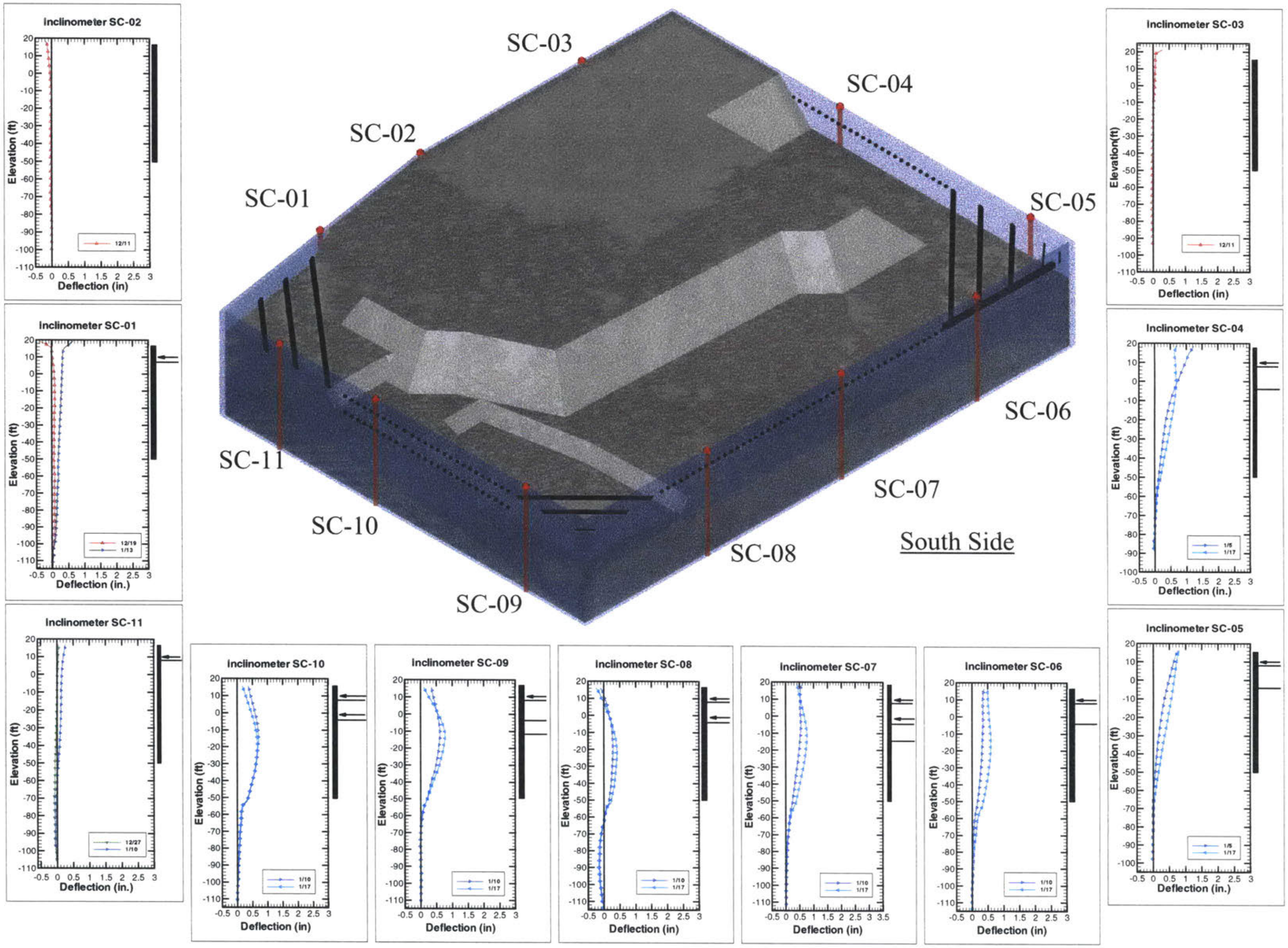


Figure 5.7: Excavation progress and wall deflections on January 17, 2001

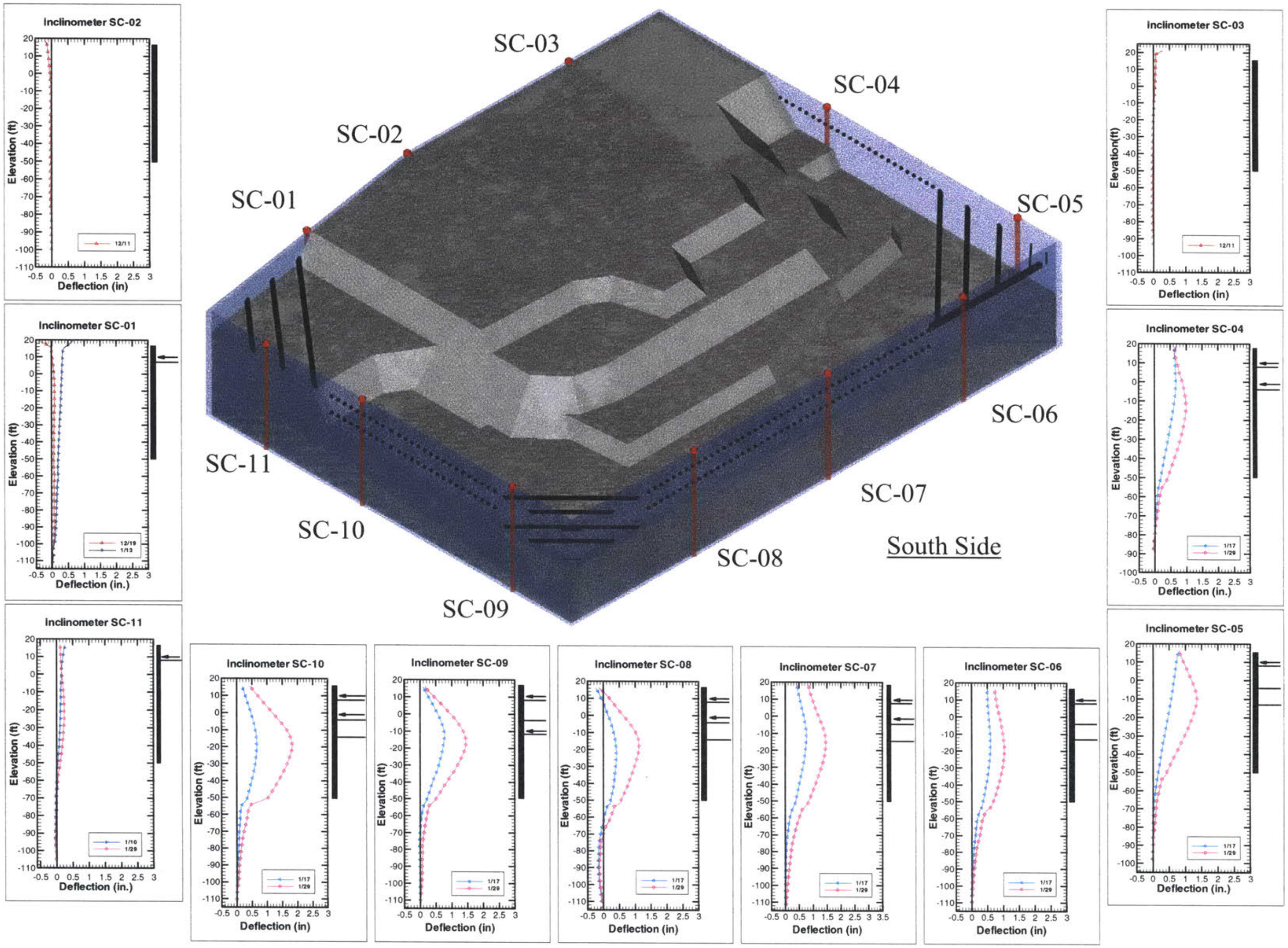


Figure 5.8: Excavation progress and wall deflections on January 29, 2001

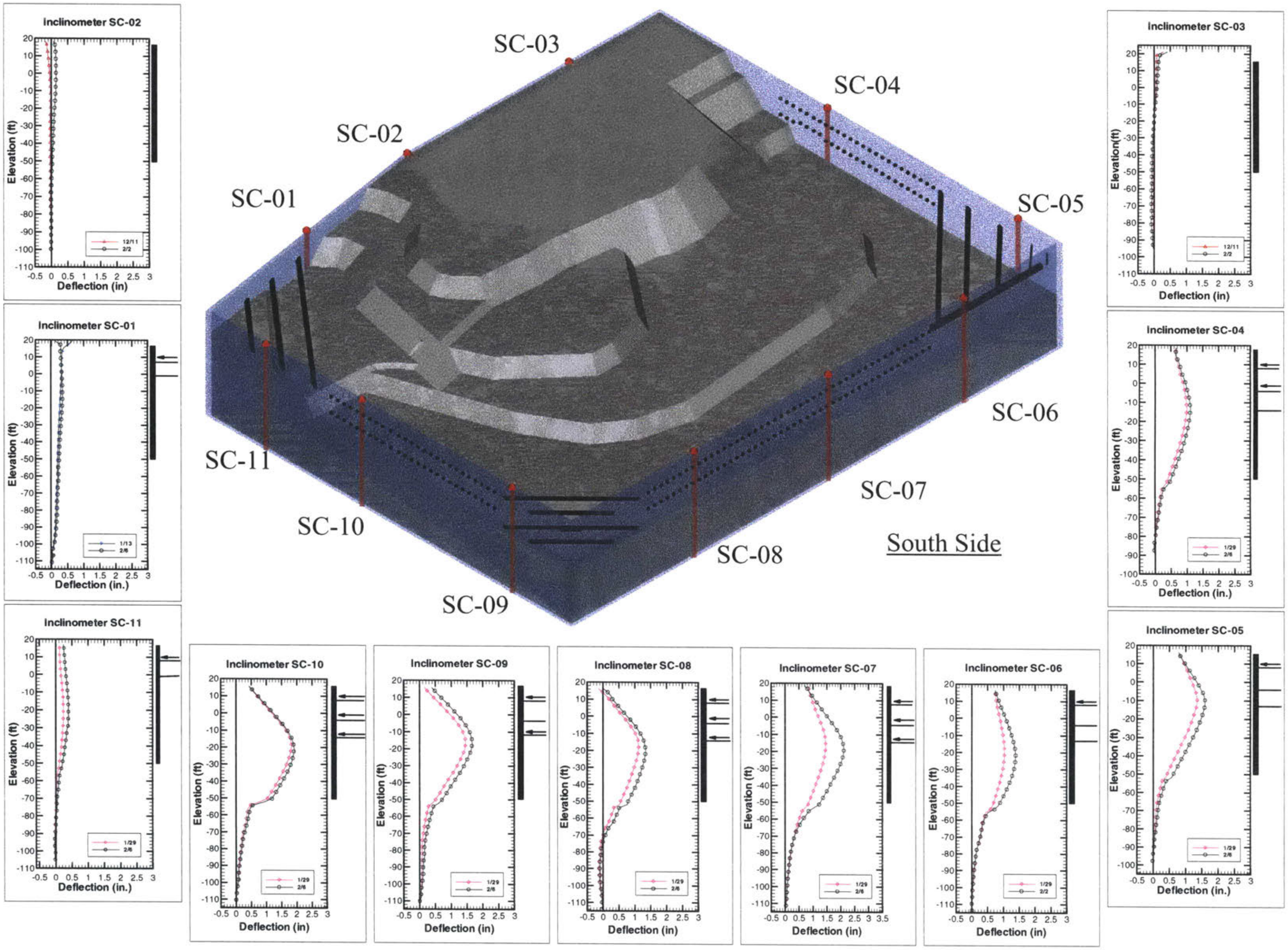


Figure 5.9: Excavation progress and wall deflections on February 6, 2001

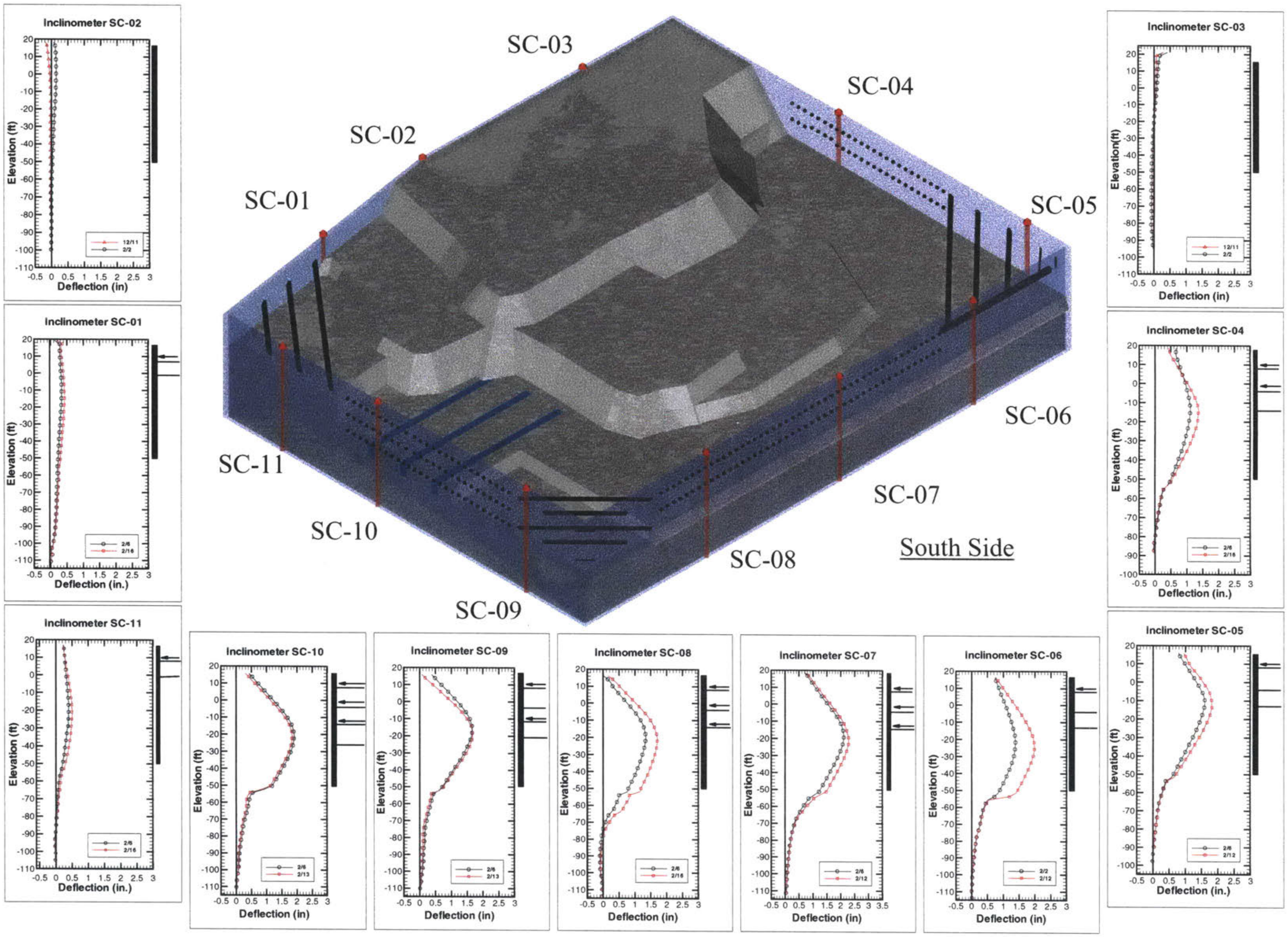


Figure 5.10: Excavation progress and wall deflections on February 13, 2001

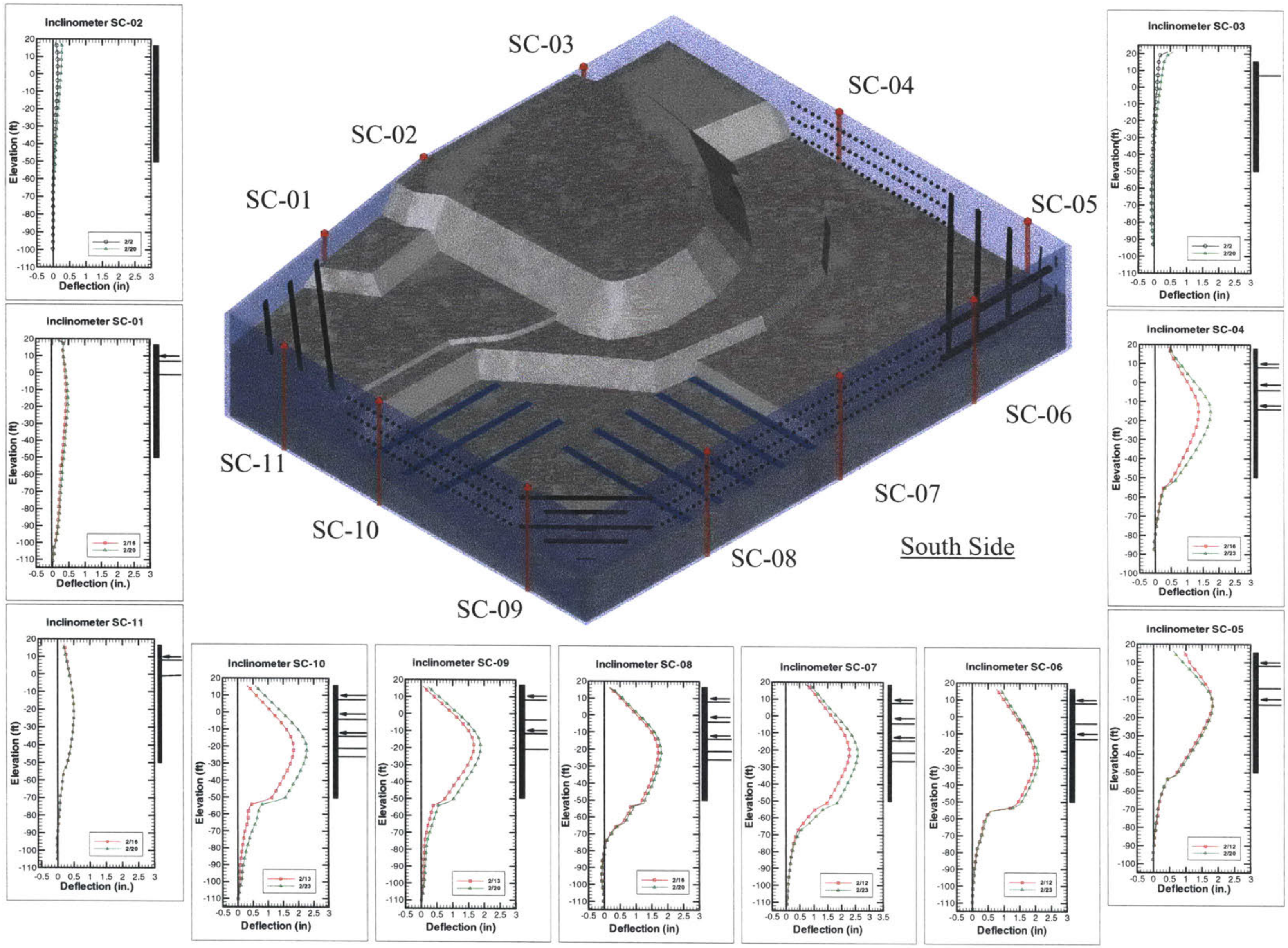


Figure 5.11: Excavation progress and wall deflections on February 23, 2001

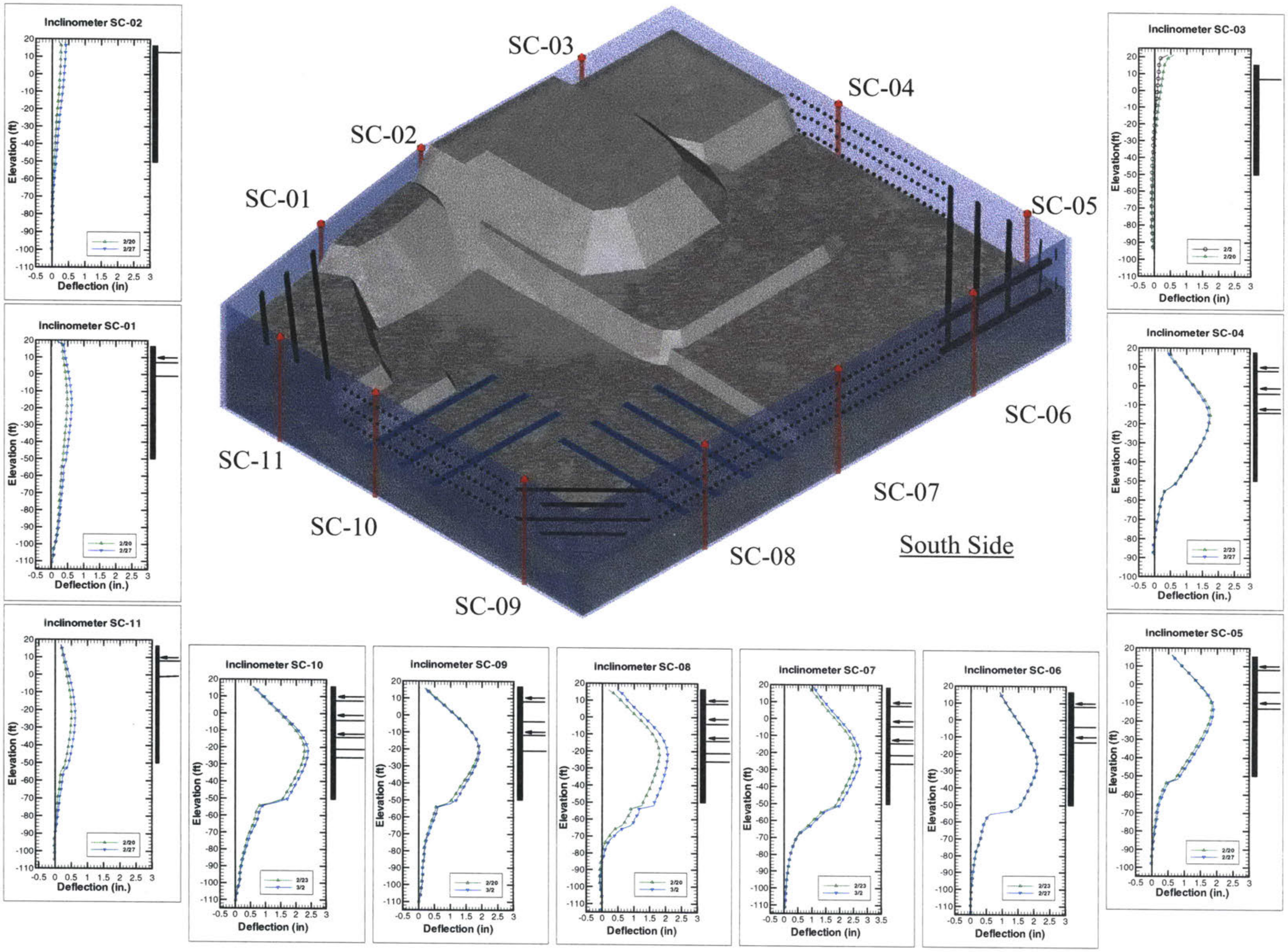


Figure 5.12: Excavation progress and wall deflections on March 2, 2001

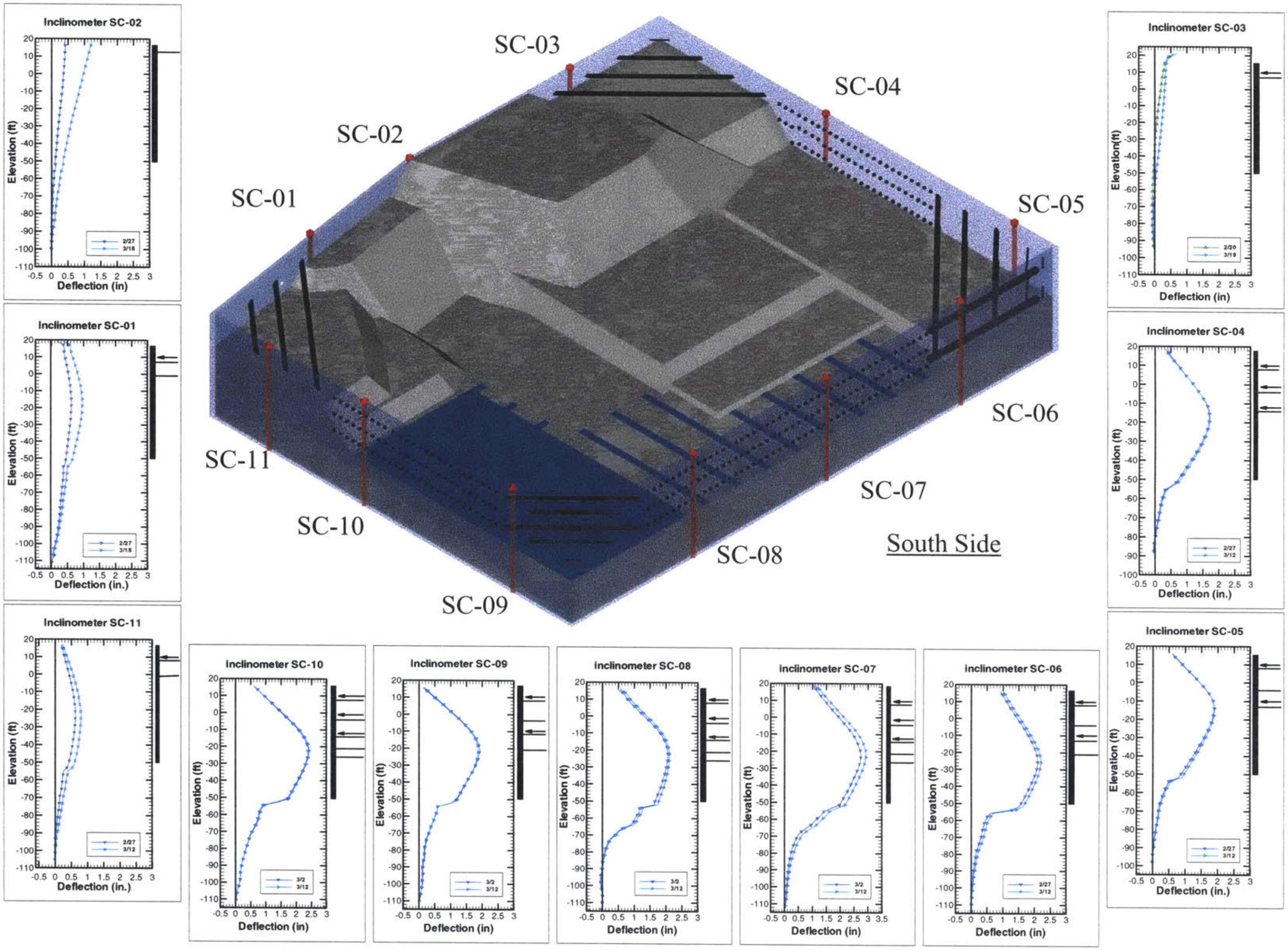


Figure 5.13: Excavation progress and wall deflections on March 14, 2001

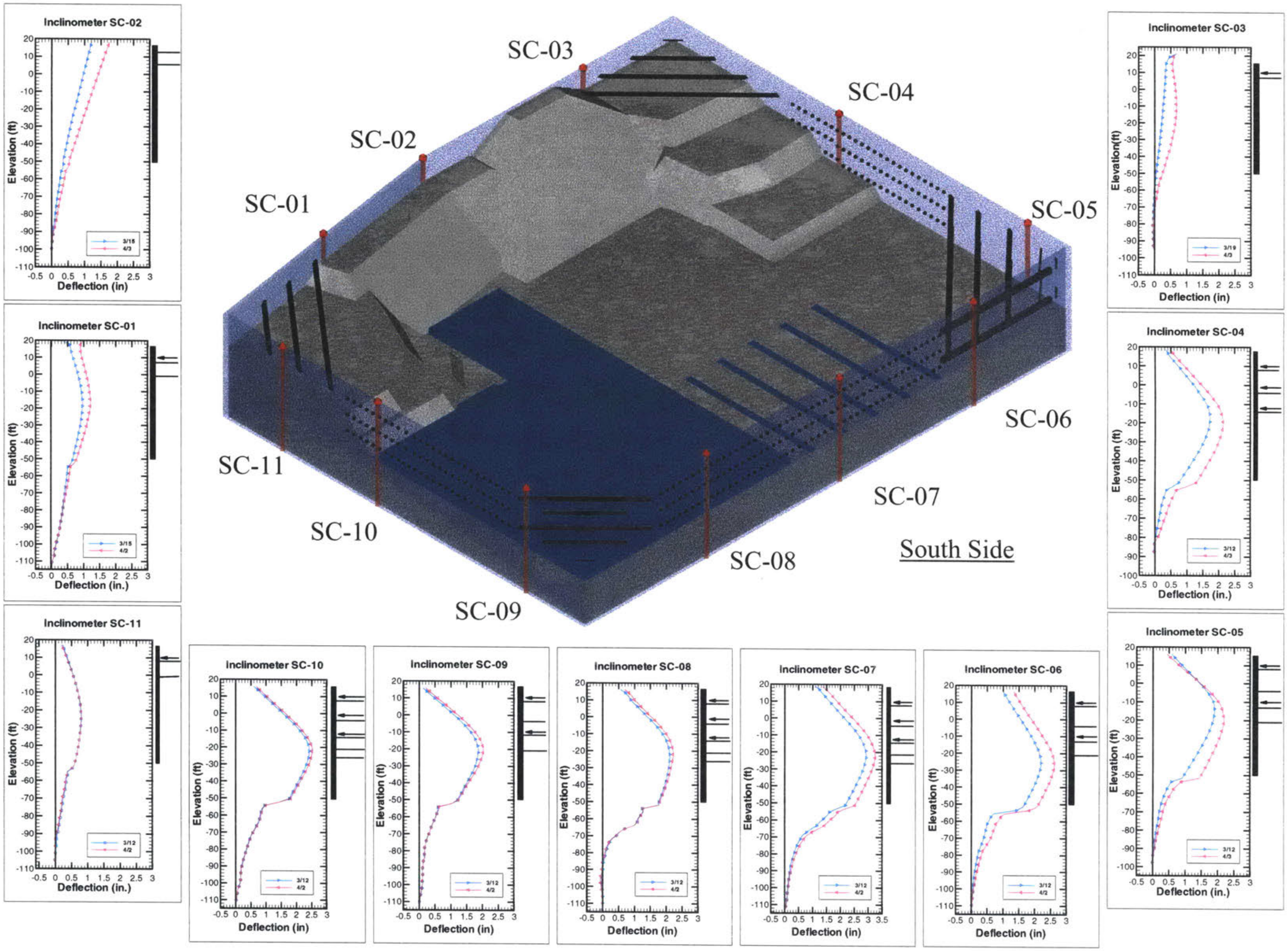


Figure 5.14: Excavation progress and wall deflections on April 3, 2001

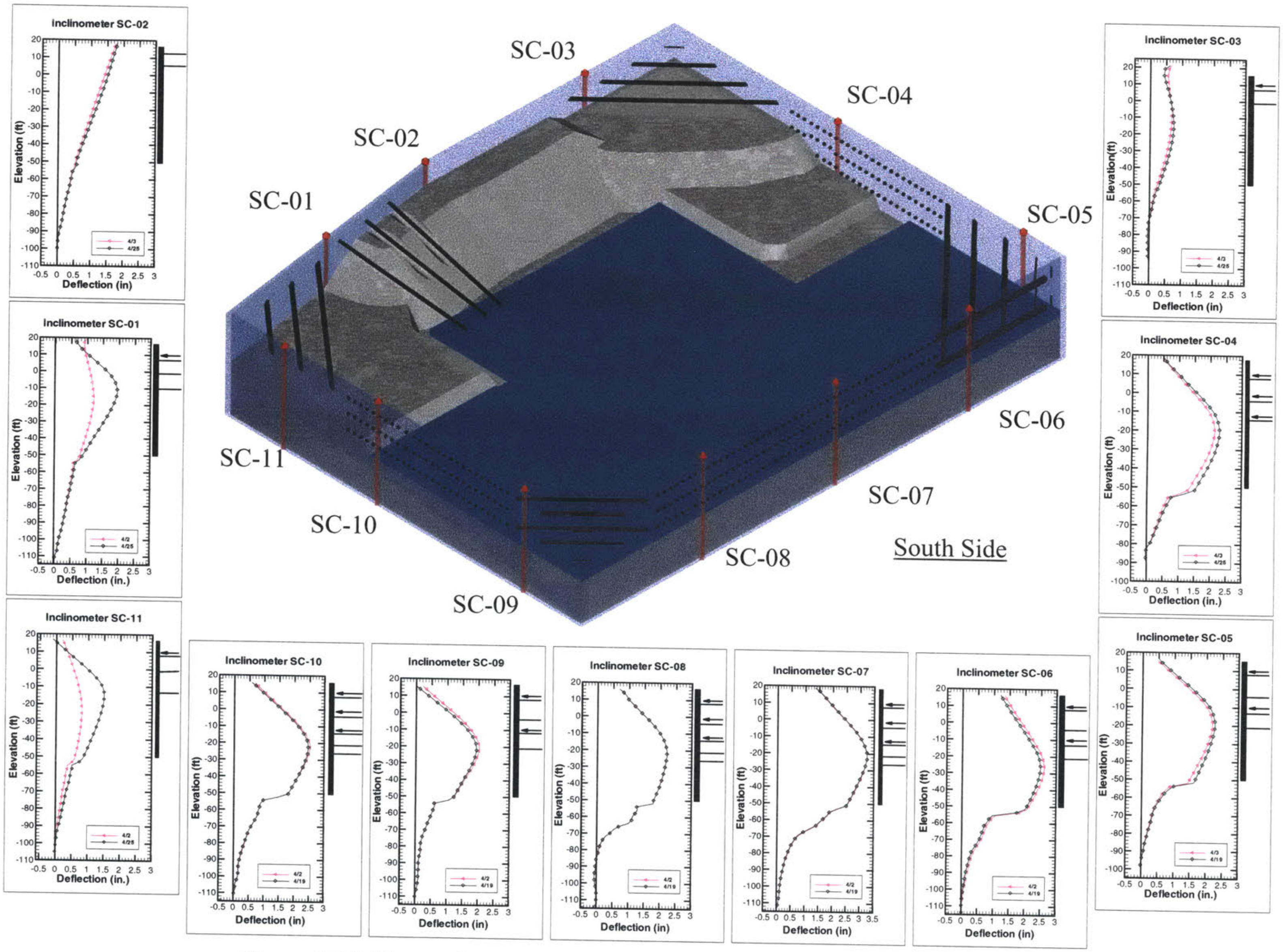


Figure 5.15: Excavation progress and wall deflections on April 25, 2001

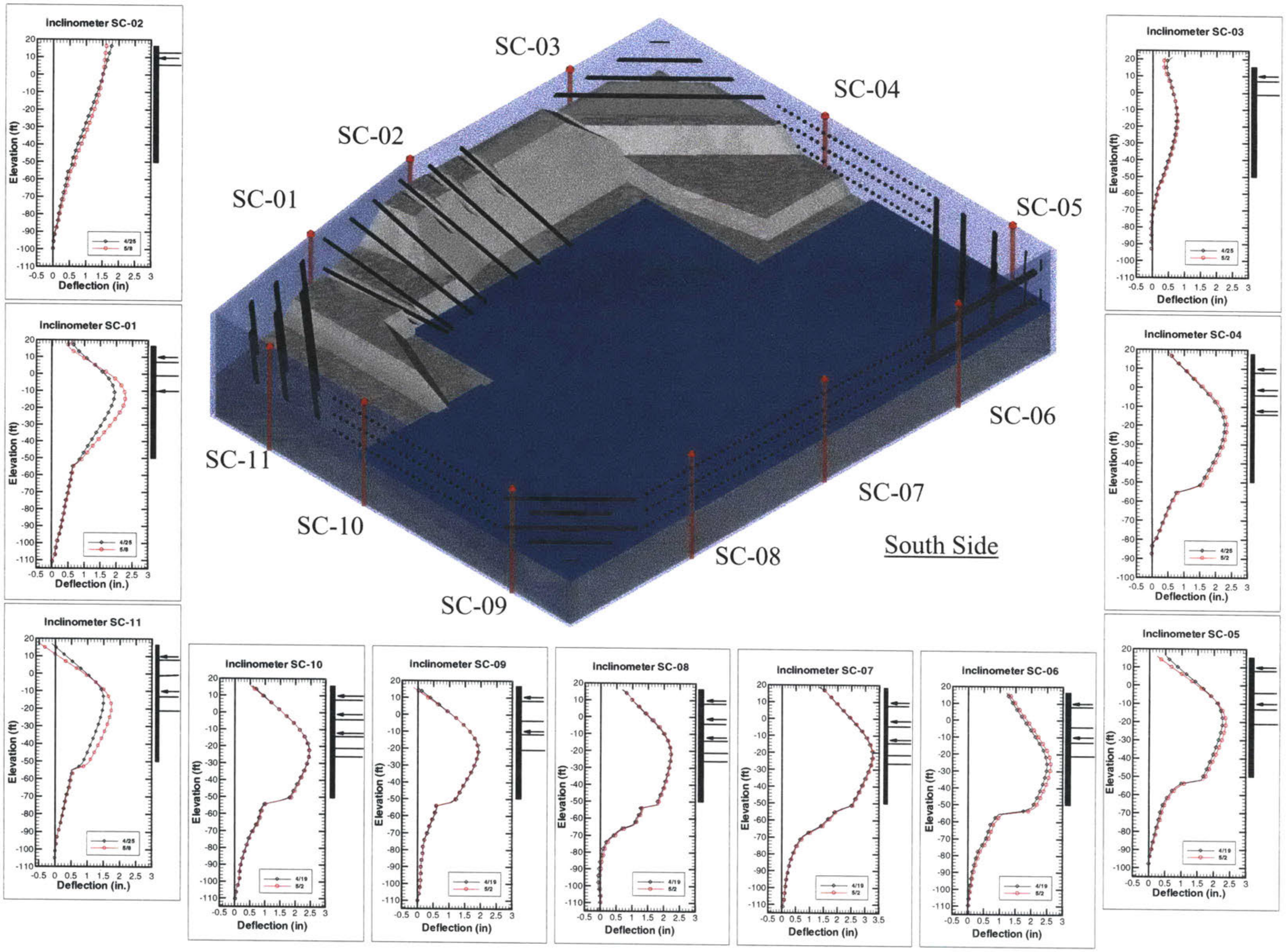


Figure 5.16: Excavation progress and wall deflections on May 8, 2001

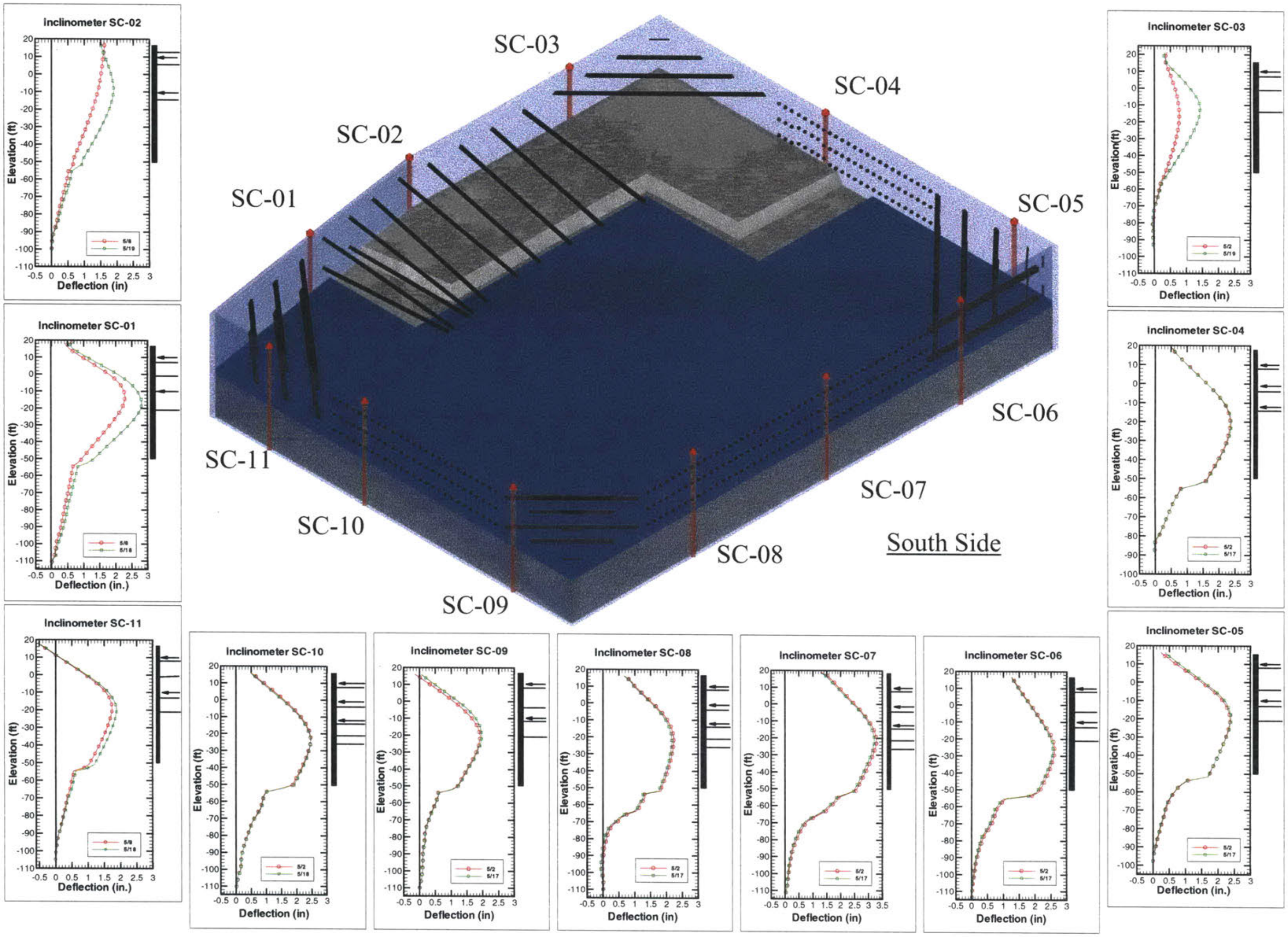


Figure 5.17: Excavation progress and wall deflections on May 17, 2001

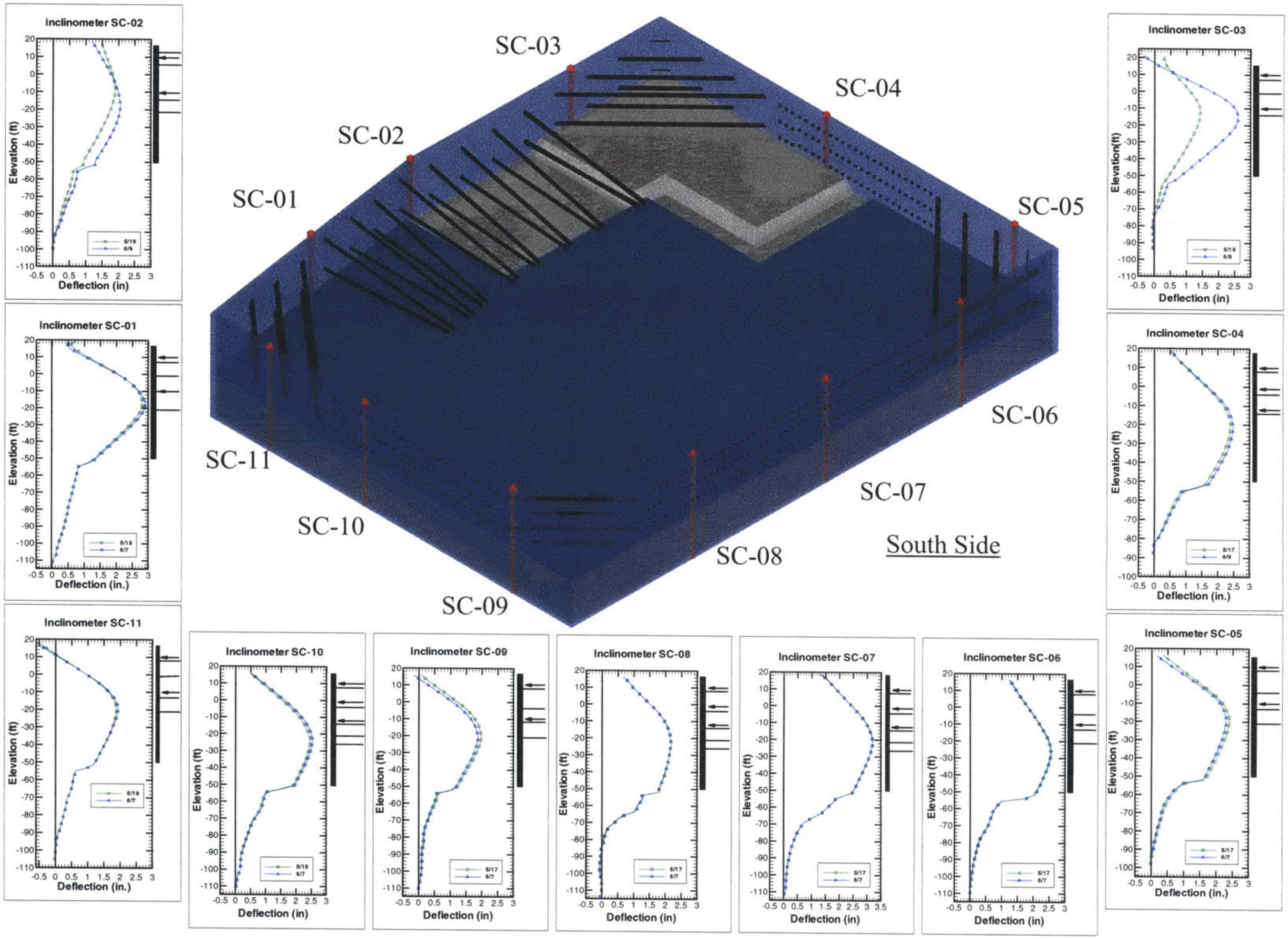


Figure 5.18: Excavation progress and wall deflections on June 7, 2001

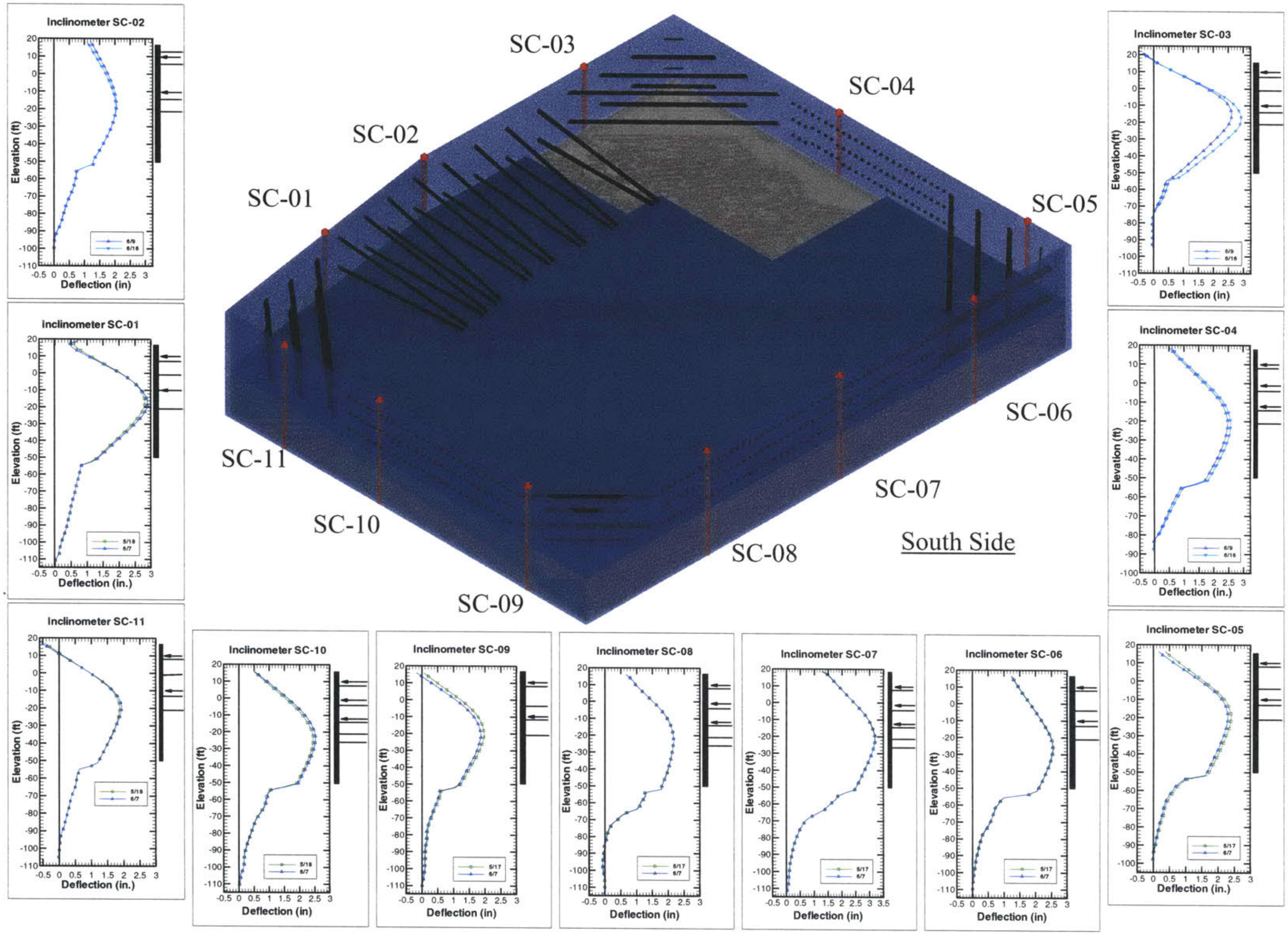


Figure 5.19: Excavation progress and wall deflections on June 16, 2001

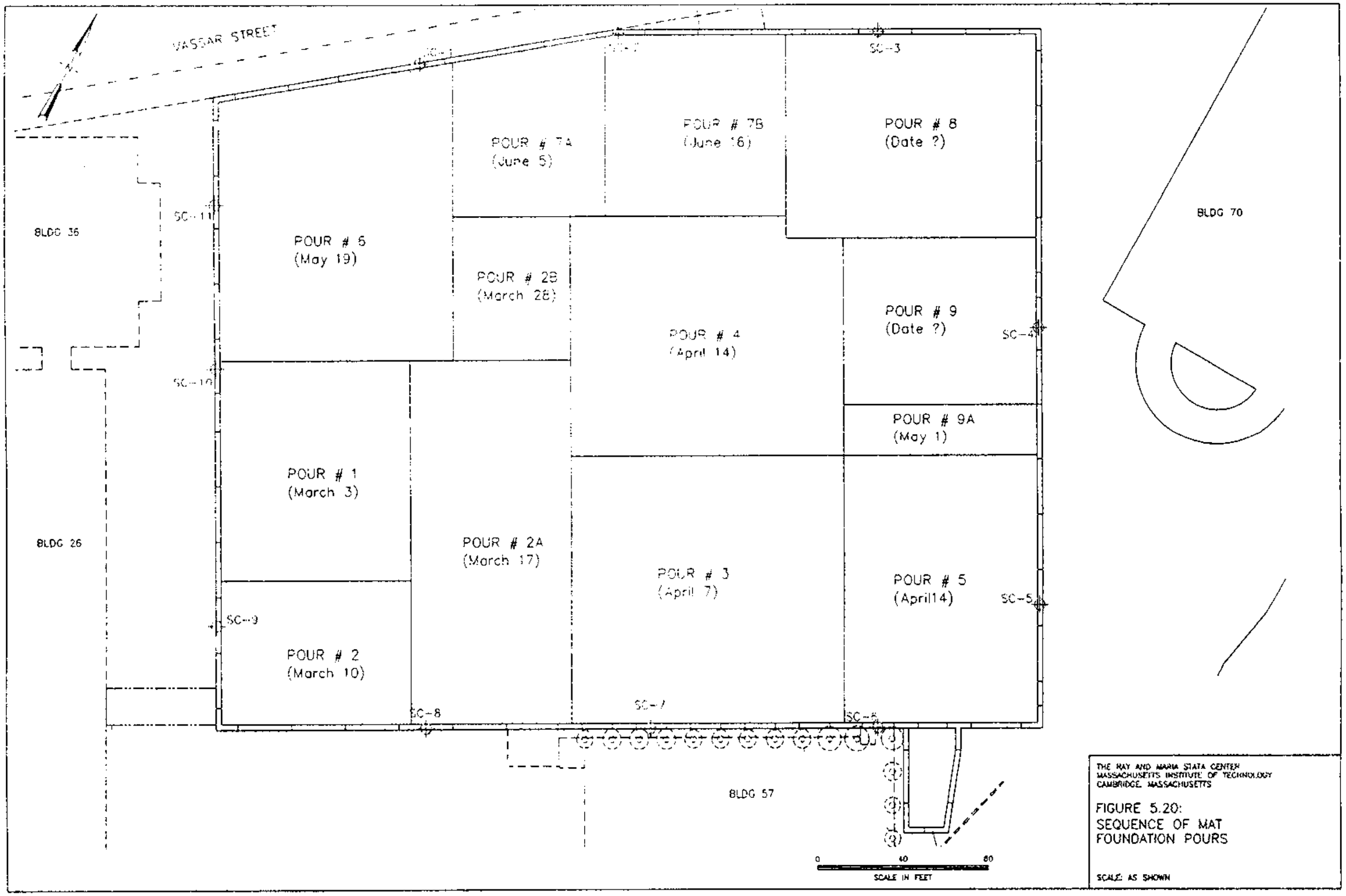


FIGURE 5.20:
SEQUENCE OF MAT
FOUNDATION POURS

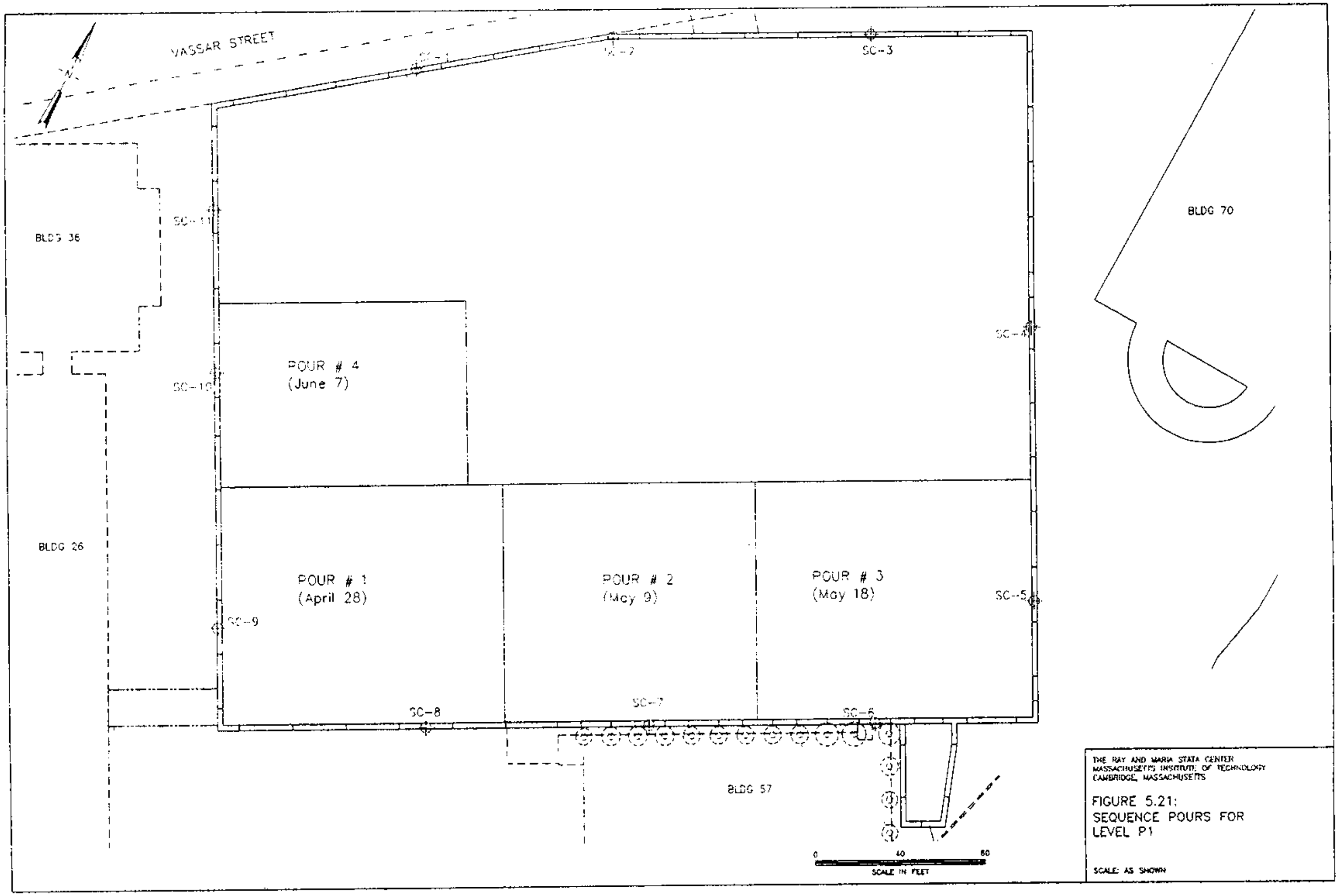
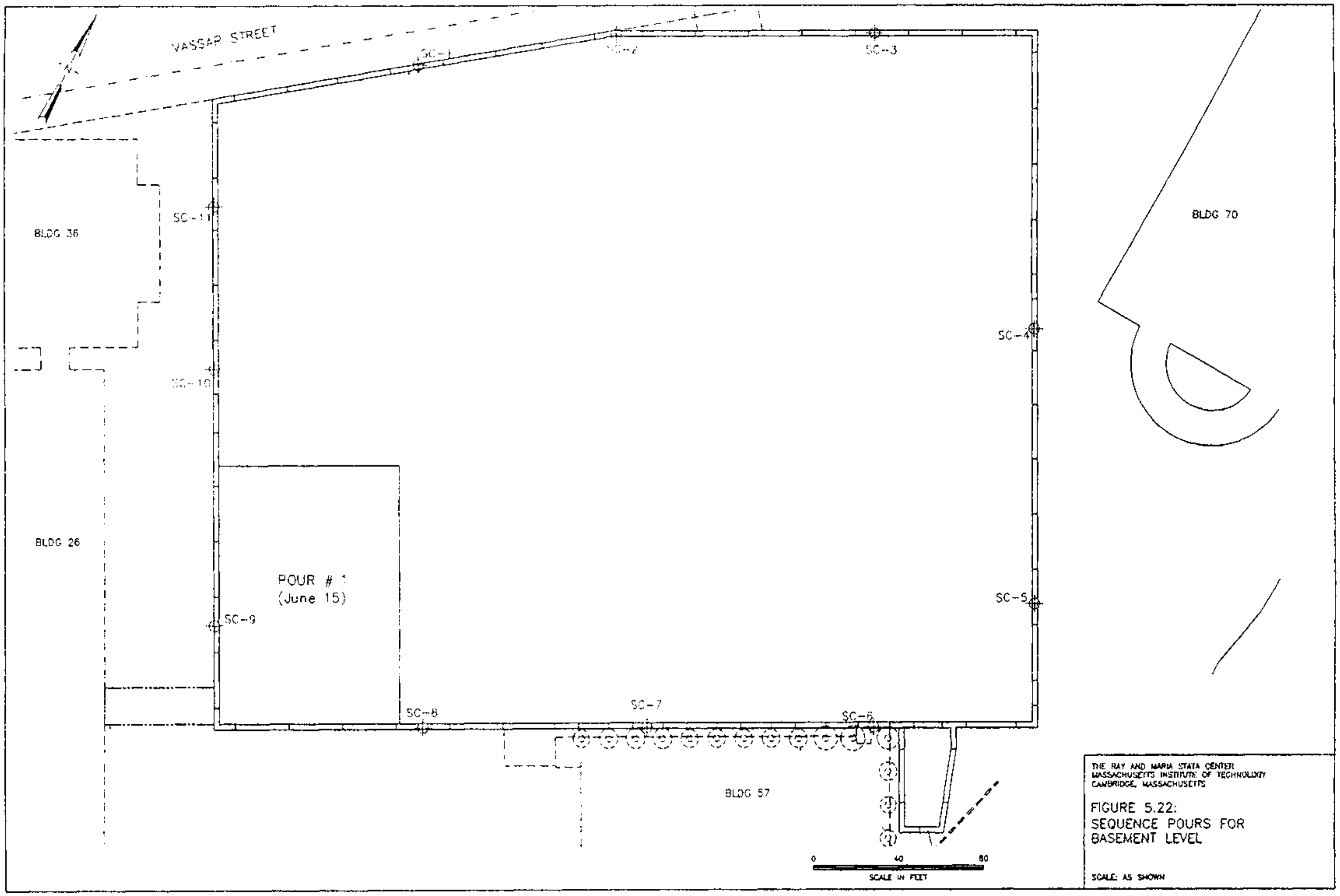


FIGURE 5.21: SEQUENCE POURS FOR LEVEL P1



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CAMBRIDGE, MASSACHUSETTS

FIGURE 5.22:
SEQUENCE POURS FOR BASEMENT LEVEL

SCALE: AS SHOWN

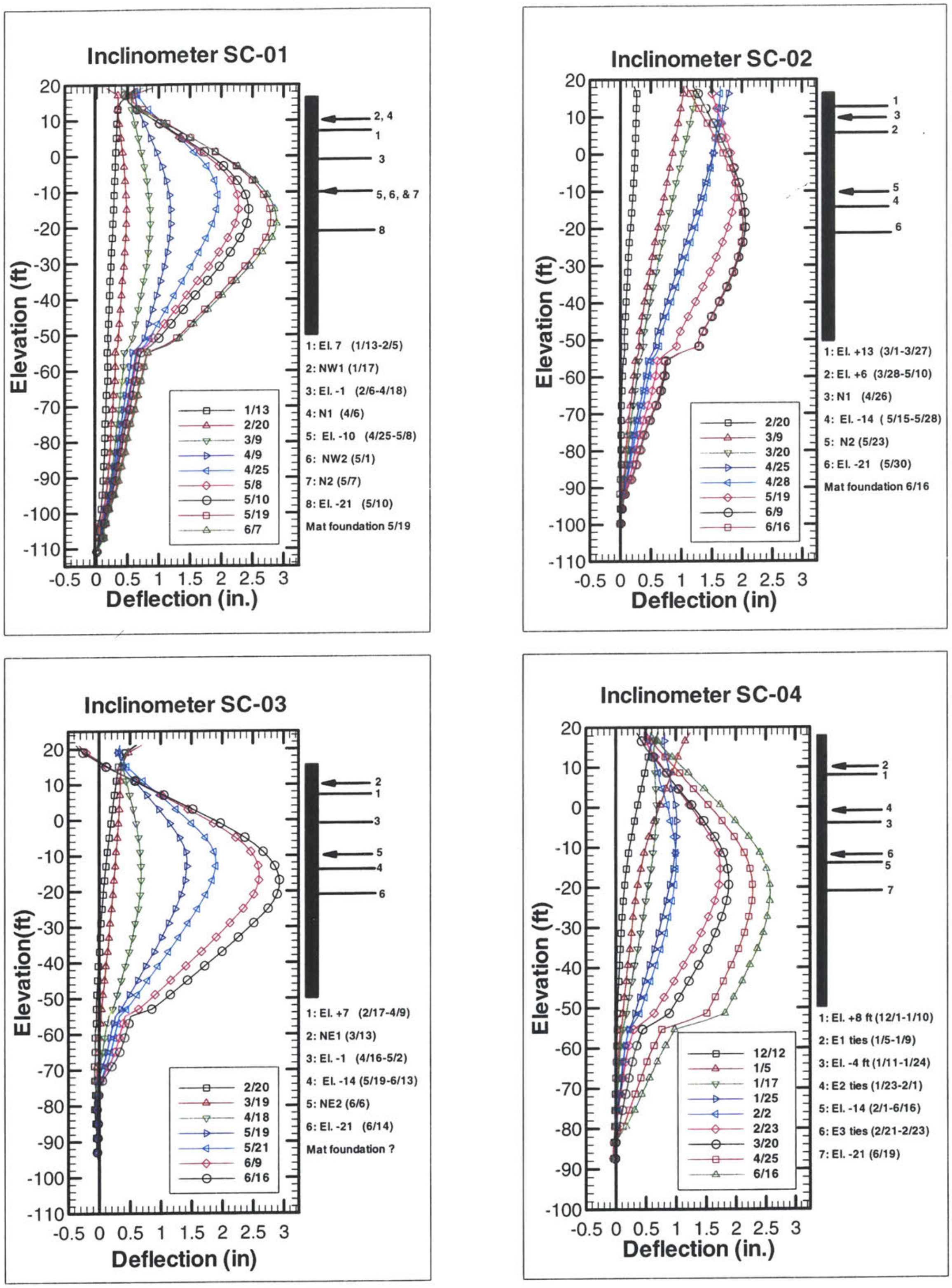


Figure 5.23: Summary of movements recorded in inclinometers SC-01 through SC-04

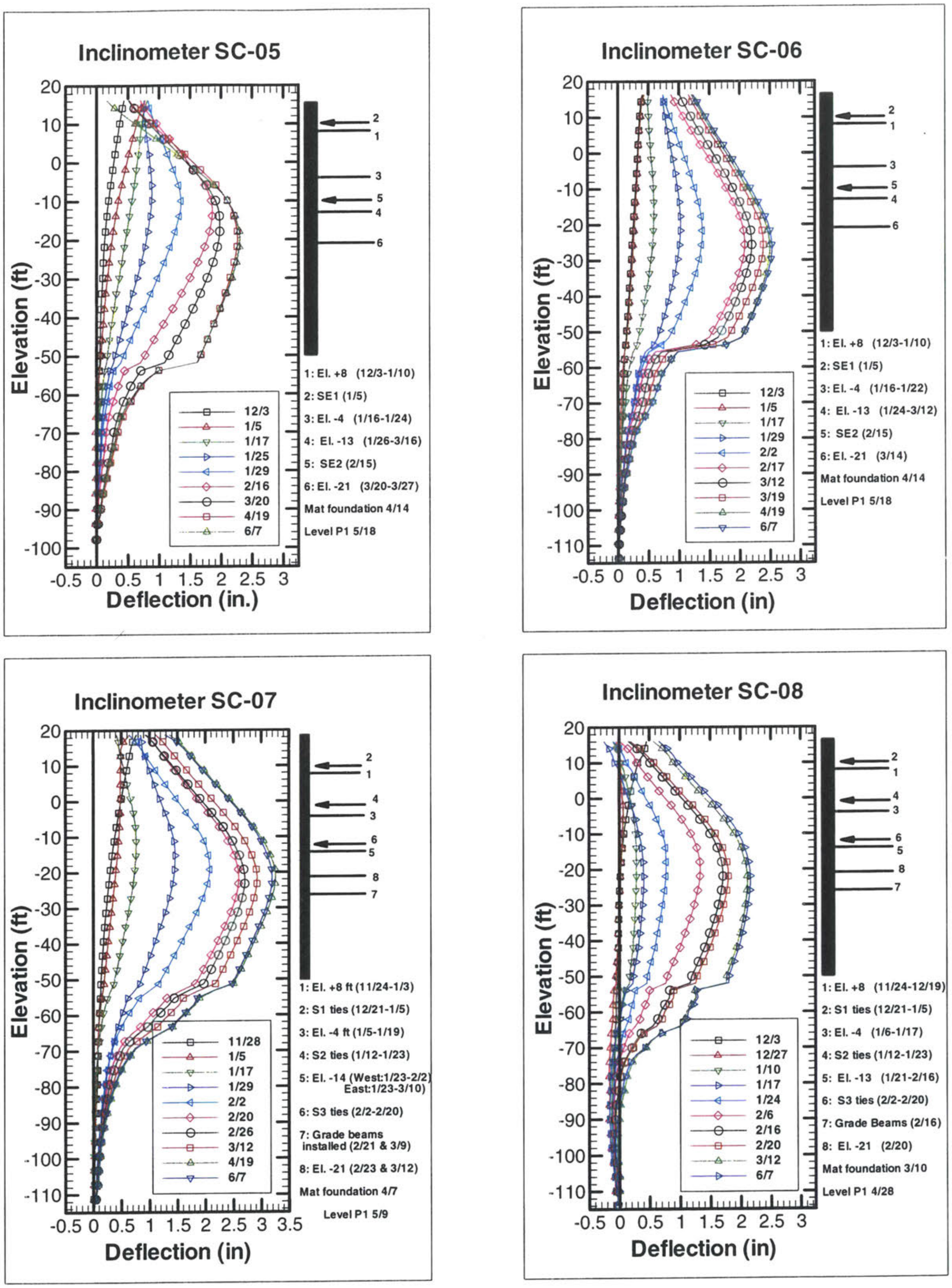
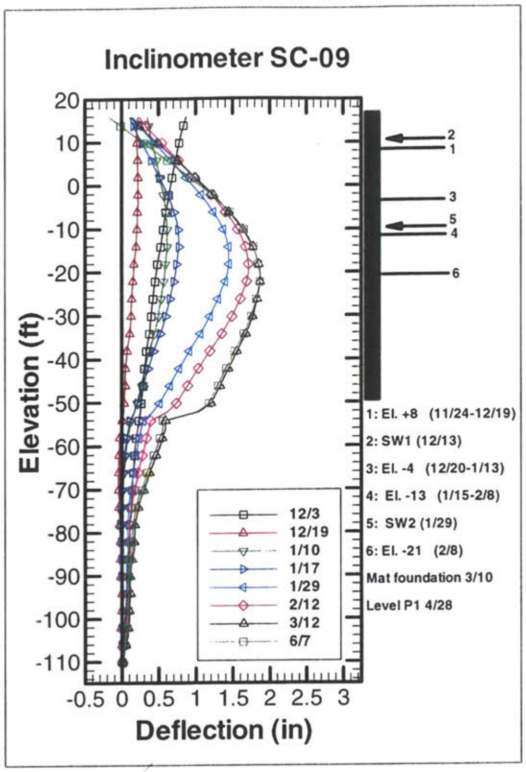
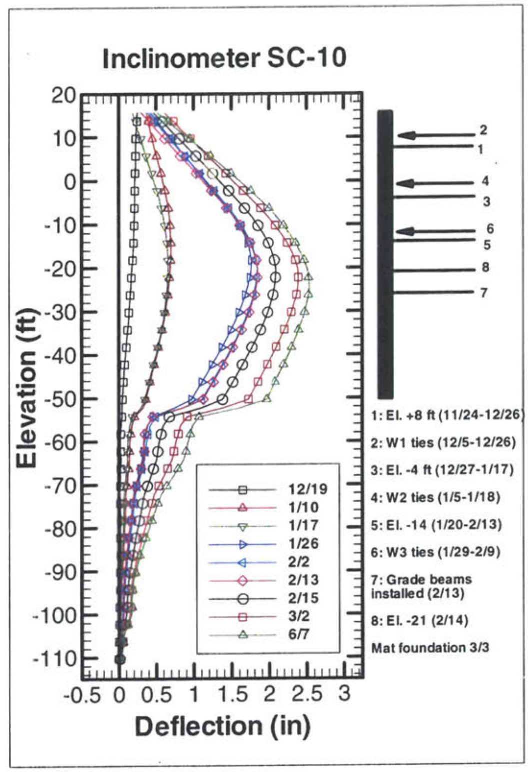


Figure 5.24: Summary of movements recorded in inclinometers SC-05 through SC-08





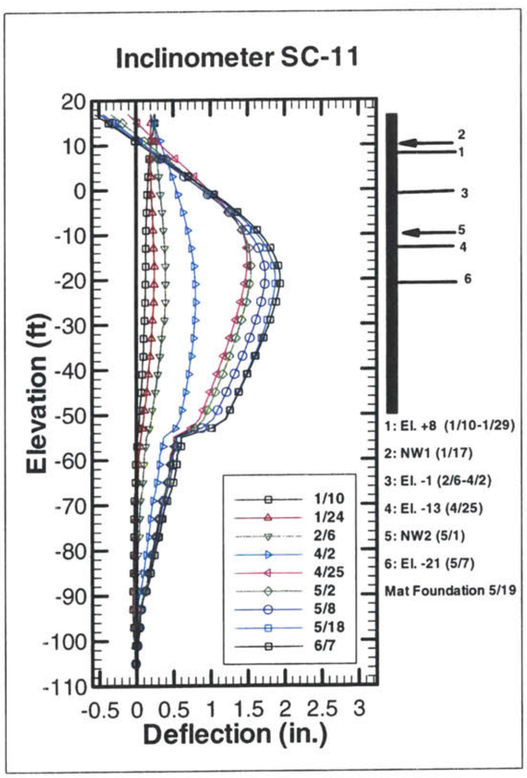


Figure 5.25: Summary of movements recorded in inclinometers SC-09 through SC-11

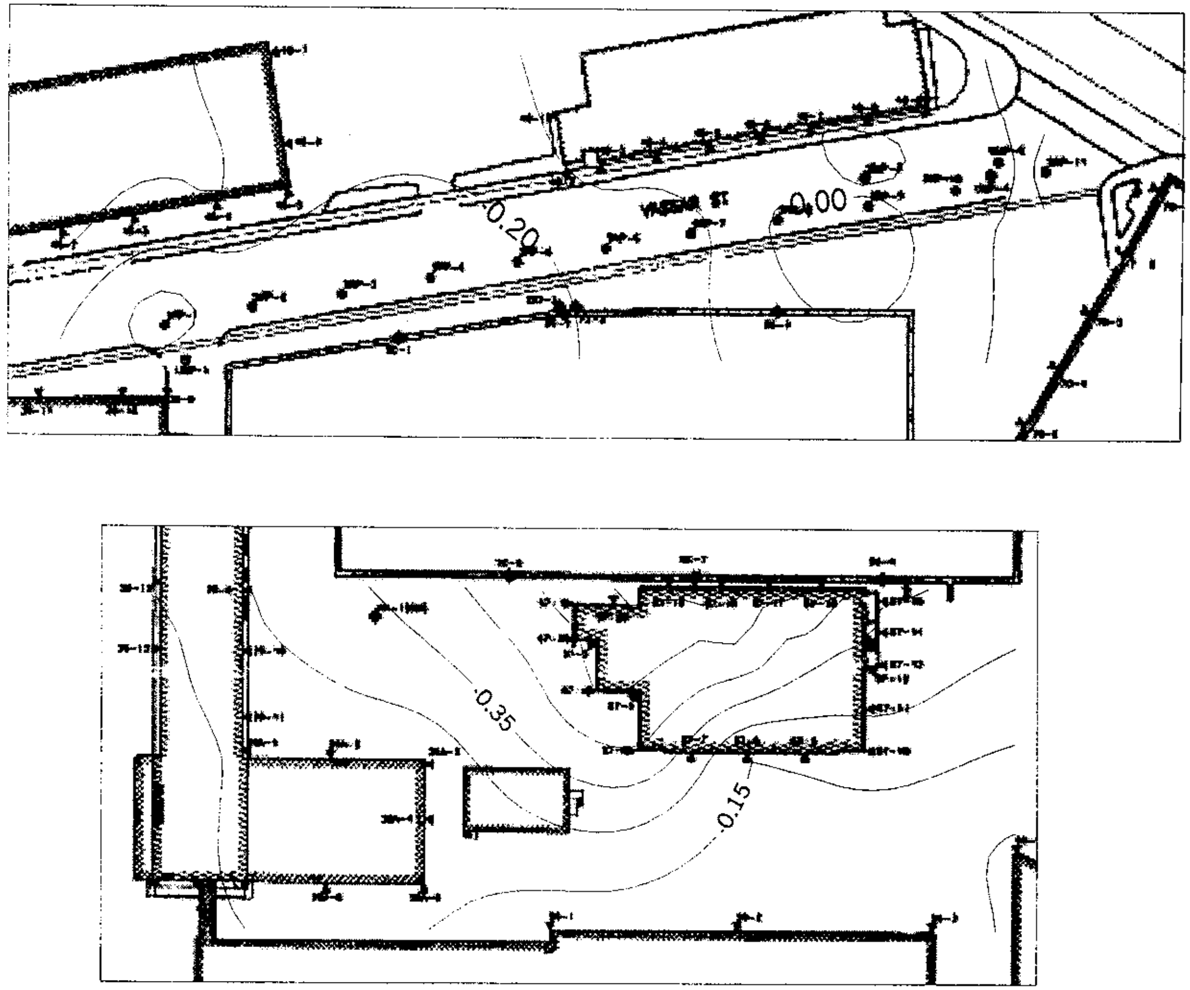


FIGURE 5.26: Settlement contours for the North and South Sides of the excavation for January 13, 2001.

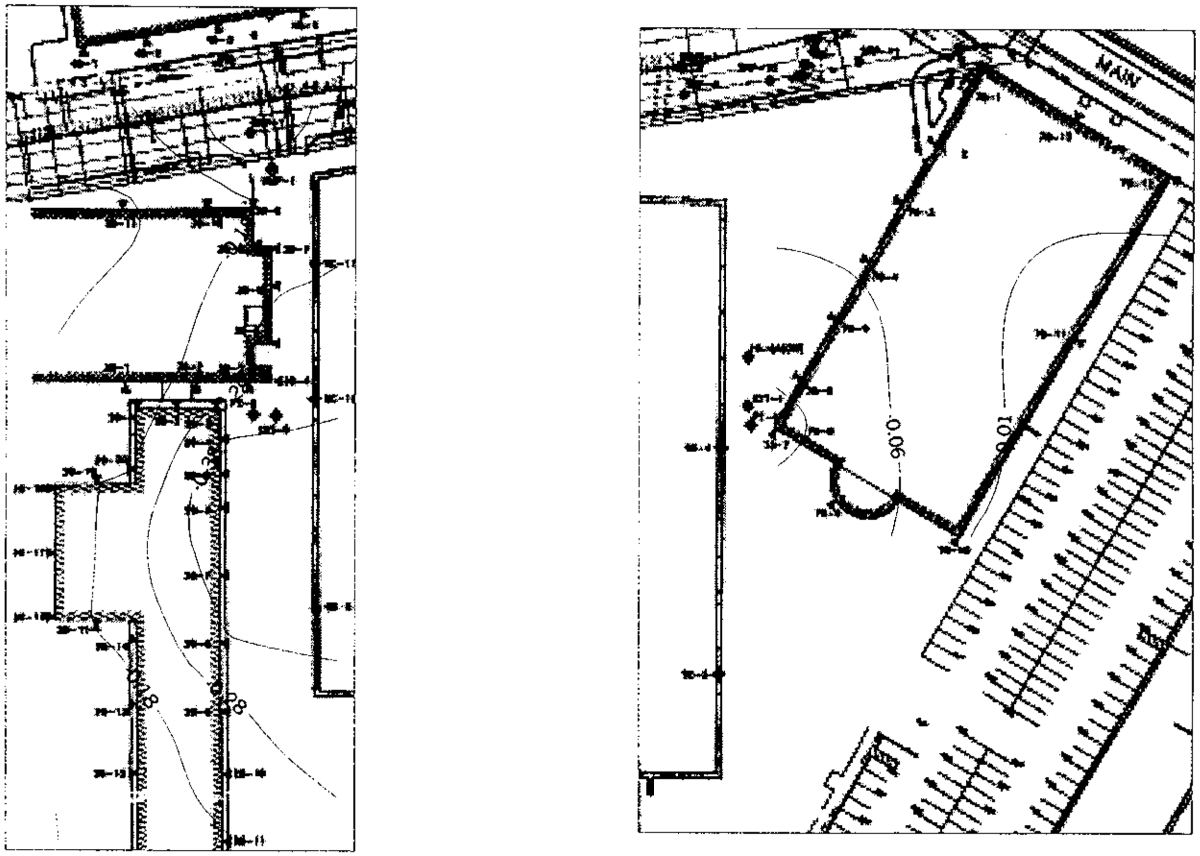


FIGURE 5.27: Settlement contours for the West and East Sides of the excavation for January 13, 2001.

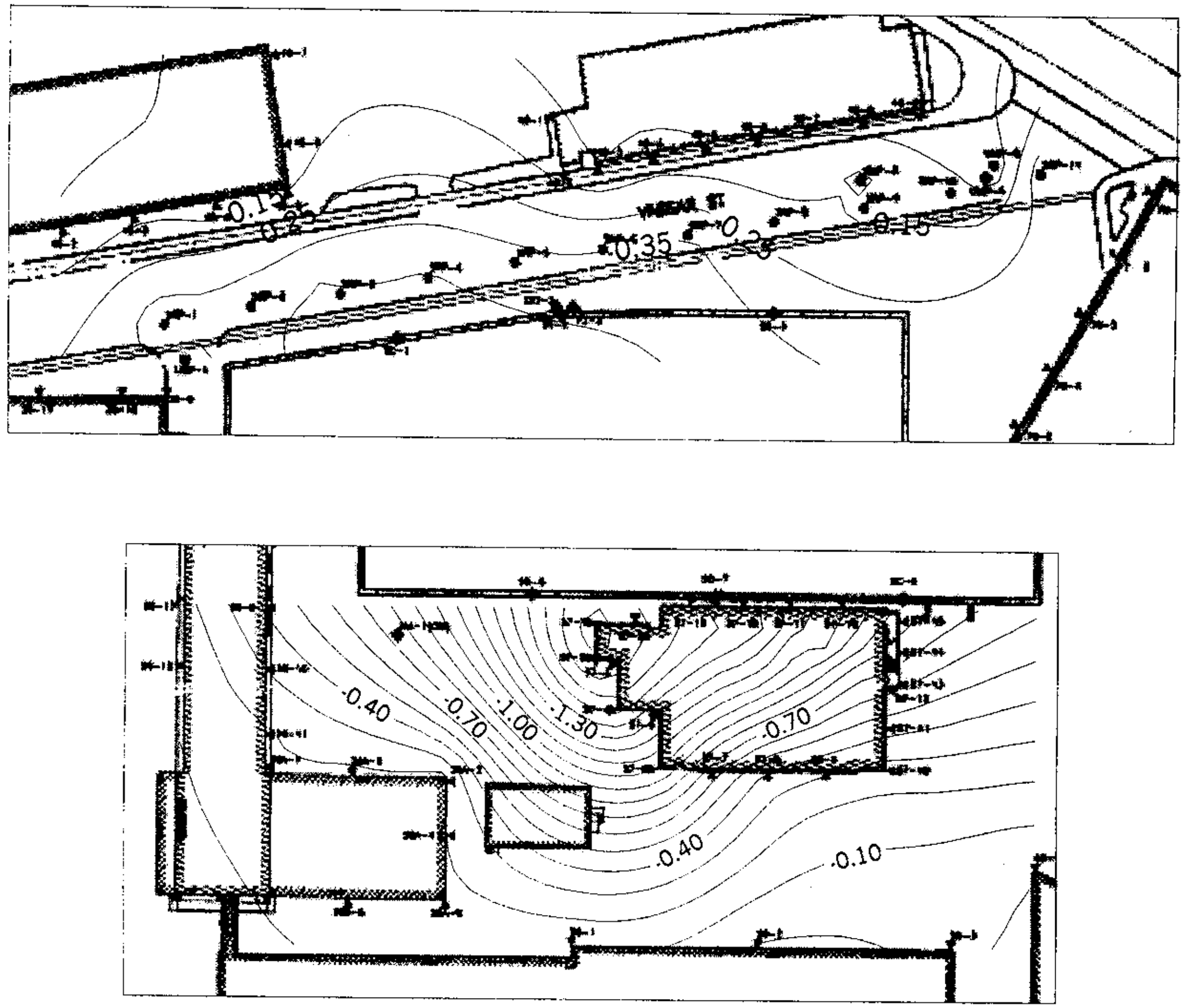


FIGURE 5.28: Settlement contours for the North and South Sides of the excavation for February 20, 2001.

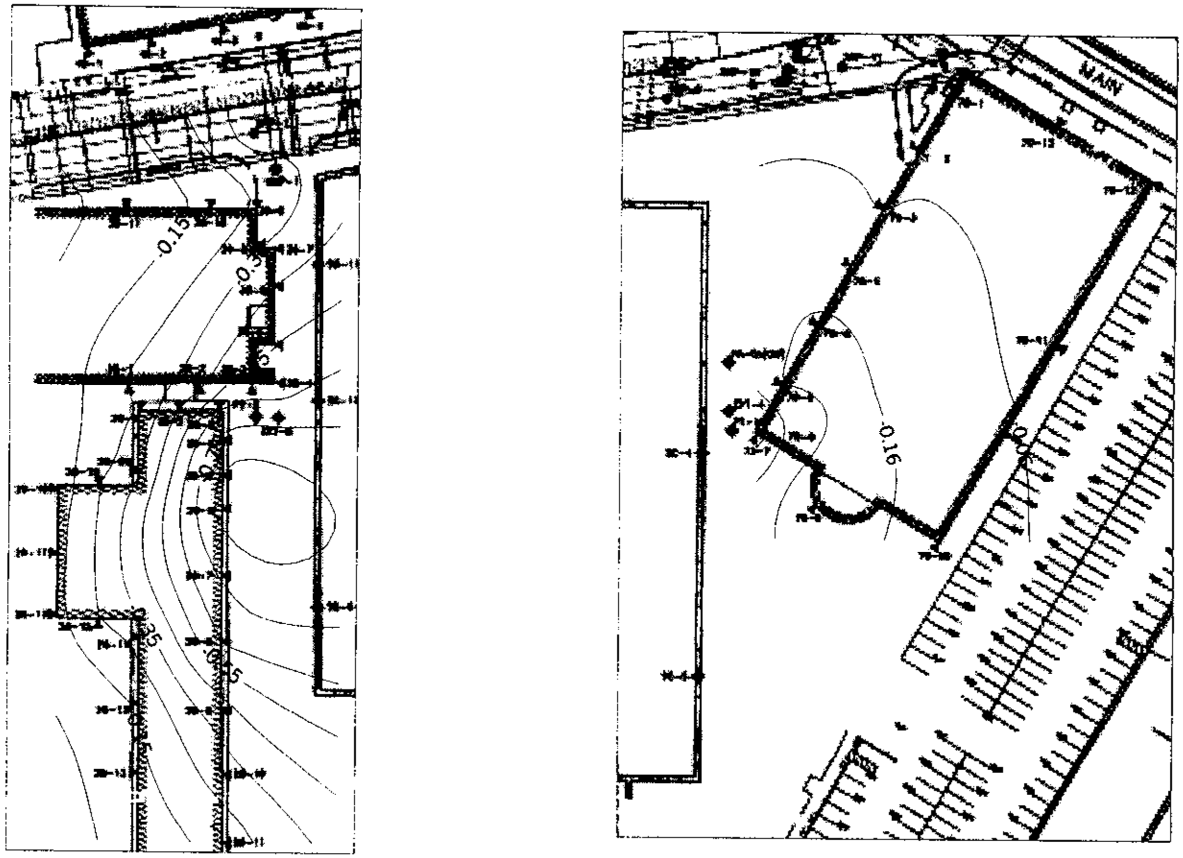


FIGURE 5.29: Settlement contours for the West and East Sides of the excavation for February 20, 2001.

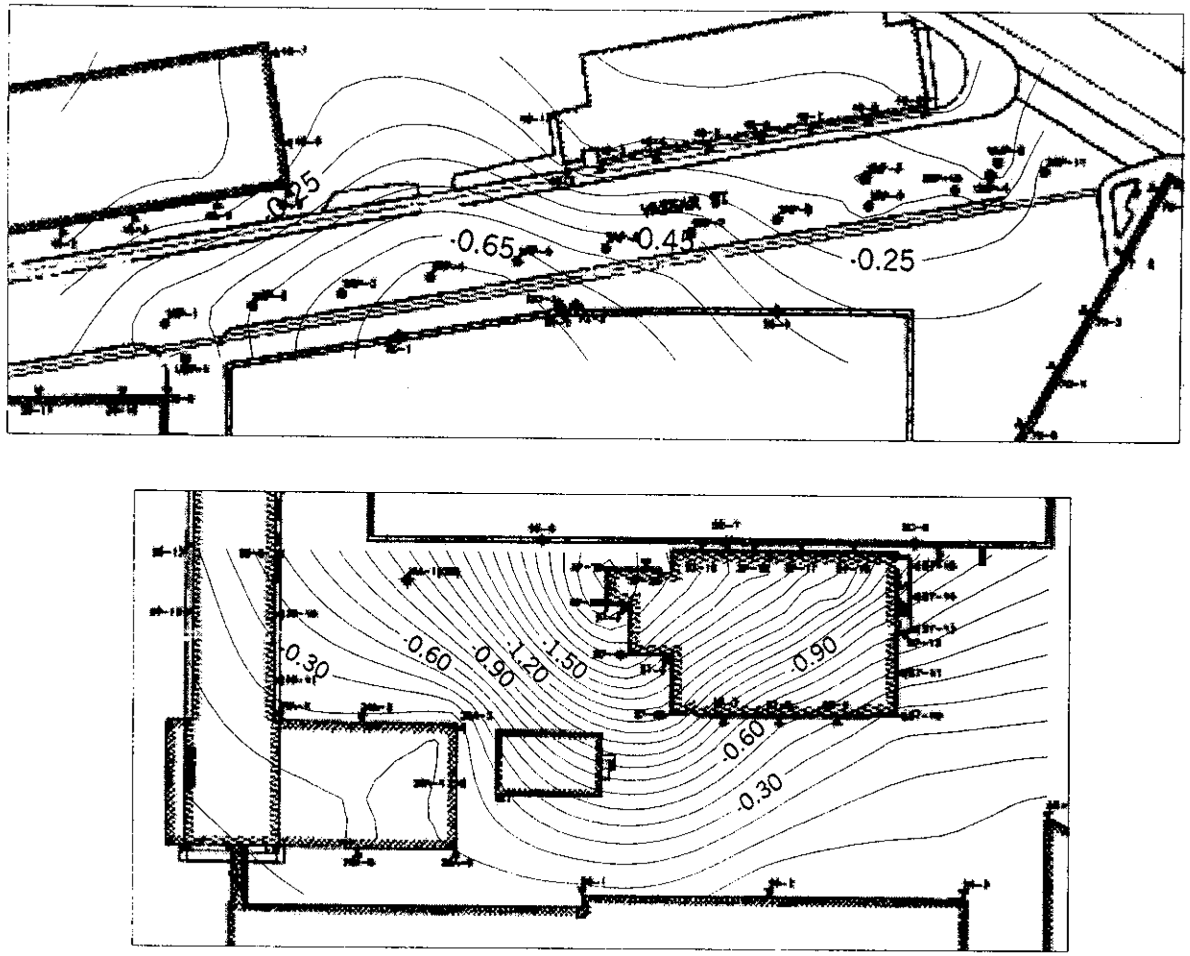


FIGURE 5.30: Settlement contours for the North and South Sides of the excavation for March 20, 2001.

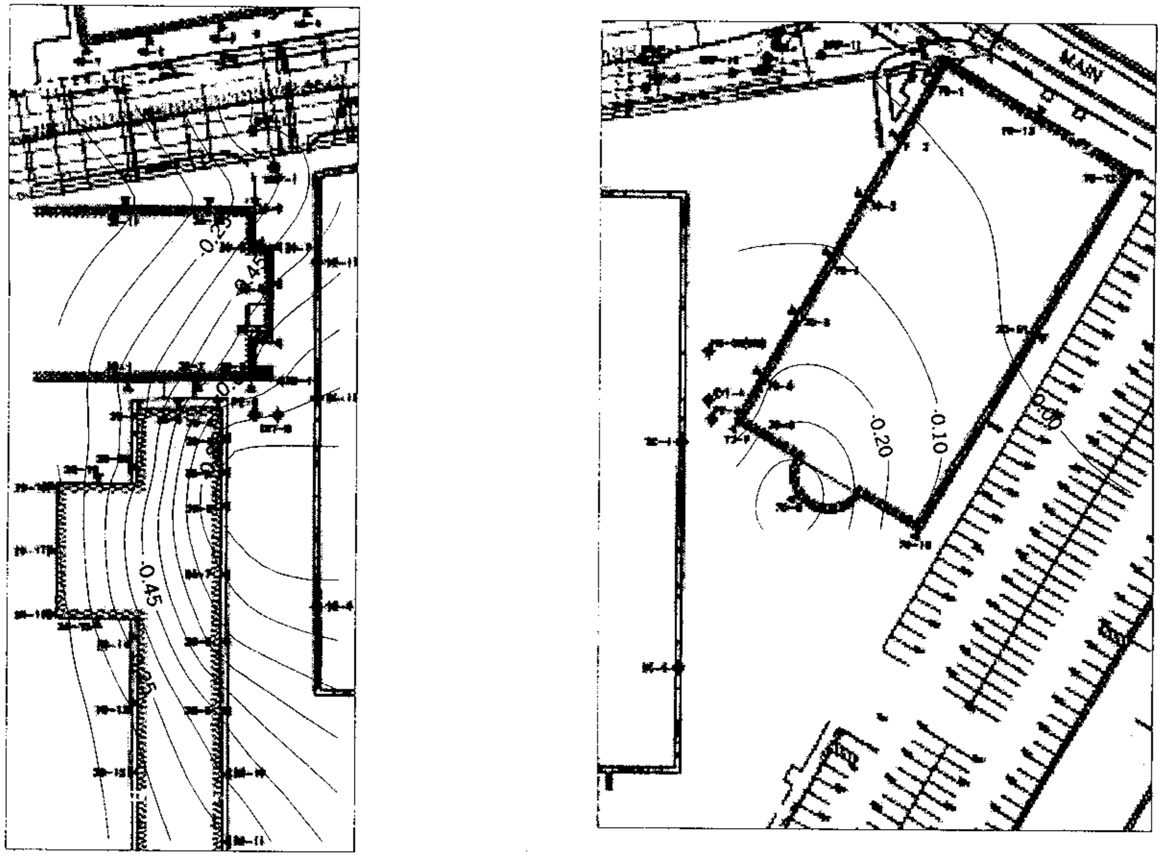


FIGURE 5.31: Settlement contours for the West and East Sides of the excavation for March 20, 2001.

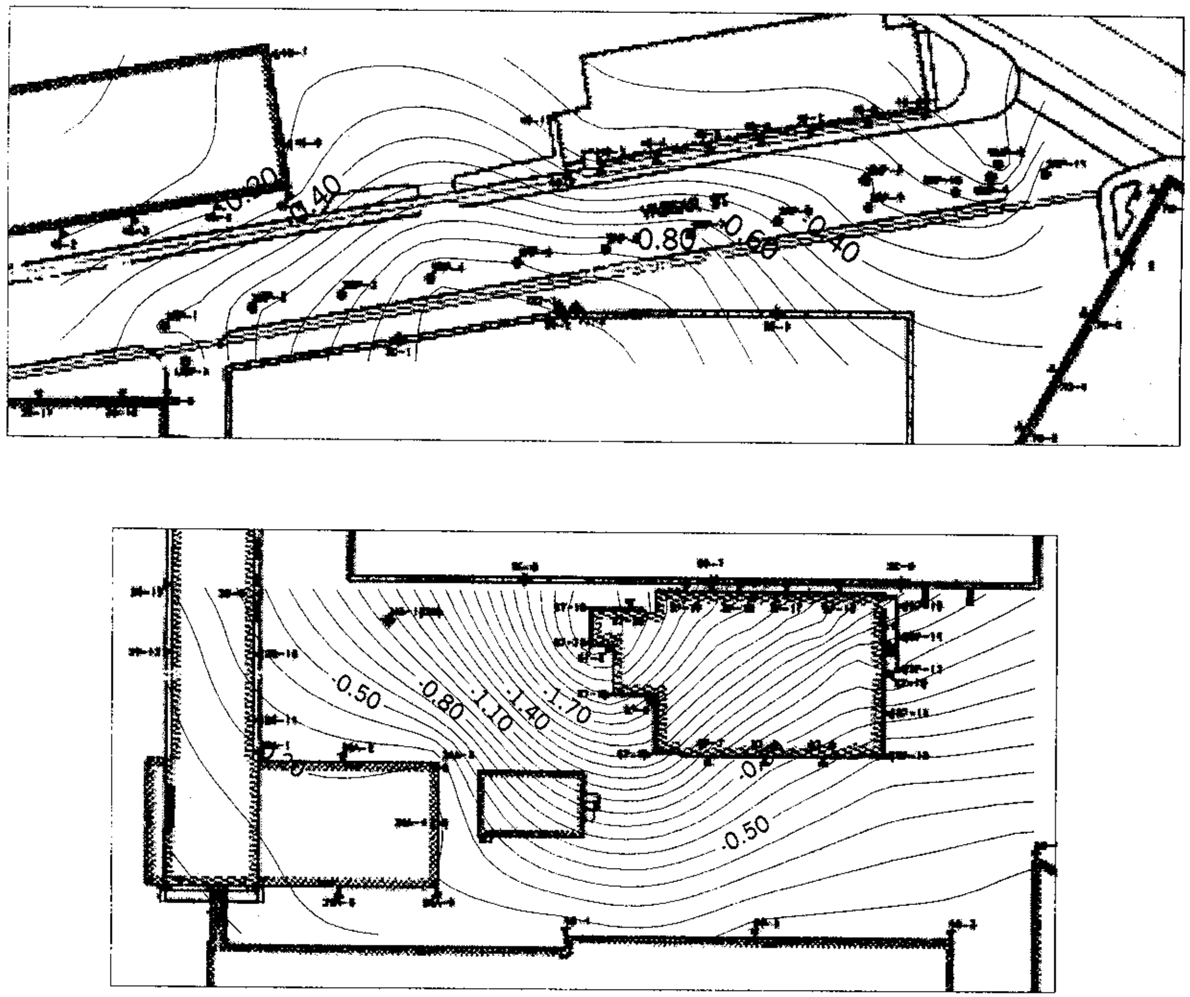


FIGURE 5.32: Settlement contours for the North and South Sides of the excavation for April 20, 2001.

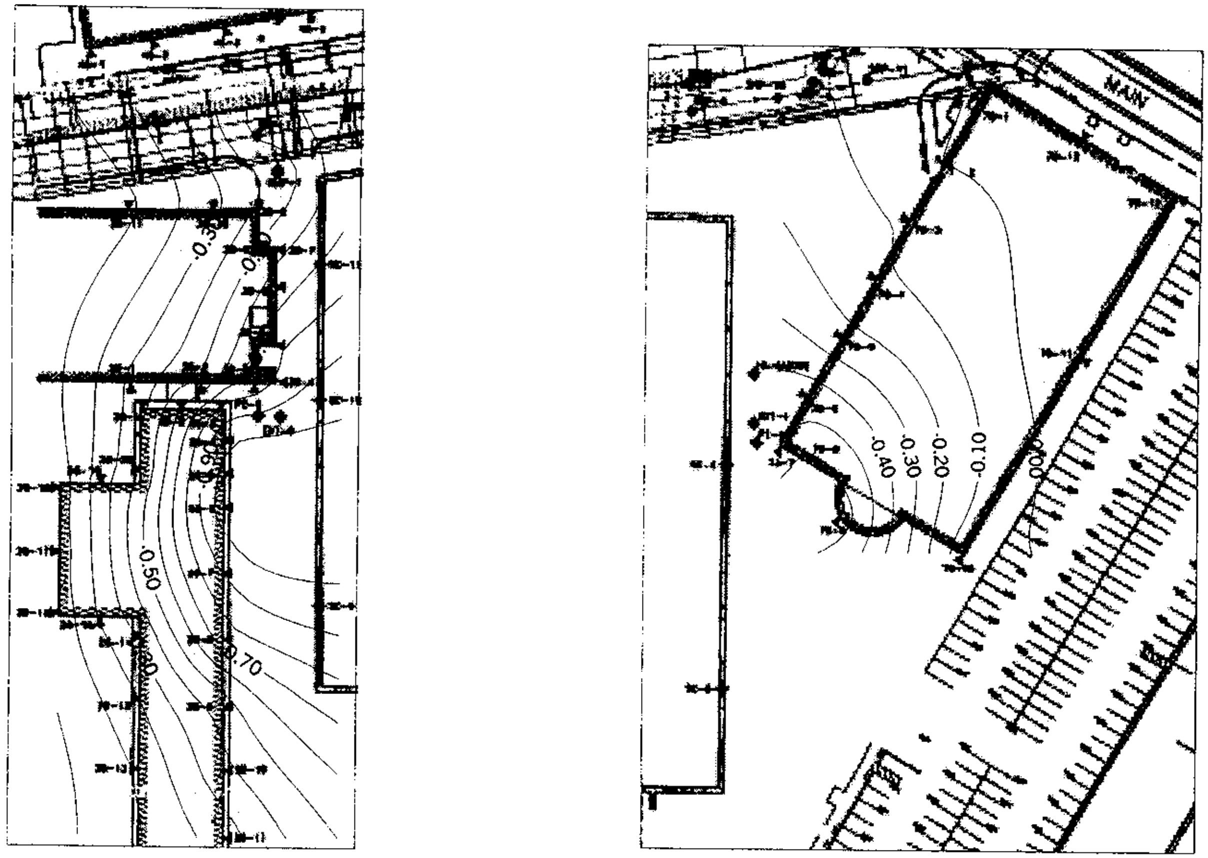


FIGURE 5.33: Settlement contours for the West and East Sides of the excavation for April 20, 2001.

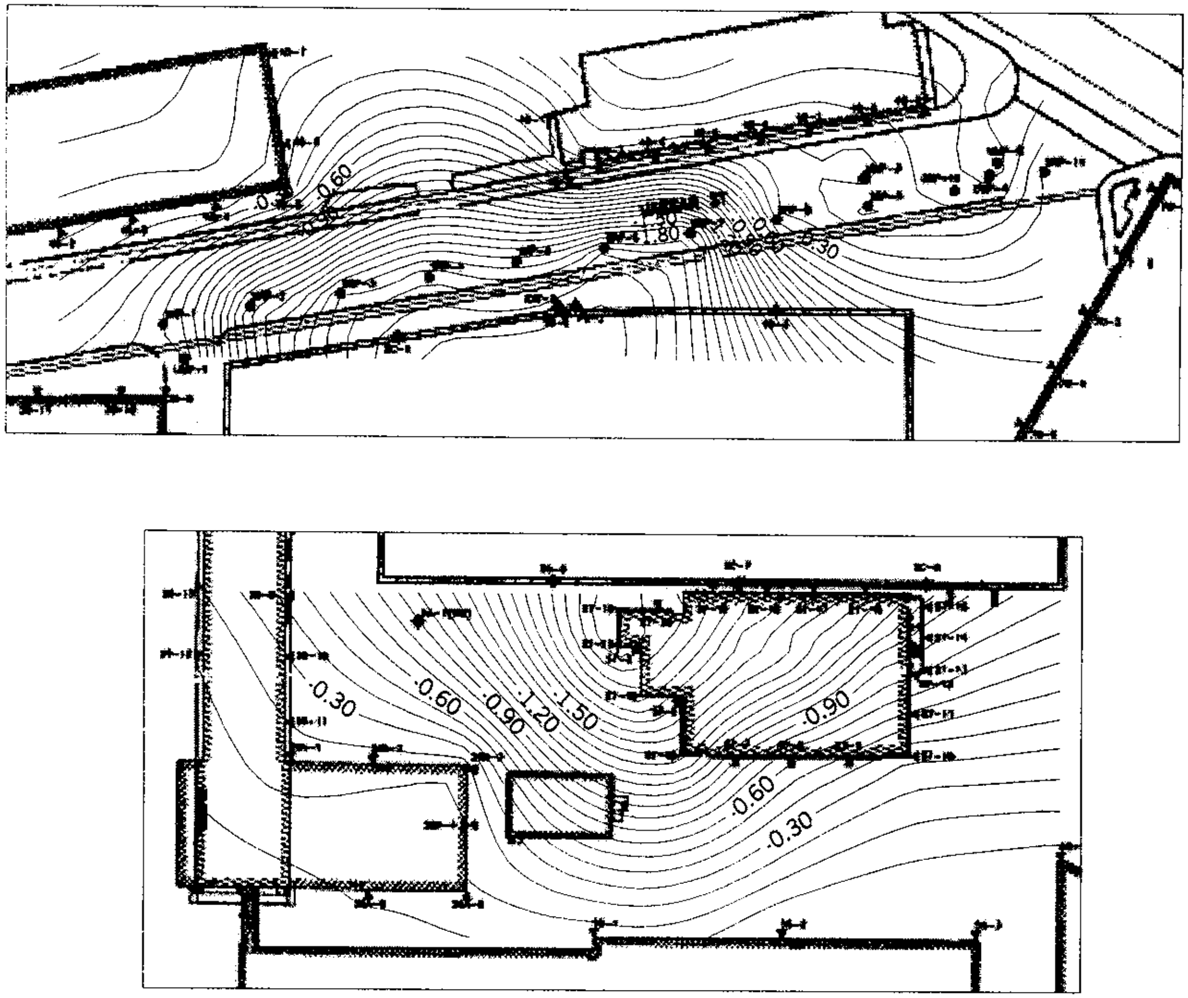


FIGURE 5.34: Settlement contours for the North and South Sides of the excavation for June 1, 2001.

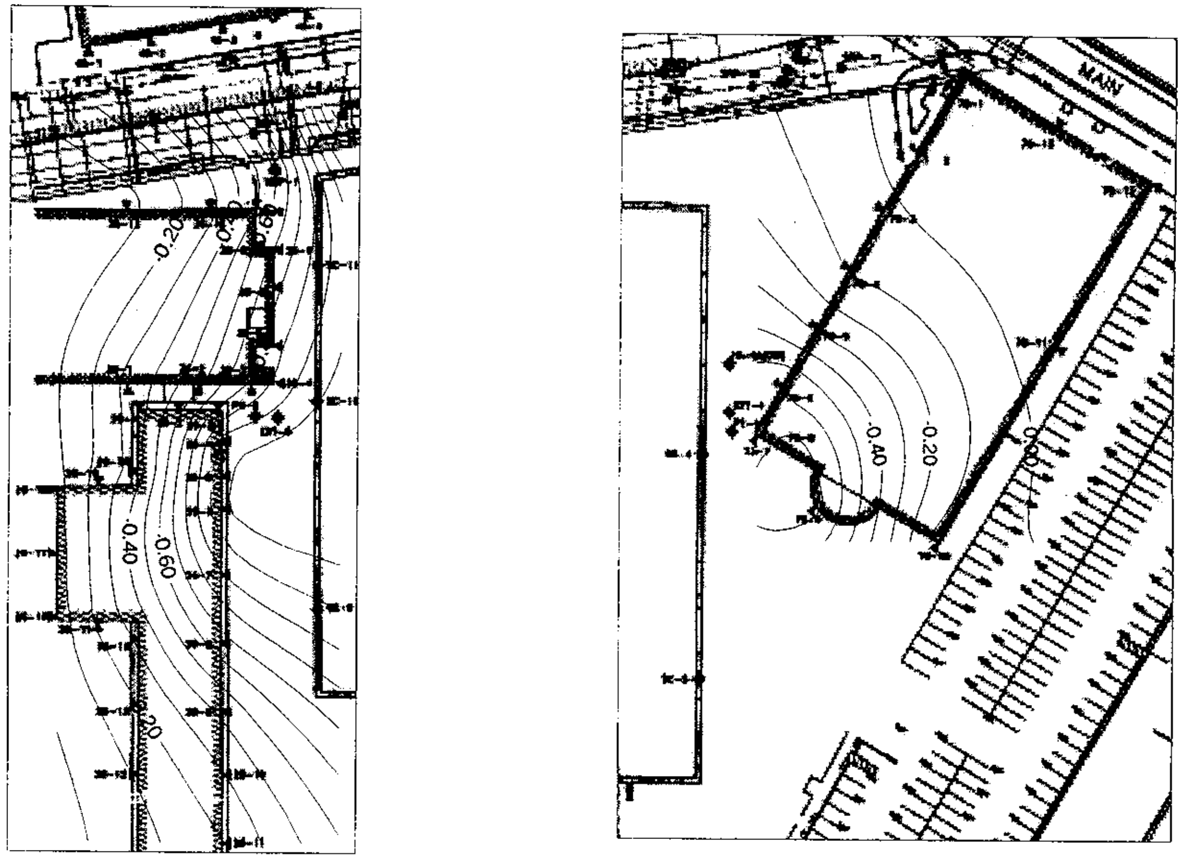


FIGURE 5.35: Settlement contours for the West and East Sides of the excavation for June 1, 2001.

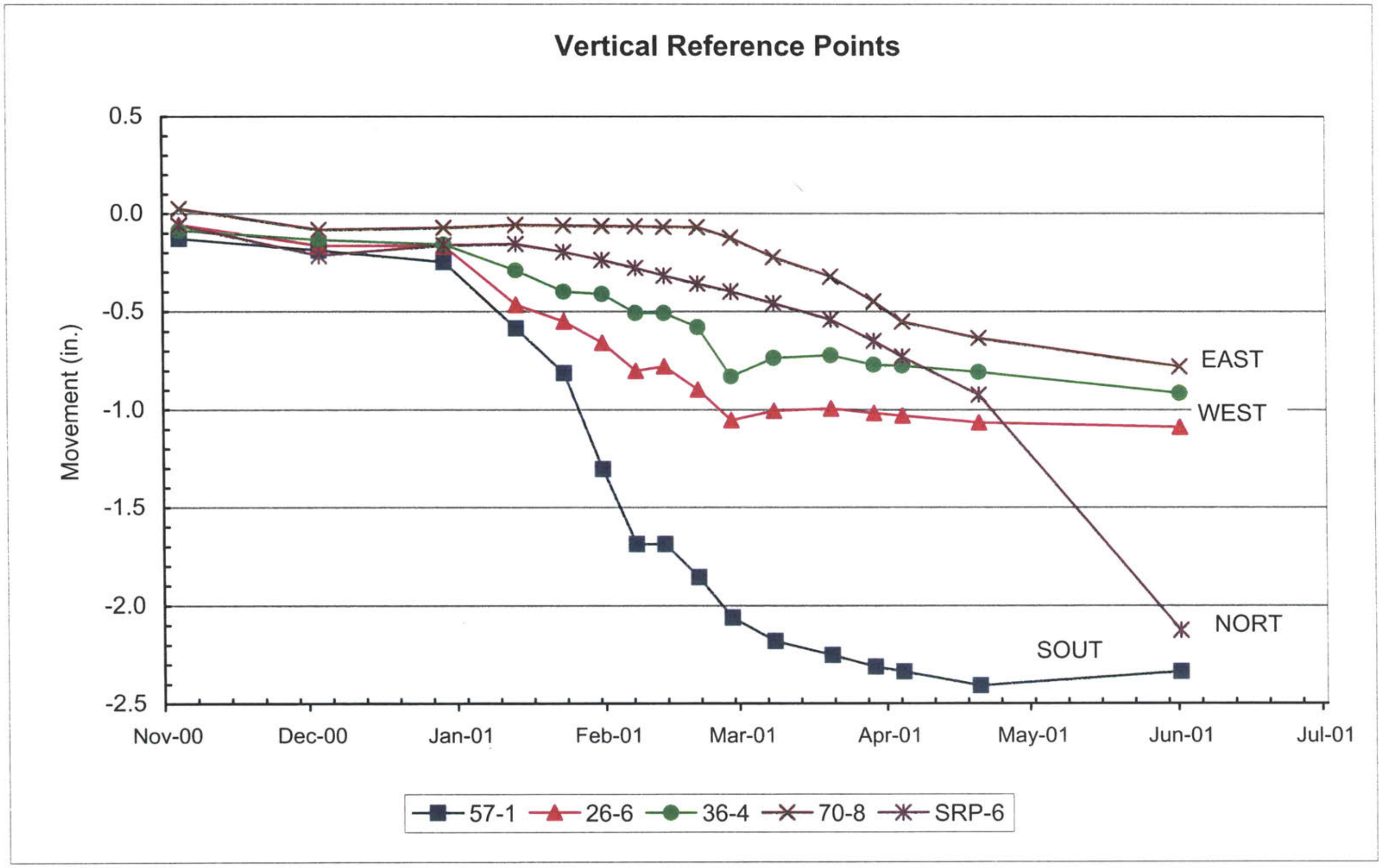
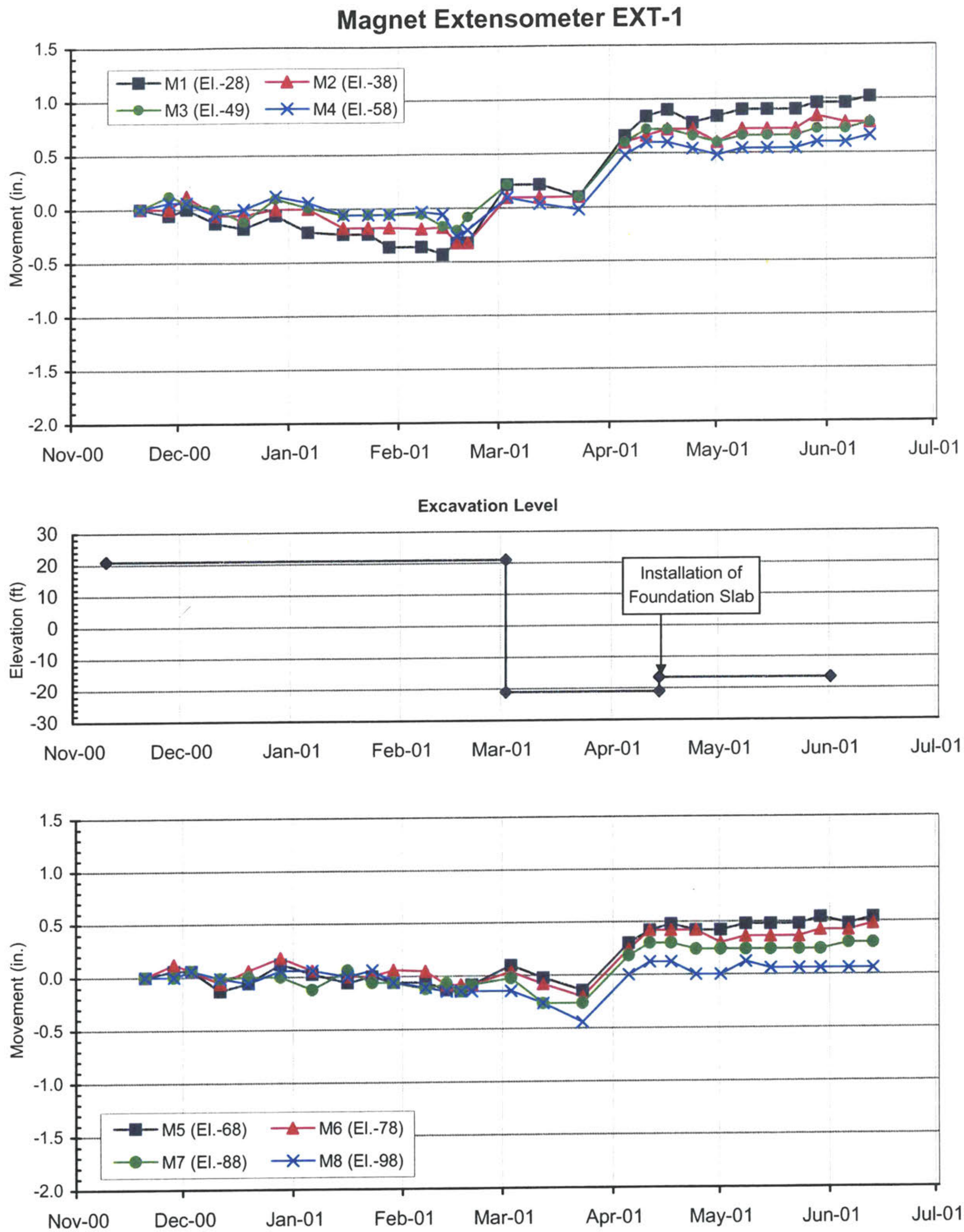


Figure 5.36: Settlement vs. Time for the points that showed the most movement on each side of the excavation. Note that 57-1 is on the south, 26-6 & 36-4 are on the west, SRP-6 is on the north, and 70-8 is on the east. (NOTE: For 70-8 and SRP-6 some points were interpolated in order to make the graph continuous.)



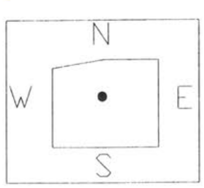


FIGURE 5.37: Summary of borehole extensometer EXT-1, excavation, and approximate location.

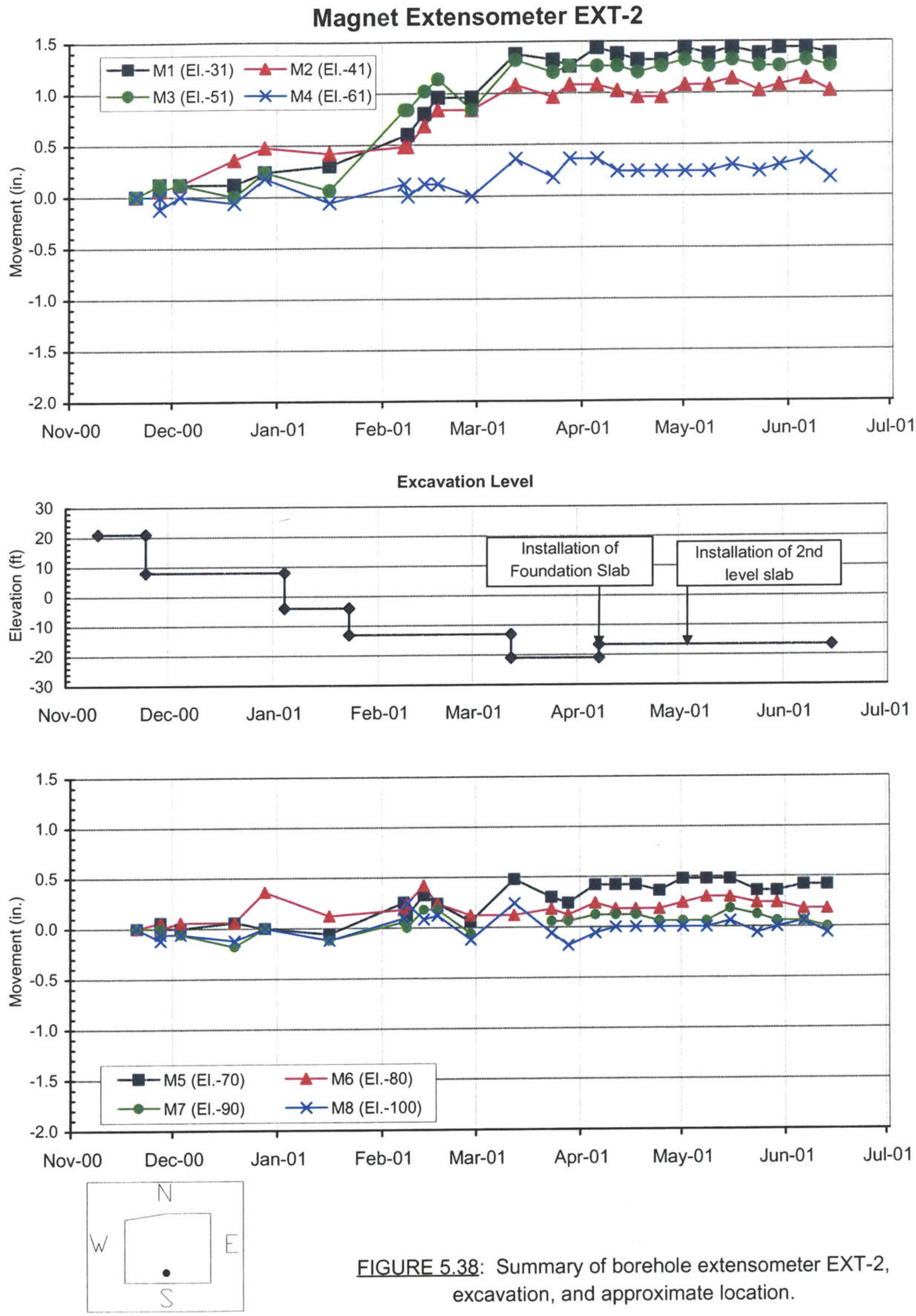
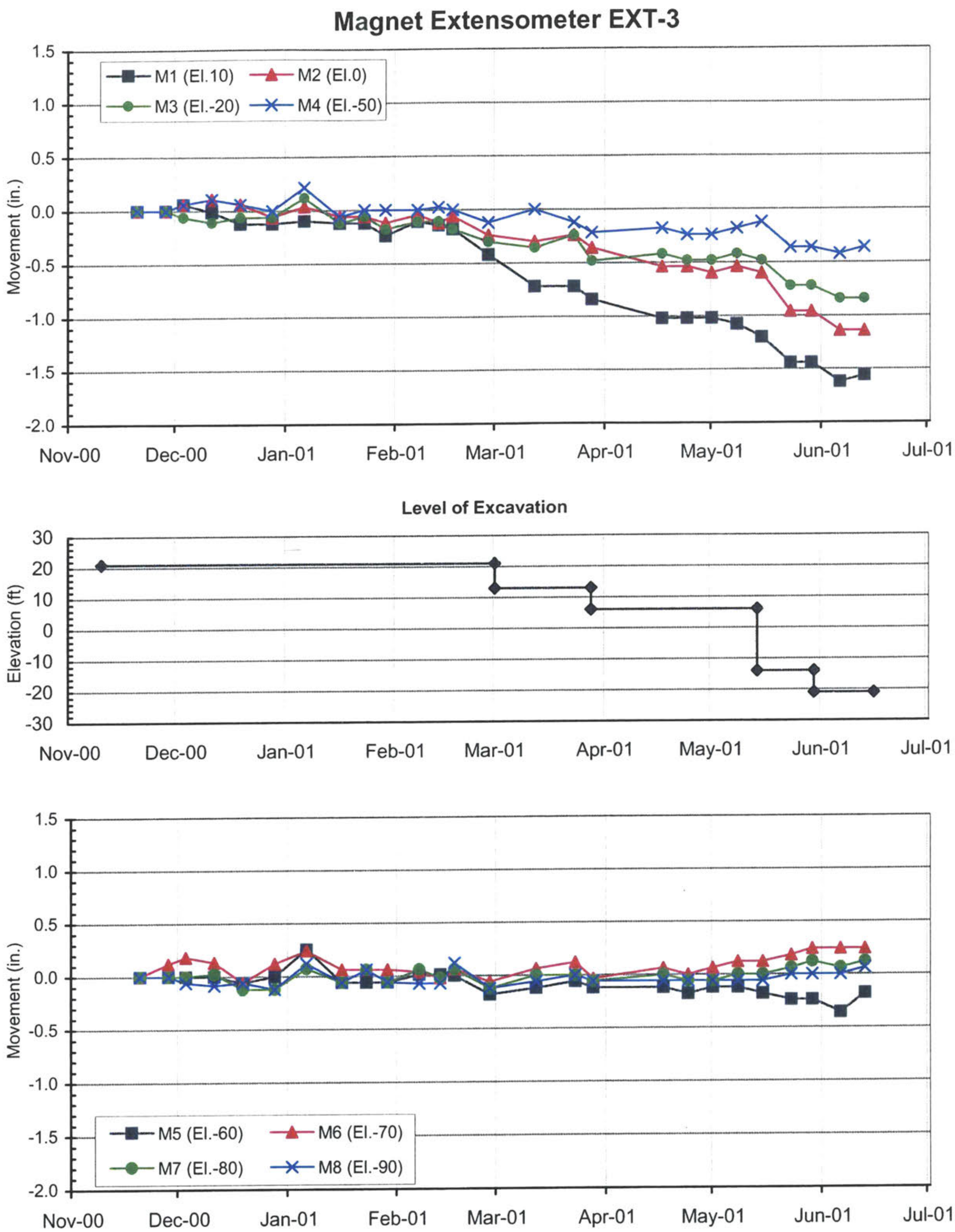


FIGURE 5.38: Summary of borehole extensometer EXT-2, excavation, and approximate location.



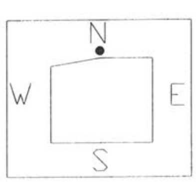


FIGURE 5.39: Summary of borehole extensometer EXT-3, excavation, and approximate location.

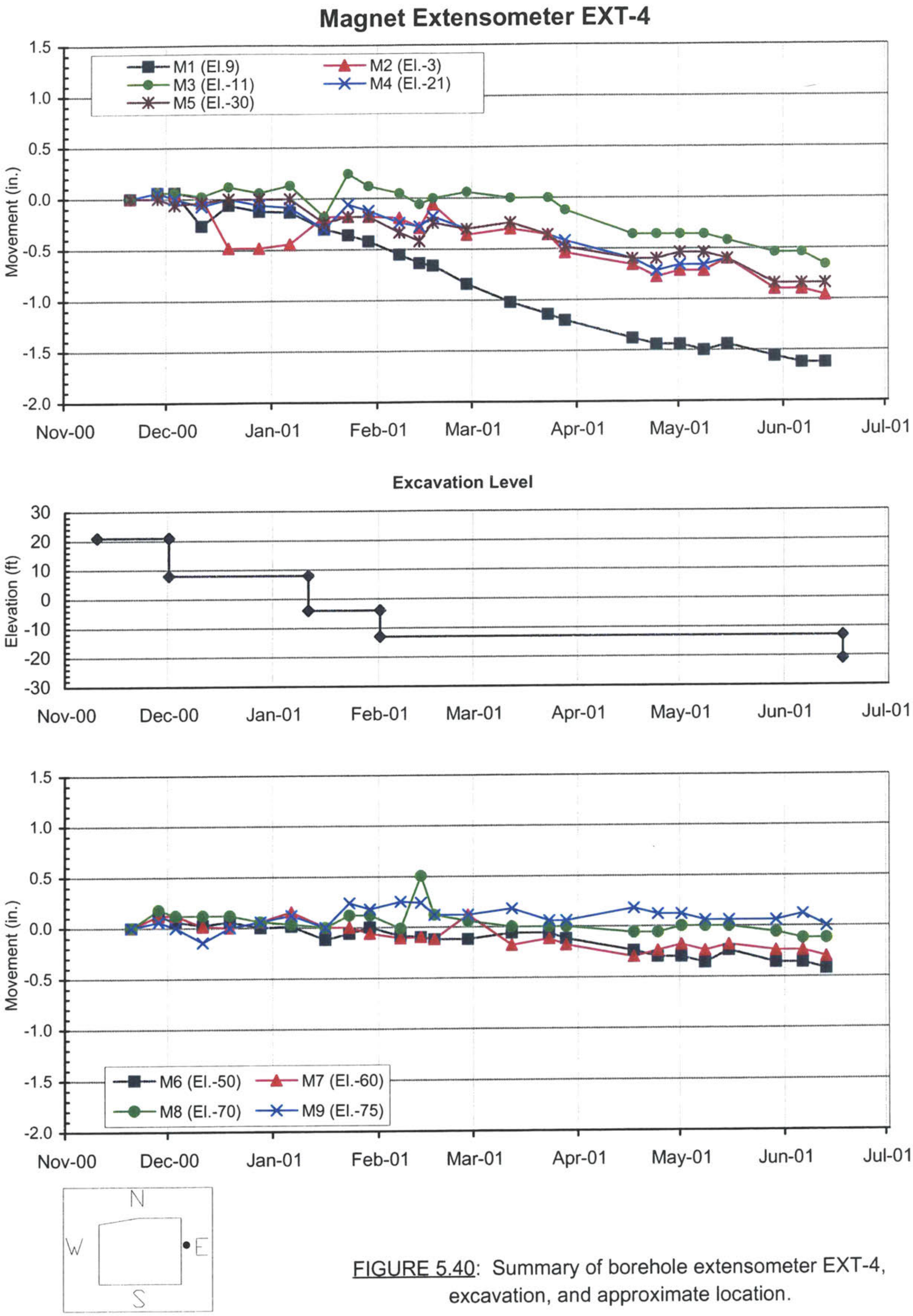


FIGURE 5.40: Summary of borehole extensometer EXT-4, excavation, and approximate location.

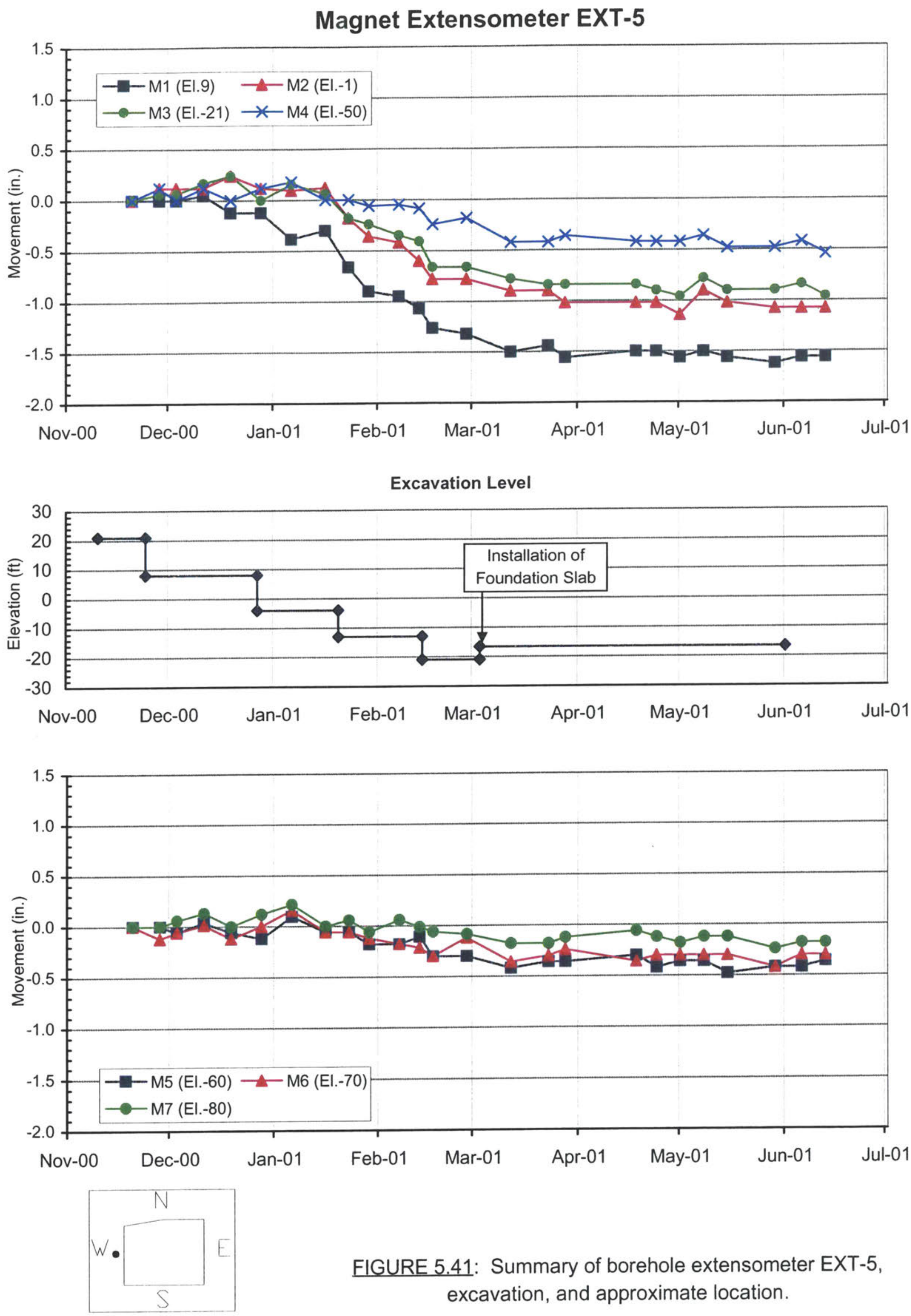


FIGURE 5.41: Summary of borehole extensometer EXT-5, excavation, and approximate location.

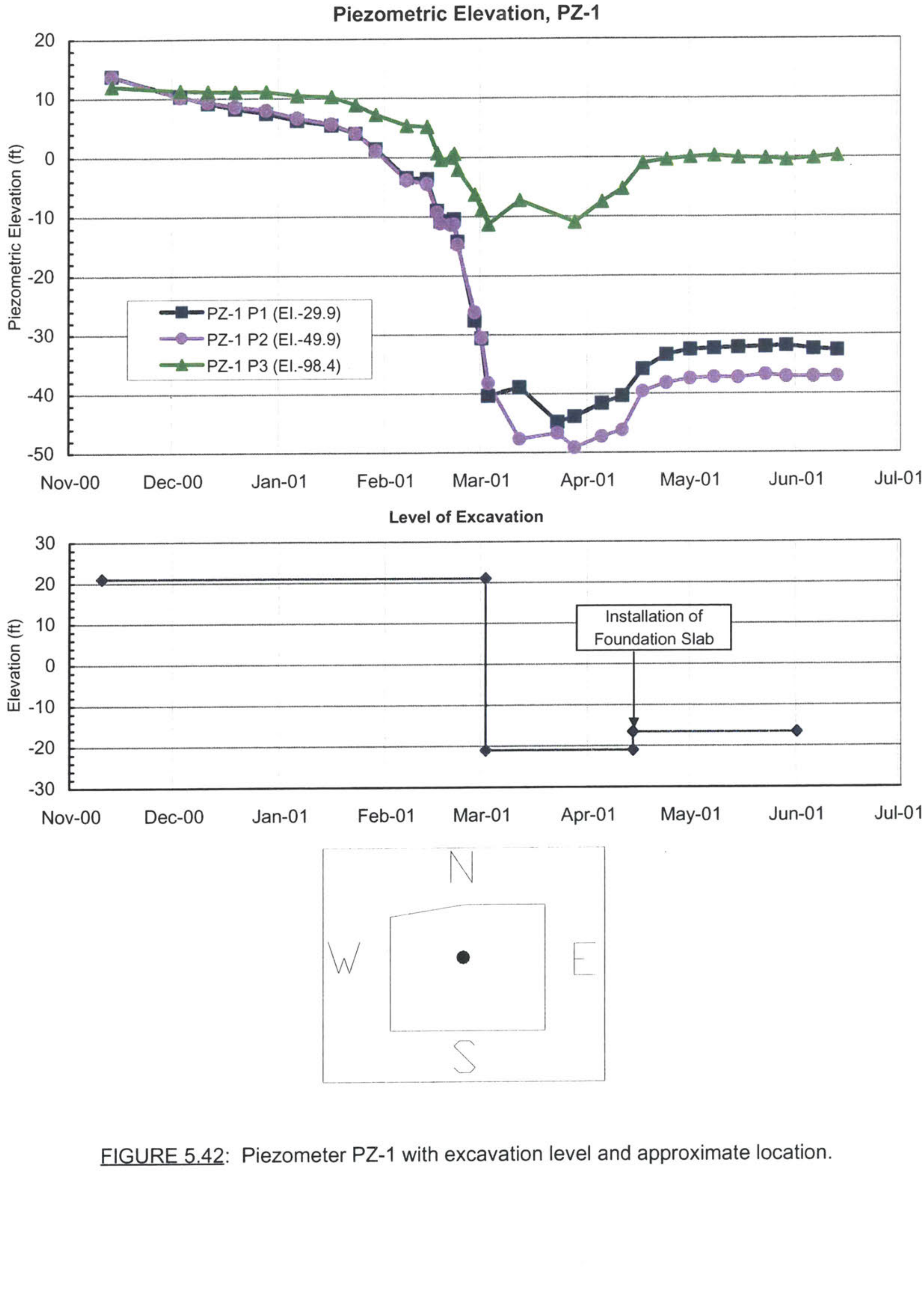


FIGURE 5.42: Piezometer PZ-1 with excavation level and approximate location.

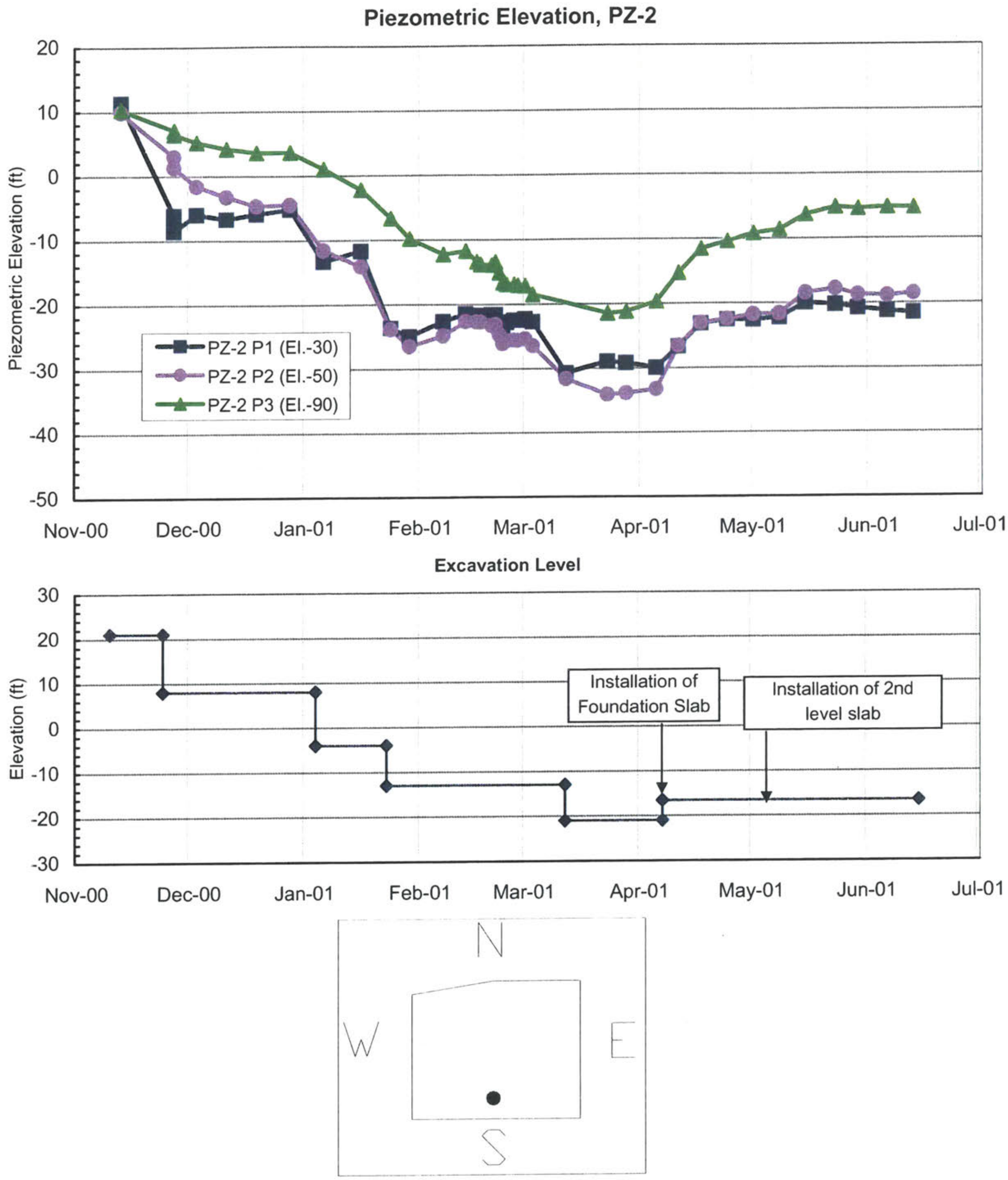


FIGURE 5.43: Piezometer PZ-2 with excavation level and approximate location.

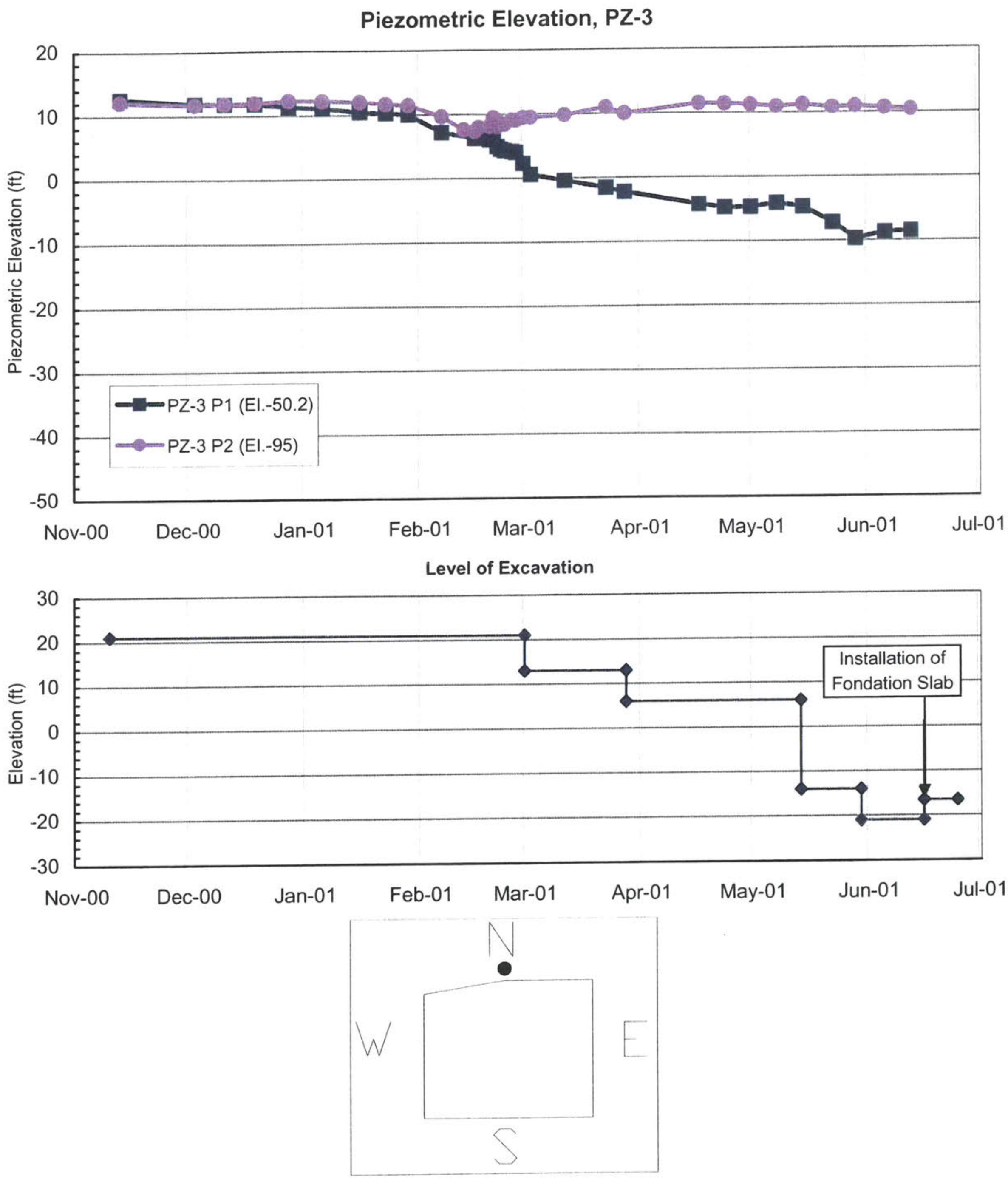
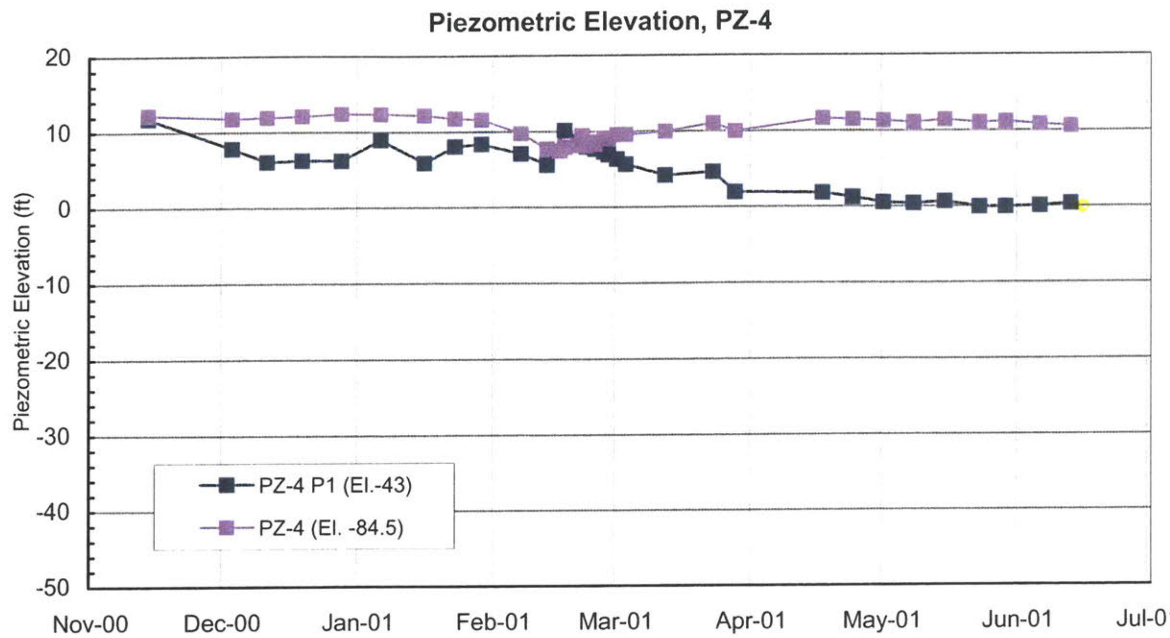
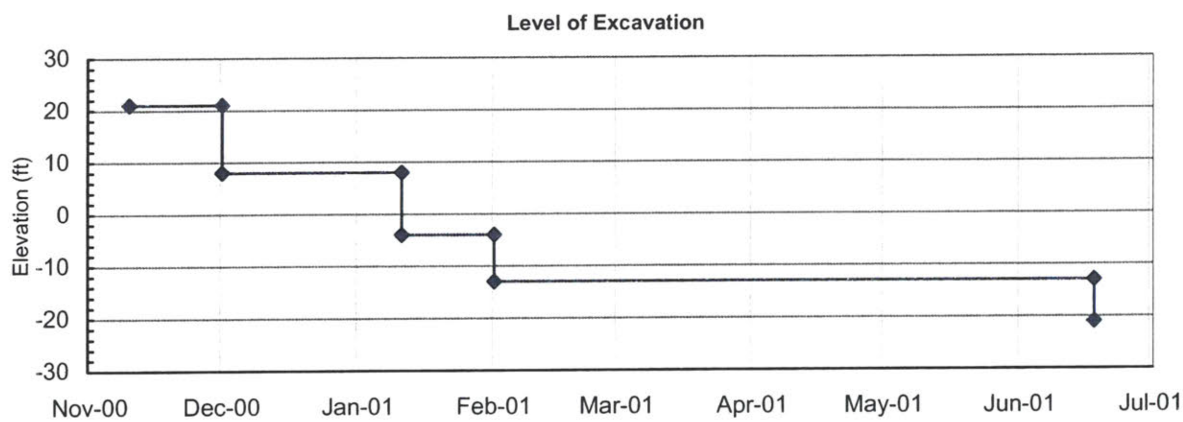


FIGURE 5.44: Piezometer PZ-3 with excavation level inside the excavation and approximate location.





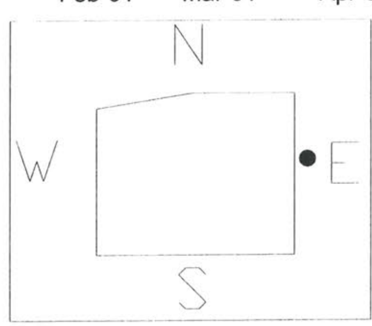


FIGURE 5.45: Piezometer PZ-4 with excavation level inside the excavation and approximate location.

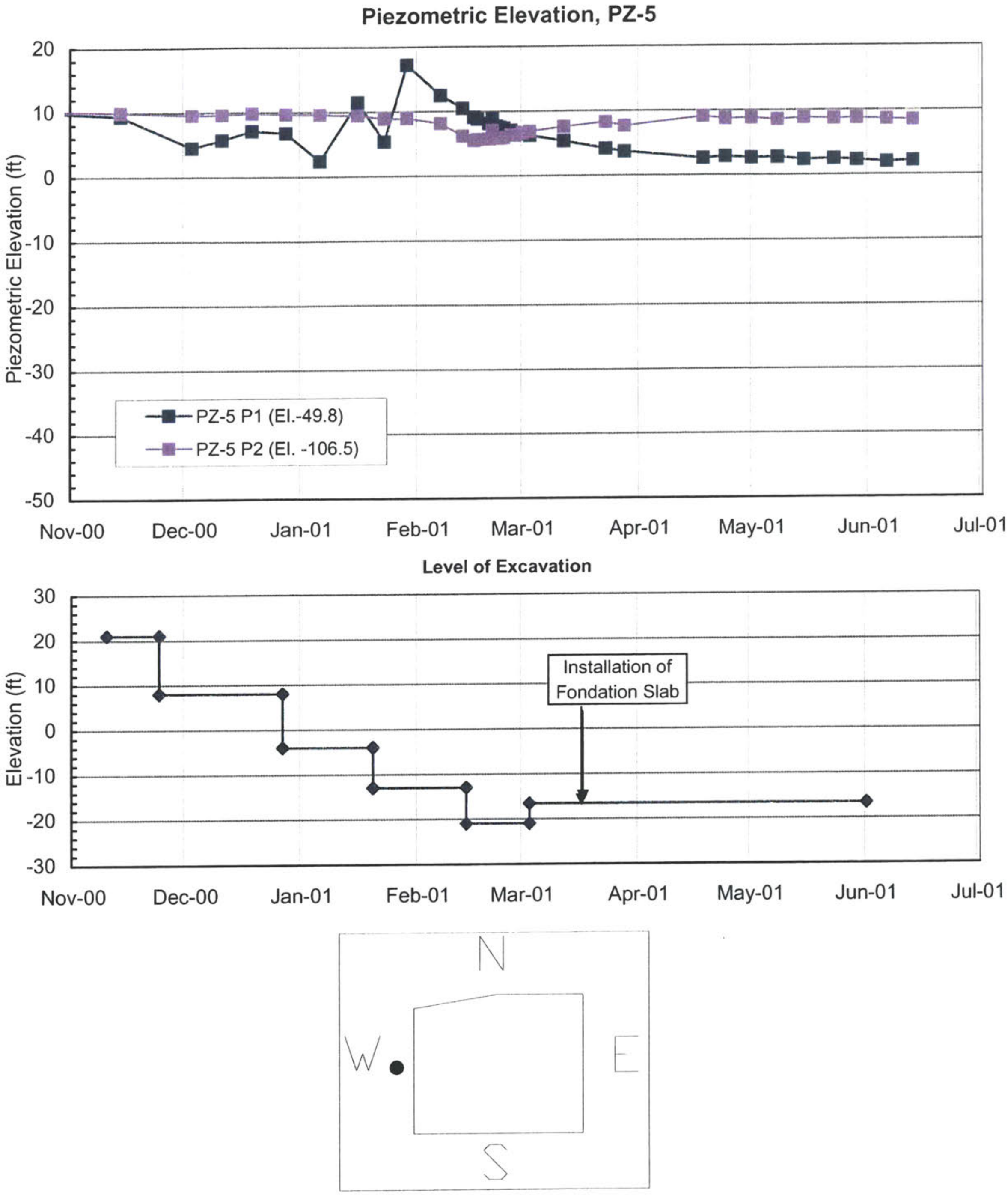
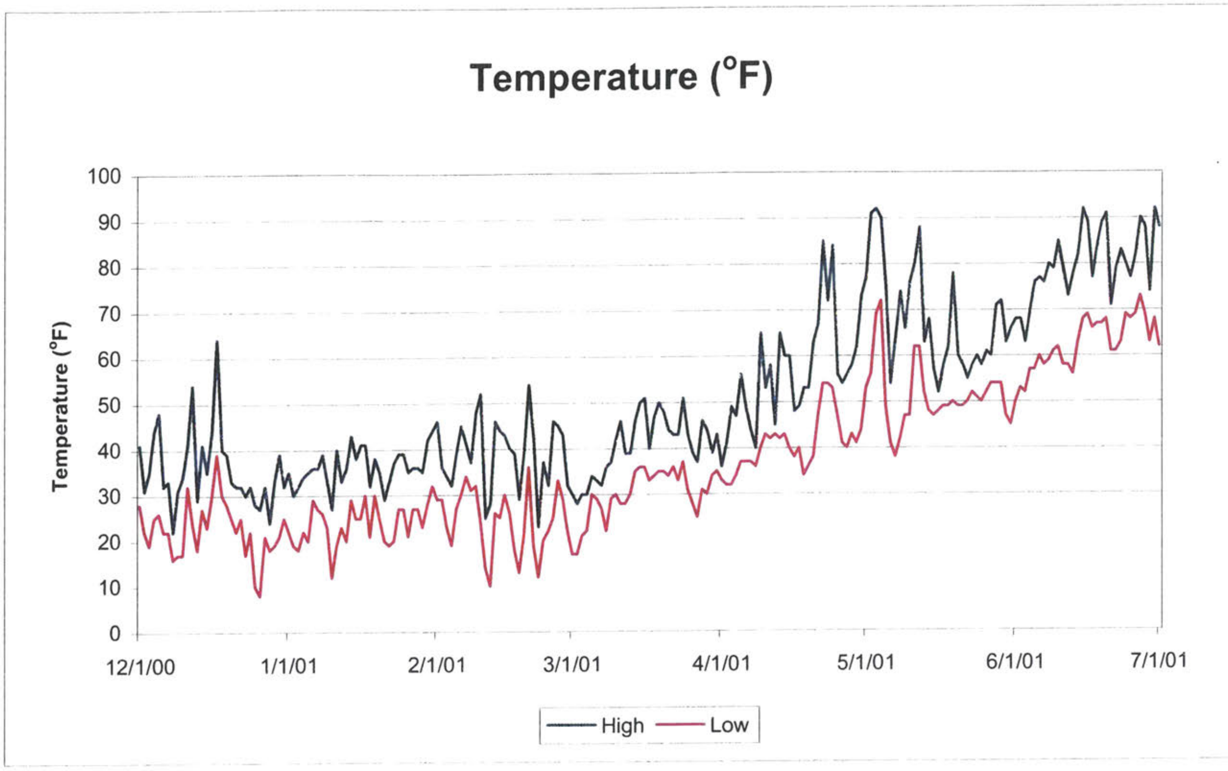


FIGURE 5.46: Piezometer PZ-5 with excavation level inside the excavation and approximate location.



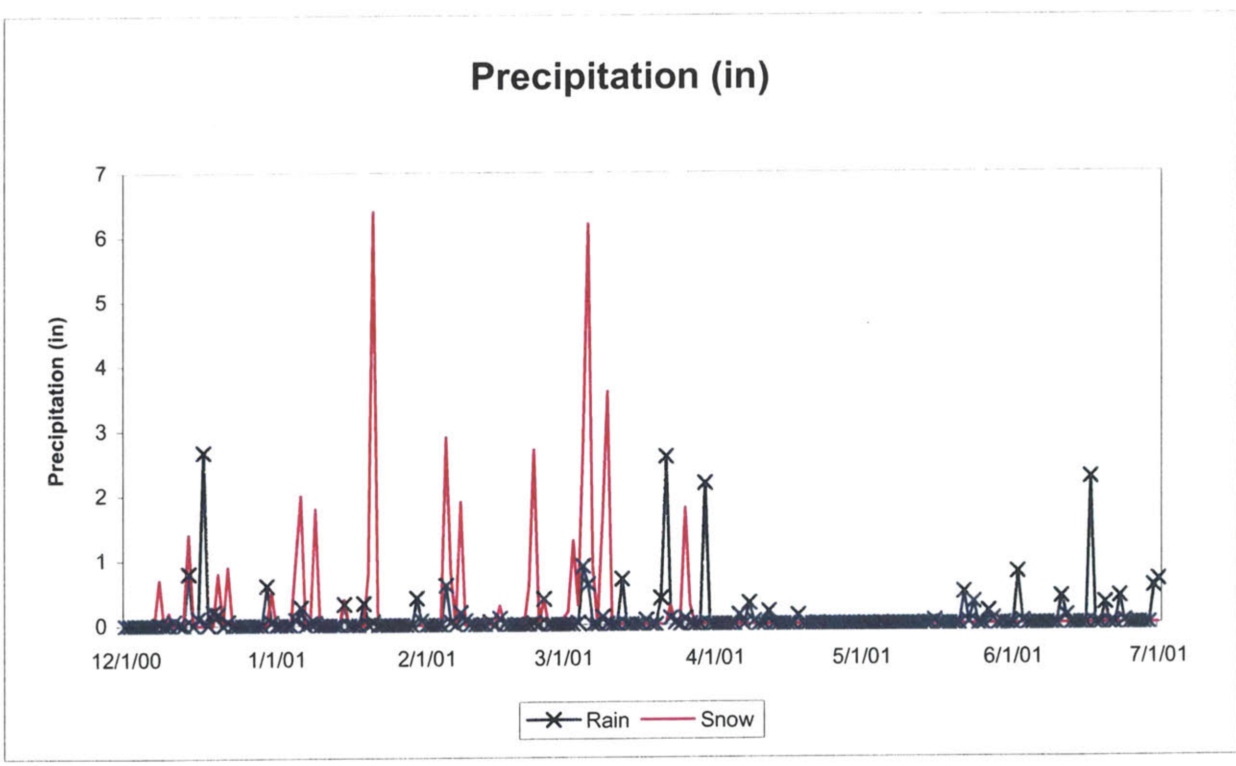


FIGURE 5.47: Weather information for Cambridge, MA from 12/1/00 to 7/1/01(NOAA)

Chapter 6
Results and Interpretation

6.1 Introduction

This chapter evaluates the measured performance of the diaphragm wall and lateral earth support system in relation to the expected performance and then in relation to other projects in the Boston area and other similar projects. The measured performance of the slurry wall consists of the lateral deflections of the wall itself and settlements of the ground surface and adjacent buildings. This chapter compares the measured data obtained during excavation for the basement of the Stata Center with prior predictions and empirical data from other slurry wall supported excavations in the Boston area and around the country.

6.2 Expected Performance

As part of the design process, a two-dimensional finite element analysis was done by Weidlinger and Associates (WAA, 2000) for five sections of the proposed wall (the middle of each of the four sides and one in the corner). These four analyses were used to analyze all three support systems (tieback anchors, rakers, and corner bracing), and to determine the effects of the excavation sequence. The sections were analyzed assuming horizontal soil layers and wall dimensions and support layout as previously discussed (three levels of tiebacks spaced 11 ft vertically, two levels of corner bracing at El. 10 ft and –10 ft, and two levels of rakers bracing the wall at El. 10 ft and –10 ft). A general Finite Element Method software package ANSYS was used to perform the calculations. A detailed description of the stratigraphy, soil properties, and surcharge loads from the surrounding buildings was not available in the version of the report that was available for review. It is understood that an averaged soil cross-section was used along with the available soil properties shown in chapter 3 and a water table at El 15 ft. In addition to surcharges from surrounding buildings, a construction surcharge of 600 psf was applied to the east and west sides for a width of 20 ft outside the excavation and a 250 psf traffic load was applied on the north side for 50 ft outside of the construction surcharge. The critical construction stages analyzed are summarized in Table 6.1.

Figures 6.1 through 6.5 compare the predicted and measured wall deflections (from the inclinometer closest to the area that was simulated). The results are also summarized in Tables 6.1 and 6.2. In looking at these figures, it should be remembered that the limiting value for maximum cumulative horizontal movement of the slurry wall set at the beginning of the project was 1.5 inches. This limiting value was determined by evaluating the allowable settlements of the surrounding structures and then calculating the amount of wall movement that would be expected to cause this settlement. The limiting settlement values for the surrounding structures were 1.5 inches (buildings 36 and 57) and 0.75 inch (all other buildings and structures).

The wall deflections shown in Figures 6.1 through 6.5 show that the mode shape of the predicted wall deflection is generally the same as the measured behavior, with maximum movement occurring at or below the lowest level of bracing and significant movements at the toe of the wall. This toe movement is typical of walls that do not have a fixed base (floating walls). The main difference that can be seen in these figures is that the magnitudes of the deflections are much larger than those predicted using 2D Finite Element analysis with the exception of the north wall. Table 6.2 shows that the actual movements ranged from 8.1% more than the value predicted to almost 105%, with four of the five over 33% larger. Notably the three tieback supported sections experienced movements more than 59% greater than predicted.

As for the settlements measured behind the diaphragm wall, the values measured were also greater the values predicted and expected (Table 6.3). In the case of the north and south sides, the settlements were about three times the values predicted. In all cases except that of the northwest corner the settlements exceeded the limiting value set out at the beginning of the project. These limits were set prior to wall design and the wall was then designed in order to meet these requirements.

There are many possible causes for differences this great including, 1) limitations in the assumptions used in the analyses, and 2) differences in the actual and assumed construction activities. Each of these issues could be the cause or more than likely it is a combination. The modeling issues include simplified selection of models and properties for the soil mass and support system[1], the validity of two dimensional assumption versus effects of three dimensional behavior, and accuracy of the construction sequence used in analysis versus the actual sequence. When any of these areas are involved it can lead to a support system that is not rigid enough for the situation. This includes support spacing, wall thickness, support material properties, and in the case of tieback anchors length. The issue of the length of tiebacks is important because they must extend deep enough into the soil mass to prevent the development of a deep-seated failure mechanism. Installation issues could include the collapse of tieback holes during installation, disturbance of the retained ground due to drilling tiebacks with air, and excessive excavation prior to installation of bracing.

Further analyses of the project including 3D finite element analysis, further investigations of soil properties, and more accurate modeling of the construction sequence could lead to a better understanding of which of these issues contributed most to the difference between predicted and actual movements.

6.3 Comparison with Other Projects

In engineering, it is important that with every project its relationship with prior projects is evaluated so that the knowledge of the profession advances. Many design procedures are based on correlations derived from experience with real projects. Most recently, Konstantakos (2000) attempted to summarize and compare the major slurry wall projects in the United States. In this thesis there is a comparison of roughly 30 projects in five cities including Boston, Chicago, New York, Washington D.C., and San Francisco. Not only are the projects in each city analyzed and compared, but all the projects were divided into four categories (floating walls, keyed tieback walls, top/down construction, and cross-lot/internally braced walls) and compared. In this section the Stata Center Project will be compared first with the Boston Projects and second with the floating walls outlined in Konstantakos (2000).

6.3.1 Boston Projects

There have been a large number of projects in the Boston area, dating back to the State Transportation Building in the 1970s, using diaphragm walls to support the excavation, often as part of the permanent structure. Another reason for their usage is the presence of a high water table which necessitates a system that is more watertight

[1] Hashash & Whittle (19996) have shown that the reliable predictions of surface settlements are critically dependent on the modeling of soil stiffness properties, while wall deflections can be reasonably estimated

(although this characteristic has not always been achieved in the Boston area). Konstantakos (2000) summarized 13 archived projects for which extensive data was available and also pointed out that there are at least 8[2] other projects that were not included (6 of these projects were constructed within 4 years of the study). These projects represent a wide range of wall lengths, excavation depths, thicknesses, and support systems. Three of the projects were floating tieback wall, two were keyed tieback walls, six were Top/Down excavations, and two were cross-lot/internally braced excavations. Most of these projects are not closely related to the Stata Center because they are either tied into bedrock or they used a top down construction sequence. Recently Whelan (1995) summarized the construction and performance of some diaphragm walls for the CA/T project in Boston. In this project the maximum movements of the wall were into the retained soil mass and exceeded 5 in of movement[3]. The project is of interest when studying the use of these walls in the Boston area but it is not closely related to this project since the diaphragm walls were keyed into the bedrock.

6.3.1.1 Wall Deflections

The 13 projects studied an overwhelming majority of the inclinometers deflected less than 1.0" with an average maximum horizontal deflection of 0.70" (Figure 6.6). It is interesting to note in figure 6.6 that over 80% of all of the inclinometers from these projects showed 1.0" of movement or less no matter the support system or construction method. Also, Figure 6.7 shows that if the deflections are normalized by the excavation

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using quite crude models of behavior.

[2] Most applications of diaphragm walls in the Boston area over the last ten years were for the CA/T project and the data is not yet available.

[3] These large reported settlements were attributed to ground loss during tieback installation.

depth that almost 70% of the inclinometers had a deflection ratio of 0.2% or less with, an average of 0.17%. The inclinometers from the Stata Center Project show a range of maximum/final deflections from 2.0" to 3.25" (average=2.5"), which places them in a range in Figure 6.6 that totals only 4.8% of all of the inclinometers from the study. These measured deflections also represent movements three to five times the average for projects in Boston. The deflection ratio for the Stata Center inclinometers ranges from 0.4% to 0.64% (average=0.49), which places them in a range in Figure 6.7 that totals only 5.7% of the inclinometers. This range is two to four times the average ratio.

Another way to compare the movements of the wall with the other projects is by the system stiffness approach as proposed by Clough et. al. (1989). As shown in Figure 6.8, the Stata project shows a high deflection ratio related to its system stiffness. The measured performance is very similar to the archived data from 500 Boylston St. This project also consisted of a floating slurry wall embedded in the top of the clay layer and is considered one of the least successful projects in the Boston area. The large wall deflections for the 500 Boylston St project attributed to poor tieback performance, and the proximity of these two projects on this graph indicates that one possible reason for large movements is inadequately designed tieback anchors. It must be noted that the value computed for the deflection ratio for tiebacks for the Stata center is for the maximum deflection experienced at inclinometer SC-07 and that if the other two inclinometers in tieback sections (SC-4 and SC-9) are computed the ratio drops to 0.42. This indicates that the tieback problems were worst on the south side than on the east or west. Even with what appears to be excessive wall movements for the Boston area, comparison of these values with Figure 6.9 (Clough et al. 1989) shows that the factor of safety against basal heave would be expected to be in the range of 1.1 to 1.4. Stability analysis was done on the Stata Center excavation prior to its construction in order to evaluate the factor of safety against basal heave (Whittle, 2000). This analysis comprised using numerical analyses that solve upper and lower bound theorems for rigid-perfectly plastic soils and the use of finite element methods to confirm the calculations. The results of the analysis of a 3 ft thick wall supported by three levels of rakers gave a factor of safety against basal heave of greater than 1.79. Then these results are compared with the estimates from Clough et al. (1989) it indicates that the chart in Figure 6.9 is not always applicable and that there are often better methods available for evaluating excavation factors of safety.

6.3.1.2 Measured Settlements

Figure 6.10 shows settlement data versus distance from the excavation for the Boston slurry wall projects studied by Konstantakos (2000) and selected data from the Stata Center project. The Boston data shows that with the exception of the 500 Boylston project (that had poor tieback performance) settlements fell within Zone I as proposed by Peck (1969) and it can be seen that settlements decrease with distance from the excavation. The data from the Stata Center Project also show the reduction with distance, but still a number of points lie outside of Zone I. This could also be an indication that, like the 500 Boylston project, the support system did not perform as would expected for a project in the Boston Area. Once again, this could be attributed to, along with other reasons, inadequately designed tieback anchors (loads, length, bonding strata, etc.) or soil loss and disturbance caused by the tieback installation

6.3.2 Floating Diaphragm Wall Projects

Diaphragm walls are considered to be floating when the toe of the wall is embedded within a soft stratum such as the Boston Blue Clay as opposed to a keyed wall where the toe of the wall is embedded in a stiff stratum like glacial till or bedrock. The analysis by Konstantakos (2000) covered three floating wall projects in Boston and six in Chicago. Two of the Chicago projects were built in the 1970s when experience with diaphragm wall construction was not very extensive and therefore the wall movements were large (2.5" and 4.6") and one of the Boston projects showed large movements (3.3") due to inadequate tieback length, tieback creep and load loss, and ground softening due to pile extraction. All other projects showed maximum wall deflections of less than 1.55" and translation of the wall base ranging from 0.2" to 0.5".

In relationship to these numbers the Stata Center Project experienced maximum wall movements of 2.0" to 3.25" and translation of the wall base ranging from 1.3" to 2.5". These numbers far exceed what would be expected from prior experience and could be expected to indicate good design and installation. Figures 6.11 and 6.12 also show this. Including the three projects that had large movements, they show that more than 90% of the inclinometers in floating walls showed less movement than those from the Stata project, and more than 80% show a deflection ratio less than those from the Stata project. This indicates that the performance of the lateral earth support system for the Stata Center performed poorly compared to prior expectations.

TABLE 6.1: Construction stages analyzed in FEM calculations of the lateral earth support performance. (WAA, 2000)

Tieback Supported Walls	Corner Bracing Supported Walls	Raker Supported Walls
1. Install walls and excavate to El 8 ft.	1. Install walls and excavate to El 8 ft.	1. Install the wall, excavate to El 8 ft on the passive side to form a berm 25 ft wide and sloping 2H:1V to El -22 ft.
2. Install tiebacks at El 10 ft.	2. Install struts at El 10 ft and pre-load.	2. Install the first level of rakers at El 10 ft and pre-load the rakers to 50% of the design load.
3. Excavate to El -3 ft.	3. Excavate to El -12 ft.	3. Excavate the berm to El -12 ft.
4. Install tiebacks at El -1 ft.	4. Install struts at El -10 ft and pre-load.	4. Install the second level of rakers at El -10 ft and pre-load them.
5. Excavate to El -14 ft.	5. Excavate to El -22 ft (bottom of excavation).	5. Excavate to El -22 ft (bottom of excavation).
6. Install tiebacks at El -12 ft.		
7. Excavate to El -22 ft (bottom of excavation).		

Wall Section	Building Number	Limiting Value (horizontal defl.)	Max. Predicted Deflection	Max. Measured Deflection	% Difference between Meas. and Predicted
West	26	1.5 in.	1.57 in.	2.5 in.	59.2
South	57	1.5 in.	1.59 in.	3.25 in.	104.4
East	70	1.5 in.	1.50 in.	2.58 in.	72.0
North	-	1.5 in.	1.85 in.	2.0 in.	8.1
Corner	36	1.5 in.	1.50 in.	2.0 in.	33.3

TABLE 6.2: Summary of predicted maximum wall deflections and maximum measured deflections.

Wall Section	Building Number	Limiting Value (Vertical Settl.)	Max. Predicted Settlement	Max. Measured Settlement	% Difference between Meas. and Predicted
West	26	0.75 in.	0.67 in.	1.1 in.	64.2
South	57	1.5 in.	0.76 in.	2.4 in.	215.8
East	70	0.75 in.	0.73 in.	0.8 in.	9.6
North	-	0.75 in.	0.75 in.	2.1 in.	180.0
Corner	36	1.5 in.	0.75 in.	0.9 in.	20.0

TABLE 6.3: Summary of predicted maximum settlements and maximum measured settlements outside of the excavation.

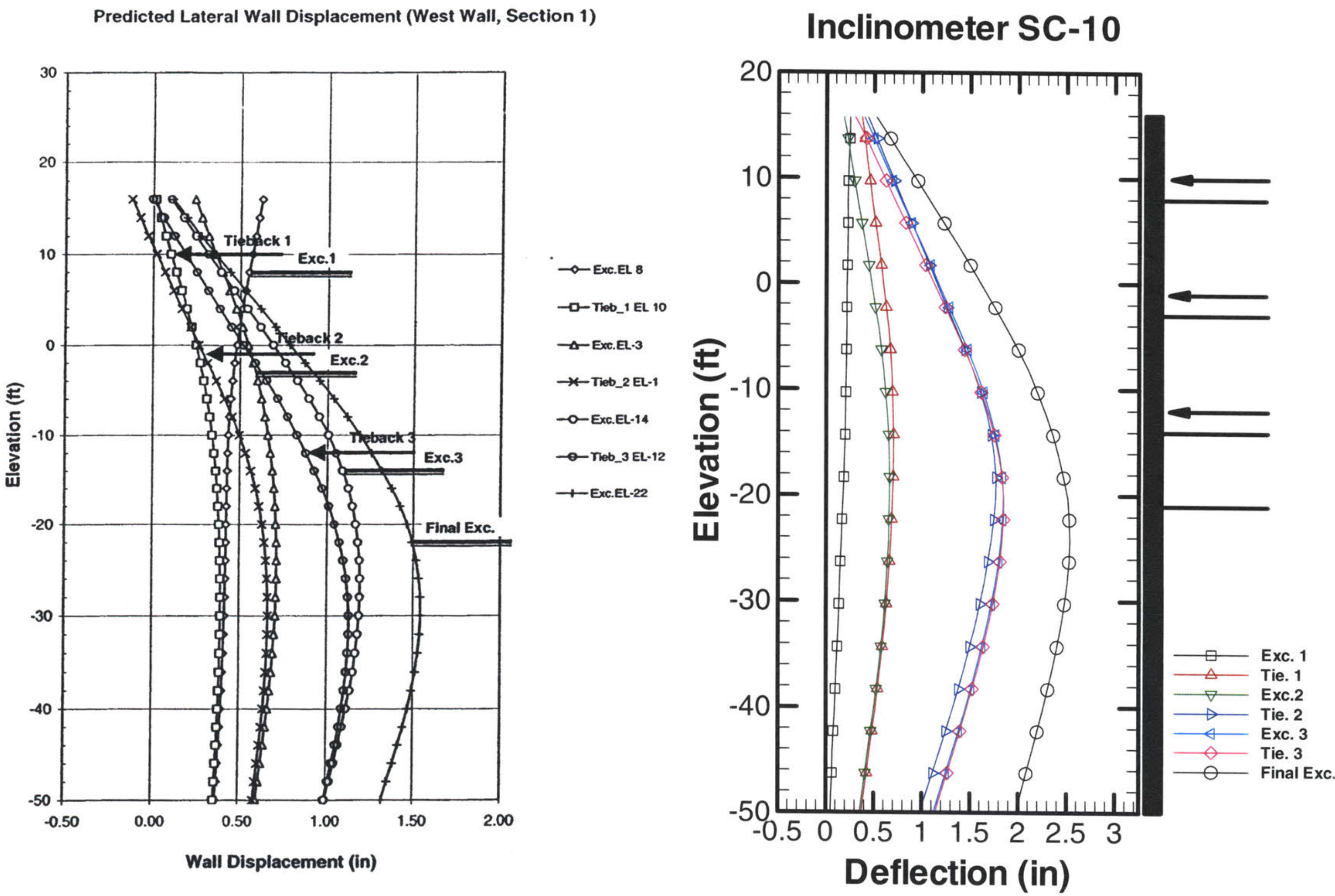


FIGURE 6.1: Comparison of 2D Finite Element predictions of horizontal wall movements (Weidlinger) during design (left), and measurements of actual wall movements (right) for the tieback anchor supported section of the West Wall.

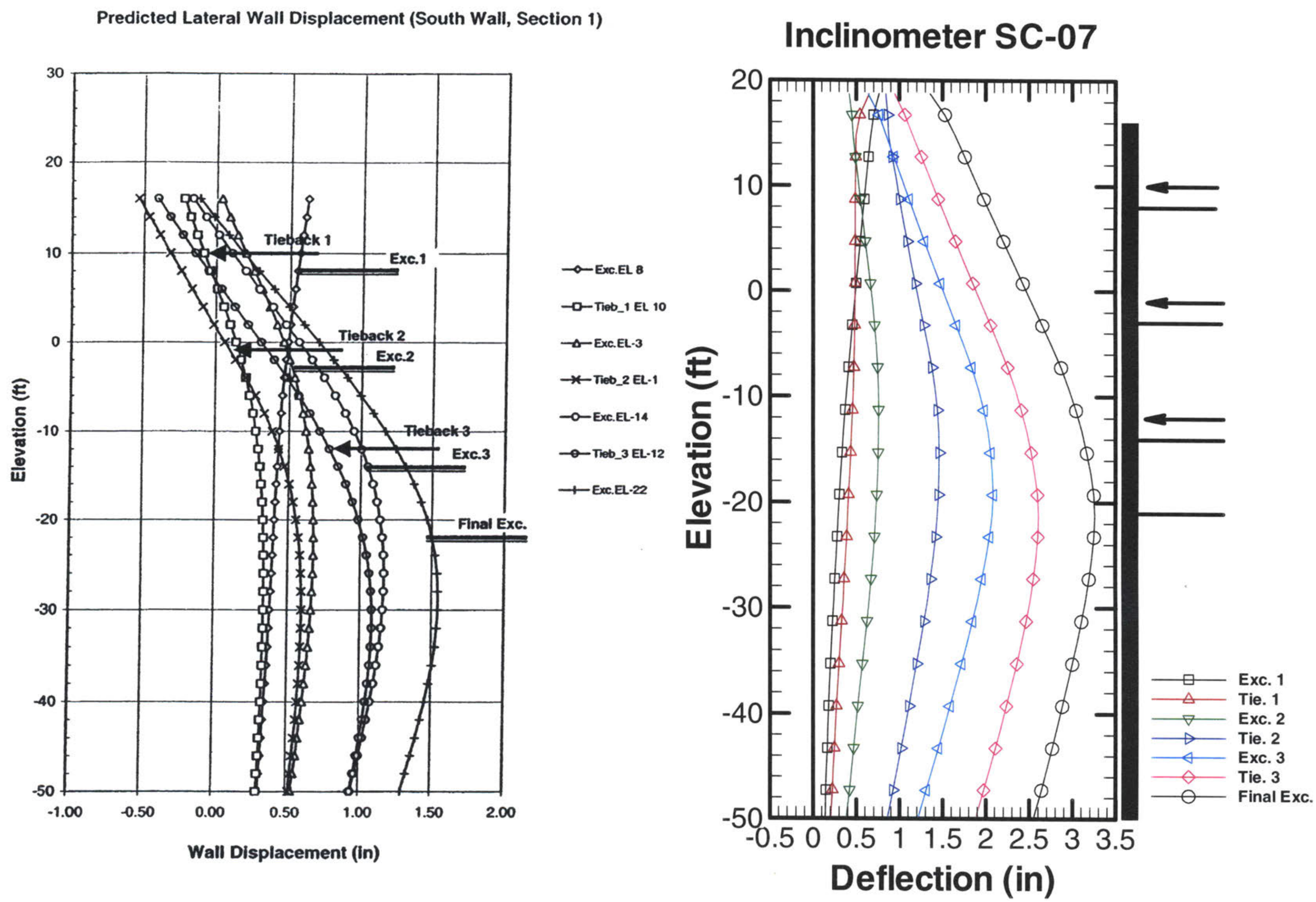


FIGURE 6.2: Comparison of 2D Finite Element predictions of horizontal wall movements (Weidlinger) during design (left), and measurements of actual wall movements (right) for the tieback anchor supported section of the South Wall.

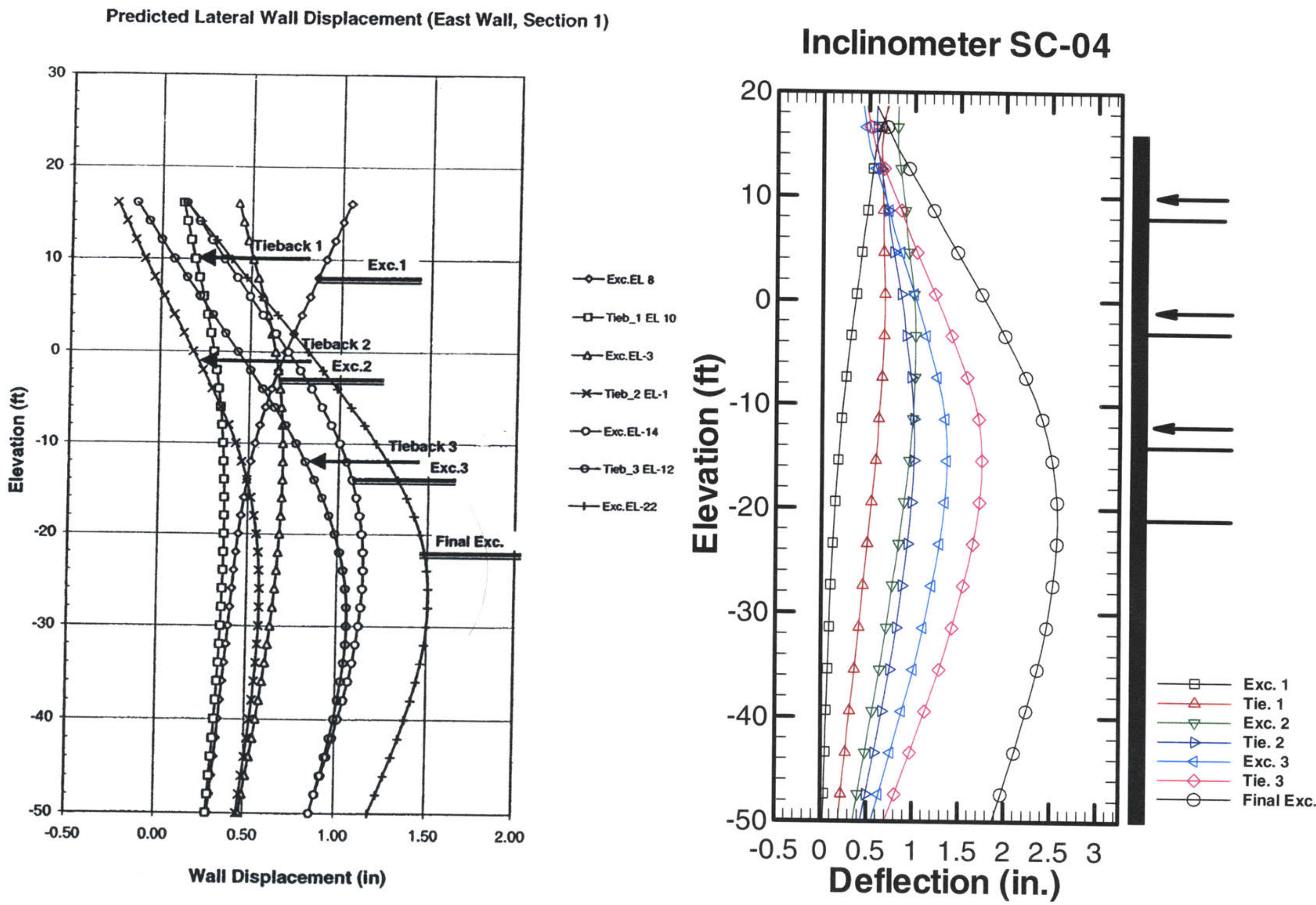


FIGURE 6.3: Comparison of 2D Finite Element predictions of horizontal wall movements (Weidlinger) during design (left), and measurements of actual wall movements (right) for the tieback anchor supported section of the East Wall.

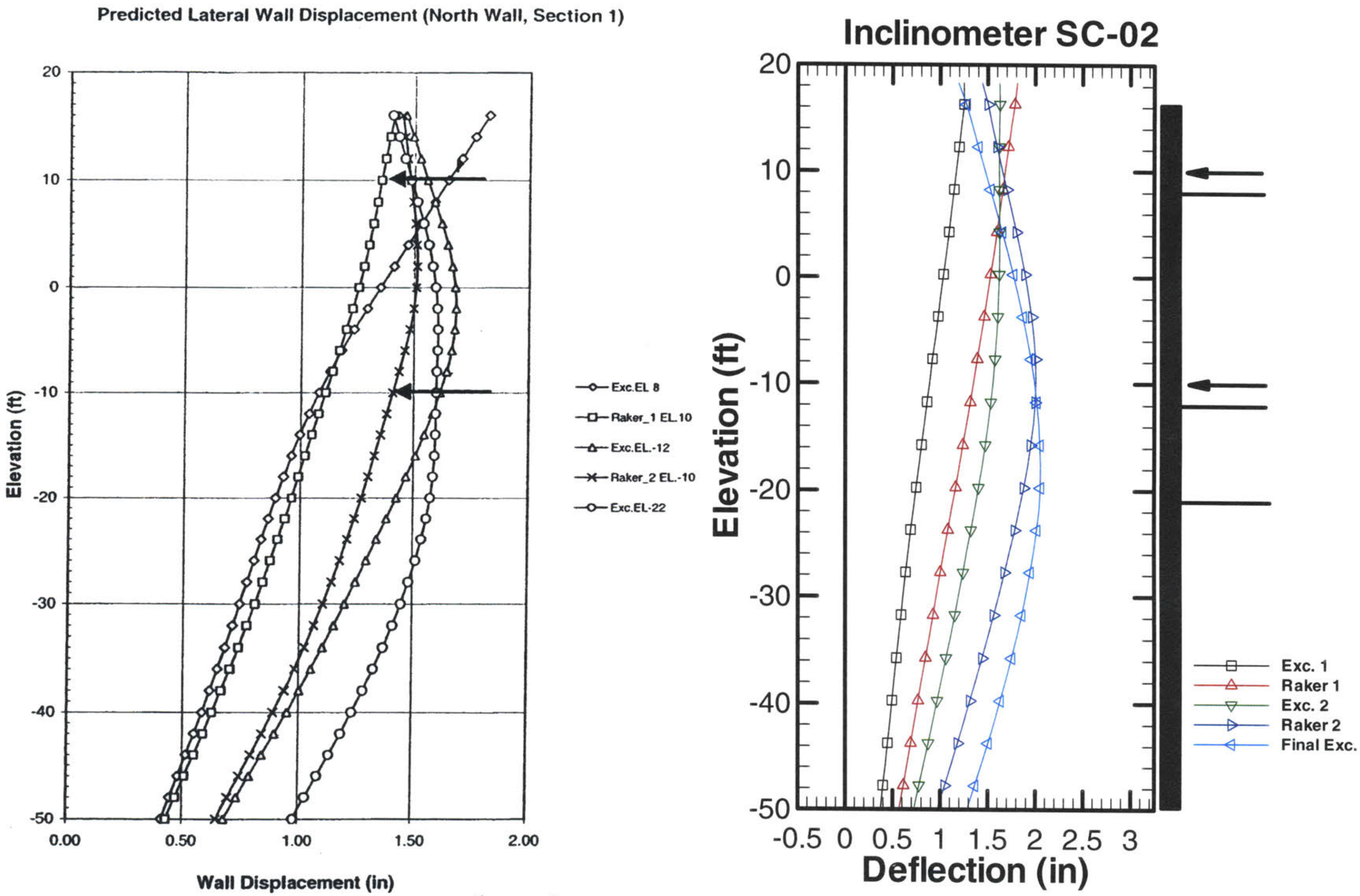


FIGURE 6.4: Comparison of 2D Finite Element predictions of horizontal wall movements (Weidlinger) during design (left), and measurements of actual wall movements (right) for the raker supported section of the North Wall.

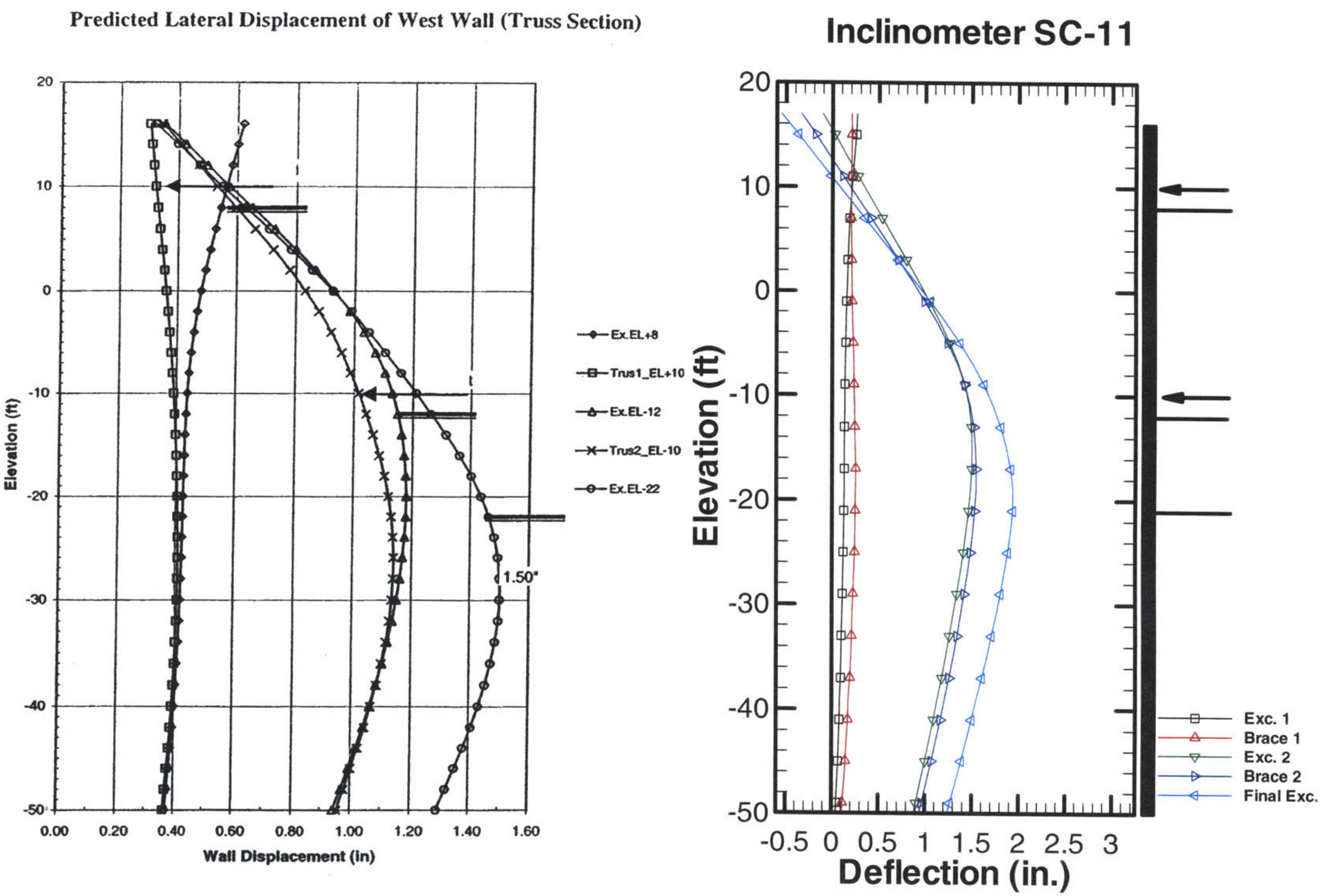


FIGURE 6.5: Comparison of 2D Finite Element predictions of horizontal wall movements (Weidlinger) during design (left), and measurements of actual wall movements (right) for the corner bracing section of the Northwest Corner.

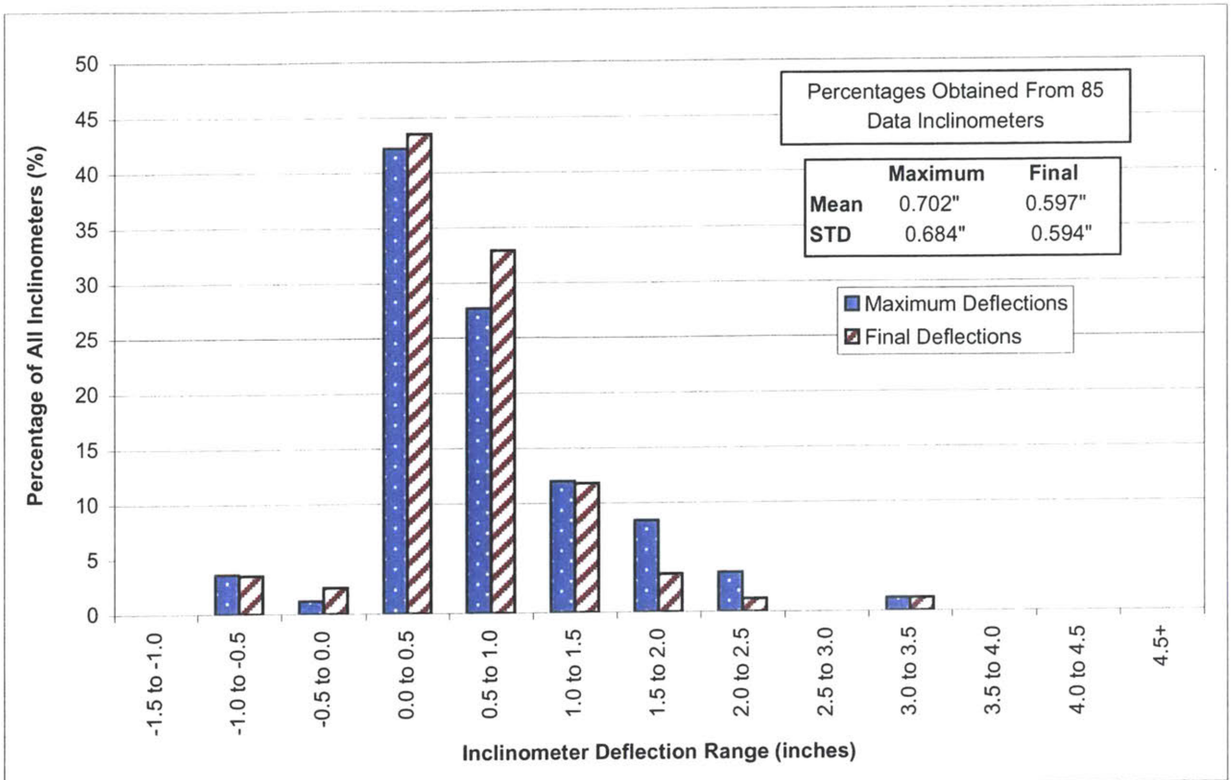


FIGURE 6.6: Statistics of maximum and final inclinometer deflections for all inclinometers in slurry wall project in Boston (Konstantakos, 2000).

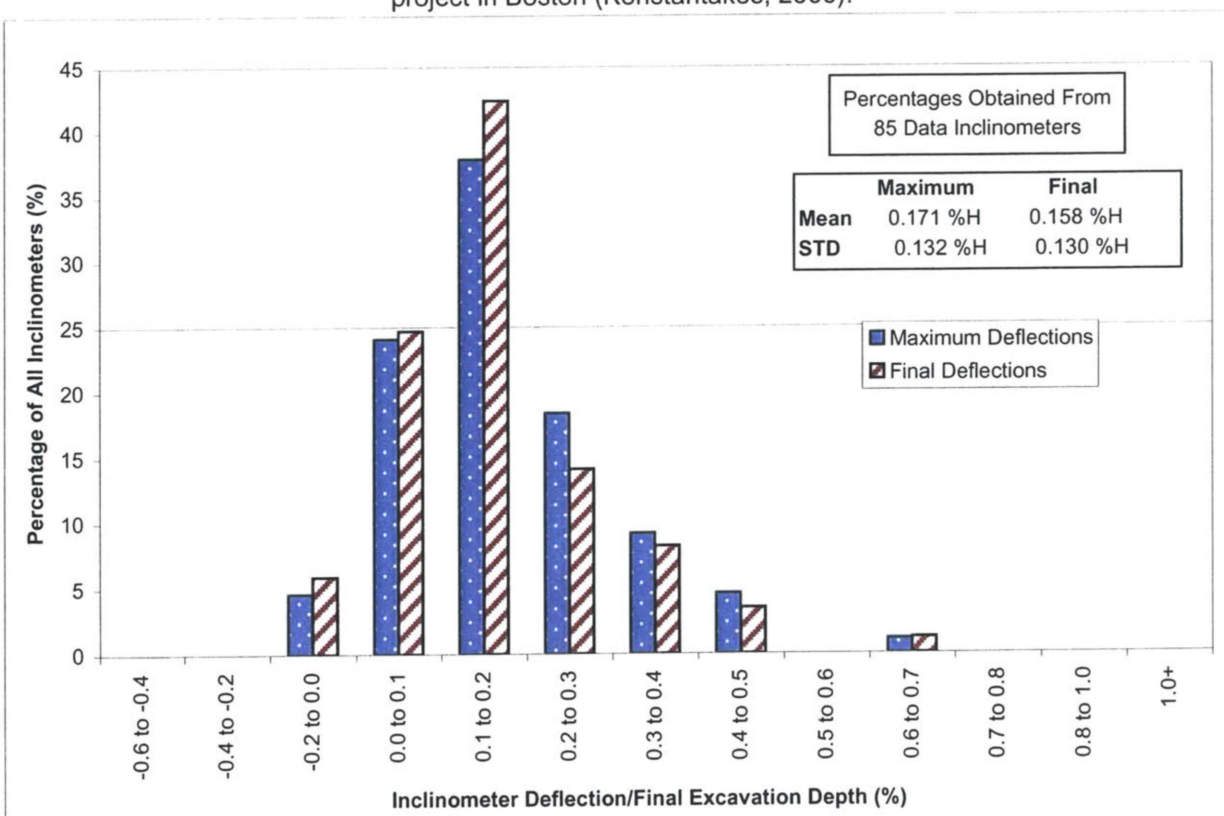
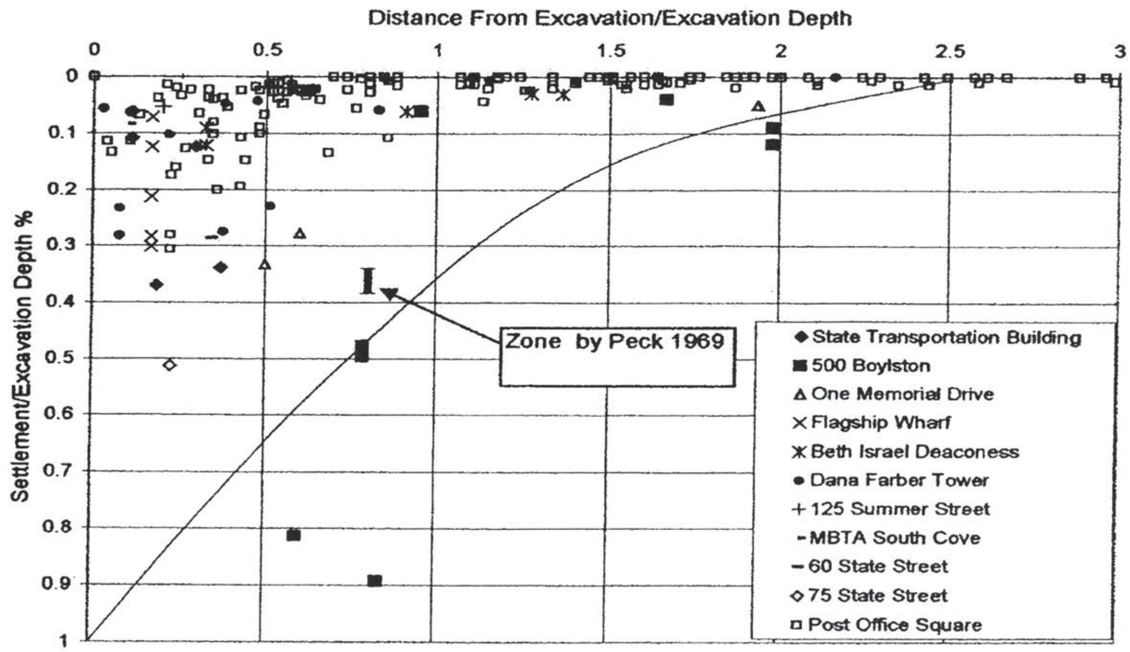


FIGURE 6.7: Statistics of maximum and final inclinometer deflections as percentages of the excavation depth for all inclinometers in slurry wall project in Boston (Konstantakos, 2000).



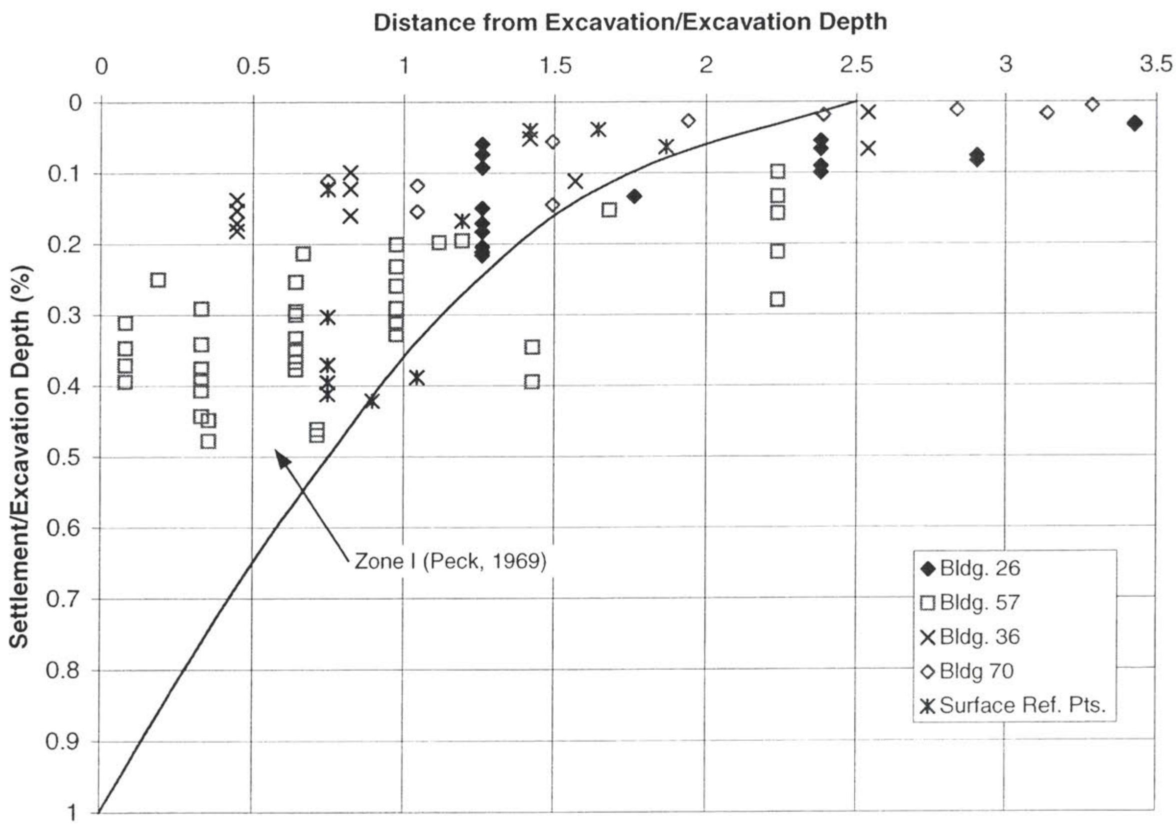


FIGURE 6.10: Settlements at distances from the excavation, standardized by the excavation depth. Top: Boston slurry wall excavations (Konstantakos, 2000). Bottom: Stata Center Project, 2001.

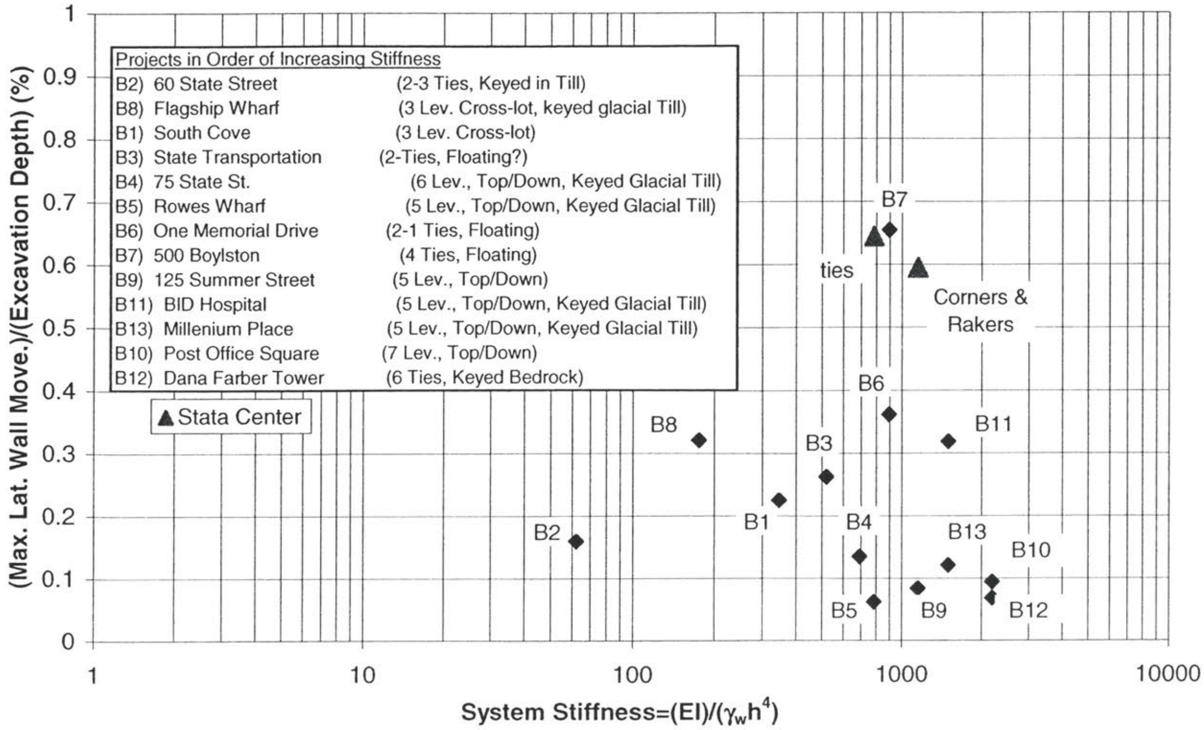


FIGURE 6.8: Maximum wall deflections for Boston projects (Konstantakos, 2000) and Stata Center plotted according to Clough's system stiffness approach (1989).

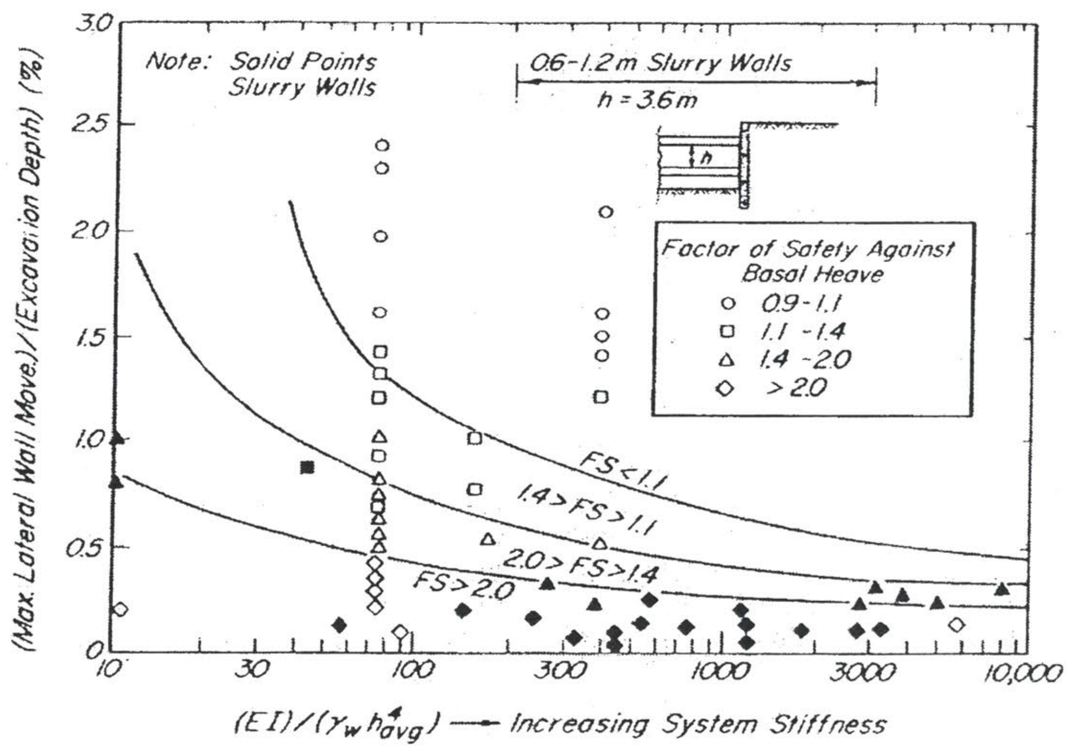


FIGURE 6.9: Wall deformations/Excavation depth versus system stiffness and factor of safety against basal heave (Clough et al., 1989). Terzaghi, Peck, & Mesri (1996) data plotted.

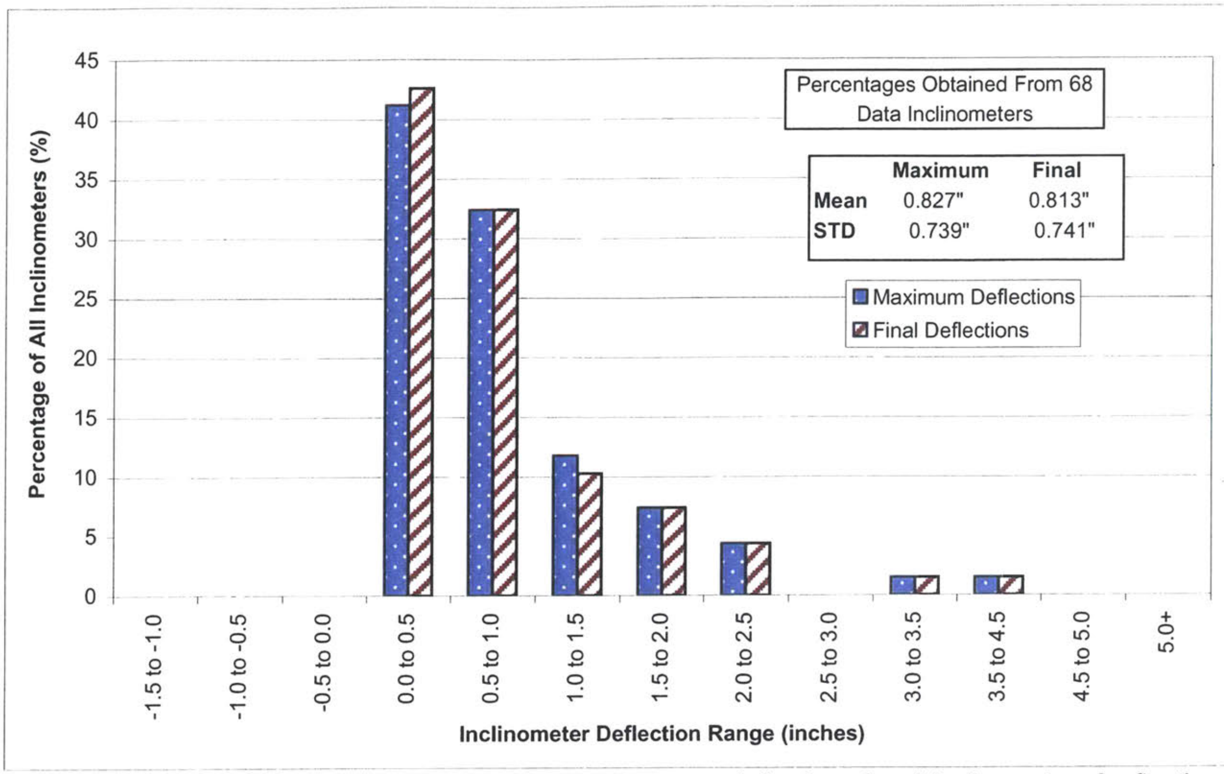


FIGURE 6.11: Statistics of maximum and final inclinometer deflections for all inclinometers for floating slurry wall excavation in Chicago & Boston (Konstantakos, 2000).

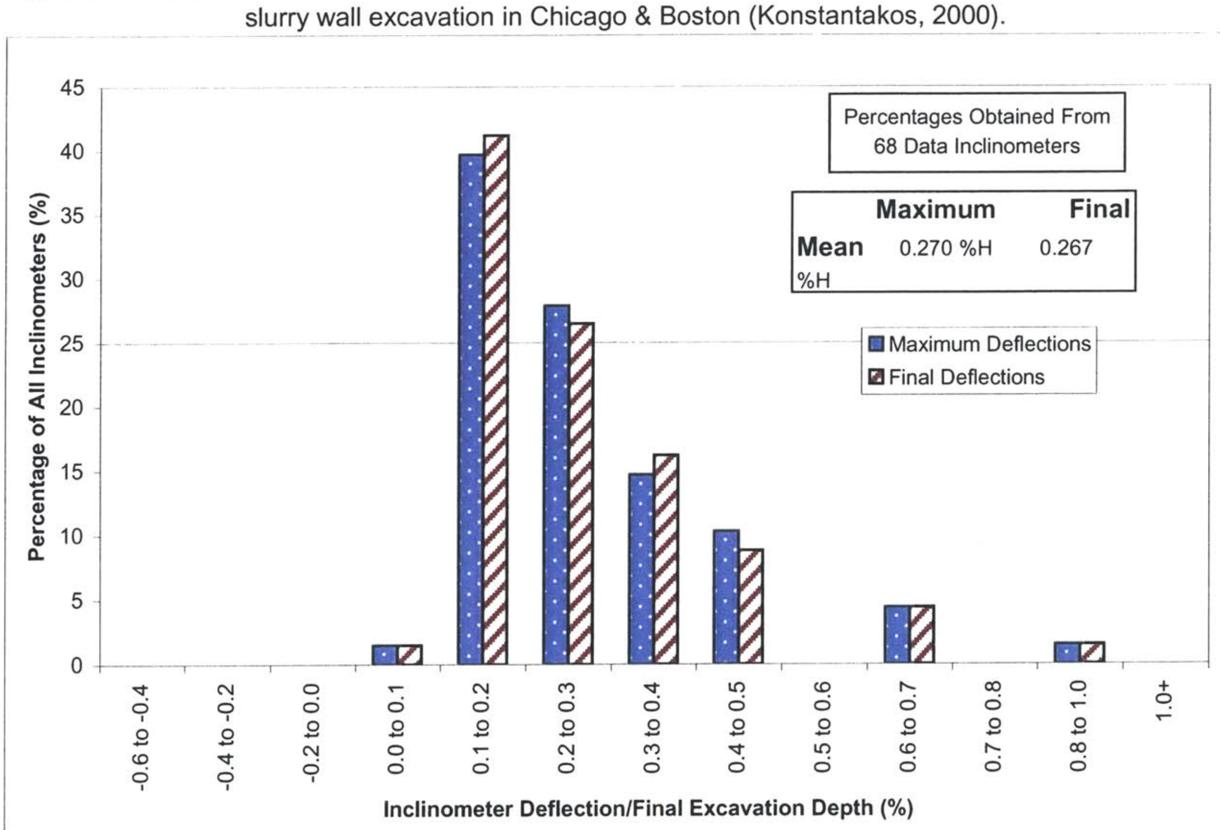


FIGURE 6.12: Statistics of maximum and final inclinometer deflections as percentages of the excavation depth for floating slurry wall excavations in Chicago & Boston (Konstantakos, 2000)

Chapter 7
Summary and Conclusions

7.1 Summary

Ground movements play an important role in the deep excavations in cohesive soils. This thesis studies the large excavation for the basement of the new Ray and Maria Stata Center on the Massachusetts Institute of Technology Campus in Cambridge, MA. The purpose of this study is to produce a case history that accurately presents the key portions of the project including the design of the lateral earth support system, the construction history, and the measured performance of the excavation support system. The performance of the excavation supports was monitored through a program of geotechnical instrumentation. Movements of the wall and soil mass were of keen interest due to the proximity of surrounding buildings including the 60 year old Alumni Pool building located 3 ft from the proposed excavation.

Chapter 3 provided a description of the site history along with a summary of the site investigations that were used to determine the subsurface stratigraphy and the engineering properties of the soil. The site is an area of reclaimed land on the north bank of the Charles River that was previously occupied by a three story wooden building. The soil profile is characteristic of the area and comprises 1) a granular fill that was placed when the area was reclaimed a century ago that varies from 8 to 19 ft in thickness, 2) a soft organic silt and peat that varies from 4 to 22 ft in thickness, 3) a marine sand that varies in thickness from 0.5 ft to 16.5 feet, 4) a marine clay (Boston Blue Clay) that varies from 60 to 90 ft in thickness below the site and varies from 20 to 100 ft within a short distance, 5) a heterogeneous layer of glacial till that is of varying thickness, and 6) below the till is bedrock that is part of the Cambridge Argillite formation. The dominant strata is the marine clay whose upper part is stiff and overconslidated due to desiccation and lower part is a soft, nearly normally consolidated clay that becomes more compressible with depth. This "soft" soil is the controlling factor in the design and performance of the lateral support system.

Chapter 4 outlines the excavation limits and the choice and design of the support system. The dimensions of the excavation in plan are 386 ft by 316 feet and the final depth of excavation is approximately 42 ft. A 30" reinforced concrete diaphragm wall was chosen due to its higher stiffness (for reducing estimated movements), water tightness, and most importantly its applicability as part of the permanent sub-structure. The wall has a total depth of approximately 70 feet and extends into the top of the clay layer and to a final embedment length of 30 ft (below basement slab). After considering many combinations of support systems, it was decided that the wall would be supported by two levels of corner bracing (in each of the corners), three levels of tiebacks on three sides of the excavation, and two levels of rakers on the north side. The choice of rakers was dictated by the potential interference of utilities under Vassar Street on the north side of the site.

The construction history and performance of the excavation support was presented in chapter five. This information was compiled from daily field reports, instrumentation data files, and personal observations. The progress of the excavation and any difficulties encountered were well documented by Haley and Aldrich, Inc. Excavation work lasted for just over 6 months and entailed excavation on three sides for installation of the tiebacks, installation of a portion of the mat foundation, installation of rakers bracing the north wall off the foundation, and finally excavation of the soil berm that was left to support the wall.

Lastly, chapter 6 compared the data obtained from the geotechnical instrumentation to the expected performance and to other diaphragm wall supported excavations in the Boston area and similar projects in around the country.

7.2 Conclusions

The principal method for evaluating the performance of the excavation support system was an analysis of the movements of the soil mass and diaphragm wall as shown by a series of 11 vertical inclinometers (within the wall), and by surface and building settlement points, and the borehole extensometers all of which were installed prior to the beginning of excavation. This performance was then compared with the limiting values set forth at the start of the project and expected performance based on empirical methods, for similar projects, and numerical calculations used in design of the project.

The deformations exceeded the limiting value of 1.5" that was set for horizontal wall movements and surface settlements. This limit was set to minimize damage that could occur to surrounding buildings. The maximum lateral movements measured varied from 2" to 3.5" and the maximum settlement was measured at over 2.3". These values did not cause any noticeable damages and therefore were eventually deemed acceptable. It would be expected that movements of this magnitude would cause damage to adjacent structures. When these movements were compared to similar projects, it was found that, strictly according to the movements, the project performed rather poorly and indeed closely resembles the experience measured in excavations at 500 Boylston St. (in the early 1980s). On the other hand, the Stata Center project also demonstrates that evaluating the performance of an excavation support system based purely on the magnitude of ground movements is quite misleading. The effects that these movements have on adjacent structures and on the safety of the project remain much more important. Excavation of the Stata basement has caused minimal damage to surrounding facilities, but modifications to the lateral earth support system (notably the installation of concrete buttresses) were necessary due to perceptions that the support stability was marginal.

Other than the large movements the support system performed as would be expected with mode shape of the wall deflections being consistent with the method of support employed improved predictions of the wall and ground movements require more detailed analysis techniques that model accurately the properties of the underlying soil.

7.3 Recommendations

This thesis attempted to present the Stata project in a manner that first, accurately relates the details of the project from support characteristics to construction history and performance and second, lends itself to use as a test case for new analysis techniques. This project is an ideal project for three dimensional analysis techniques because of its physical dimensions, variability in soil stratigraphy, variety of structural supports, and excavation sequence. This project showed to the author that the gathering and interpretation of concurrent activities that occurred during construction is not an easy task, but it is essential to the analysis of a project of this magnitude whether it be by finite element analysis or some other technique. It also showed that many of the common assumptions used in empirical design techniques and two dimensional numerical analyses may not be entirely accurate.

Due to these factors, work should continue on the development of methods for accurately compiling and archiving large well instrumented excavation projects and the development of new analysis techniques (such as 3D analysis) that will accurately account for the method and sequence of construction.

Chapter 8
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APPENDIX A: SOIL INFORMATION

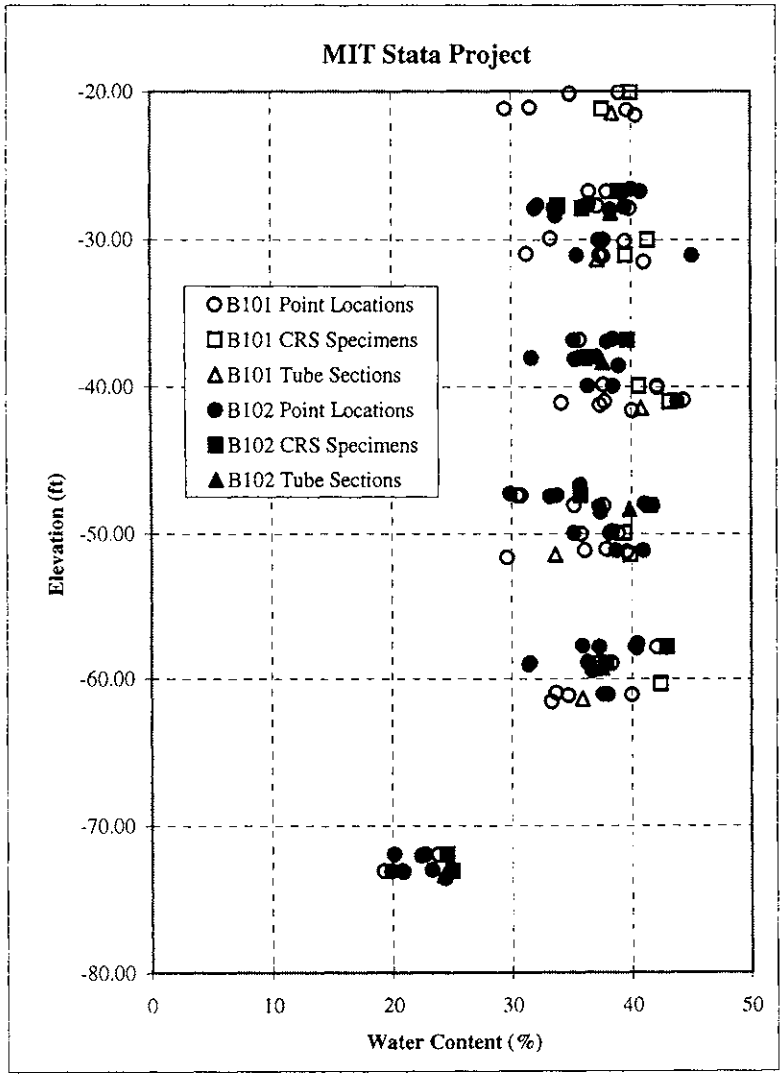


FIGURE A-1: Water Content vs. Elevation

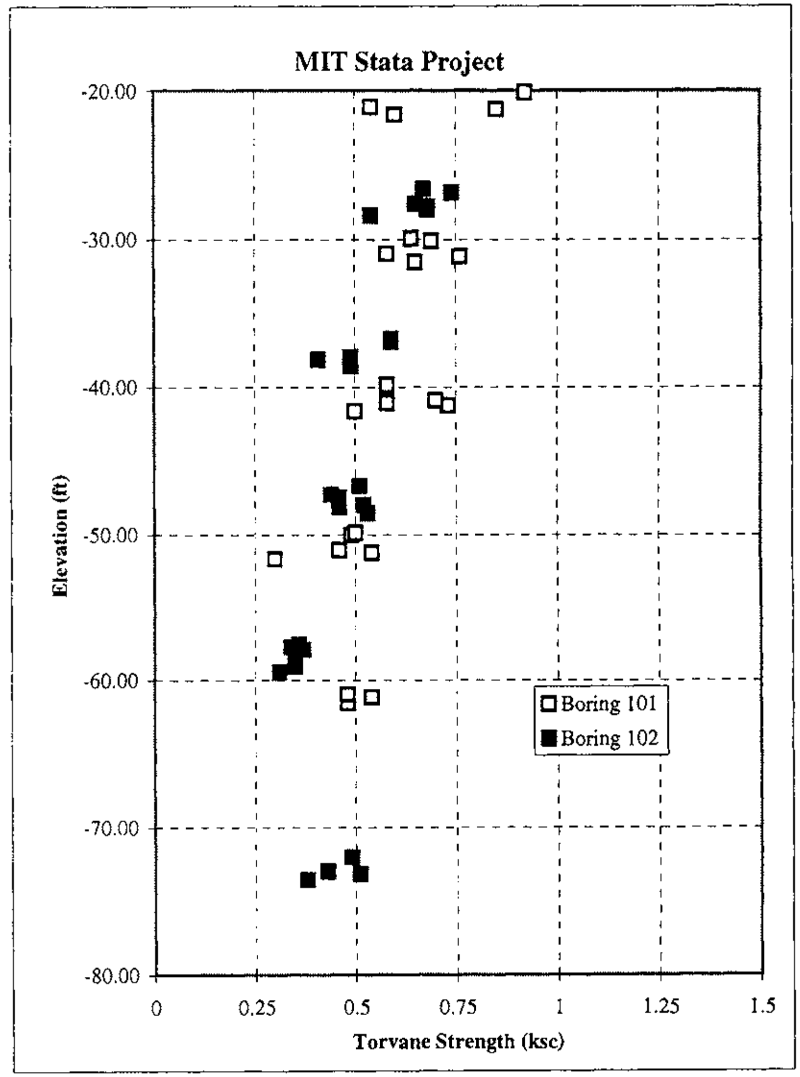


FIGURE A-2: Torvane Stregth vs. Elevation

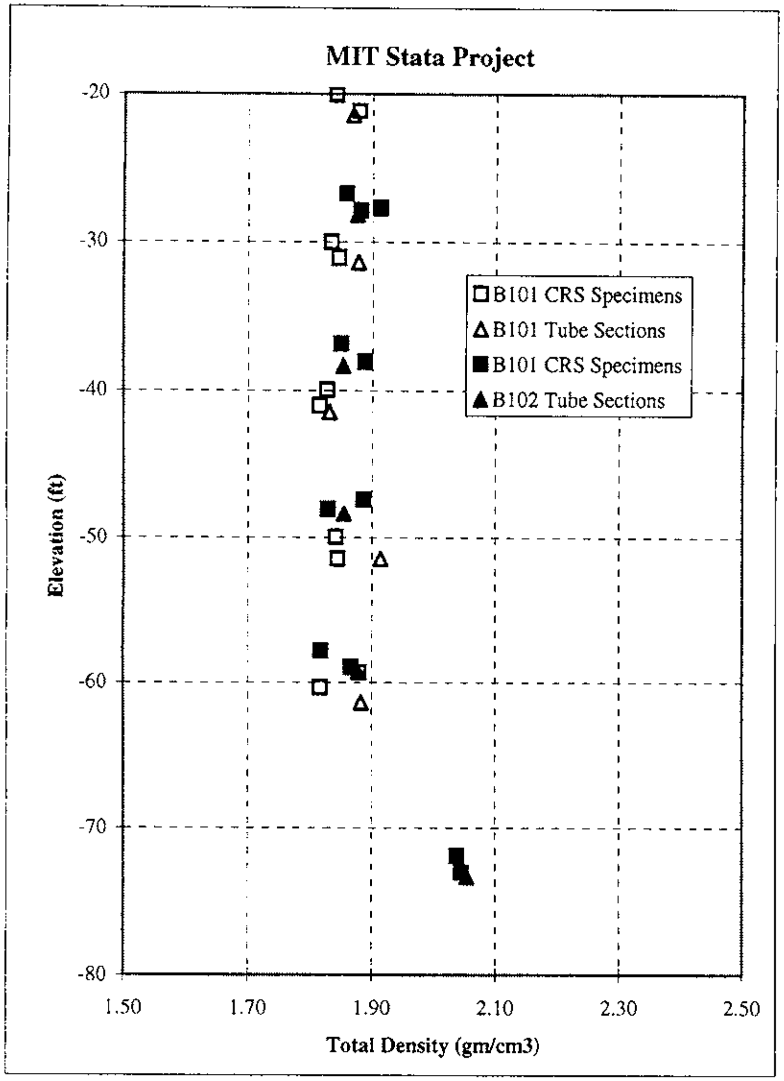


FIGURE A-3: Total Density vs. Elevation

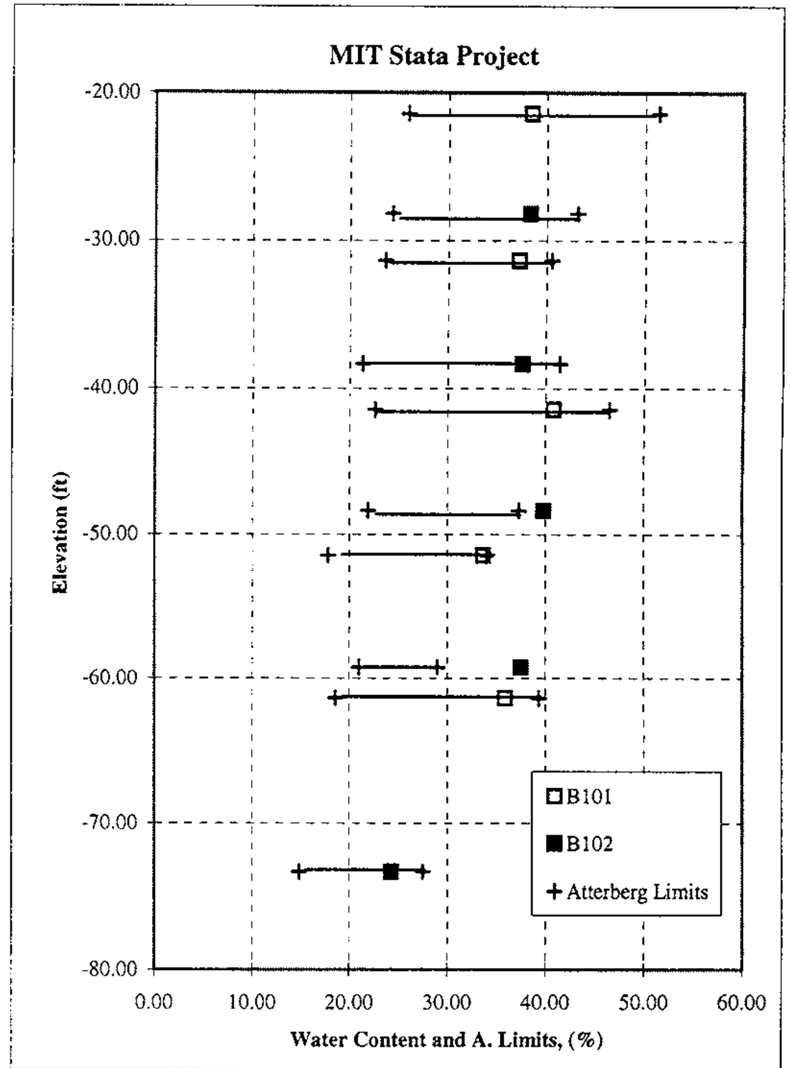


FIGURE A-4: Water Content and Atterberg Limits vs. Elevation

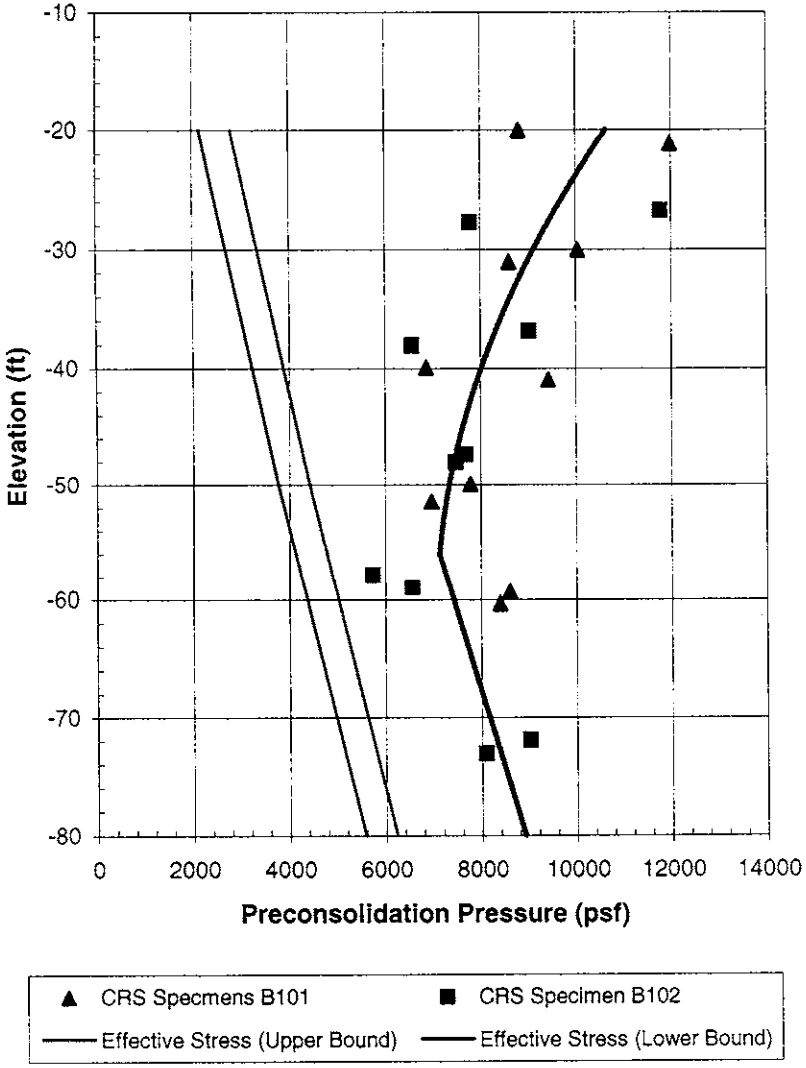


FIGURE A-5: Preconsolidation Pressure vs. Elevation

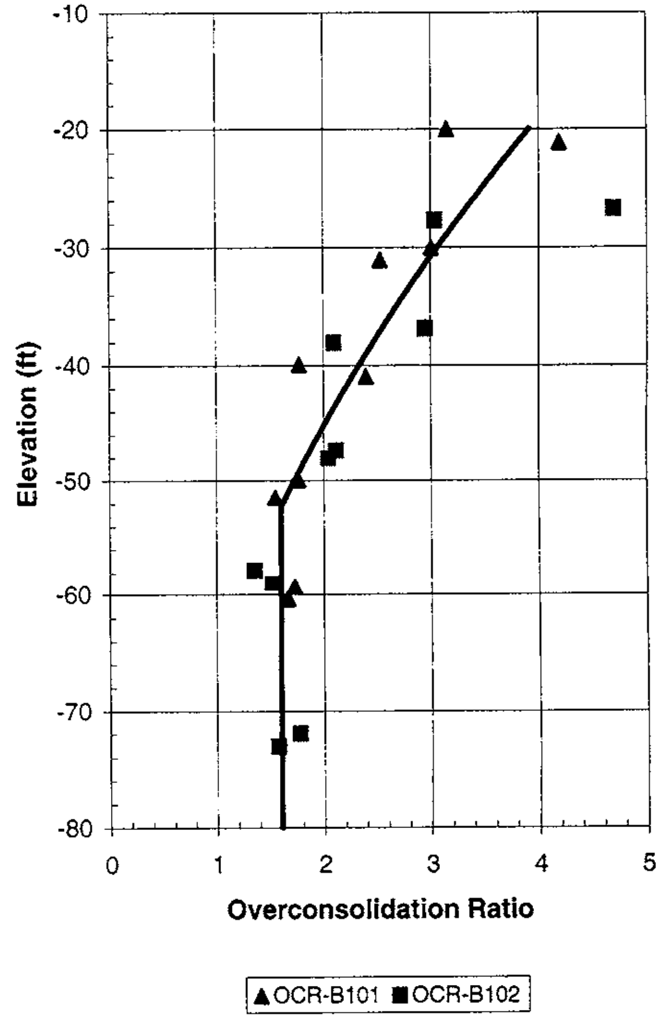


FIGURE A-6: Overconsolidation Ratio vs. Elevation

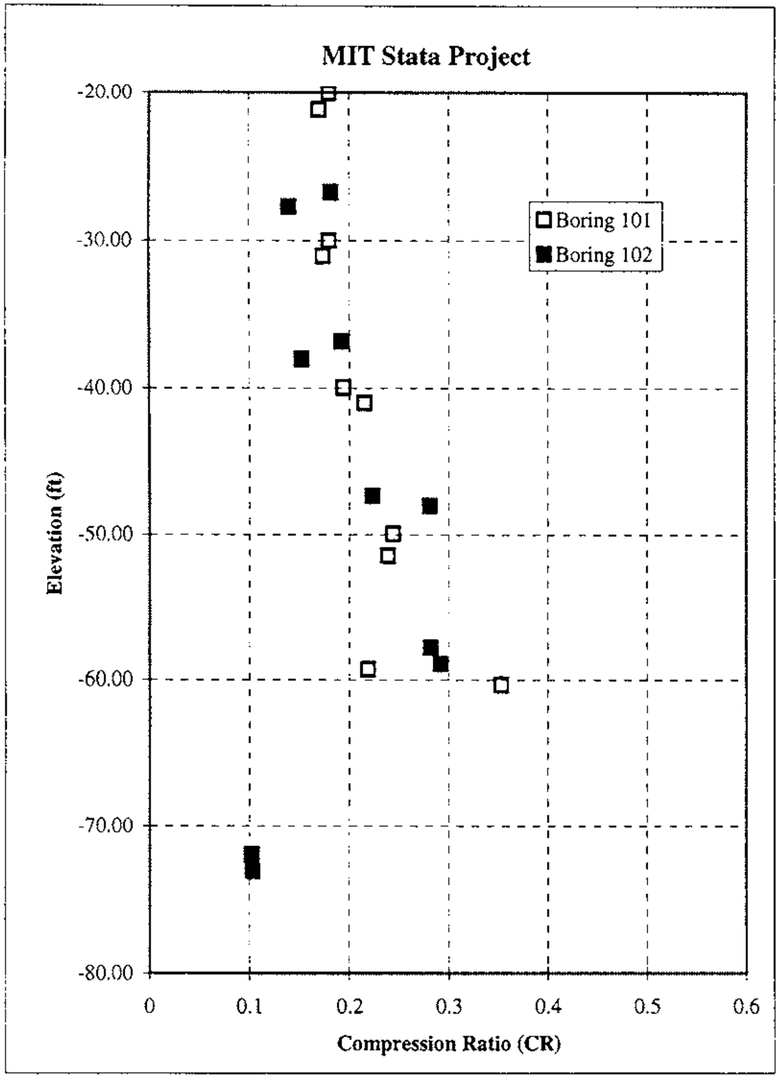


FIGURE A-7: Compression Ratio vs. Elevation

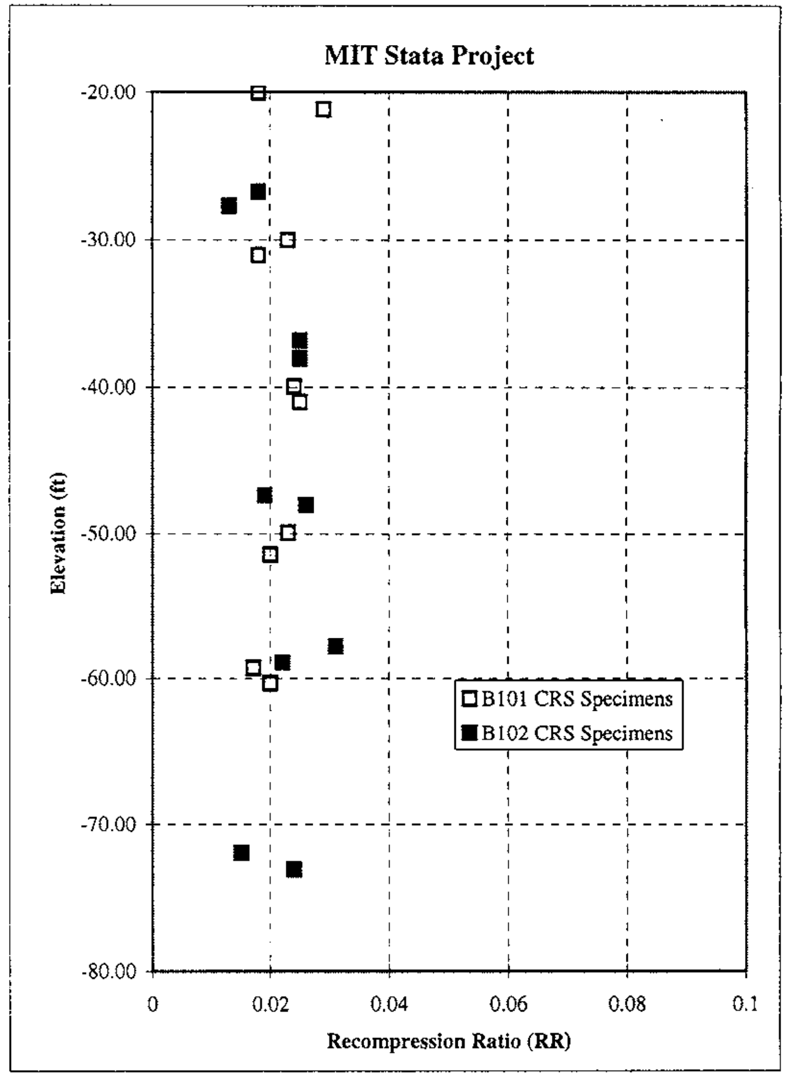


FIGURE A-8: Recompression Ration vs. Elevation

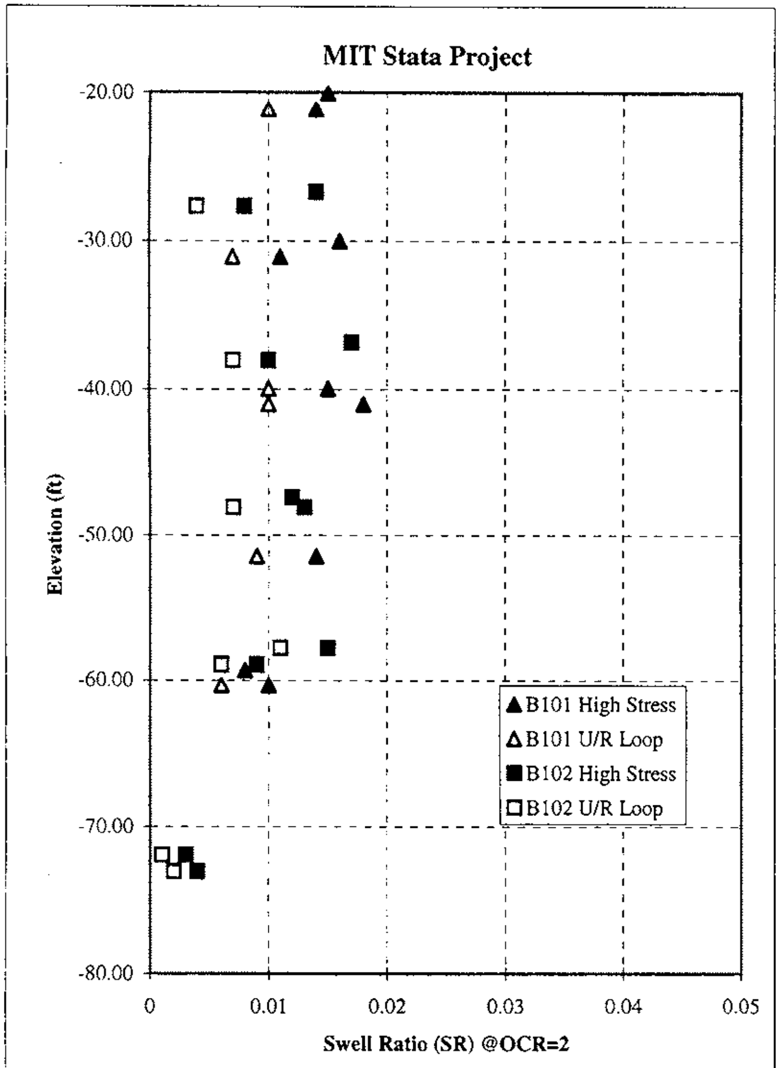


FIGURE A-9: Swell Ration at OCR=2 vs. Elevation

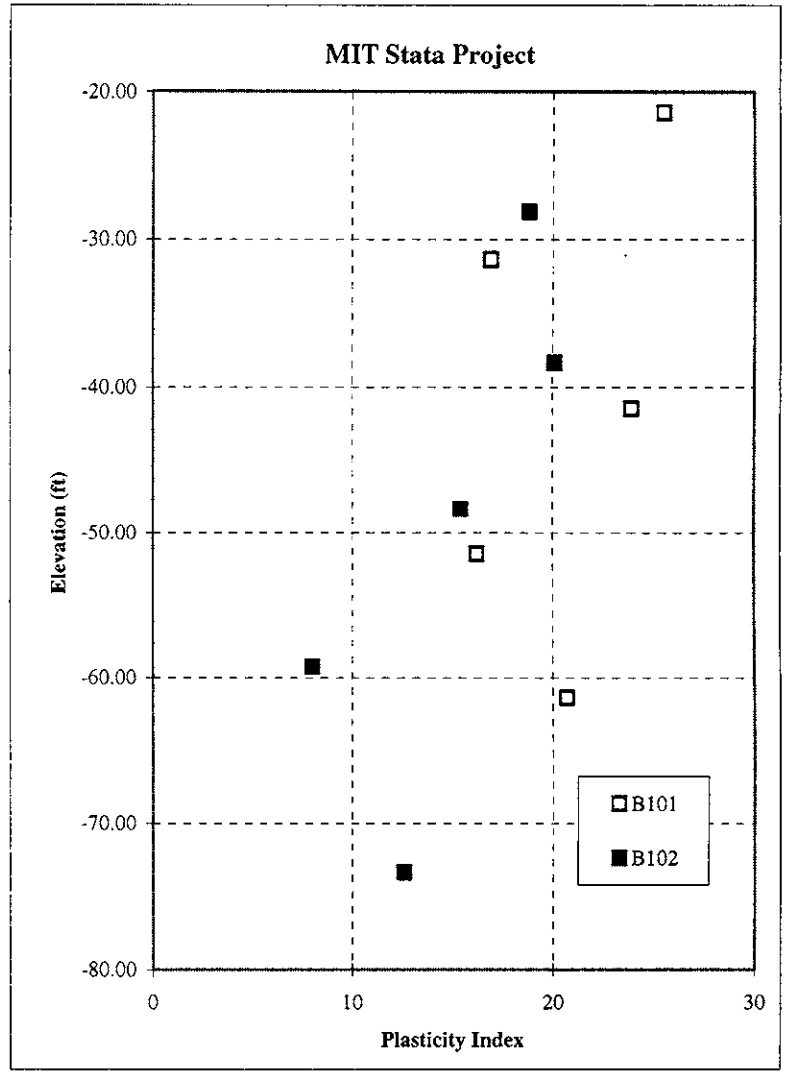


FIGURE A-10: Plasticity Index vs. Elevation

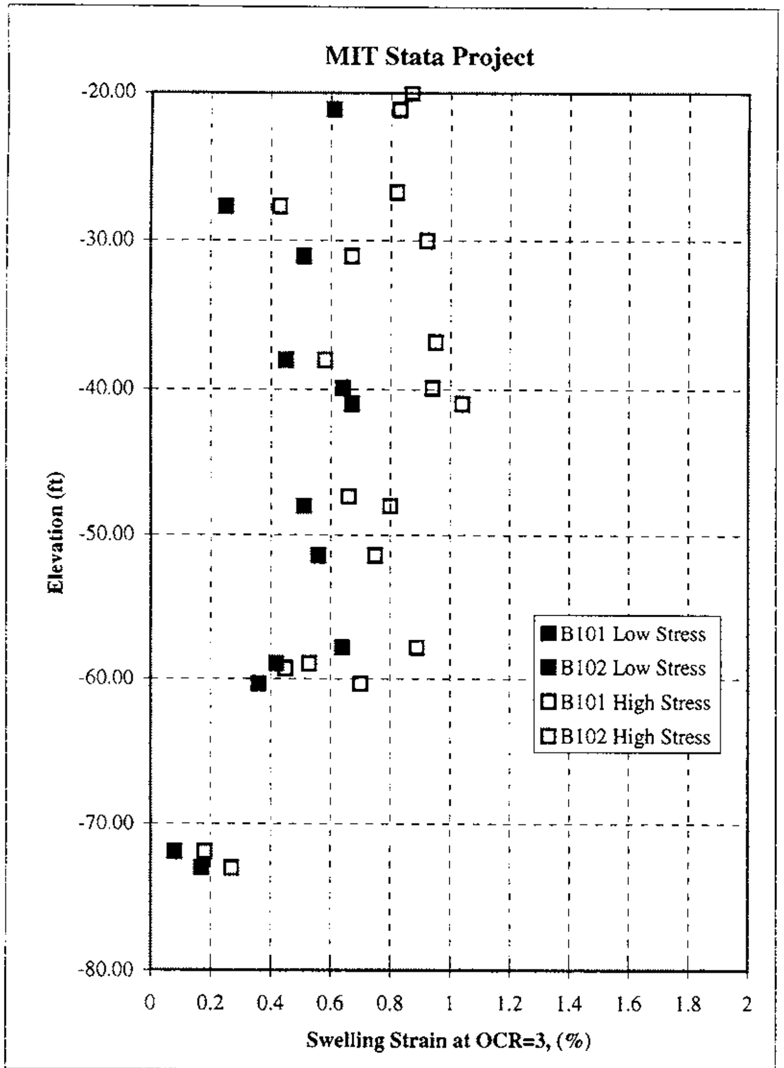


FIGURE A-11: Swelling Strain at OCR=3 vs. Elevation

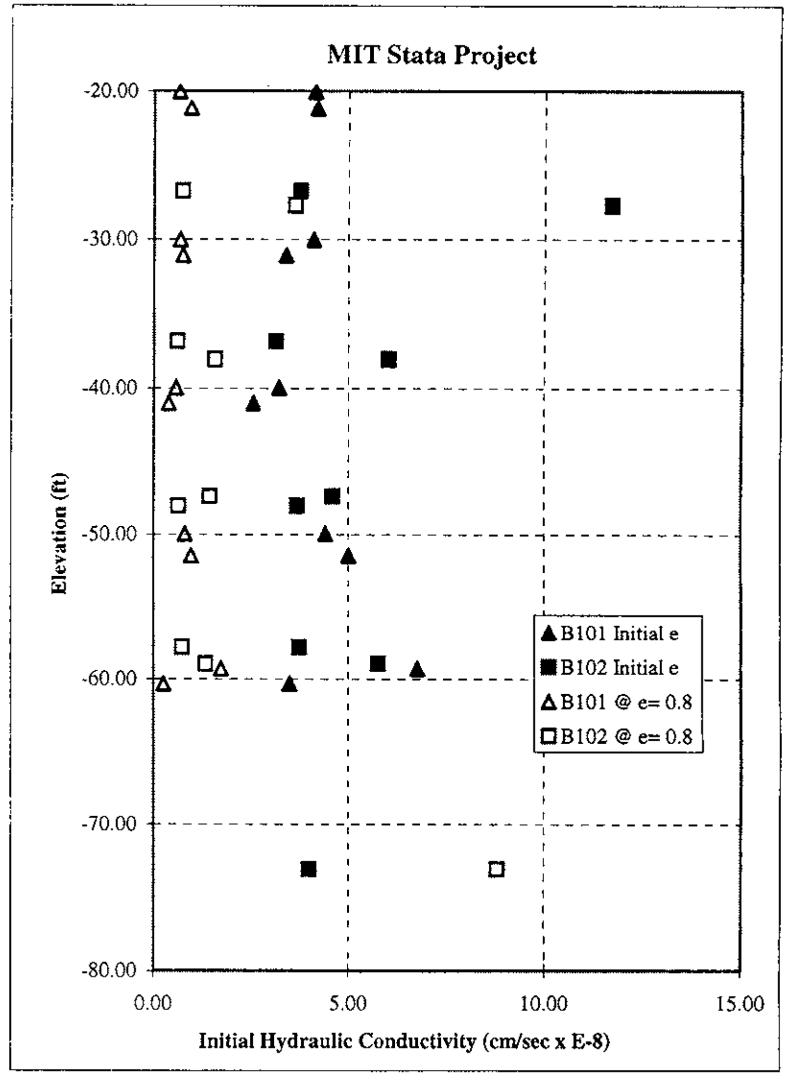


FIGURE A-12: Initial Hydraulic Conductivity vs. Elevation

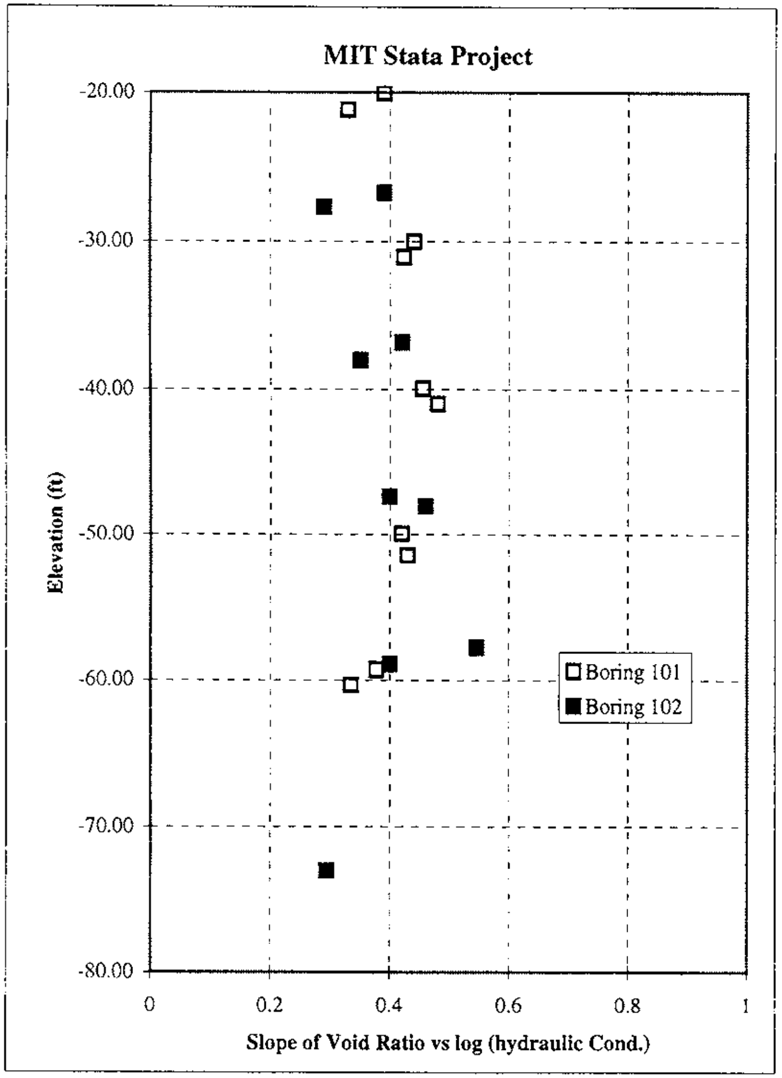


FIGURE A-13: Slope of Void Ratio vs. log(hyd. Cond.) vs. Elevation

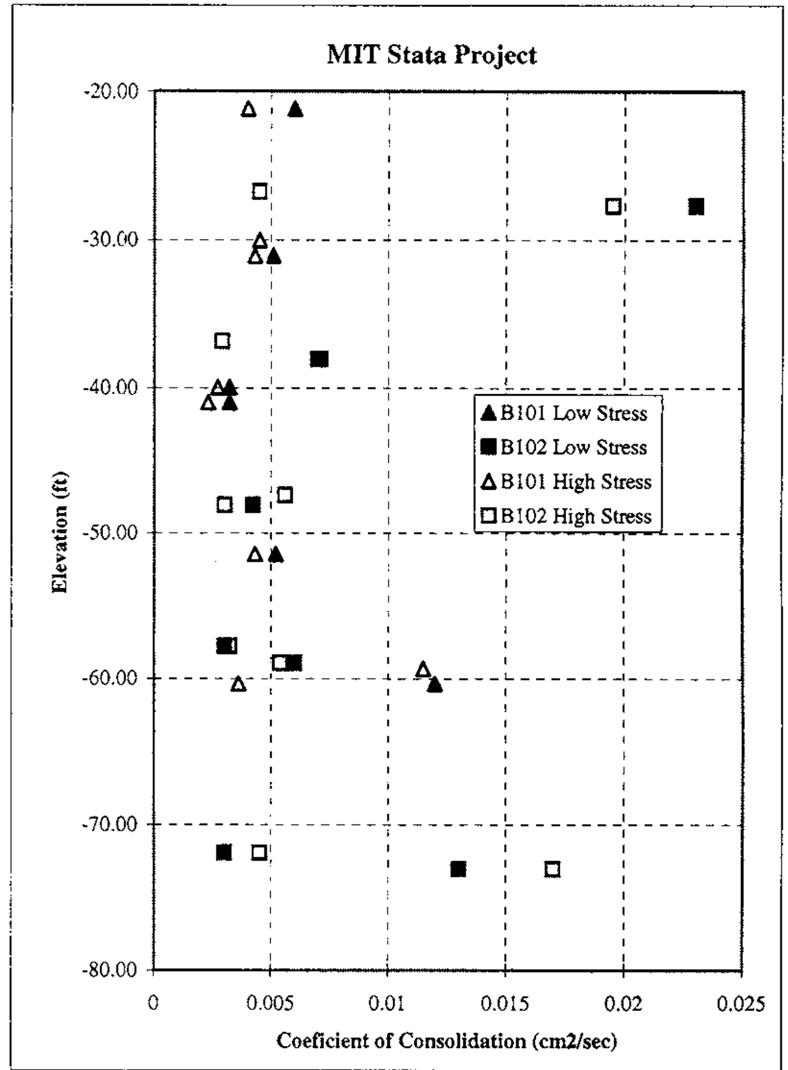


FIGURE A-14: Coefficient of Consolidation vs. Elevation

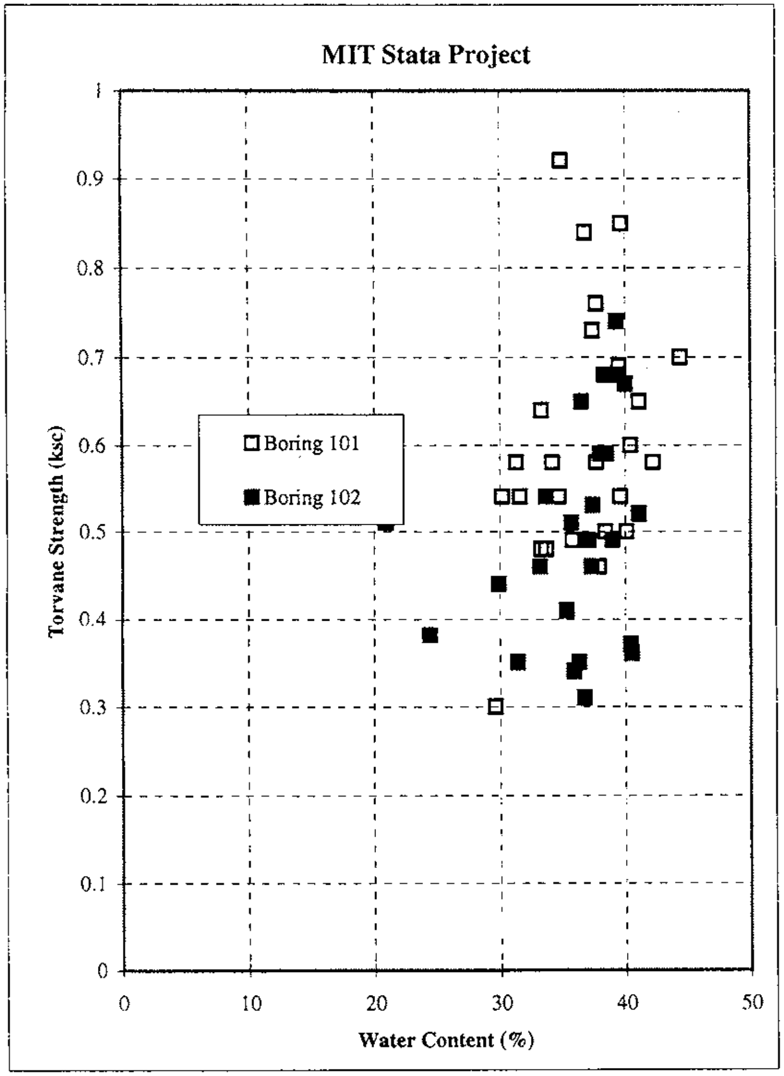


FIGURE A-15: Water Content vs. Torvane Strength

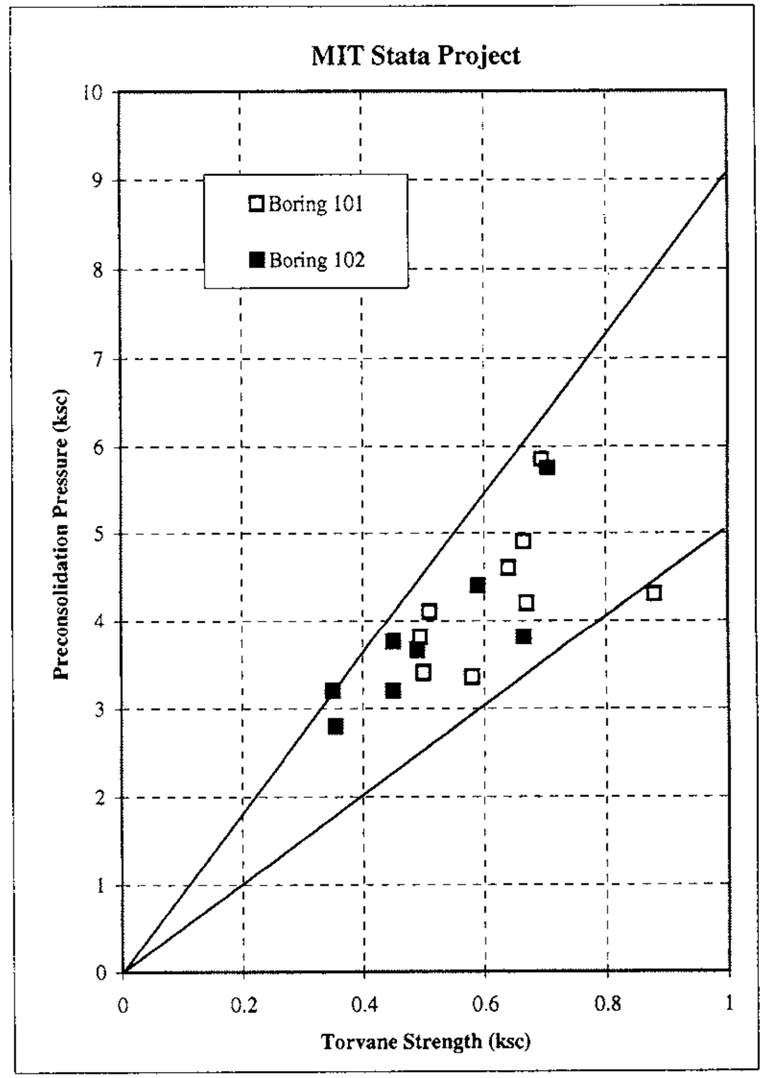


FIGURE A-16: Torvane Strength vs. Preconsolidation Pressure

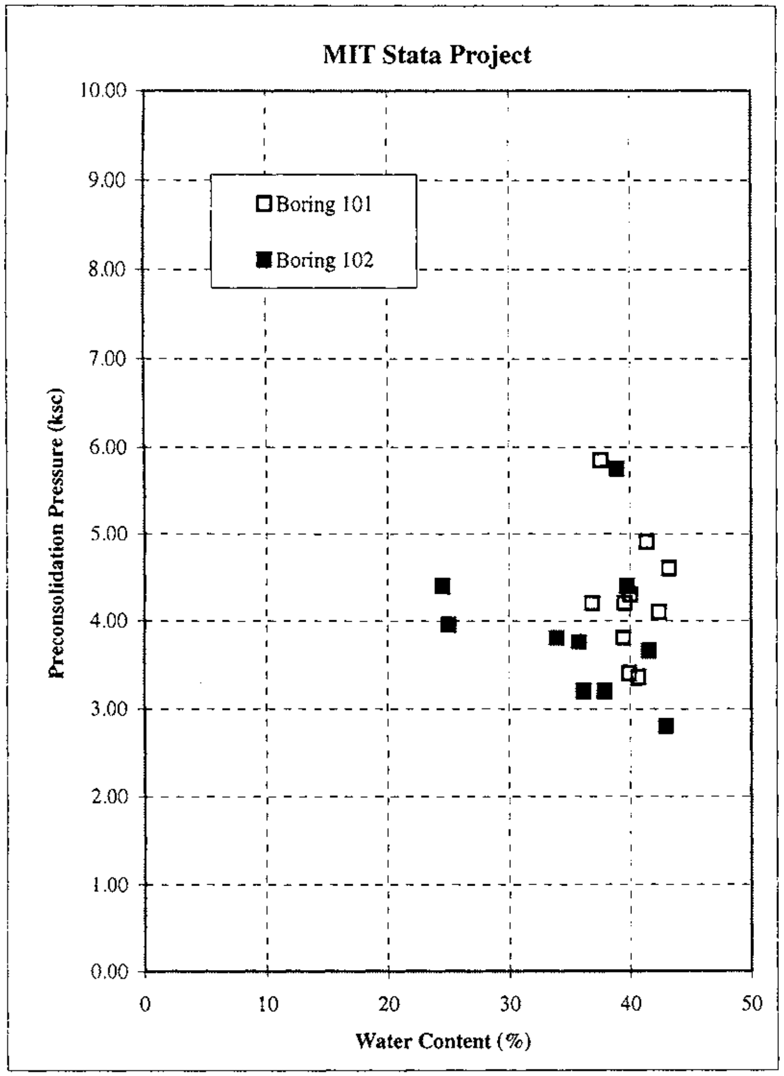


FIGURE A-17: Water Content vs. Preconsolidation Pressure

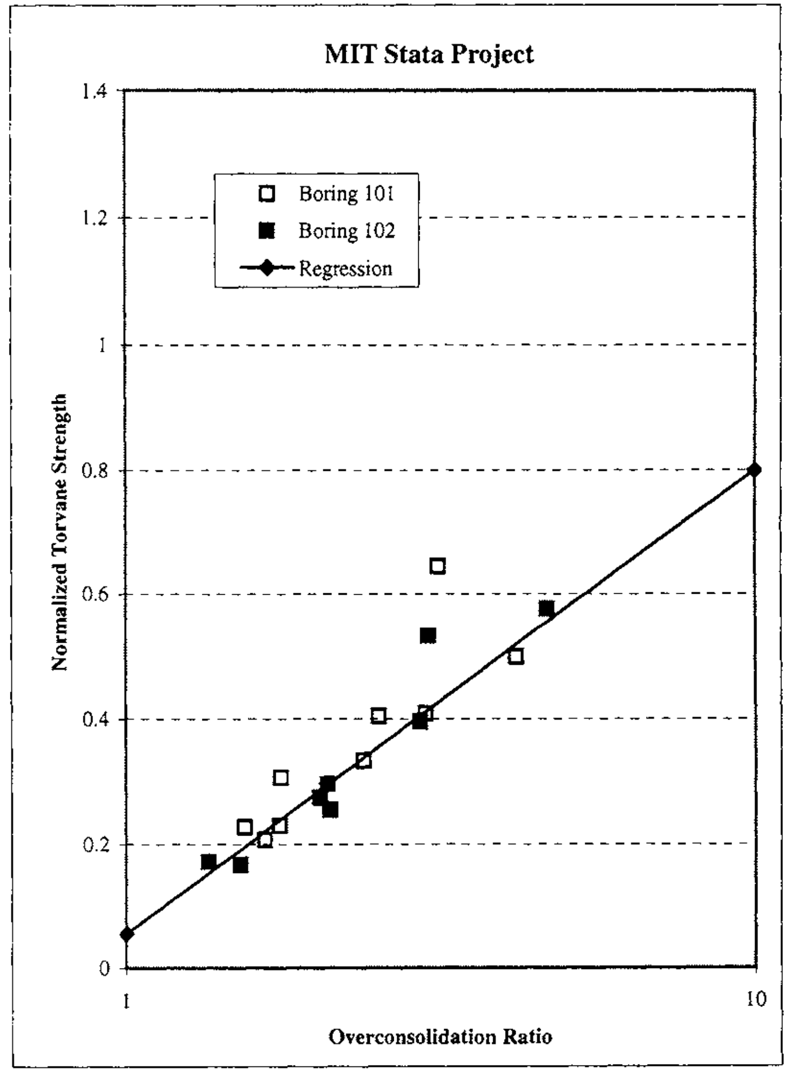


FIGURE A-18: Overconsolidation Ratio vs. Normalized Torvane Strength

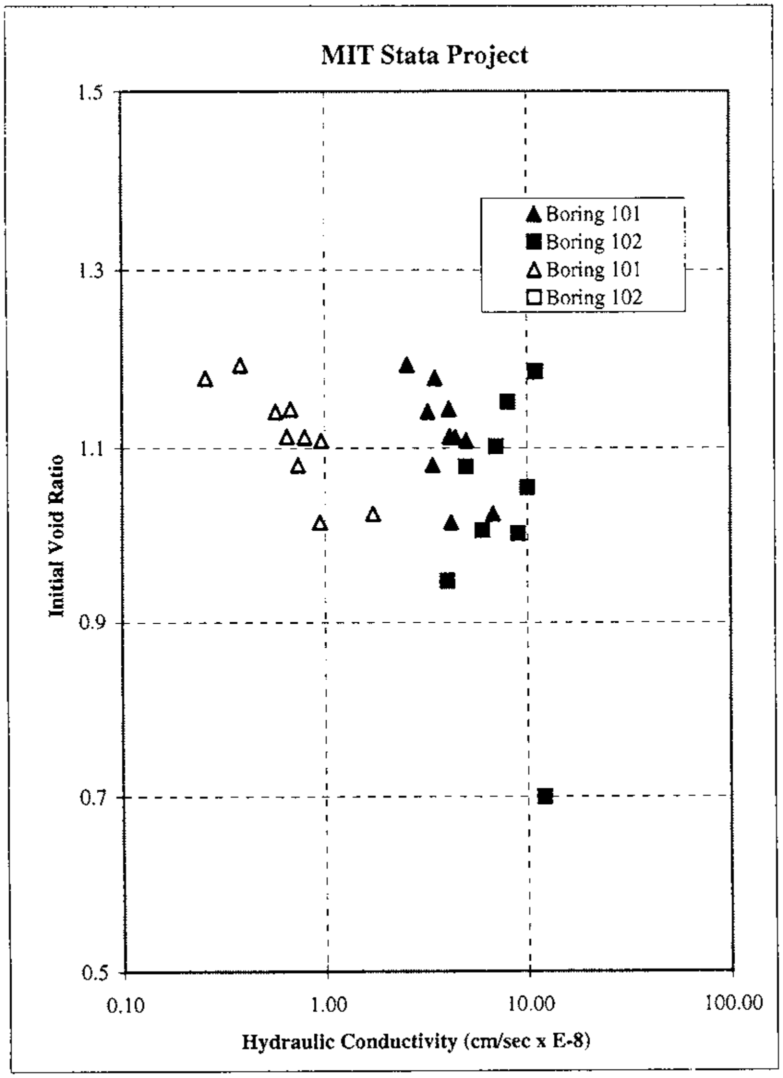


FIGURE A-19: Hydraulic Conductivity vs. Initial Void Ratio

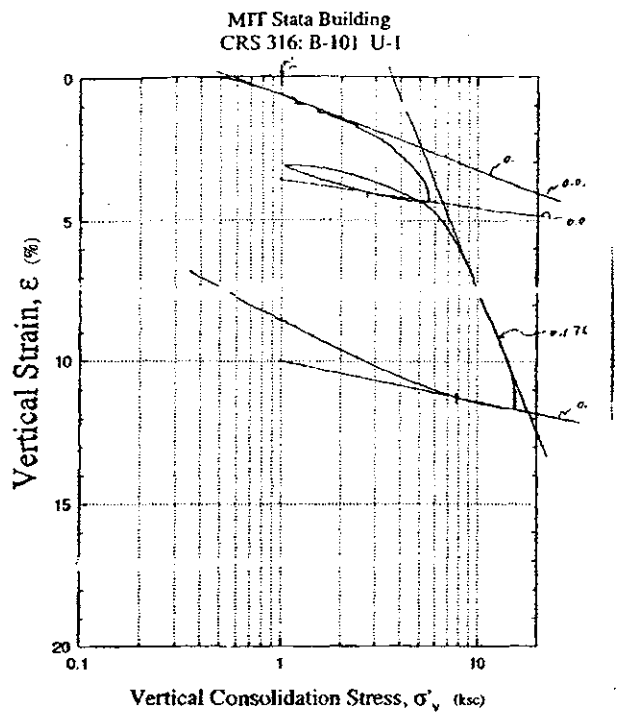


FIGURE A-20: CRS @ El. -21.16

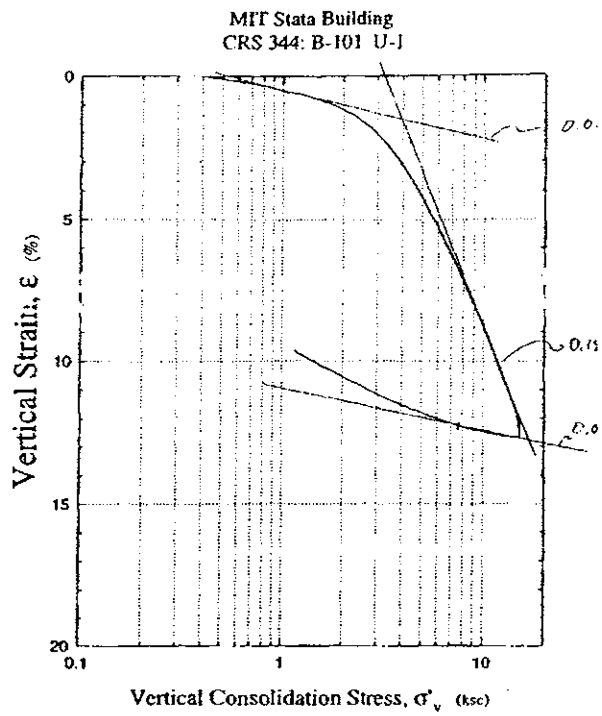


FIGURE A-21: CRS @ El. -20.03

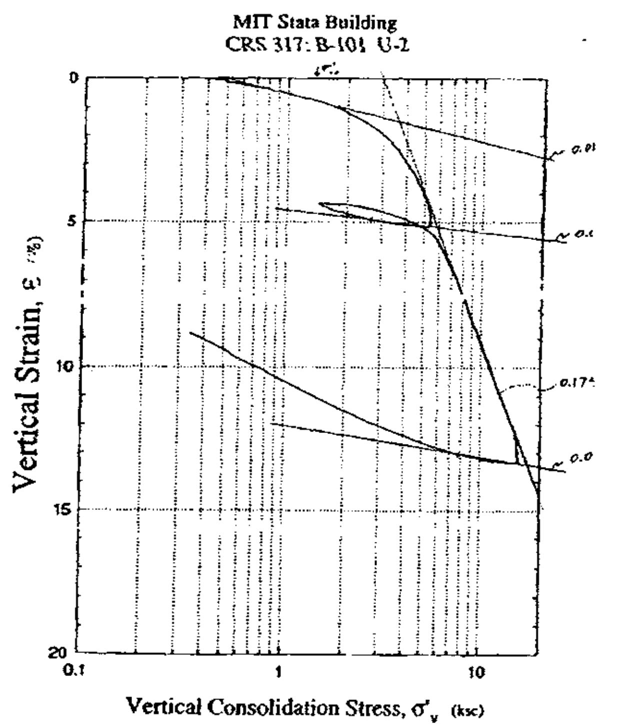


FIGURE A-22: CRS @ El. -31.07

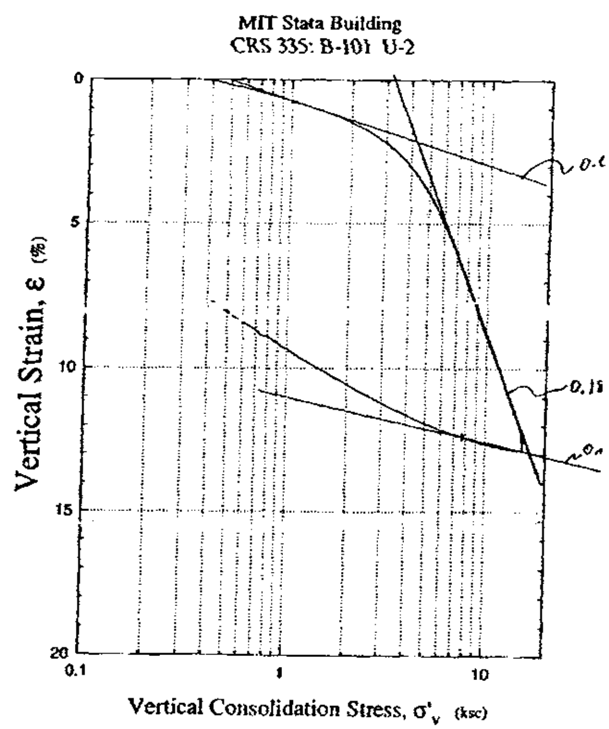


FIGURE A-23: CRS @ El. -30.03

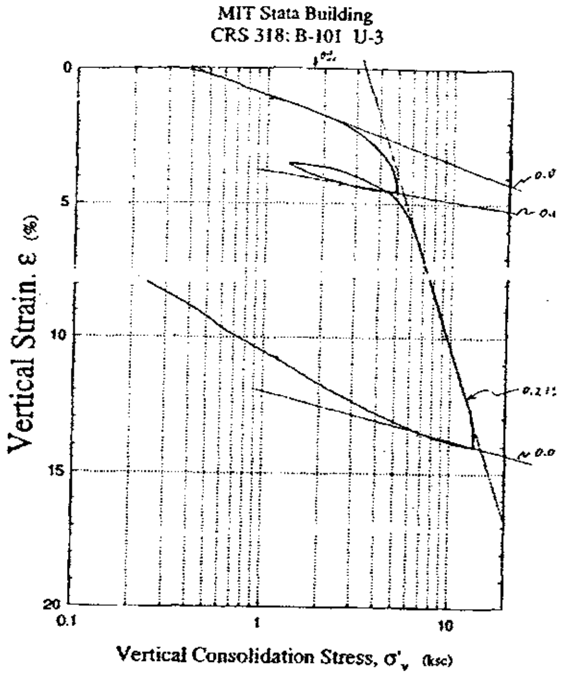


FIGURE A-24: CRS @ El. -40.99

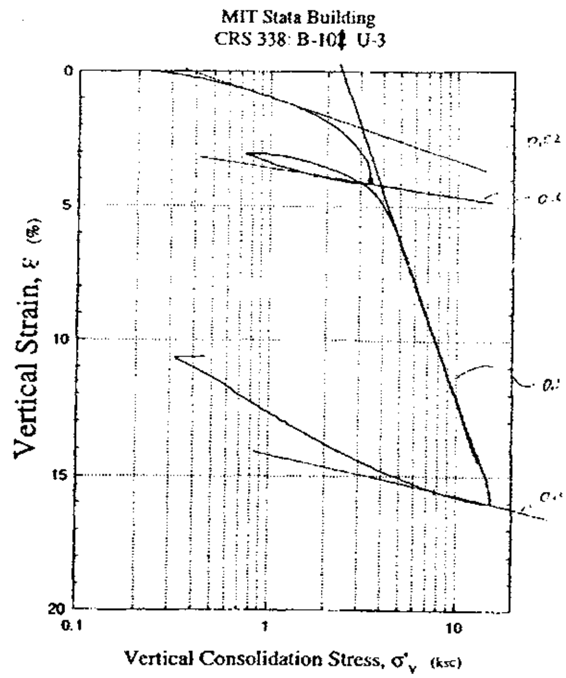


FIGURE A-25: CRS @ El. -39.95

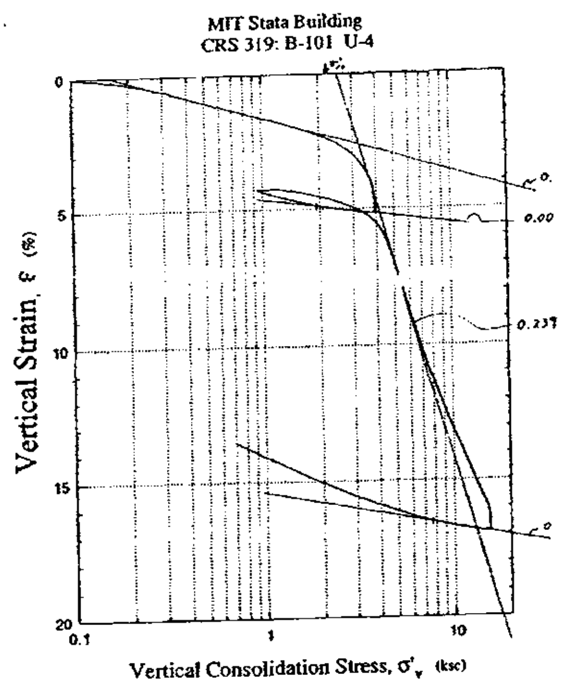


FIGURE A-26: CRS @ El. -51.45

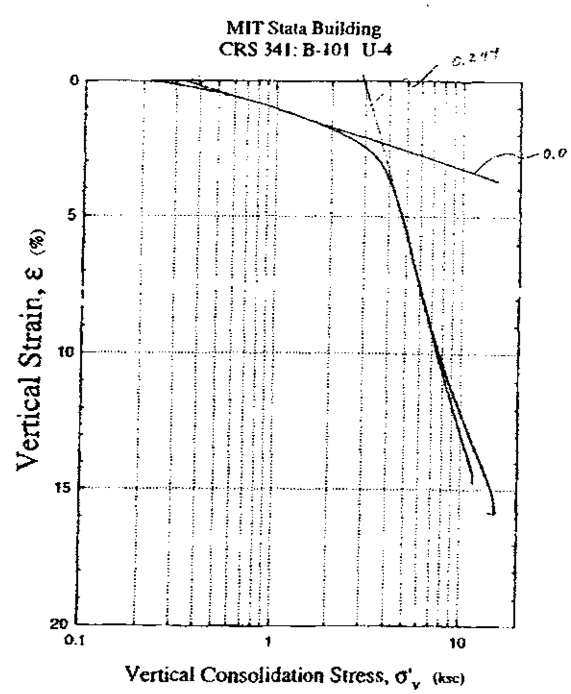
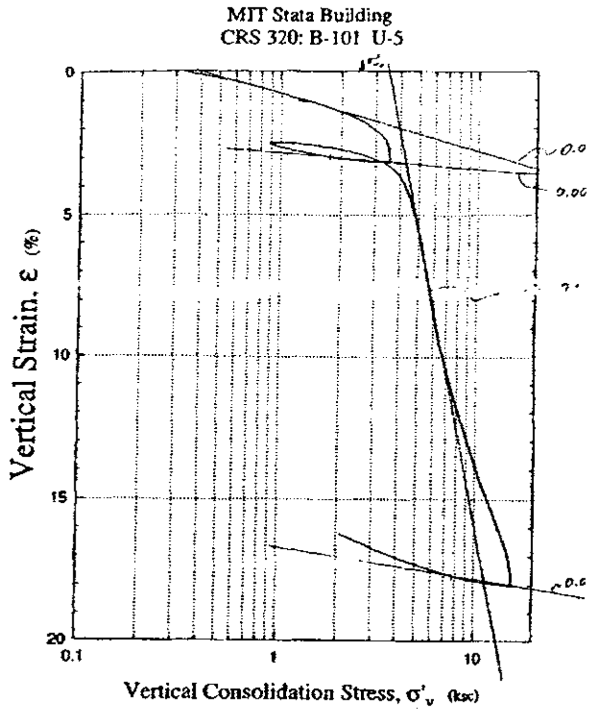
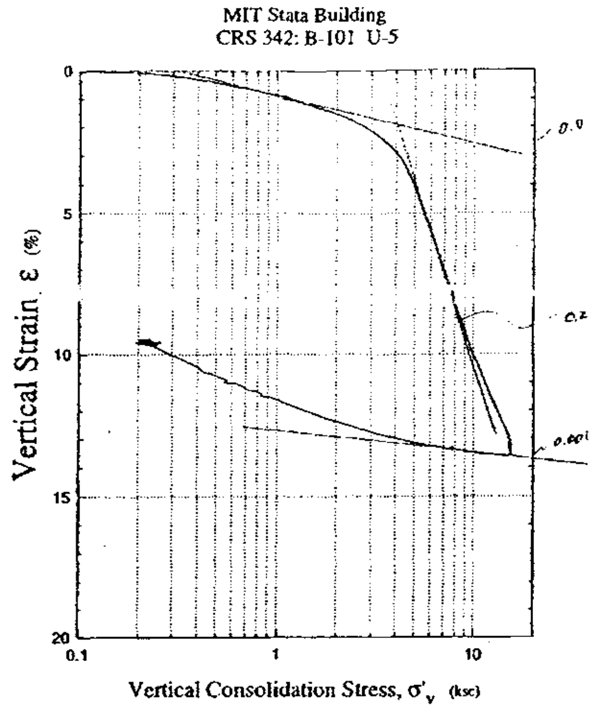


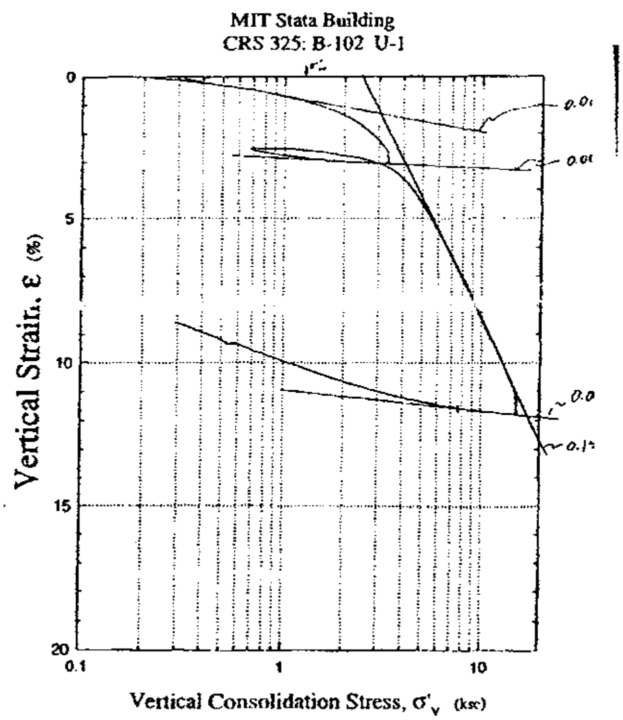
FIGURE A-27: CRS @ El. -49.95



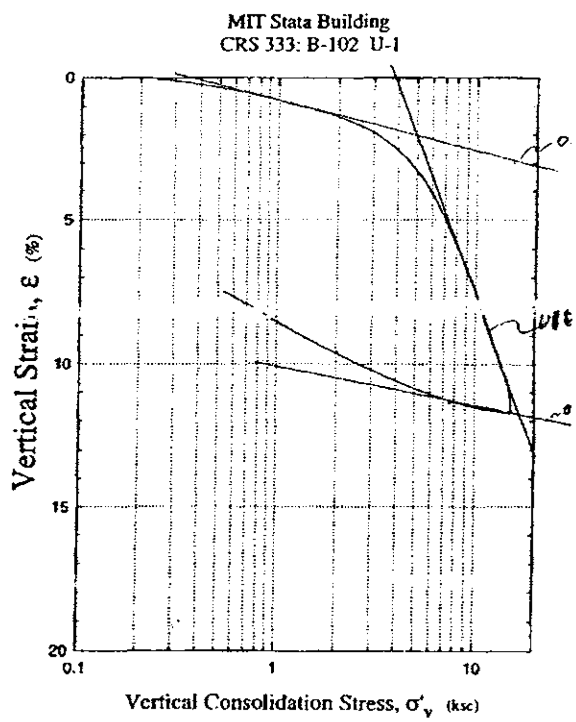
<u>FIGURE A-28:</u> CRS @ El. -60.35



<u>FIGURE A-29:</u> CRS @ El. -59.31



<u>FIGURE A-30:</u> CRS @ El. -27.72



<u>FIGURE A-31:</u> CRS @ El. -26.76

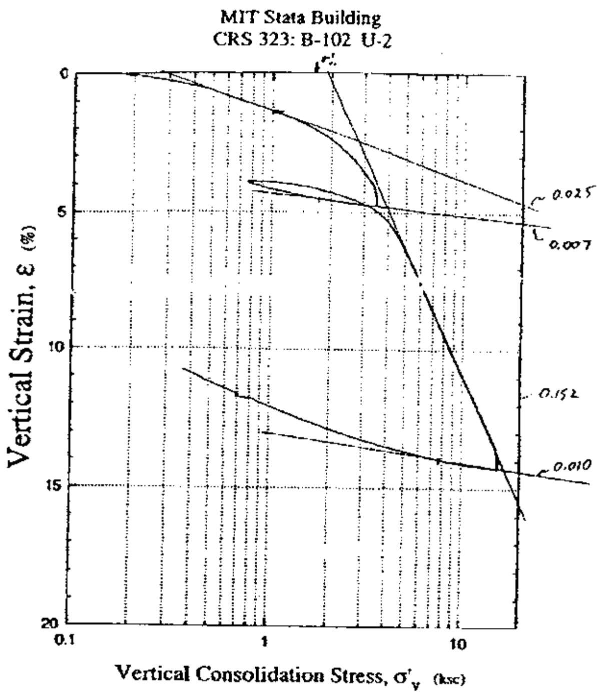


FIGURE A-32: CRS @ El. -38.06

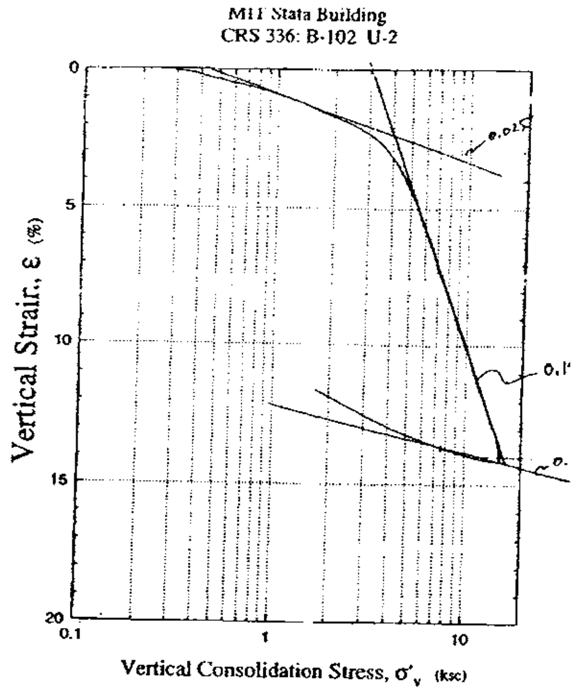


FIGURE A-33: CRS @ El. -36.85

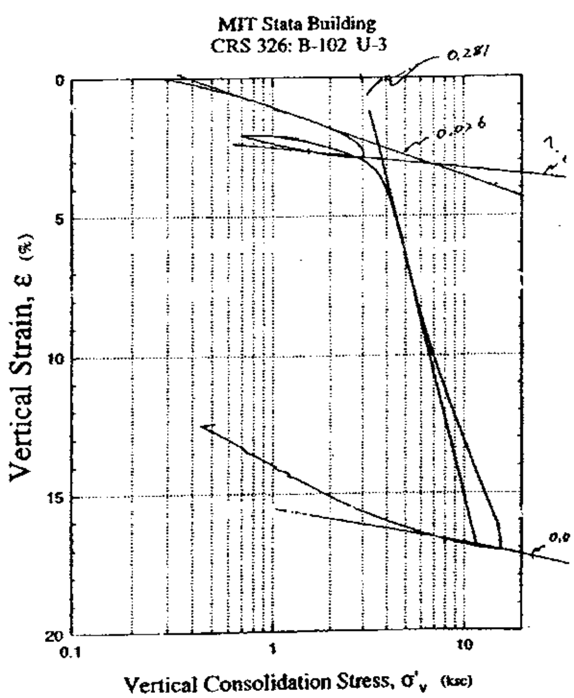


FIGURE A-34: CRS @ El. -48.06

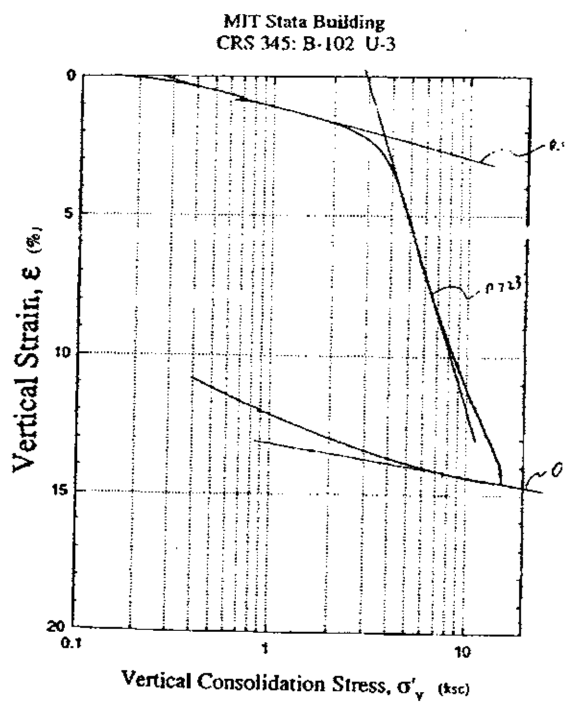


FIGURE A-35: CRS @ El. -47.39

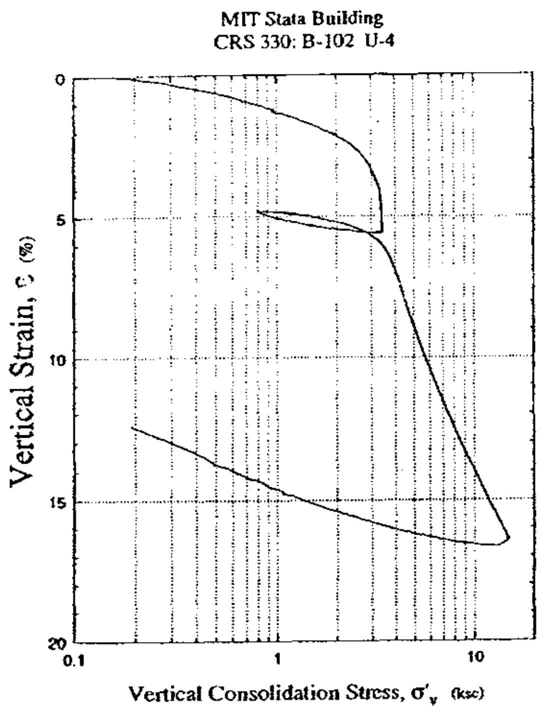


FIGURE A-36: CRS @ El. -58.97

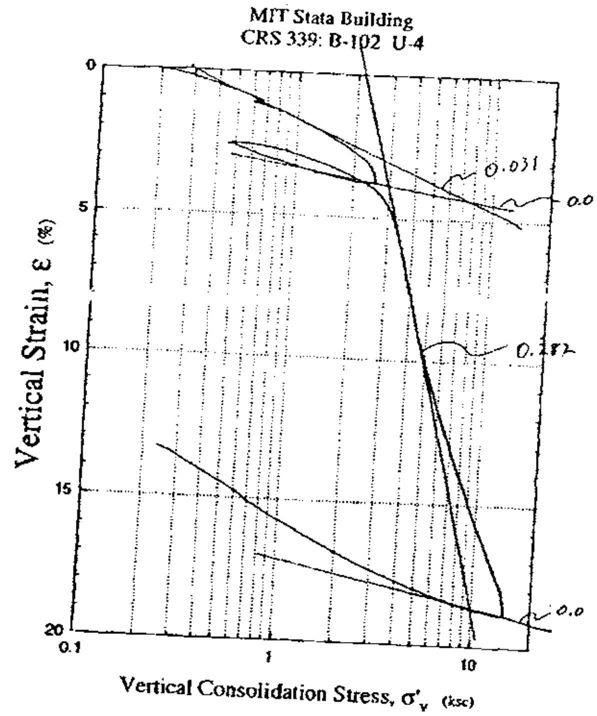


FIGURE A-37: CRS @ El. -57.84

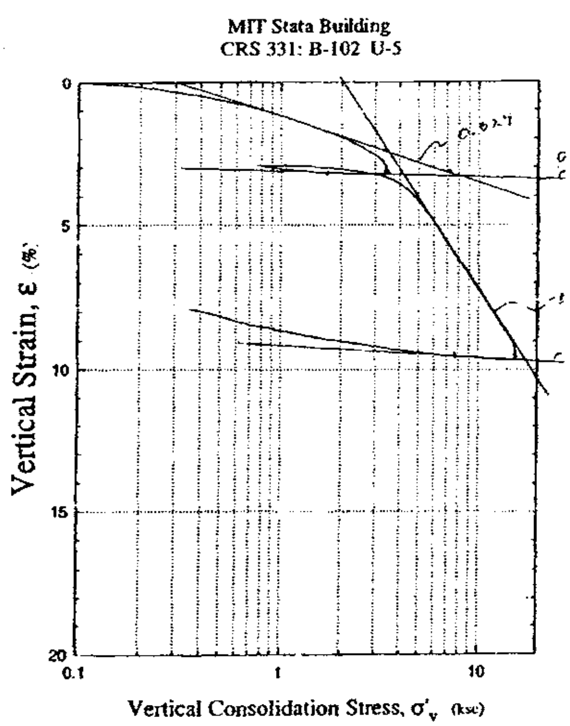


FIGURE A-38: CRS @ El. -73.06

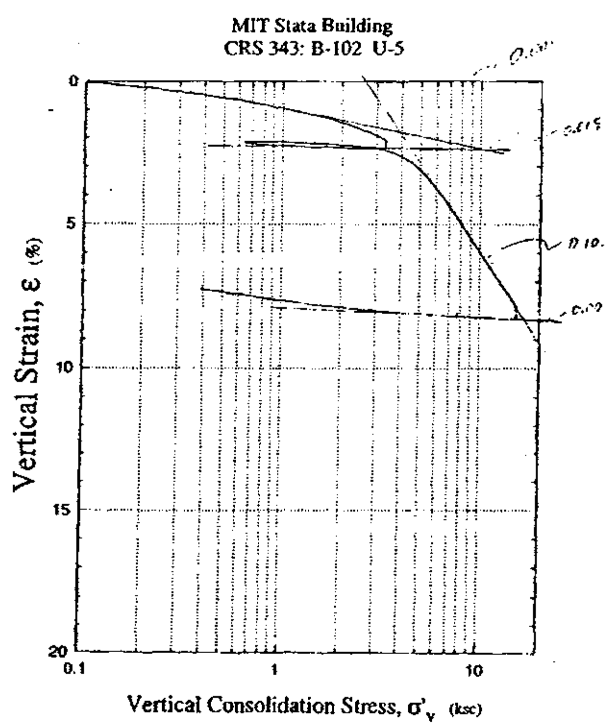


FIGURE A-39: CRS @ El. -71.93

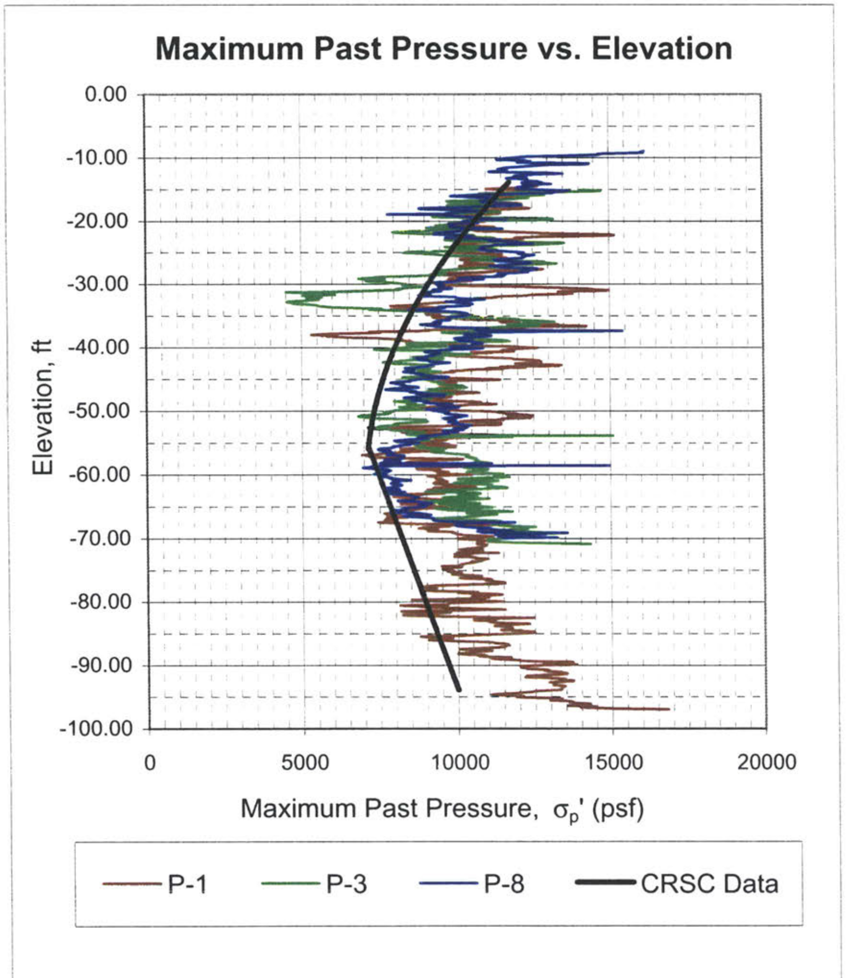


FIGURE A-40: Maximum Past Pressure vs. Elevation for cones P-1, P-3, and P-8.

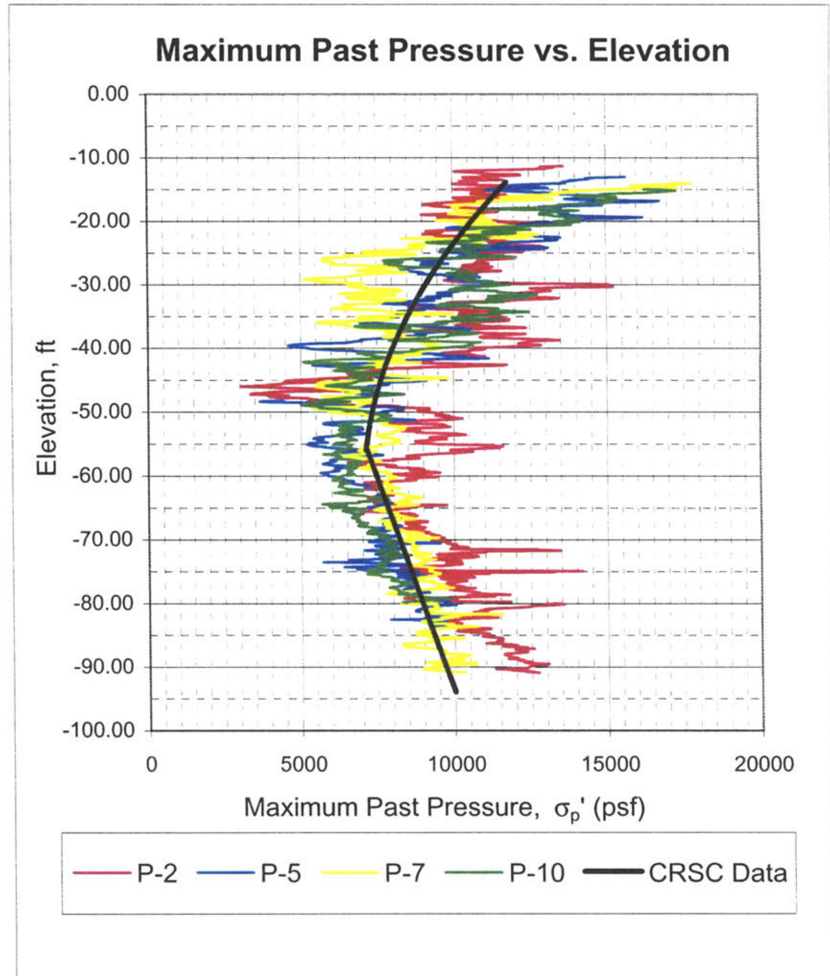


FIGURE A-41: Maximum Past Pressure vs. Elevation for cones P-2, P-5, P-7, and P-10.

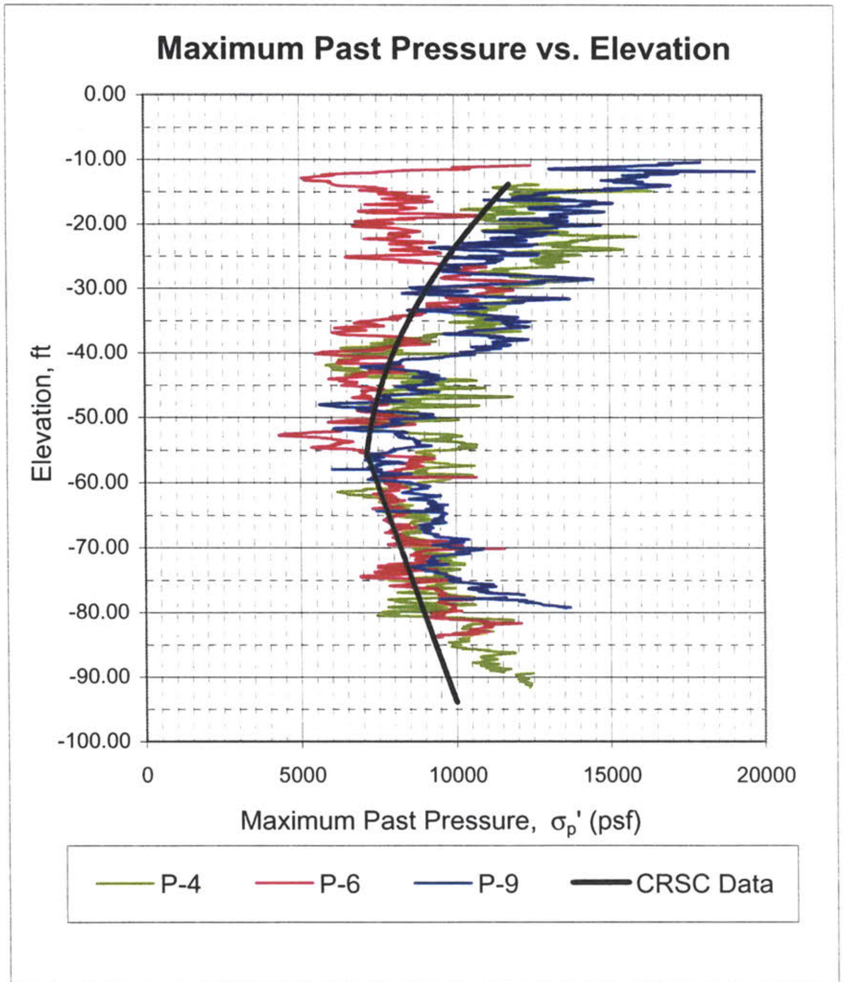


FIGURE A-42: Maximum Past Pressure vs. Elevation for cones P-4, P-6, and P-9.

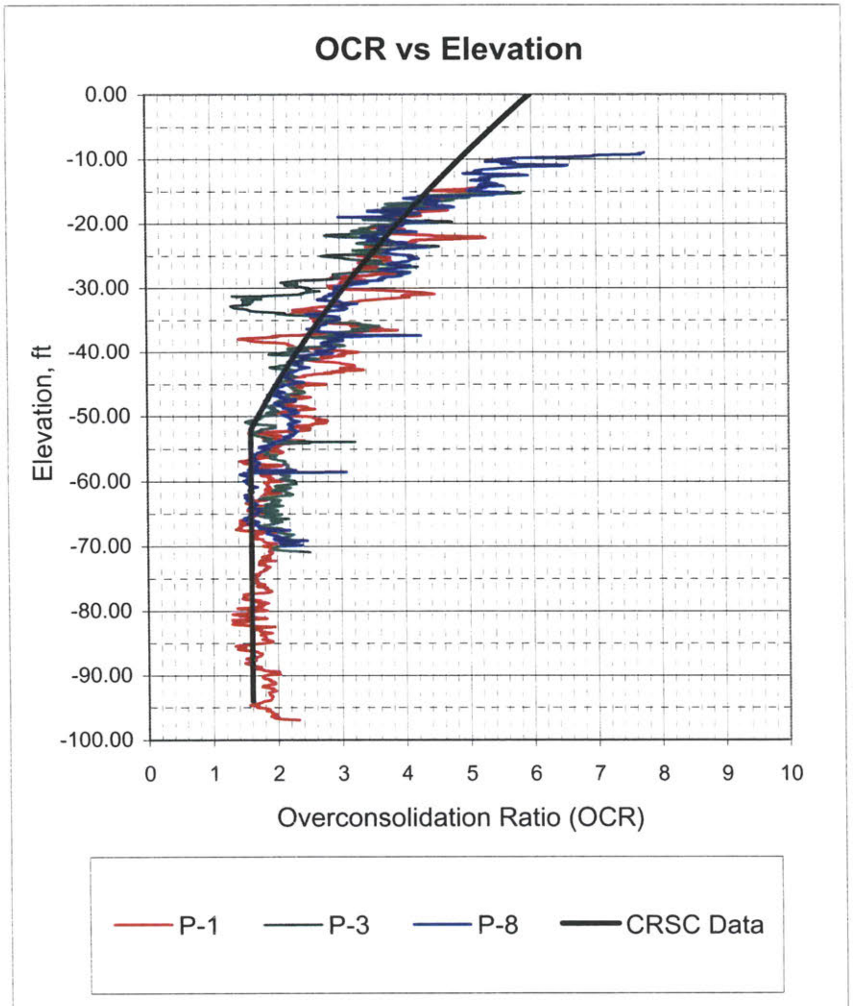


FIGURE A-43: Overconsolidation Ratio vs. Eelevation for cones P-1, P-3, and P-8.

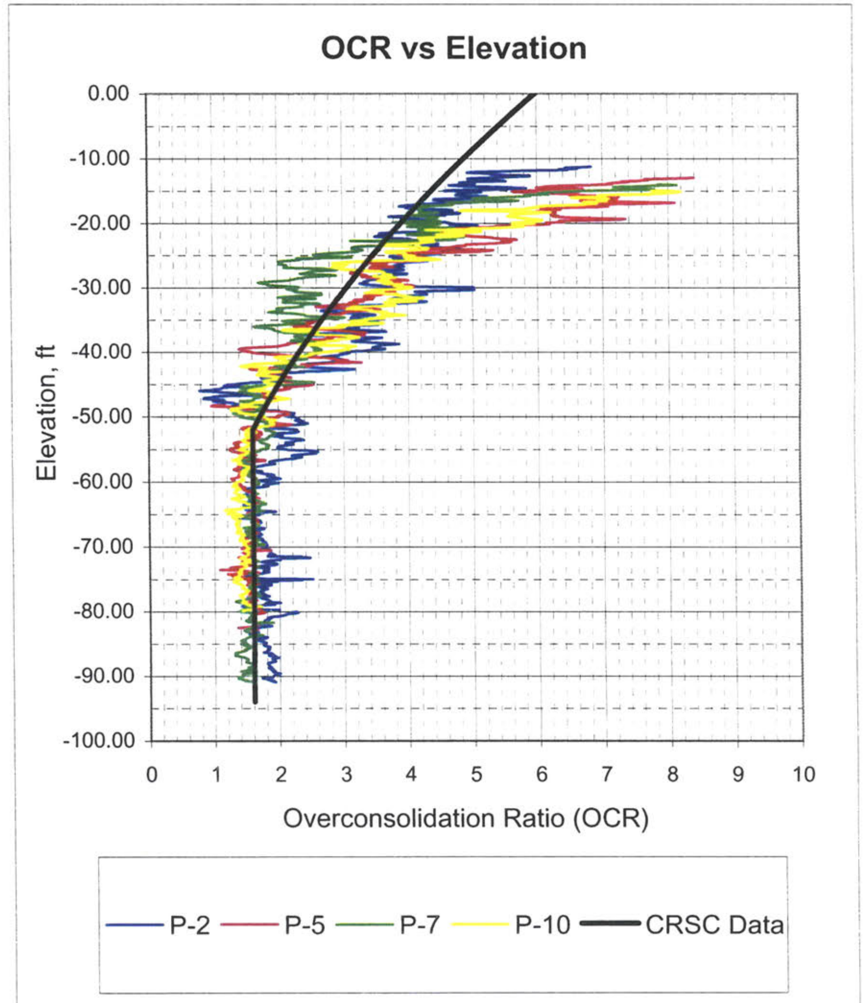


FIGURE A-44: Overconsolidation Ratio vs. Eelevation for cones P-2, P-5, P-7, and P-10.

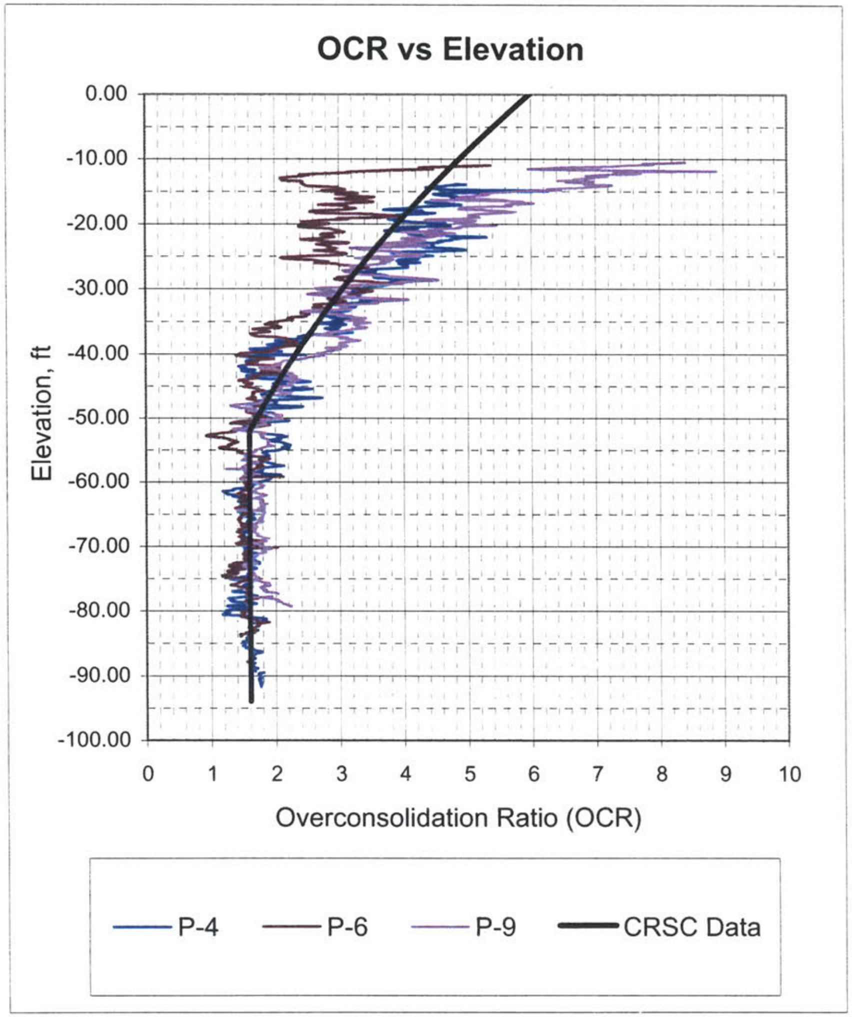


FIGURE A-45: Overconsolidation Ratio vs. Eelevation for cones P-4, P-6, and P-9.

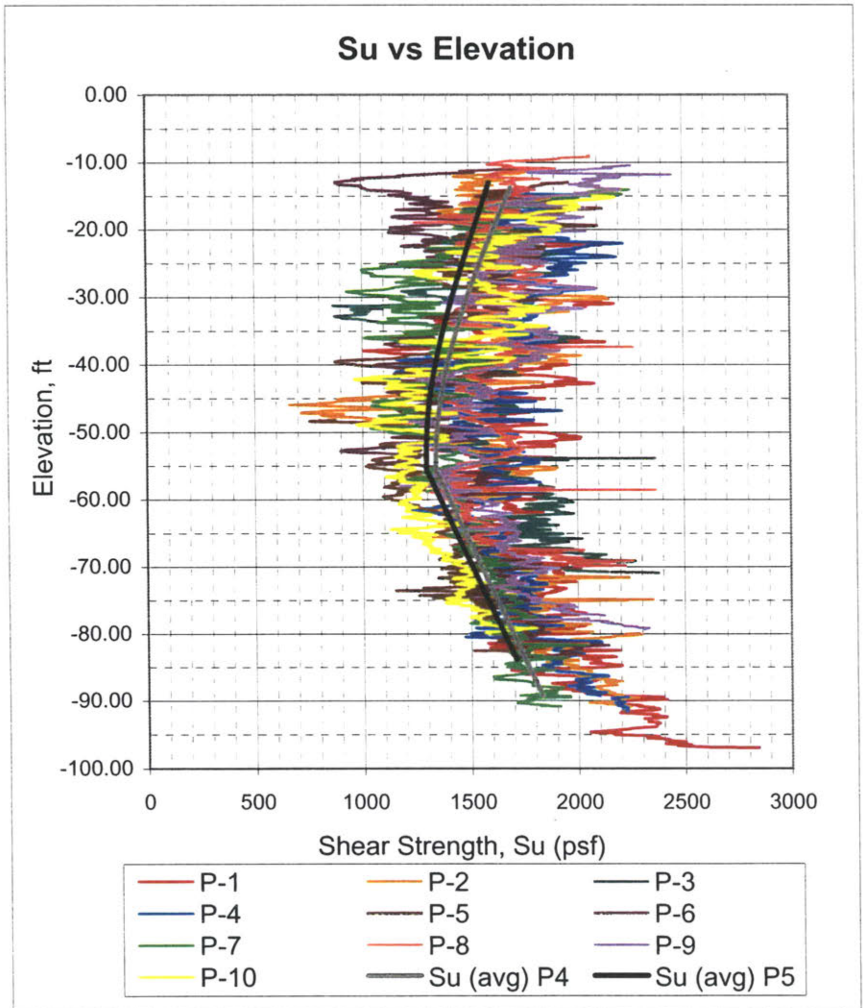


FIGURE A-46: Undrained Shear Strength vs. Elevation
For all of the cones.

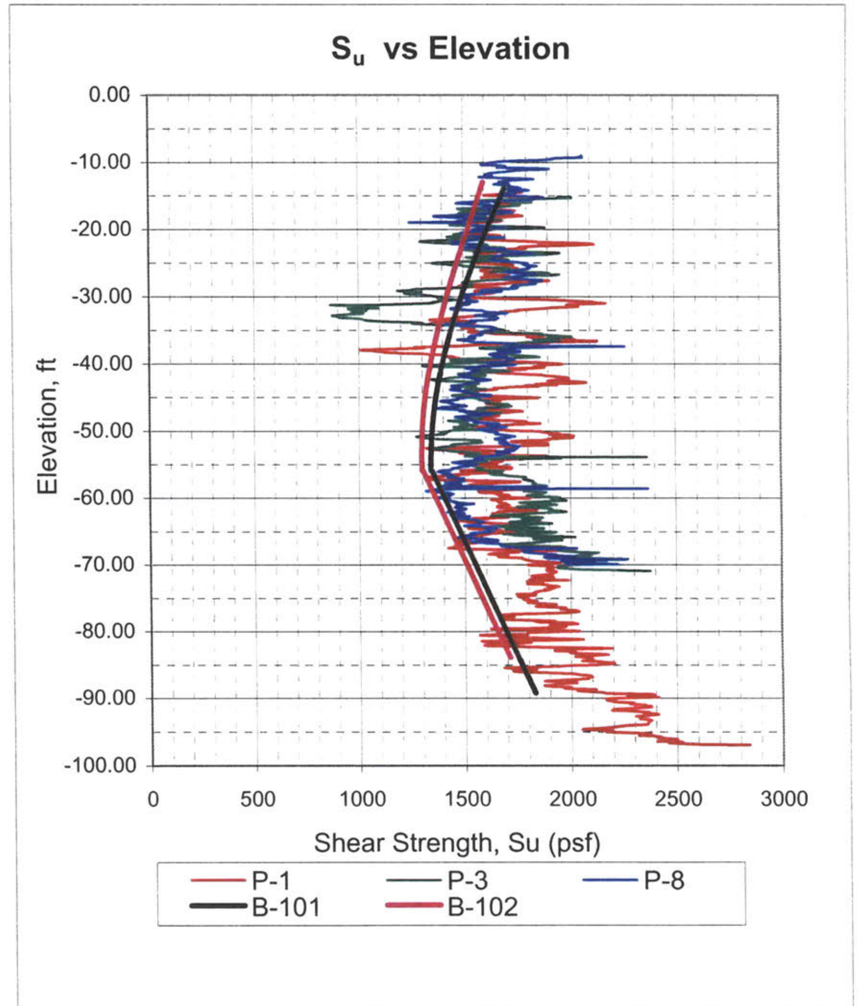


FIGURE A-47: Undrained Shear Strength vs. Elevation
for cones P-1, P-3, and P-8.

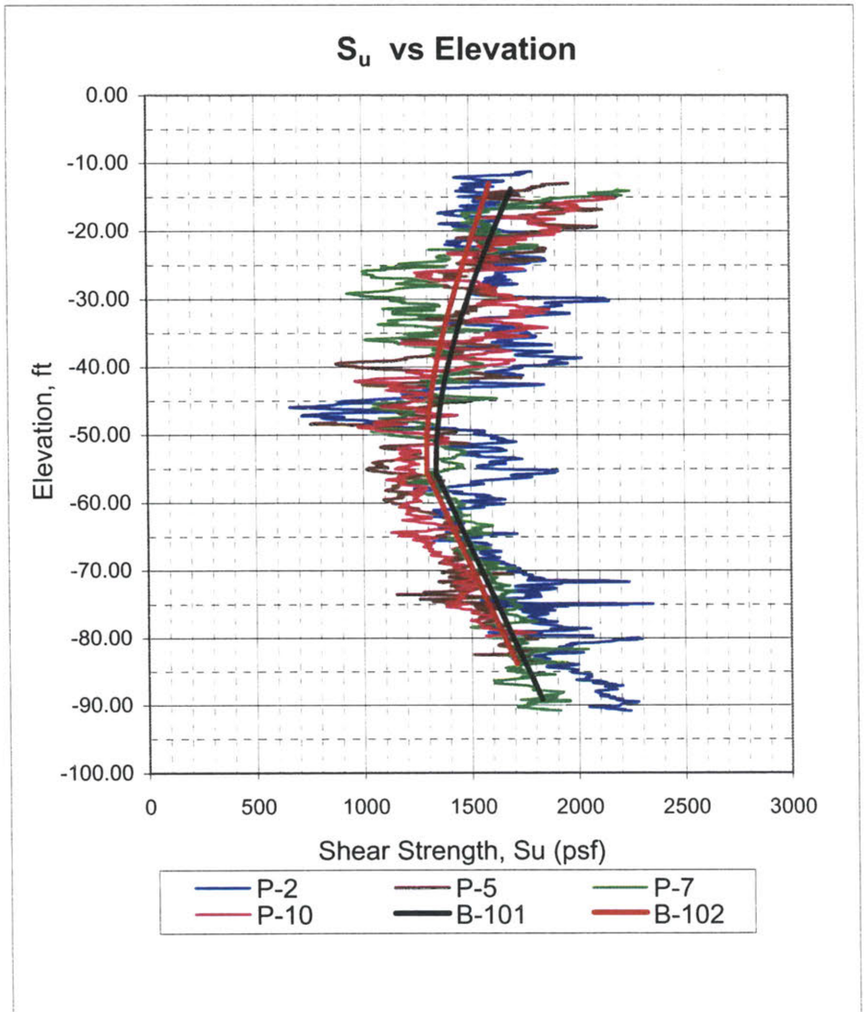


FIGURE A-48: Undrained Shear Strength vs. Elevation for cones P-2, P-5, P-7, and P-10.

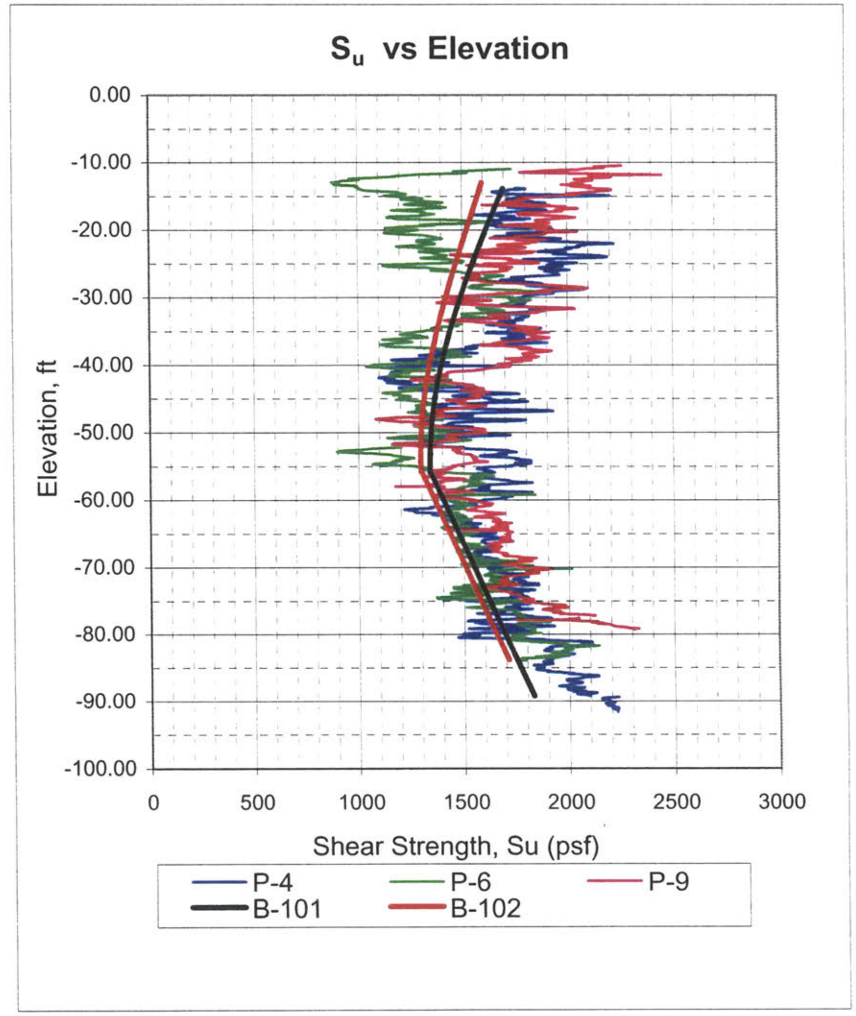
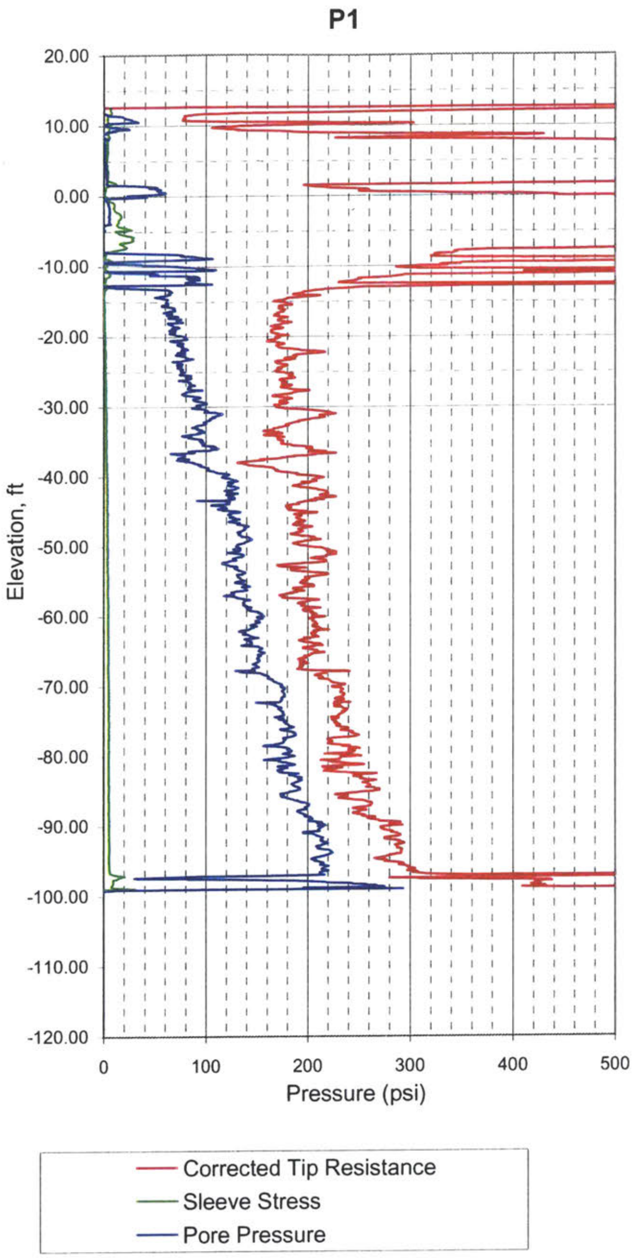
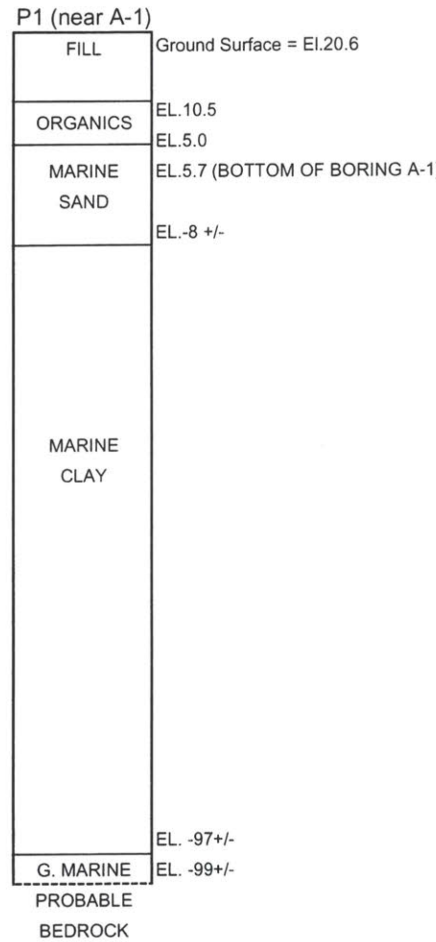


FIGURE A-49: Undrained Shear Strength vs. Elevation for cones P-4, P-6, and P-19.





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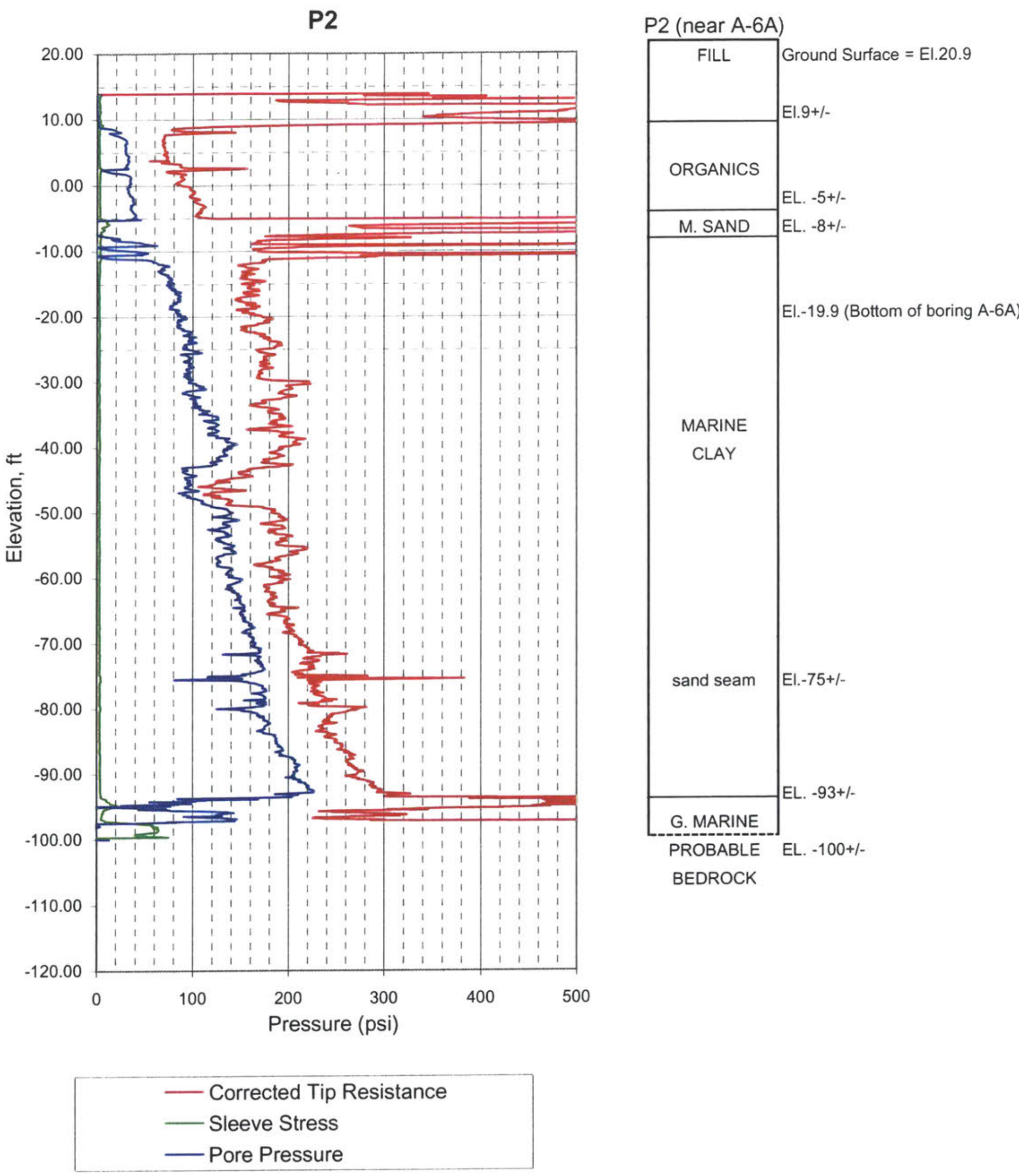
CONE PENETROMETER P1
TIP RESISTANCE, SLEEVE STRESS, AND
PORE PRESSURE VS. ELEVATION
(From H & A)

FIGURE A-50



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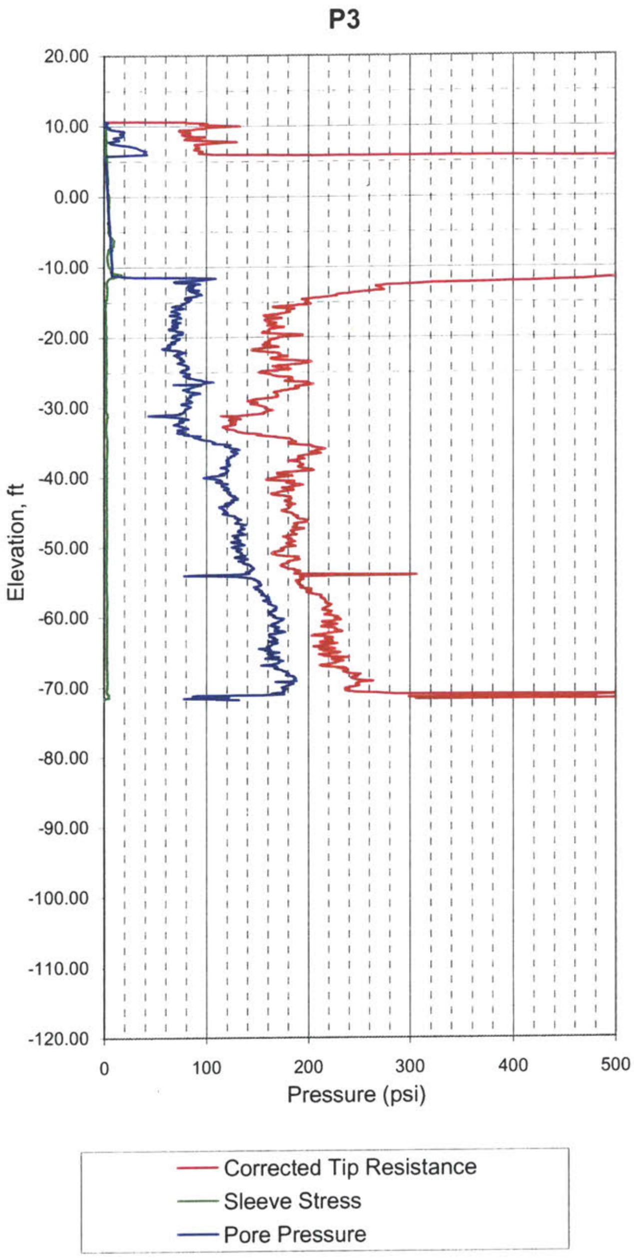
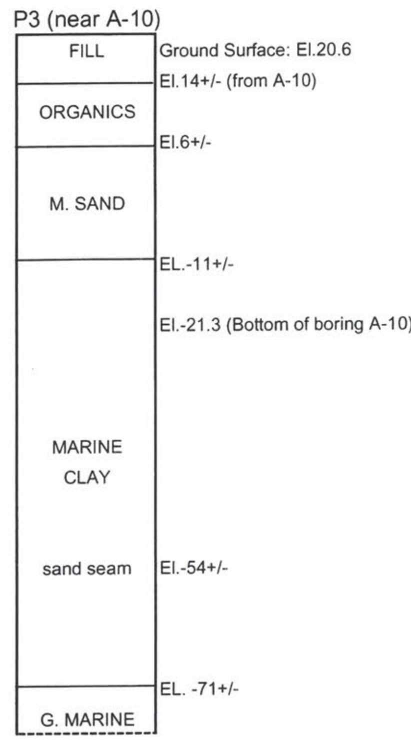
CONE PENETROMETER P2
TIP RESISTANCE, SLEEVE STRESS, AND
PORE PRESSURE VS. ELEVATION
(From H & A)

FIGURE A-51





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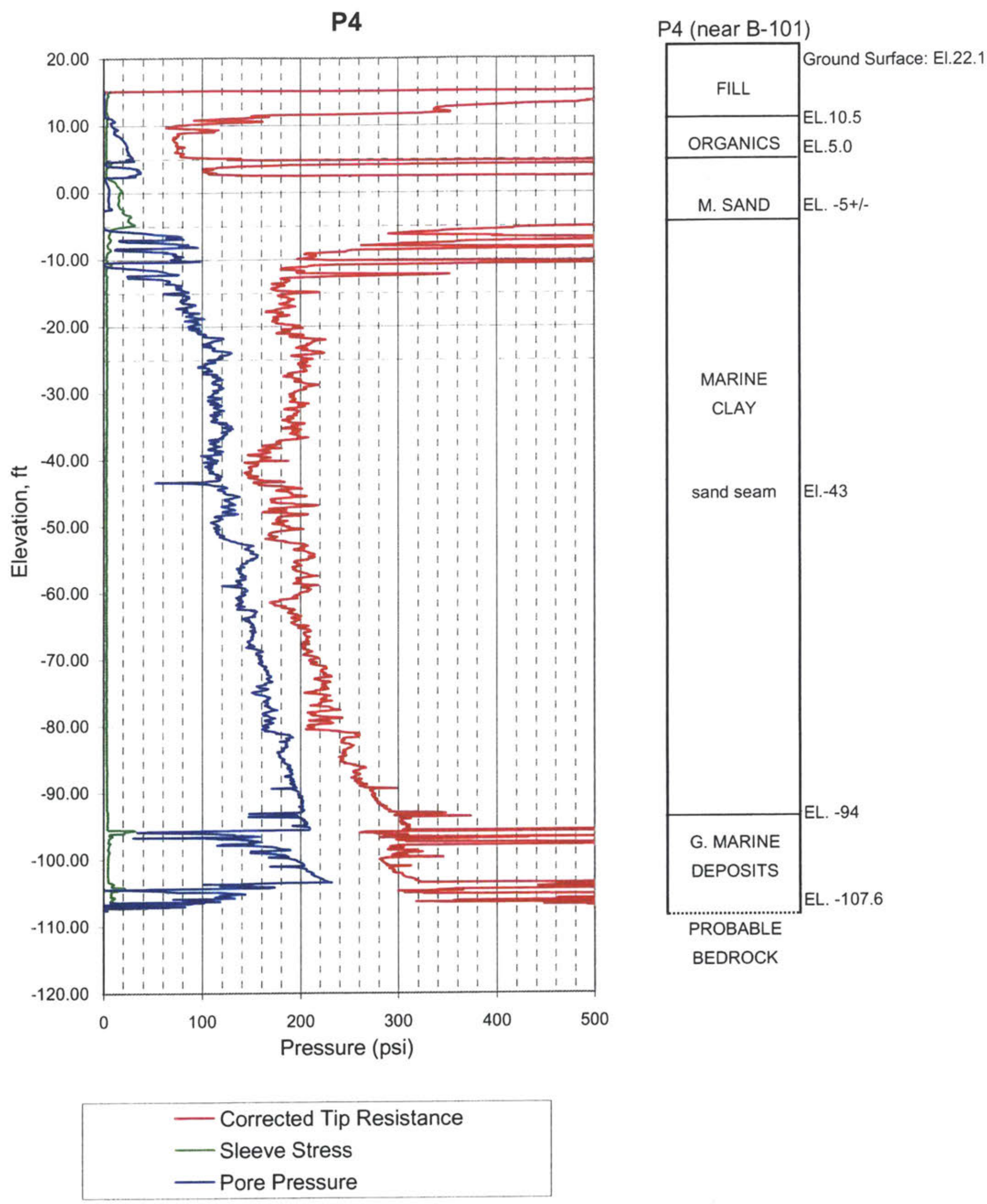
CONE PENETROMETER P3
TIP RESISTANCE, SLEEVE STRESS, AND
PORE PRESSURE VS. ELEVATION
(From H & A)

FIGURE A-52



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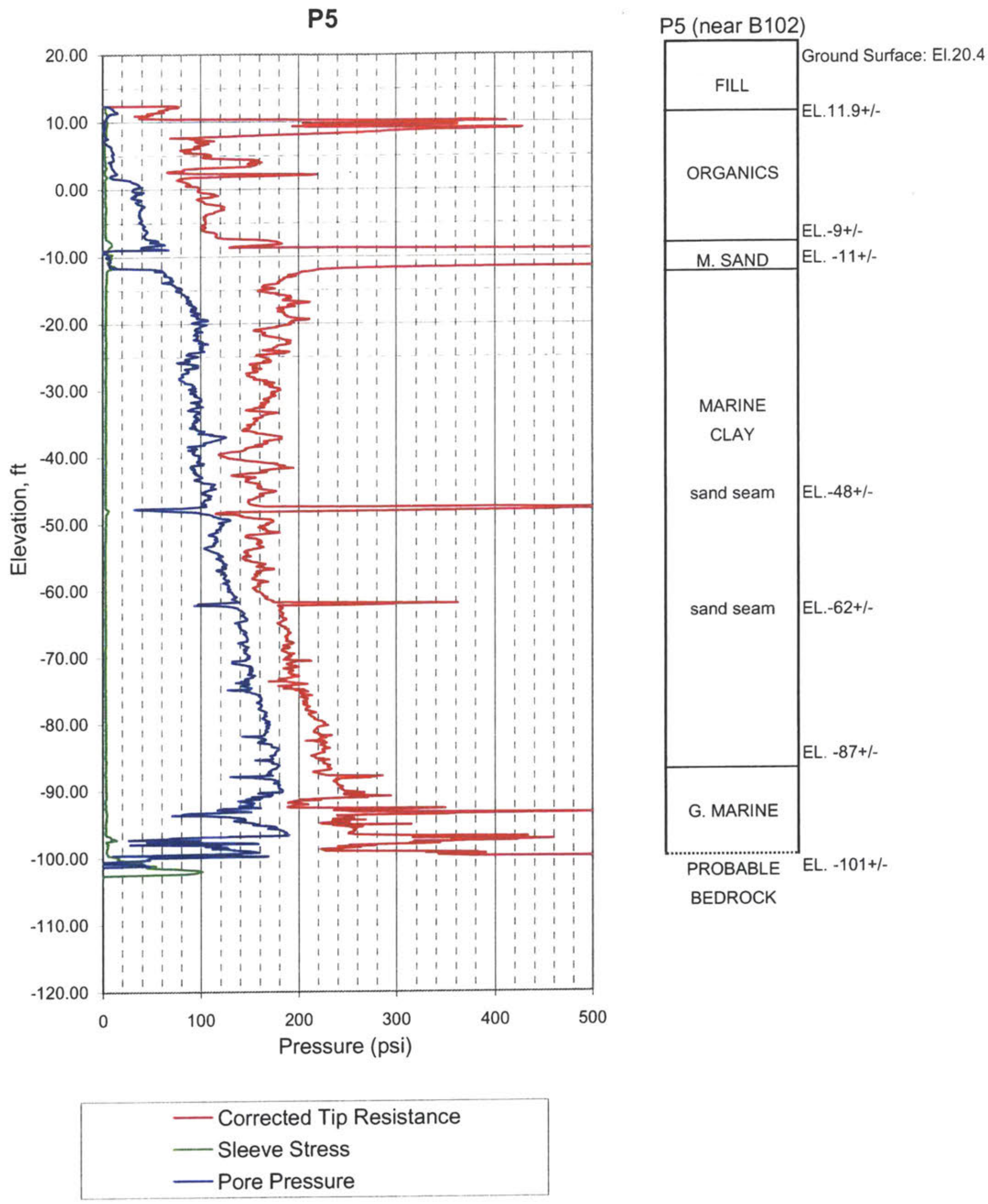
CONE PENETROMETER P4
TIP RESISTANCE, SLEEVE STRESS, AND
PORE PRESSURE VS. ELEVATION
(From H & A)

FIGURE A-53



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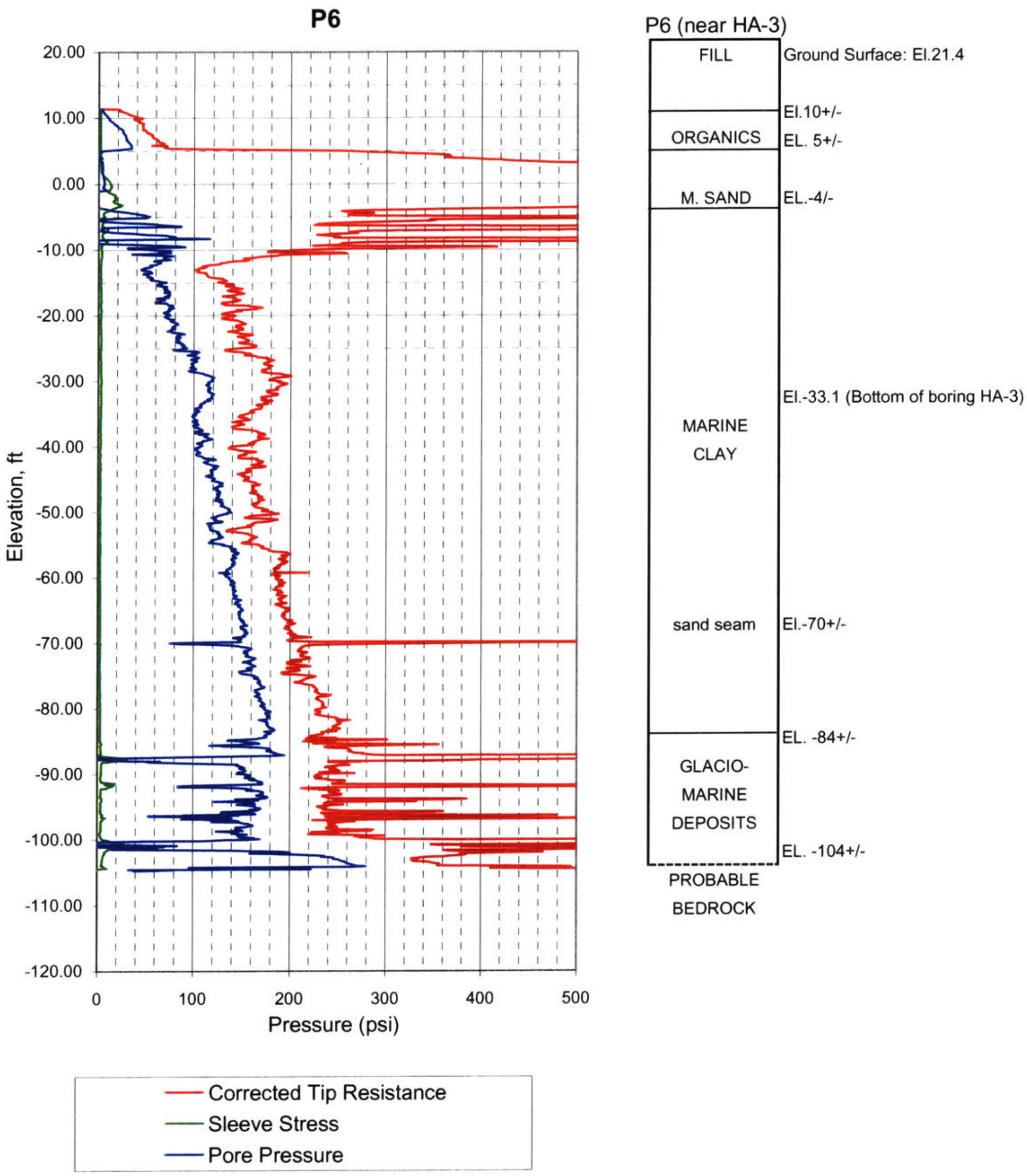
CONE PENETROMETER P5
TIP RESISTANCE, SLEEVE STRESS, AND
PORE PRESSURE VS. ELEVATION
(From H & A)

FIGURE A-54



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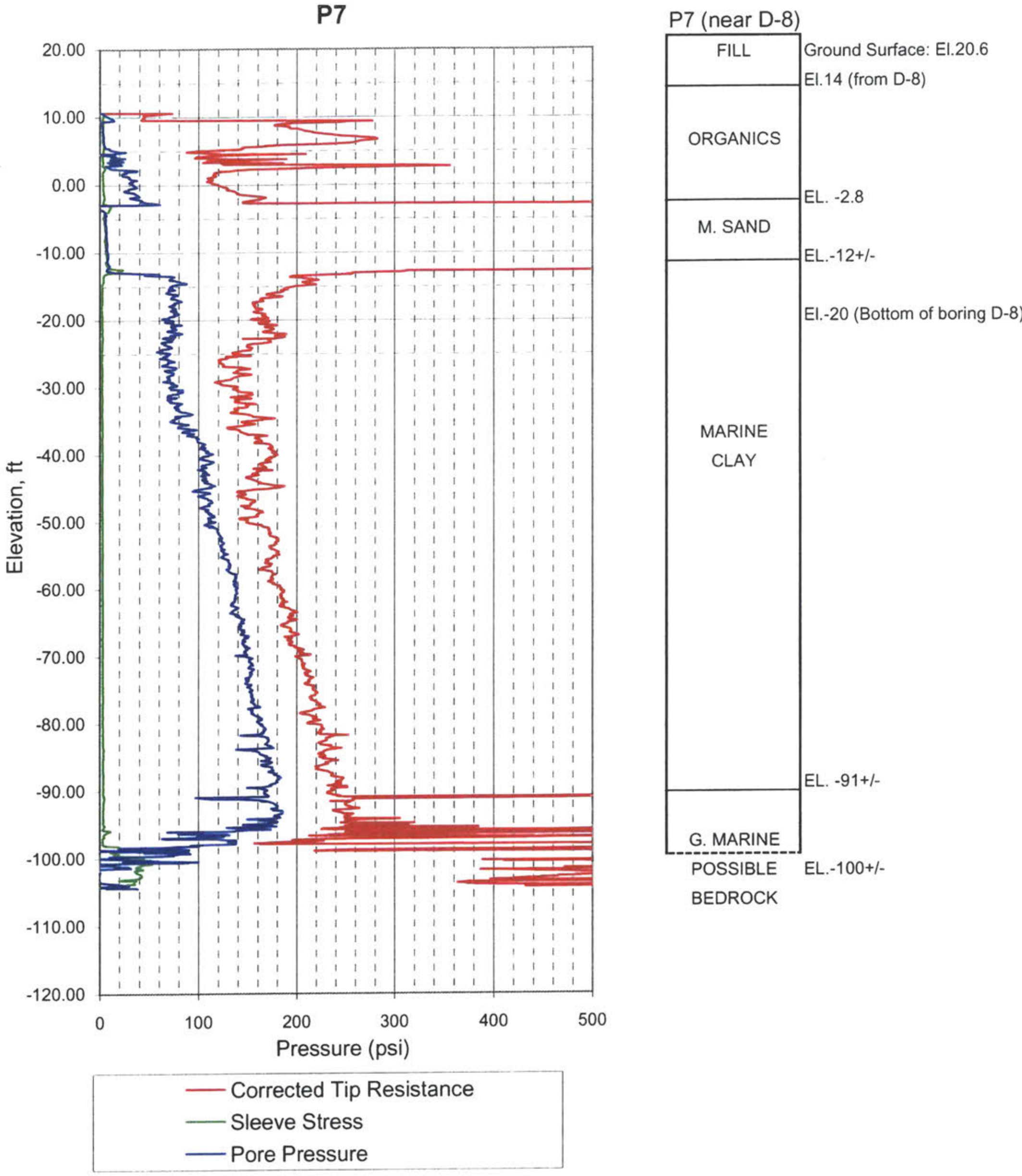
CONE PENETROMETER P6
TIP RESISTANCE, SLEEVE STRESS, AND
PORE PRESSURE VS. ELEVATION
(From H & A)

FIGURE A-55



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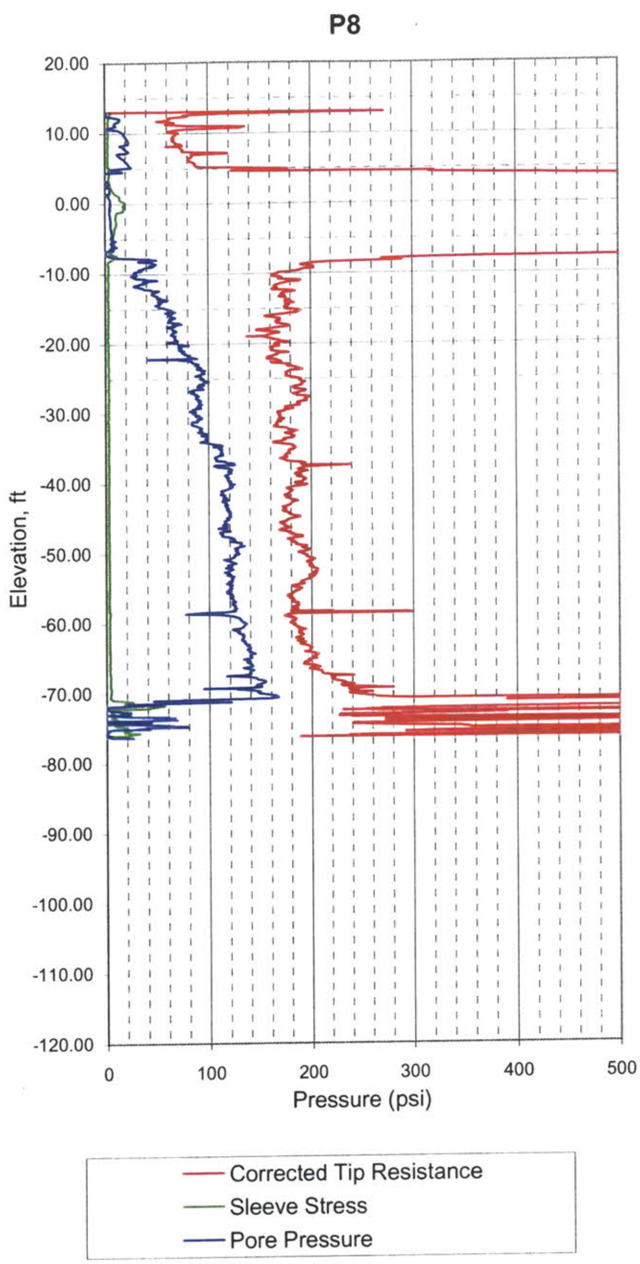
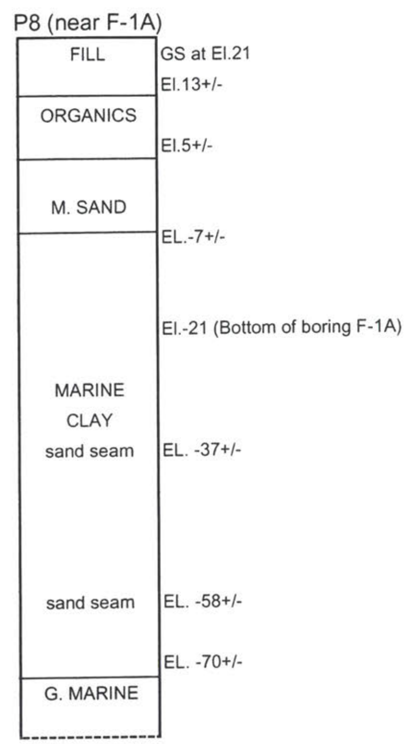
CONE PENETROMETER P7
TIP RESISTANCE, SLEEVE STRESS, AND
PORE PRESSURE VS. ELEVATION
(From H & A)

FIGURE A-56





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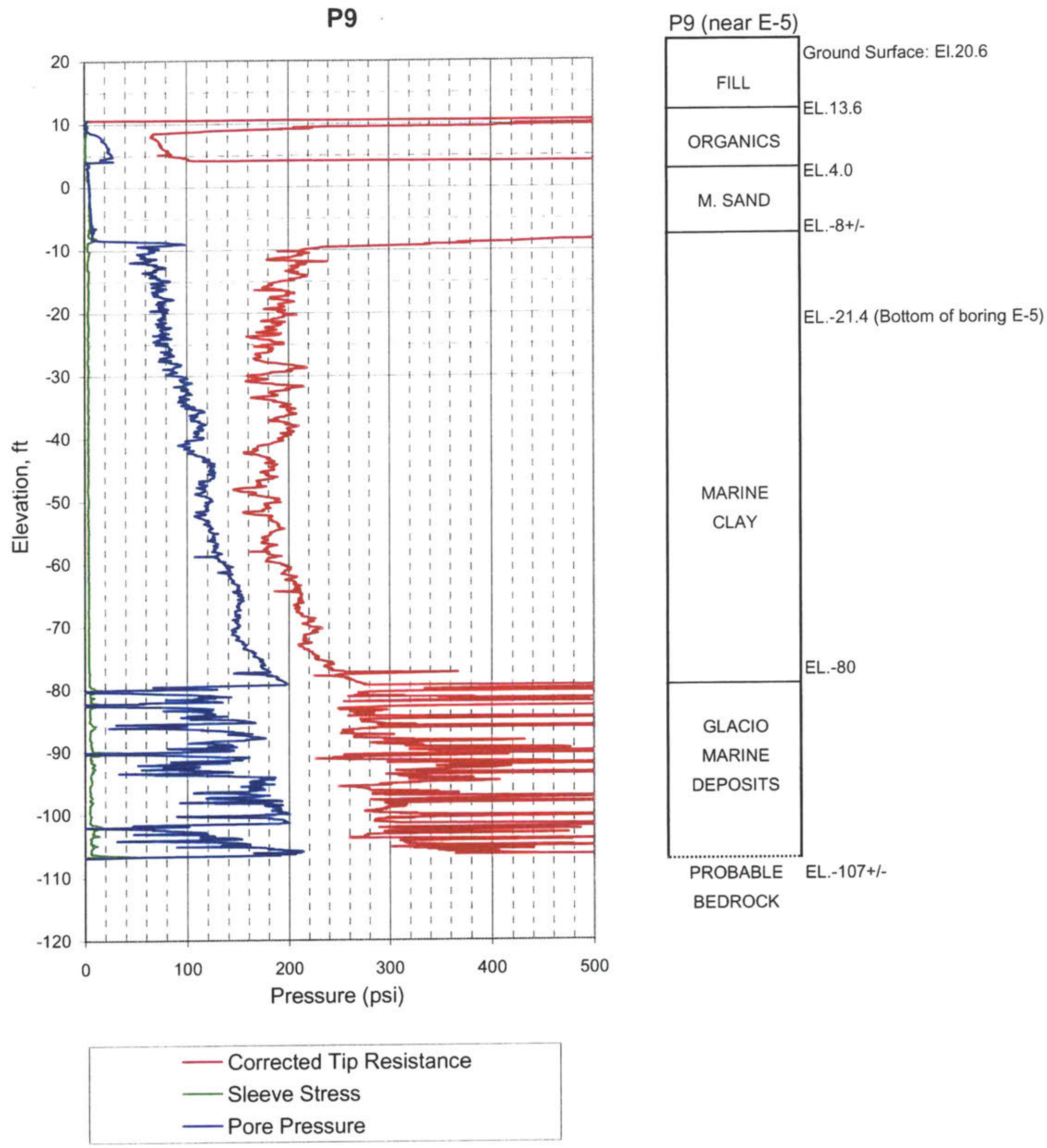
CONE PENETROMETER P8
TIP RESISTANCE, SLEEVE STRESS, AND PORE PRESSURE VS. ELEVATION
(From H & A)

FIGURE A-57



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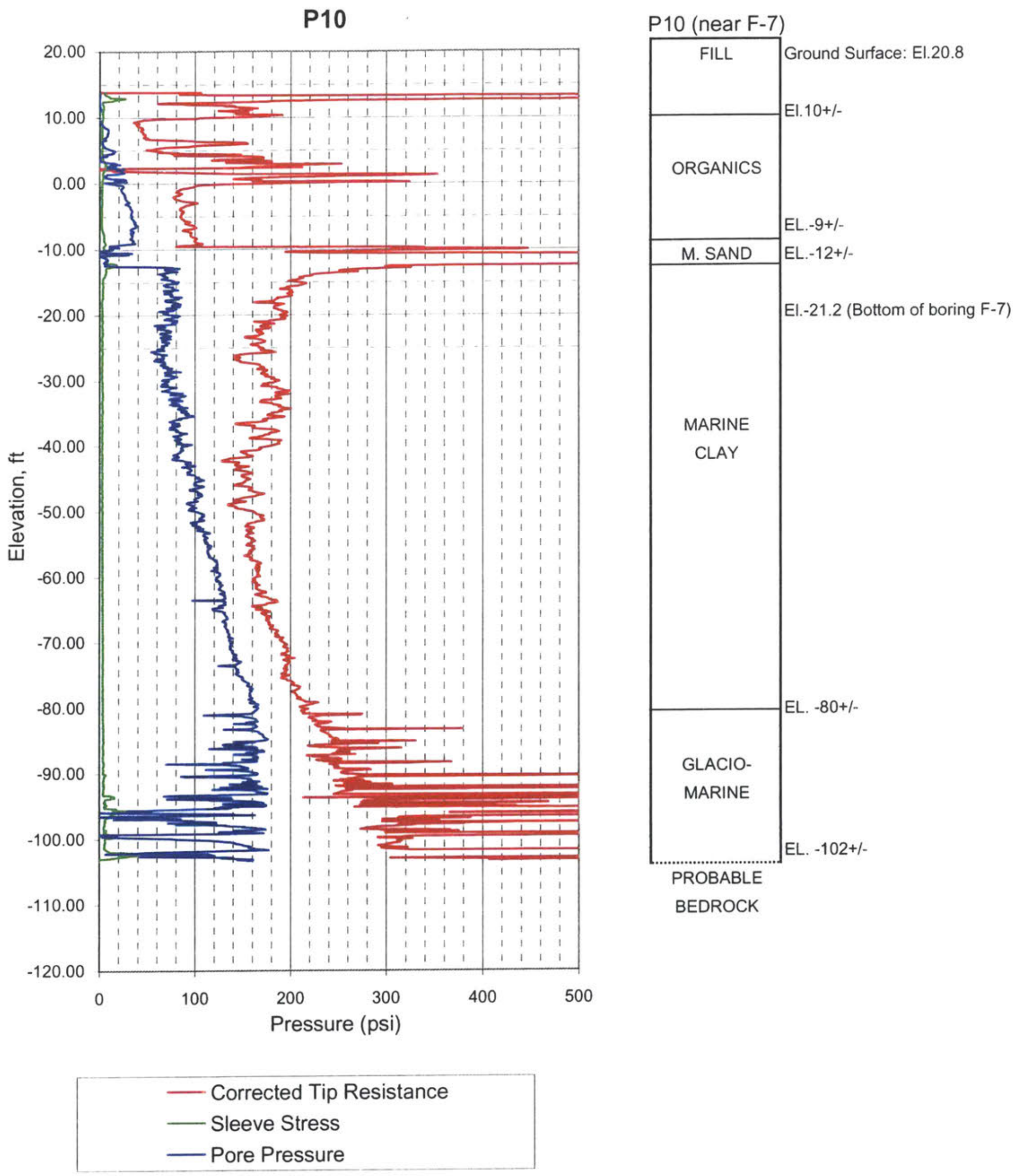
CONE PENETROMETER P9
TIP RESISTANCE, SLEEVE STRESS, AND
PORE PRESSURE VS. ELEVATION
(From H & A)

FIGURE A-58



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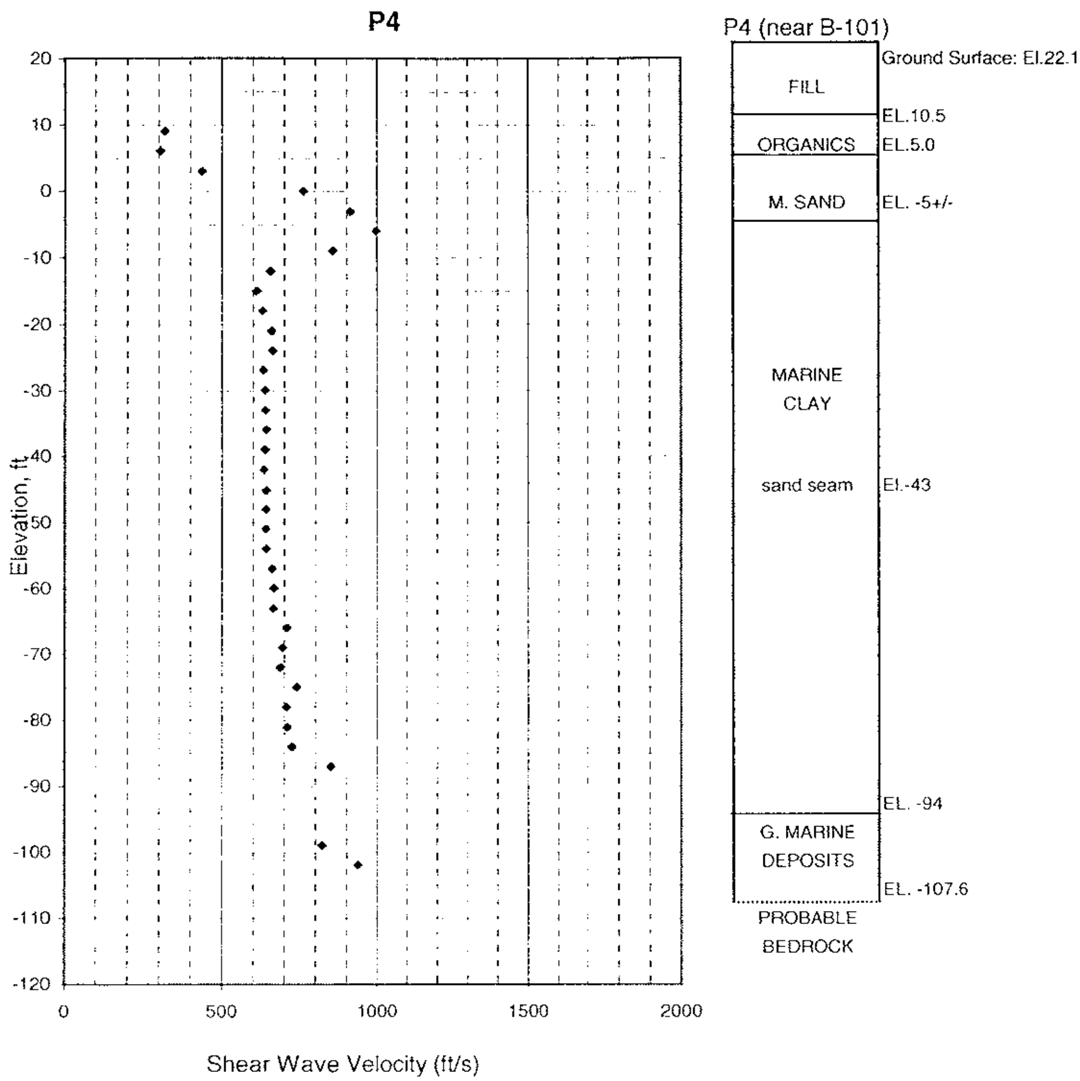
CONE PENETROMETER P10

TIP RESISTANCE, SLEEVE STRESS, AND PORE PRESSURE VS. ELEVATION

(From H & A)

FIGURE A-59



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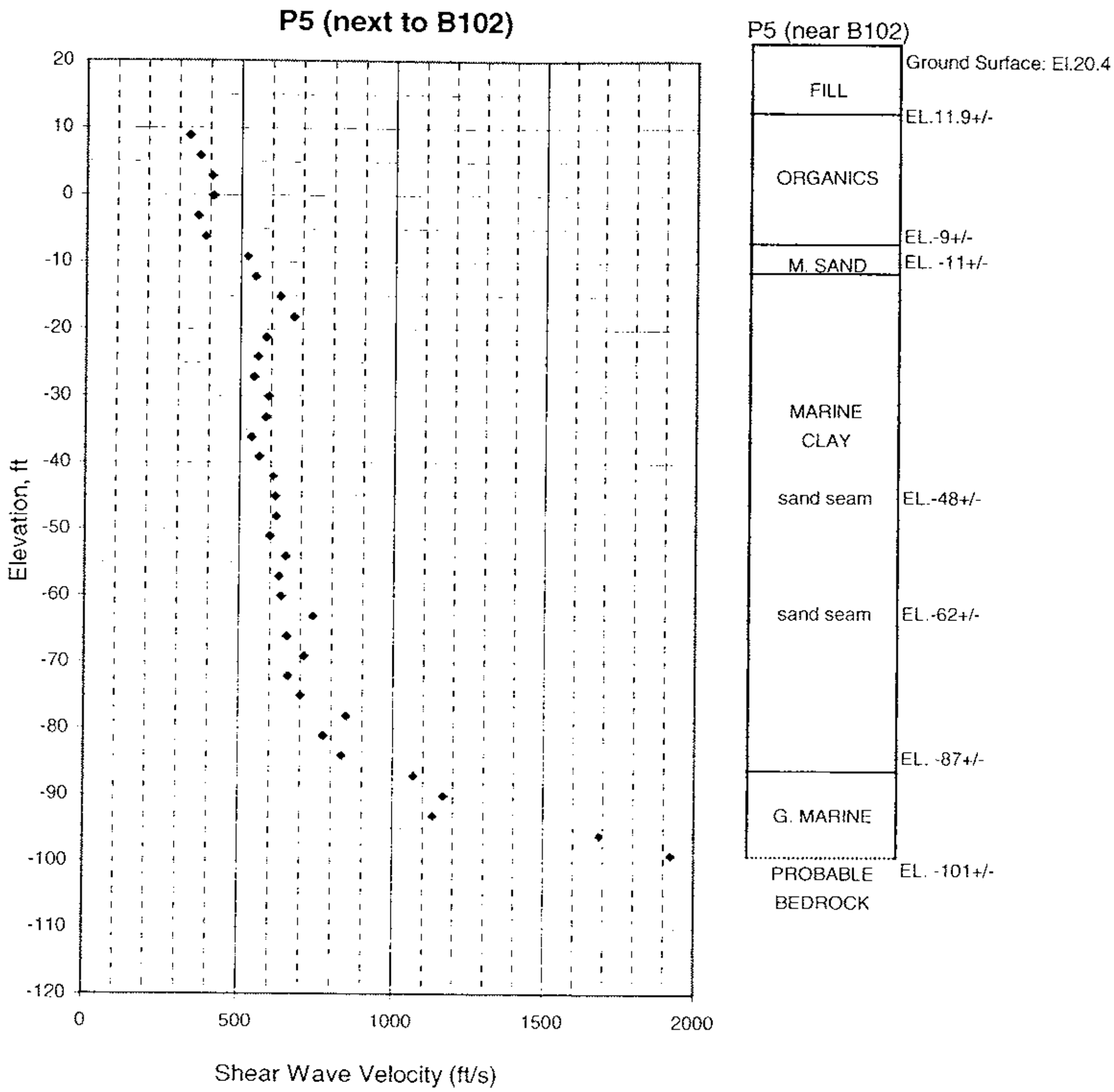
CONE PENETROMETER P4
Shear Wave Velocity vs. Elevation
(From H & A)

FIGURE A-60



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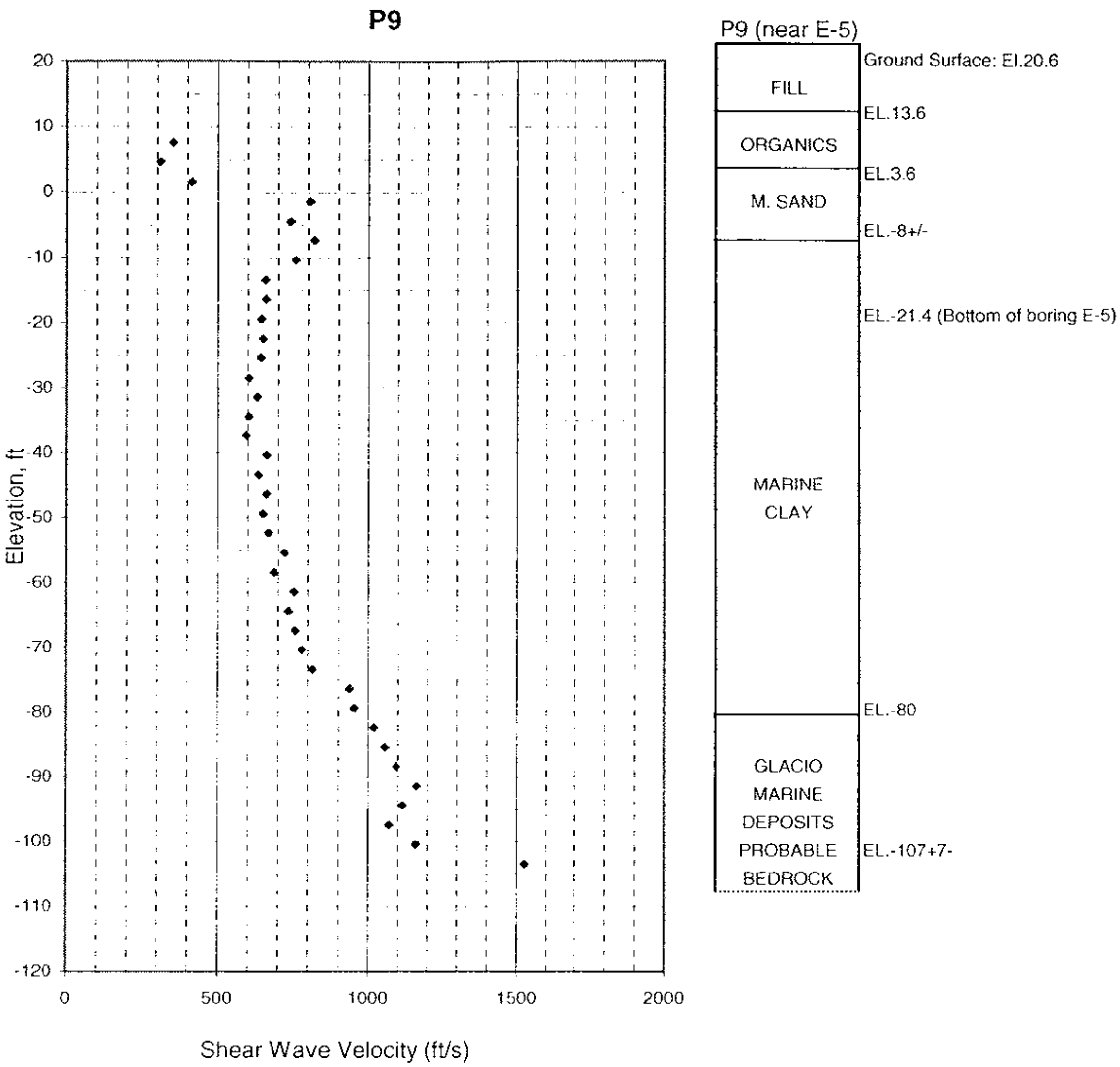
CONE PENETROMETER P5
Shear Wave Velocity vs. Elevation
(From H & A)

FIGURE A-61



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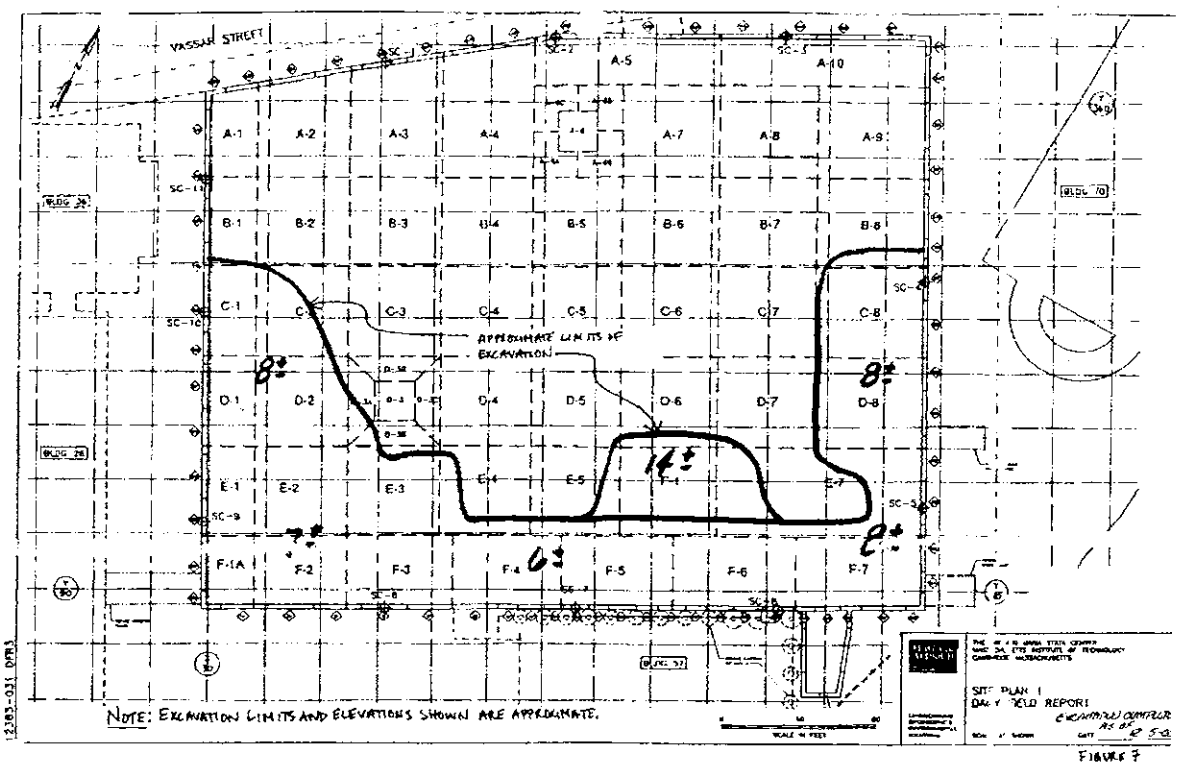
CONE PENETROMETER P9
Shear Wave Velocity vs. Elevation
(From H & A)

FIGURE A-62

APPENDIX B: EXCAVATION PROGRESS (PLAN VIEW)



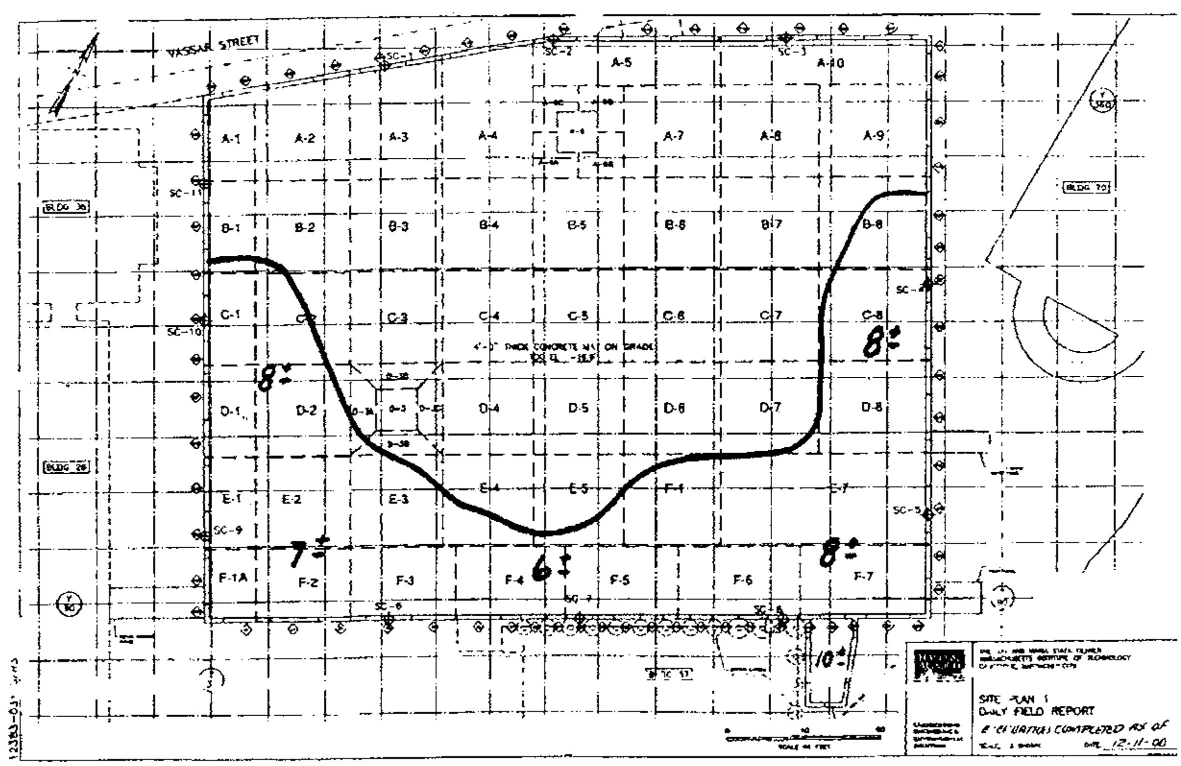


FIGURE B-1: Plan view of approximate excavation limits on December 5, 2000 (top) and December 11, 2000 (bottom). (Haley & Aldrich)

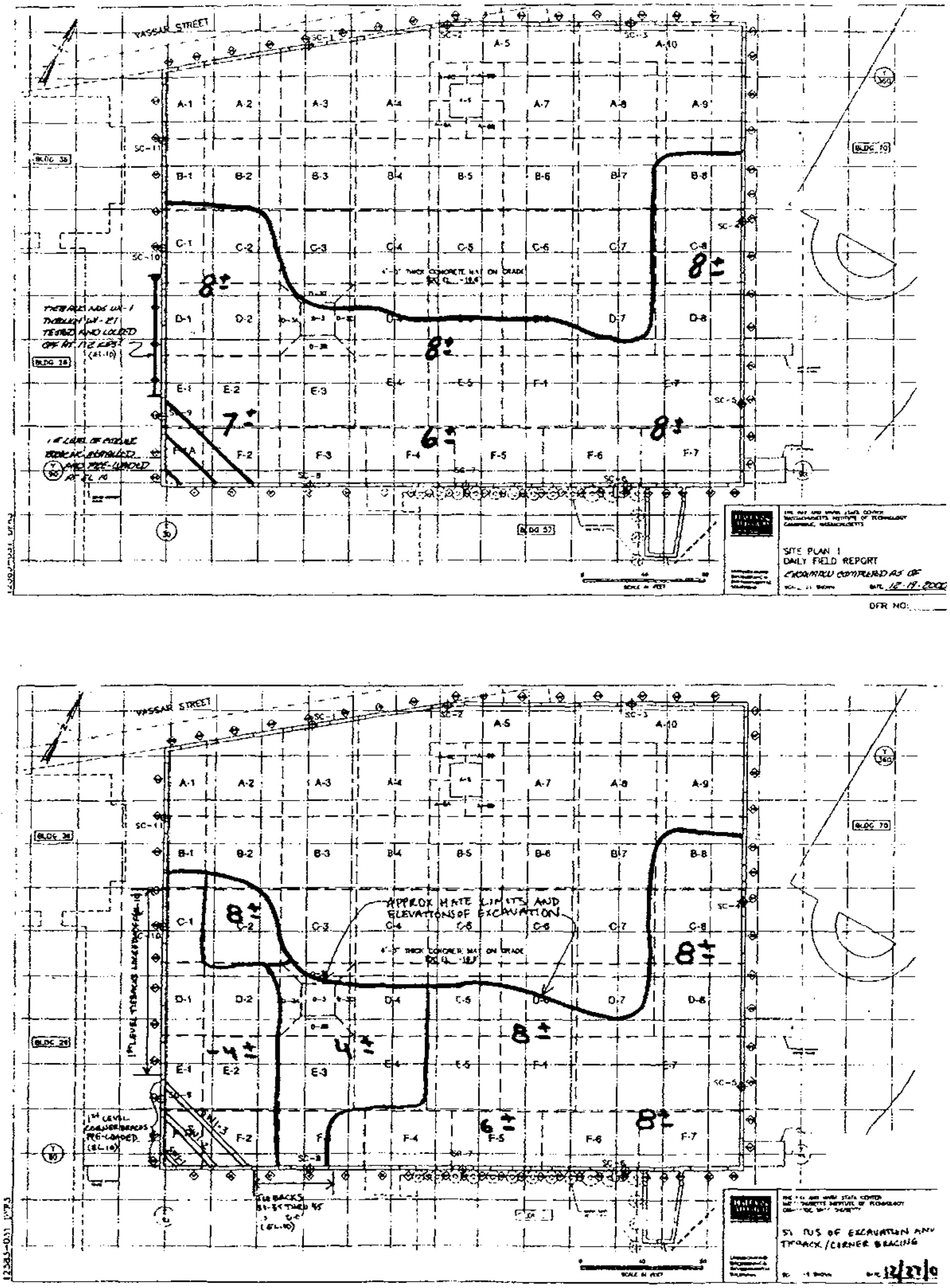
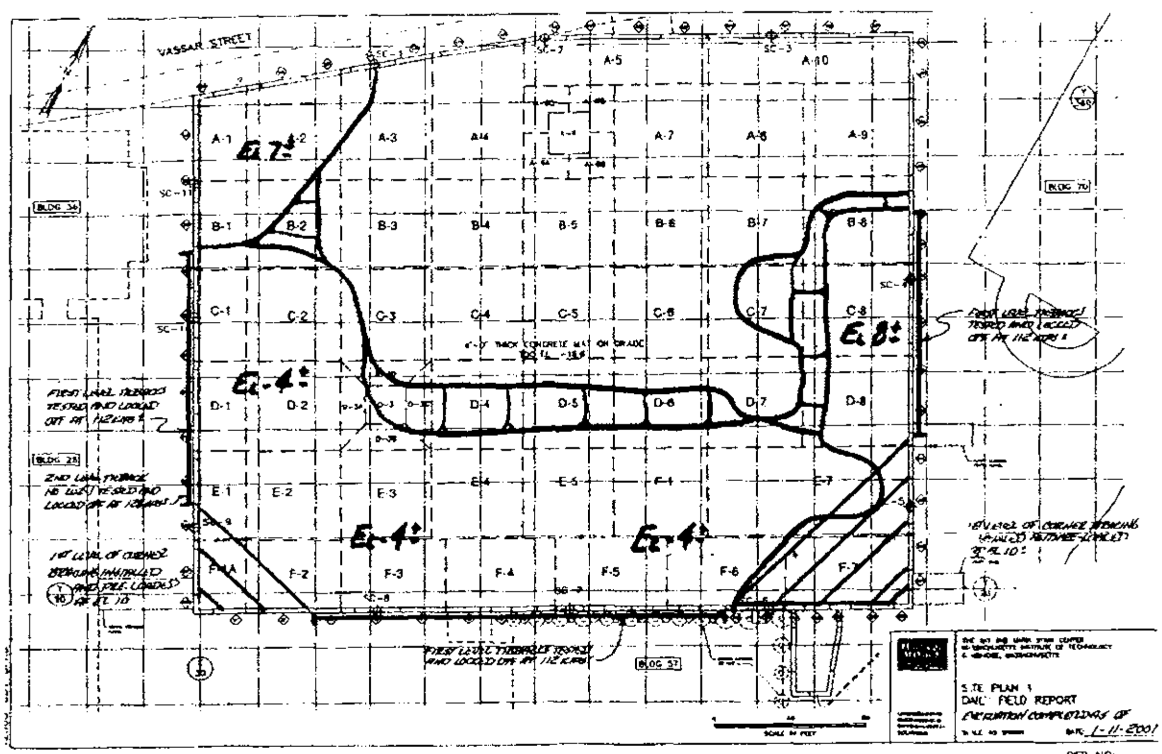


FIGURE B-2: Plan view of approximate excavation limits on December 19, 2000 (top) and December 27, 2000 (bottom). (Haley & Aldrich)



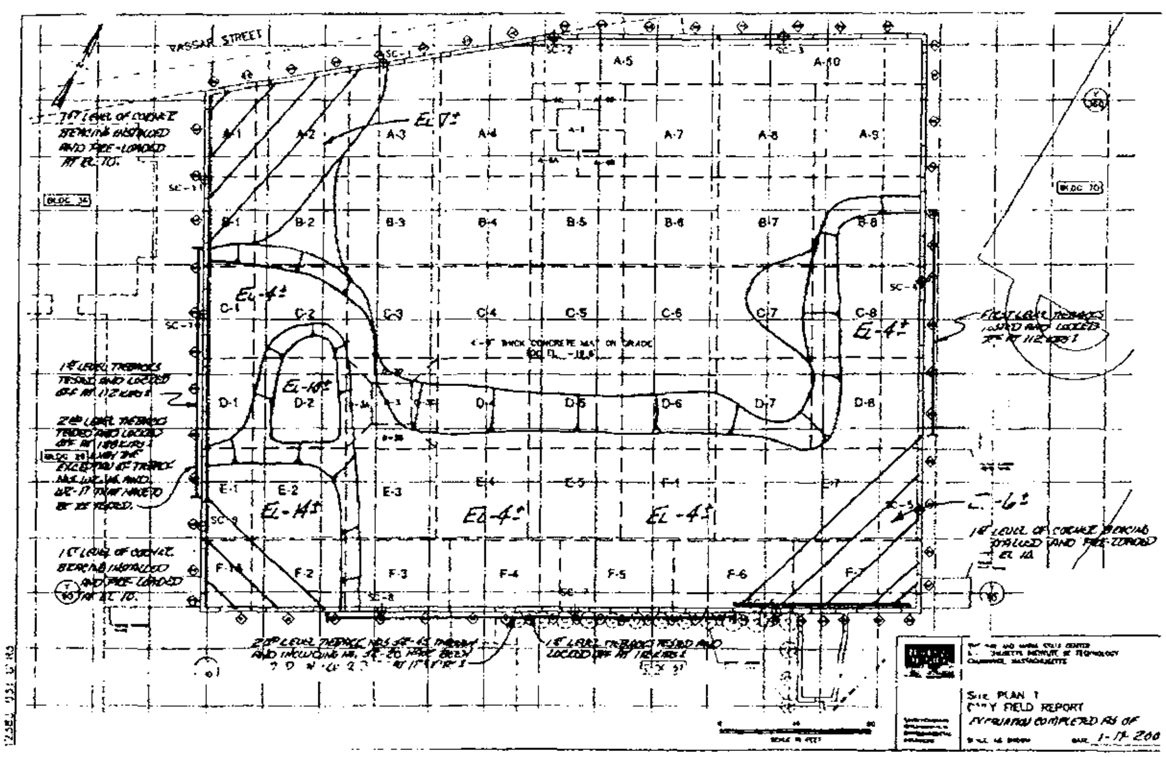
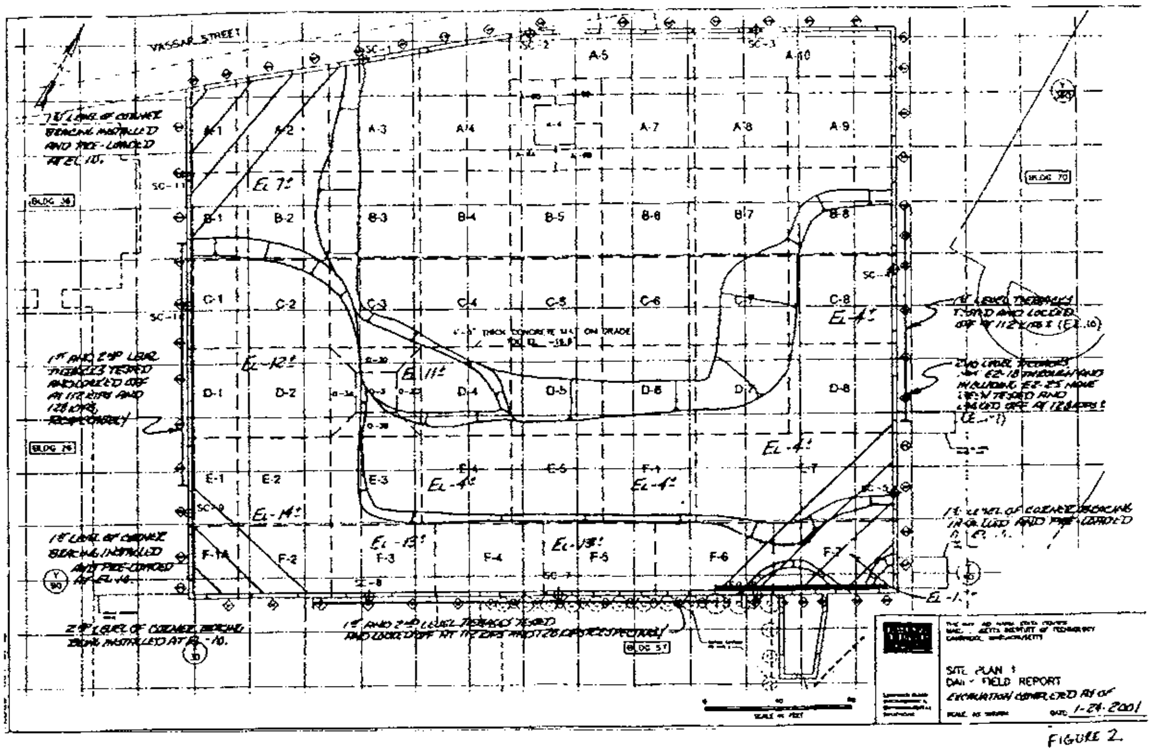


FIGURE B-3: Plan view of approximate excavation limits on January 11, 2001 (top) and January 17, 2001 (bottom). (Haley & Aldrich)



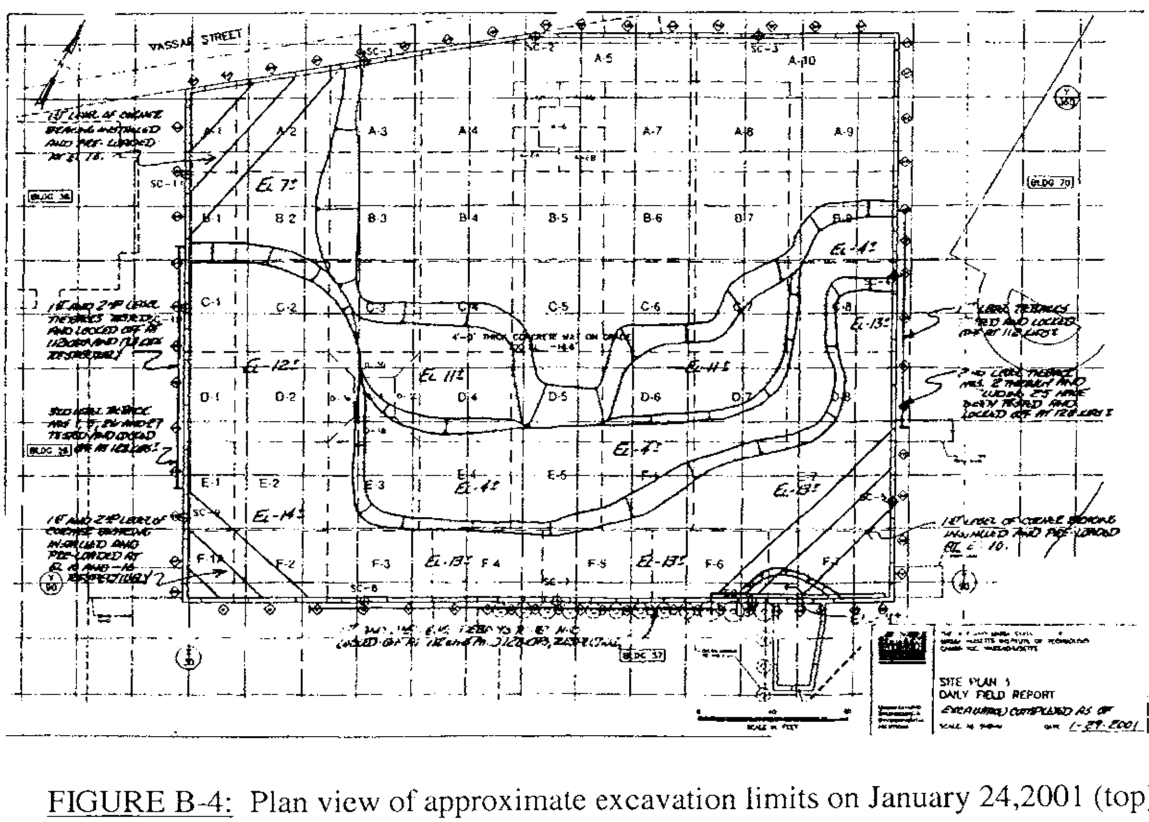
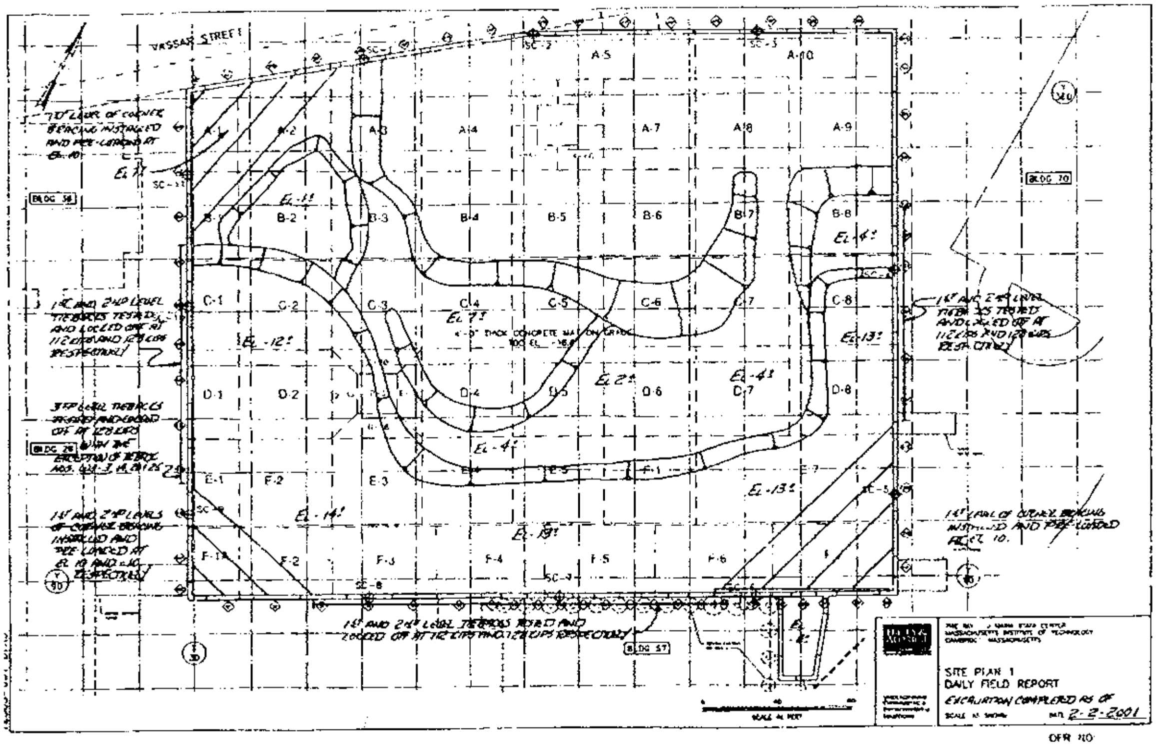


FIGURE B-4: Plan view of approximate excavation limits on January 24,2001 (top) and January 29, 2001 (bottom). (Haley & Aldrich)



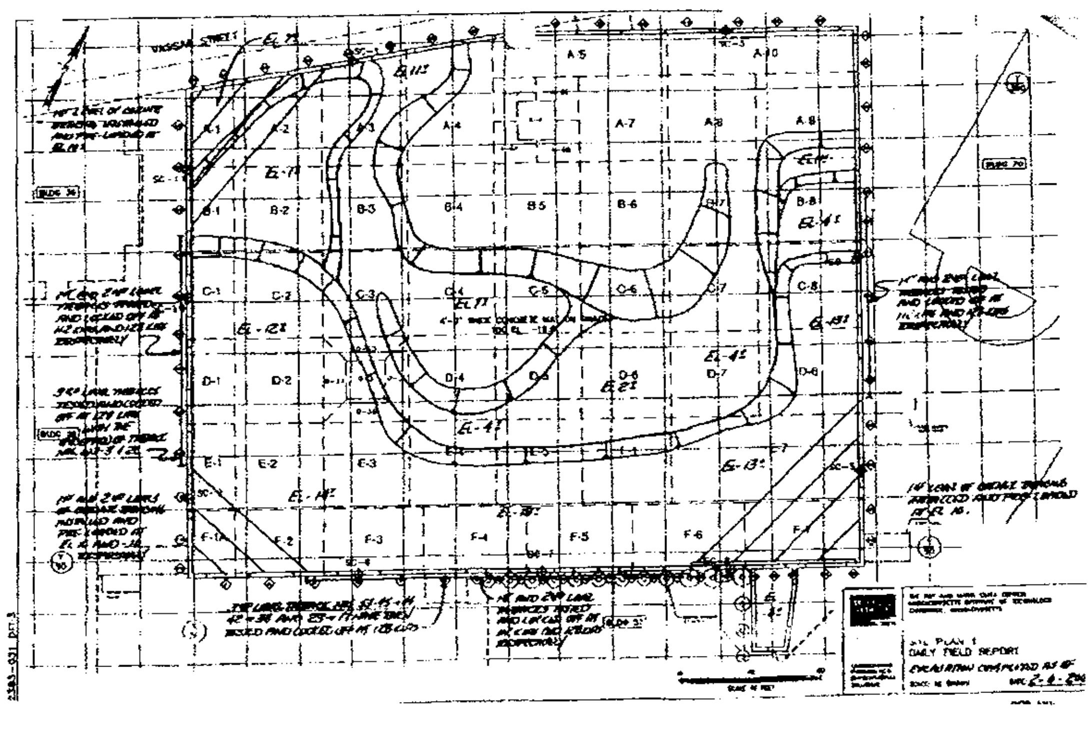
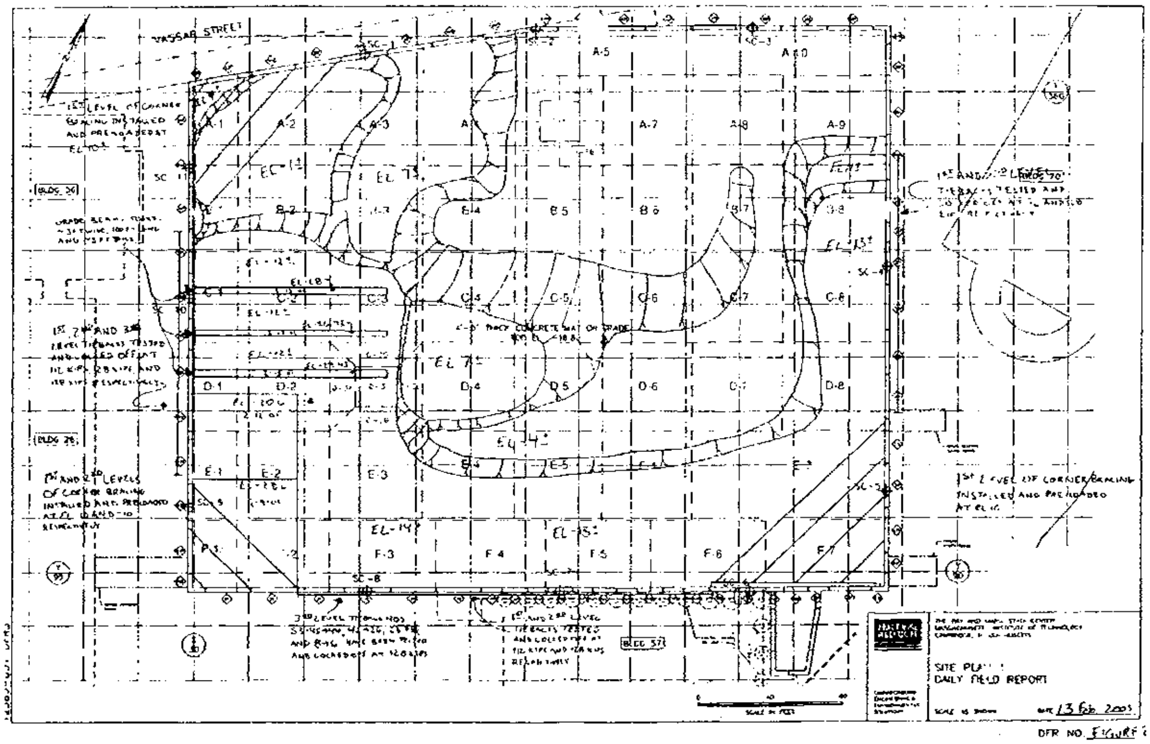


FIGURE B-5: Plan view of approximate excavation limits on February 2, 2001 (top) and February 6, 2001 (bottom). (Haley & Aldrich)



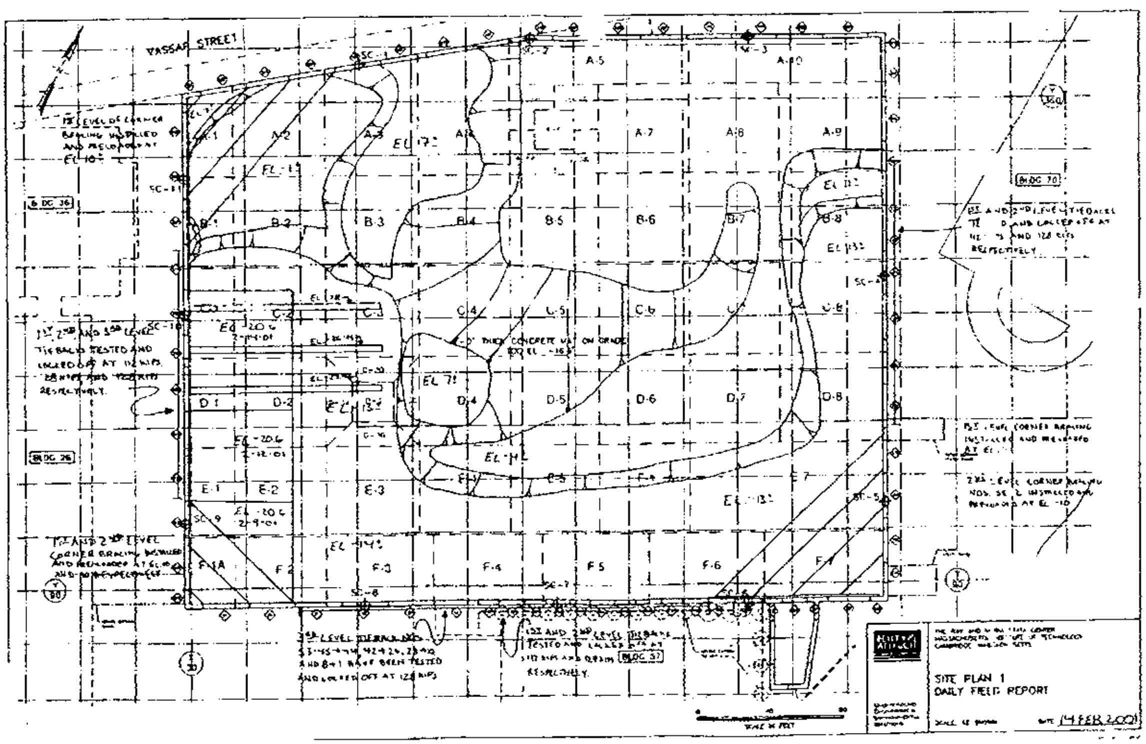
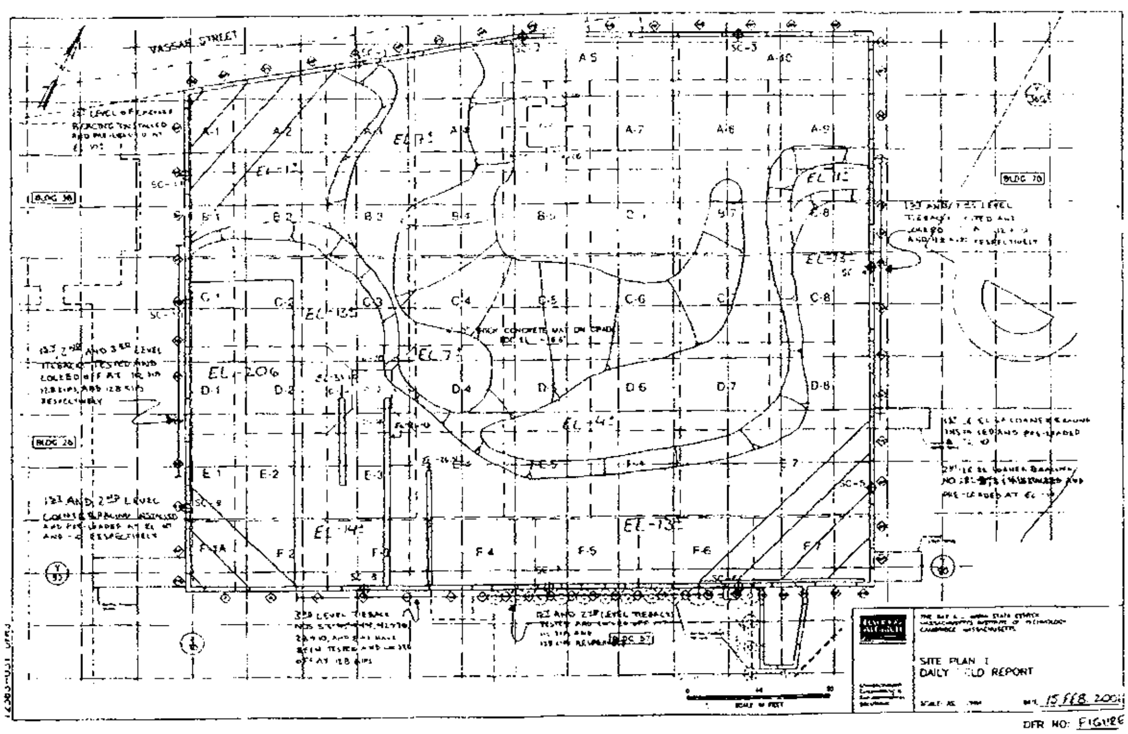


FIGURE B-6: Plan view of approximate excavation limits on February 13, 2001 (top) and February 14, 2001 (bottom). (Haley & Aldrich)



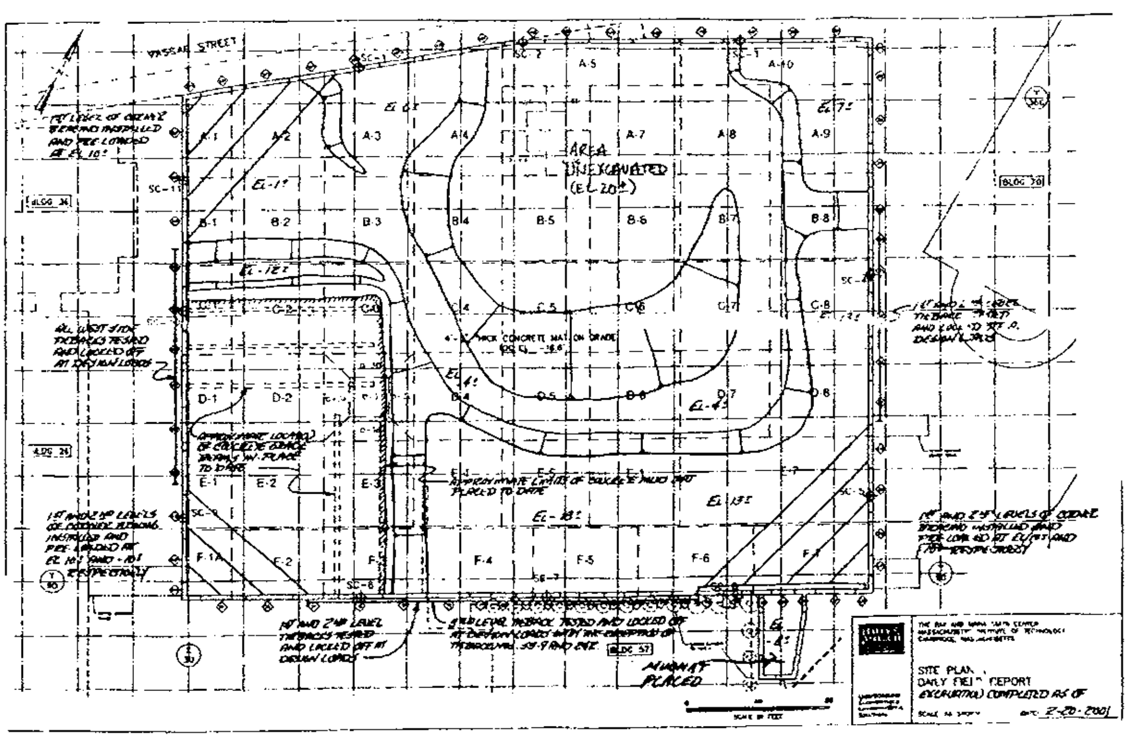
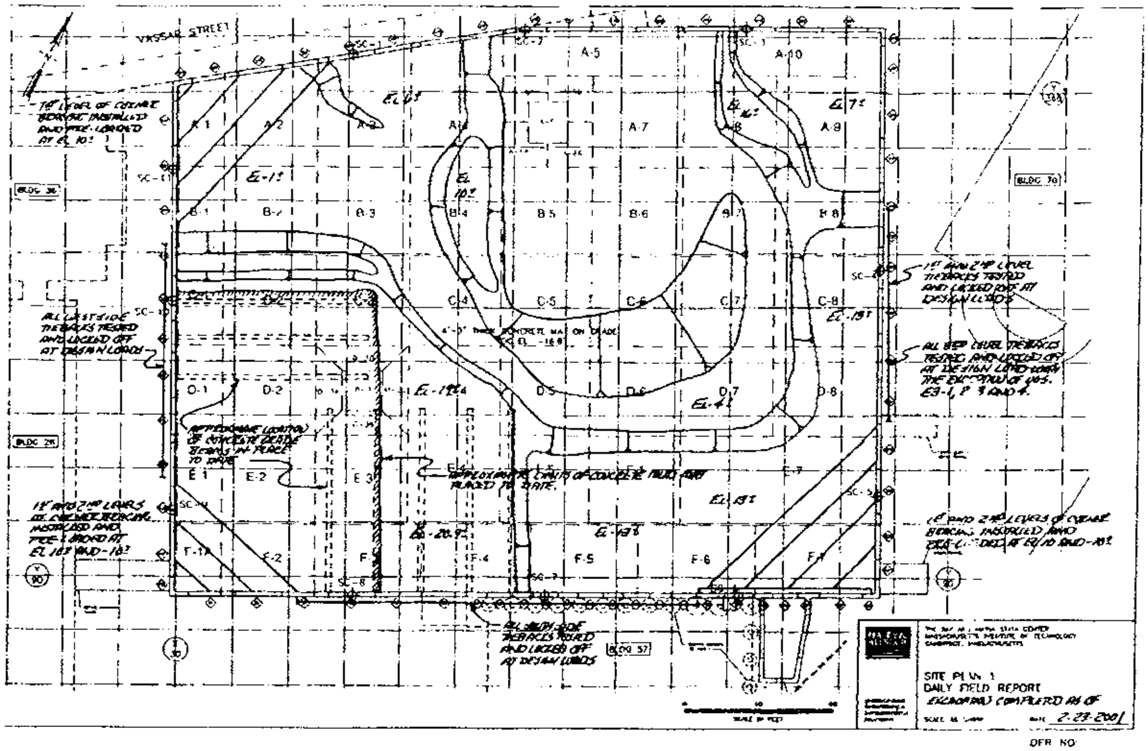


FIGURE B-7: Plan view of approximate excavation limits on February 15, 2001 (top) and February 20, 2001 (bottom). (Haley & Aldrich)



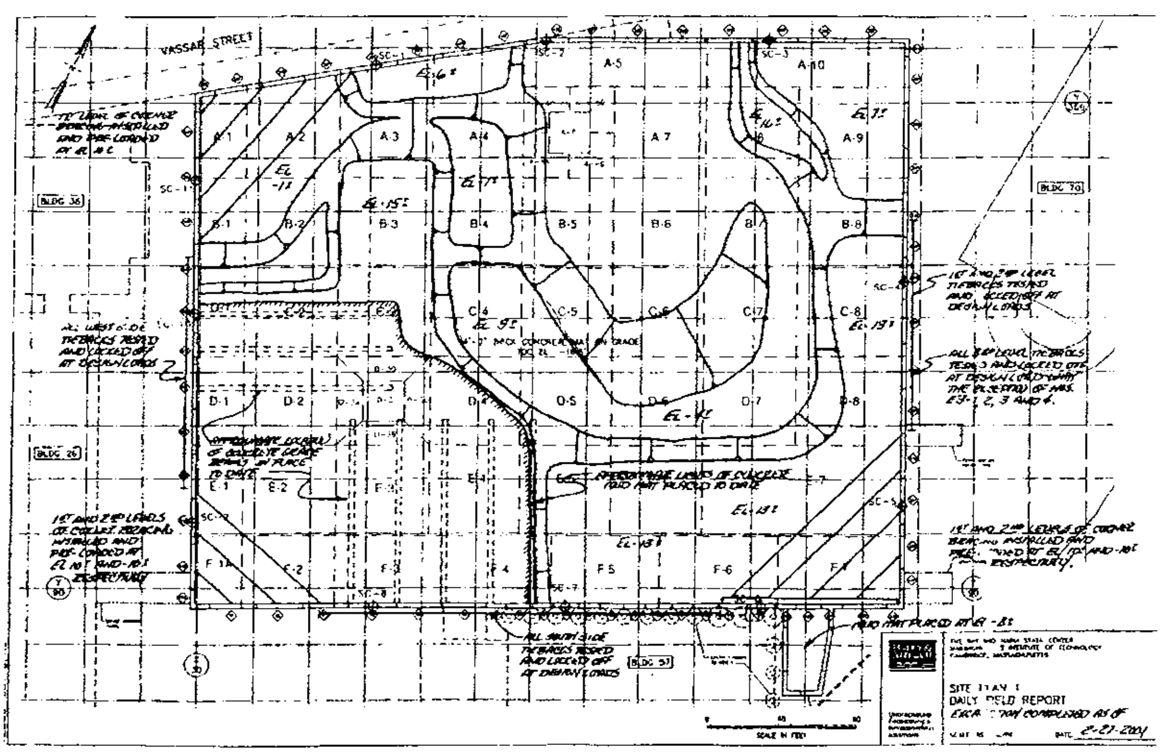
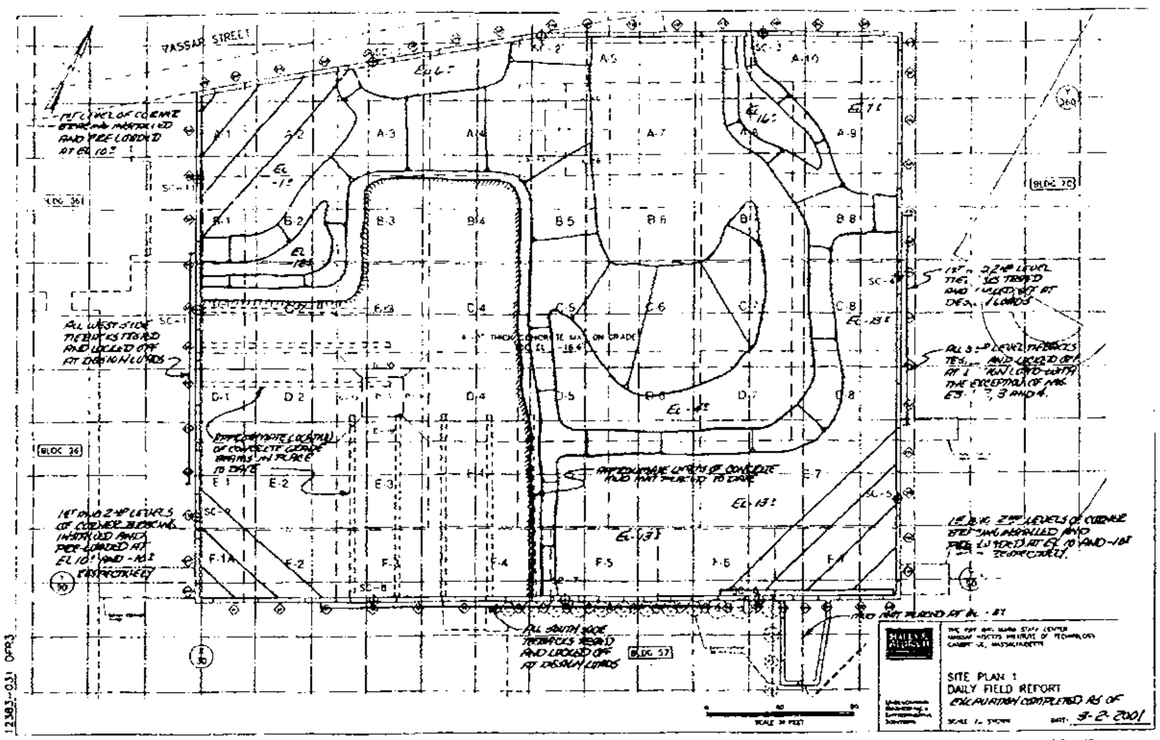


FIGURE B-8: Plan view of approximate excavation limits on February 23, 2001 (top) and February 27, 2001 (bottom). (Haley & Aldrich)



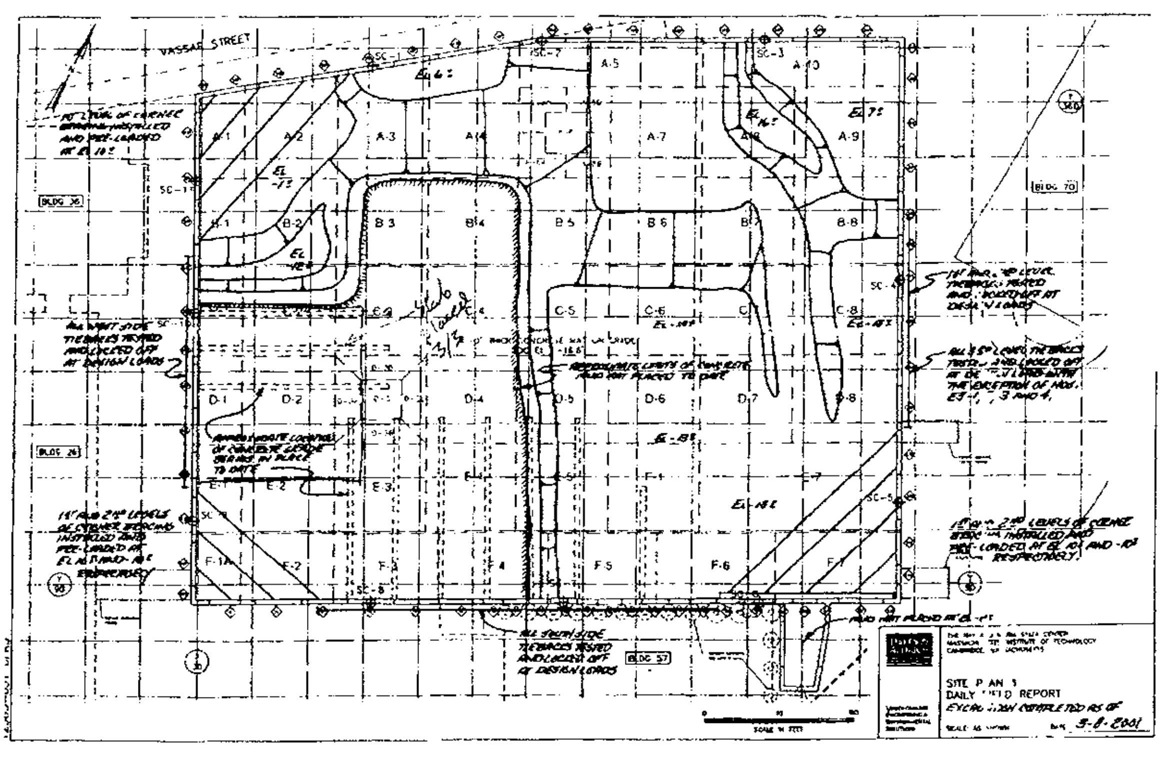
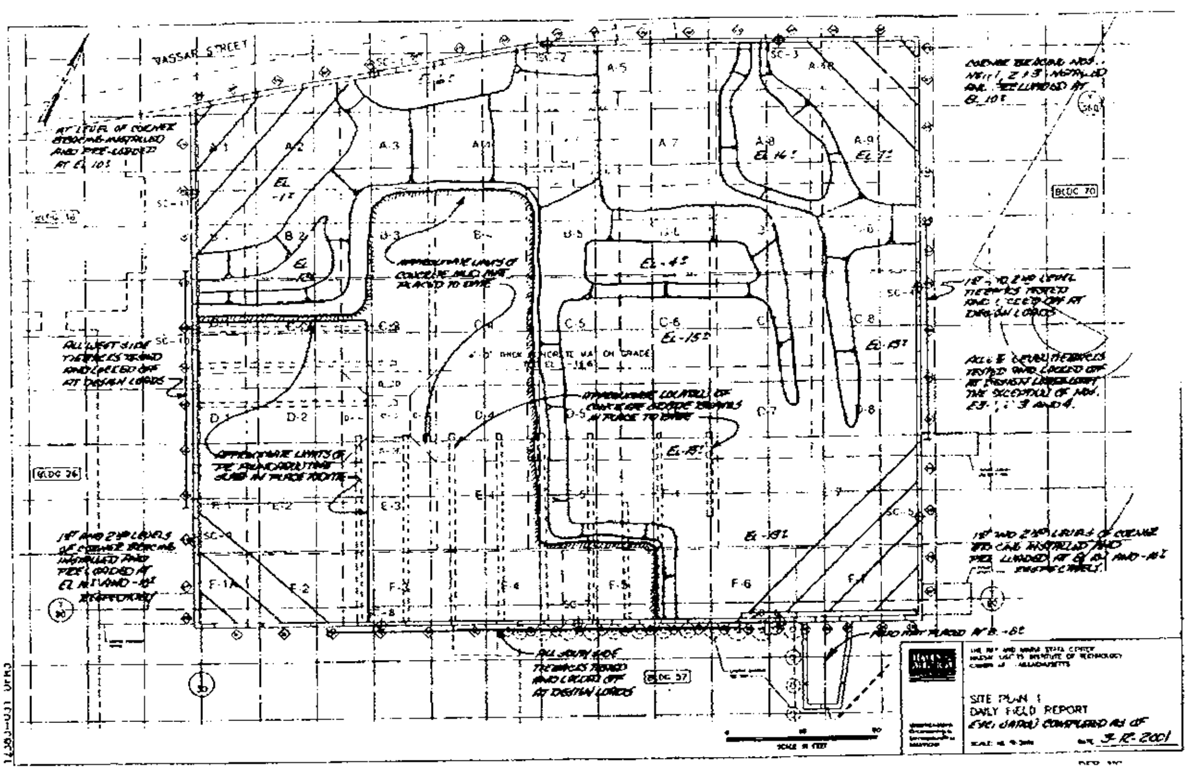


FIGURE B-9: Plan view of approximate excavation limits on March 2, 2001 (top) and March 8, 2001 (bottom). (Haley & Aldrich)



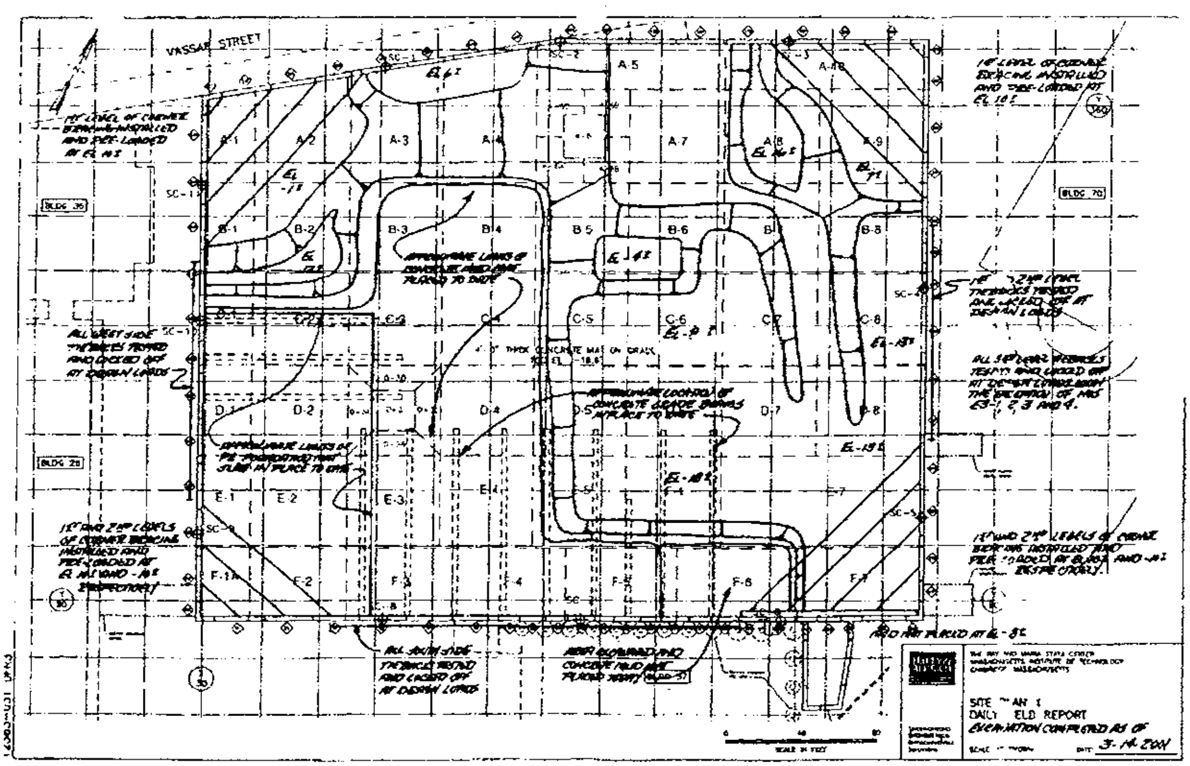
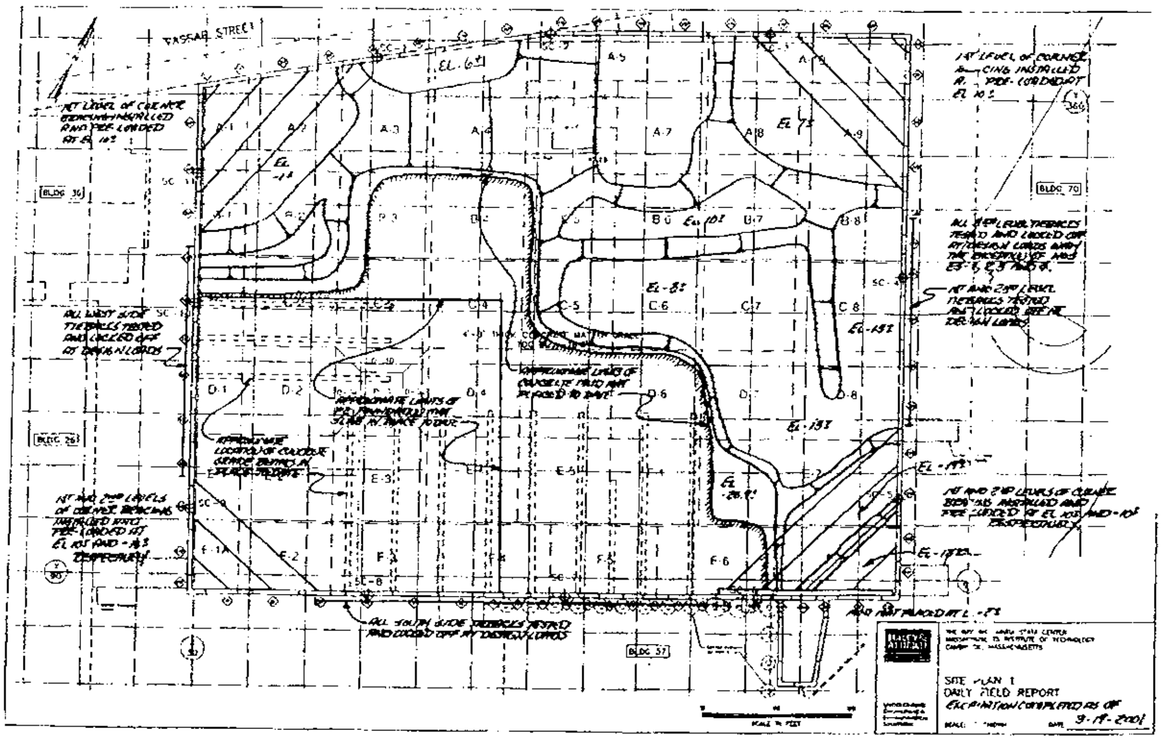


FIGURE B-10: Plan view of approximate excavation limits on March 12, 2001 (top) and March 14, 2001 (bottom). (Haley & Aldrich)



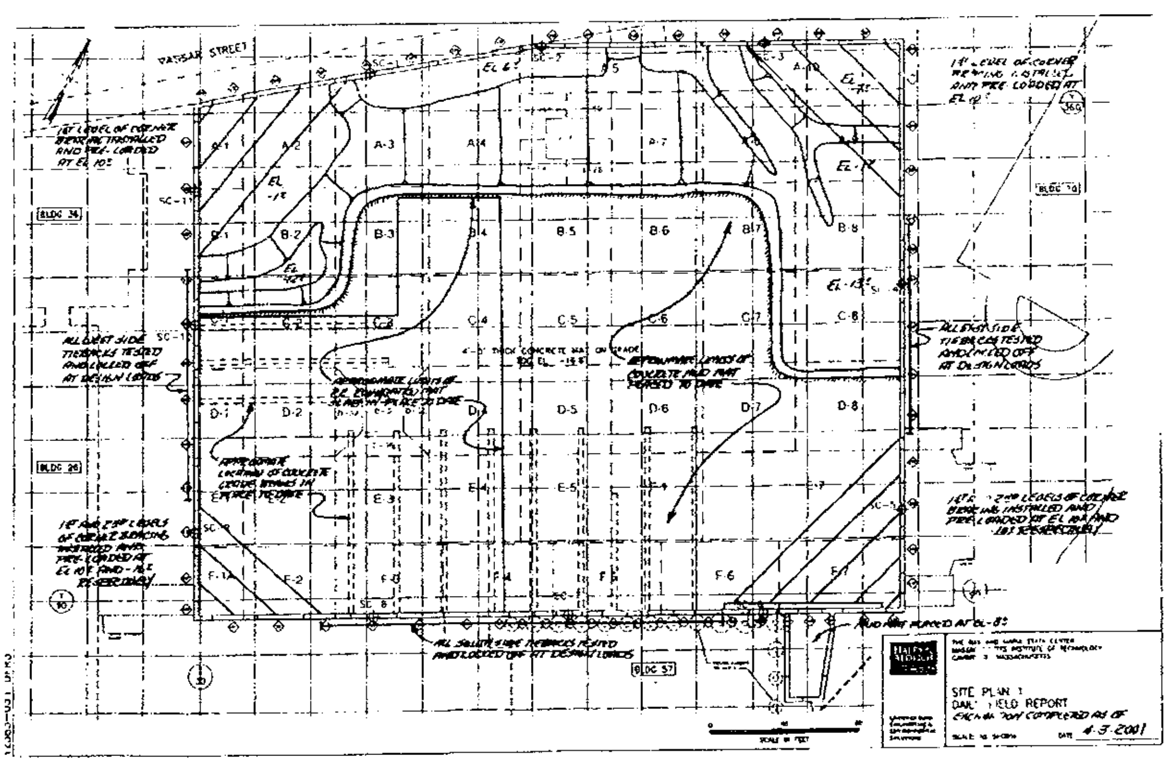
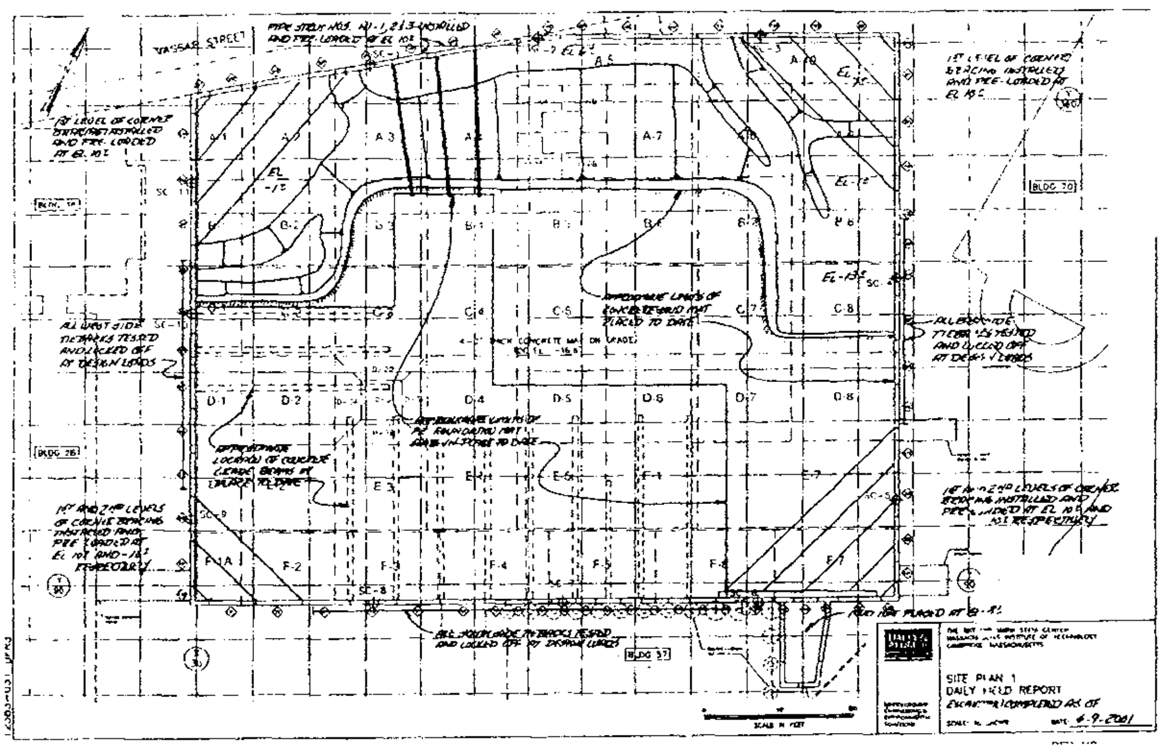


FIGURE B-11: Plan view of approximate excavation limits on March 19, 2001 (top) and April 3, 2001 (bottom). (Haley & Aldrich)



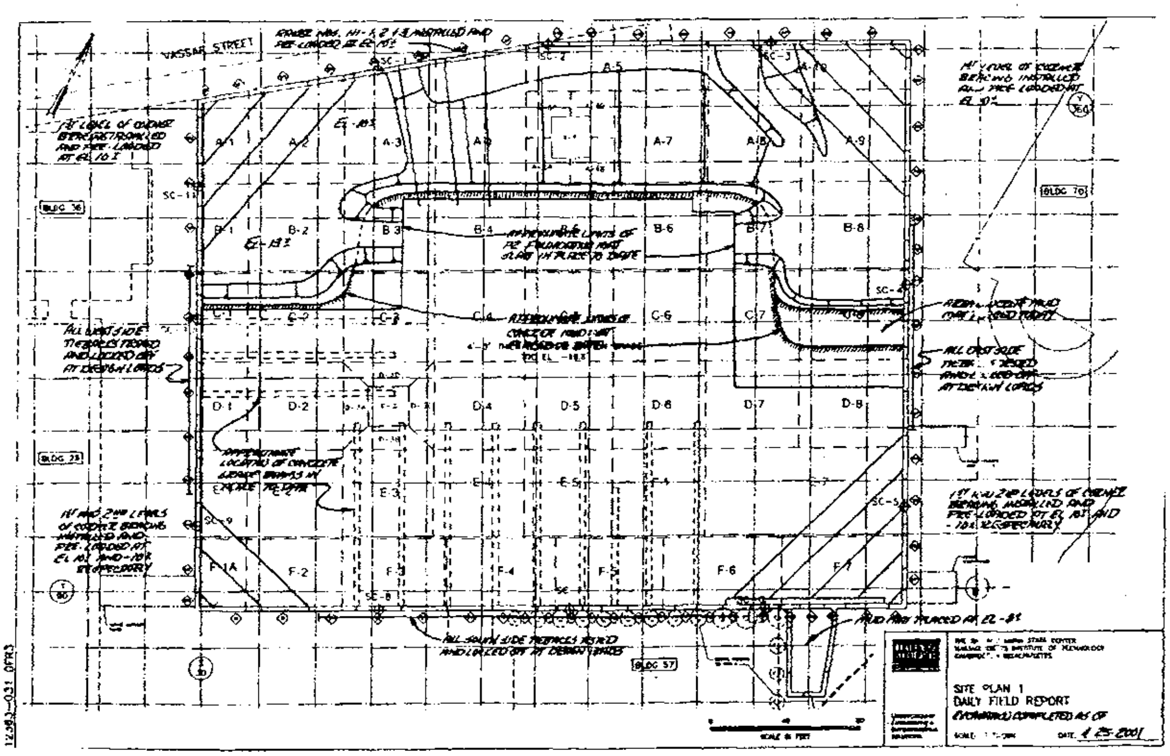


FIGURE B-12: Plan view of approximate excavation limits on April 9, 2001 (top) and April 25, 2001 (bottom). (Haley & Aldrich)

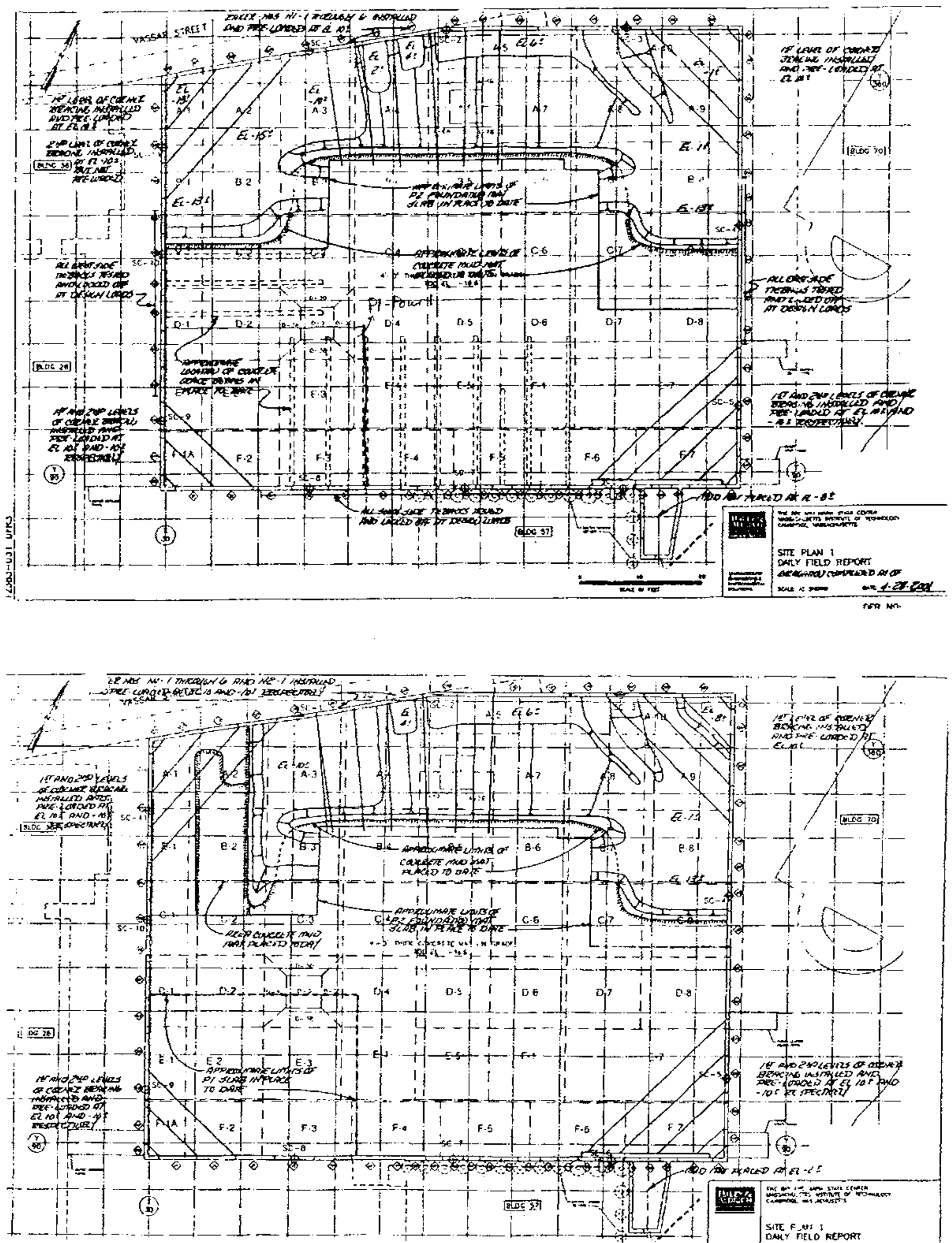
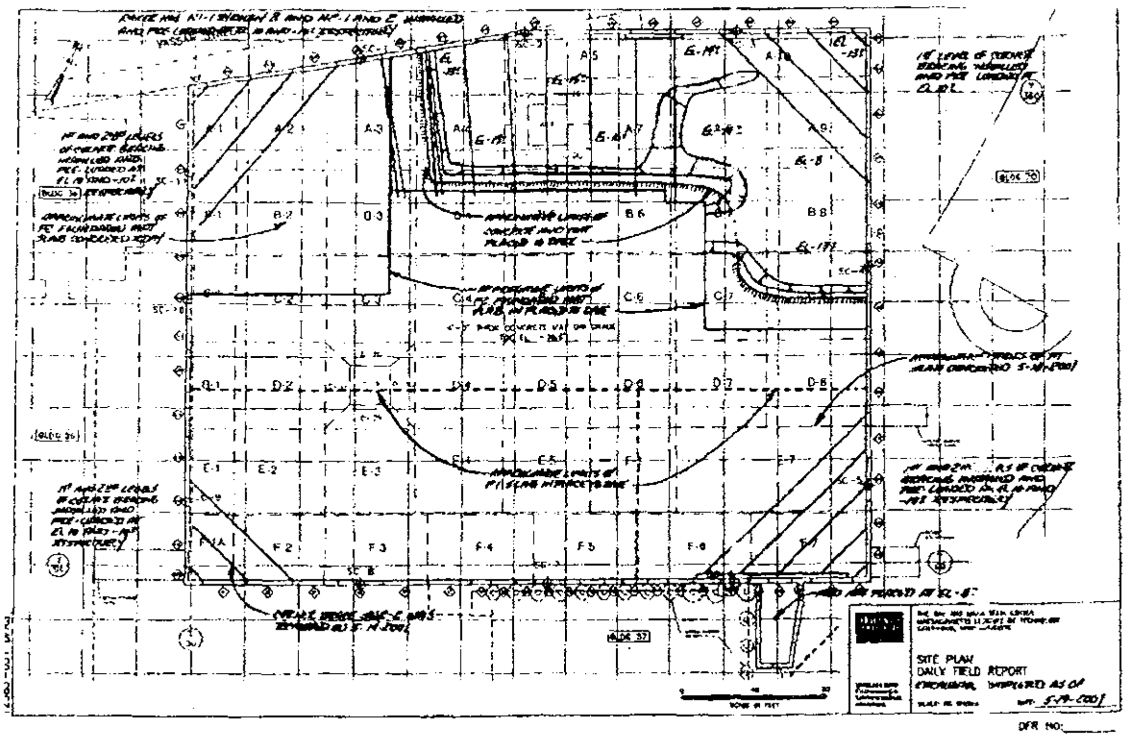


FIGURE B-13: Plan view of approximate excavation limits on April 28, 2001 (top) and May 8, 2001 (bottom). (Haley & Aldrich)



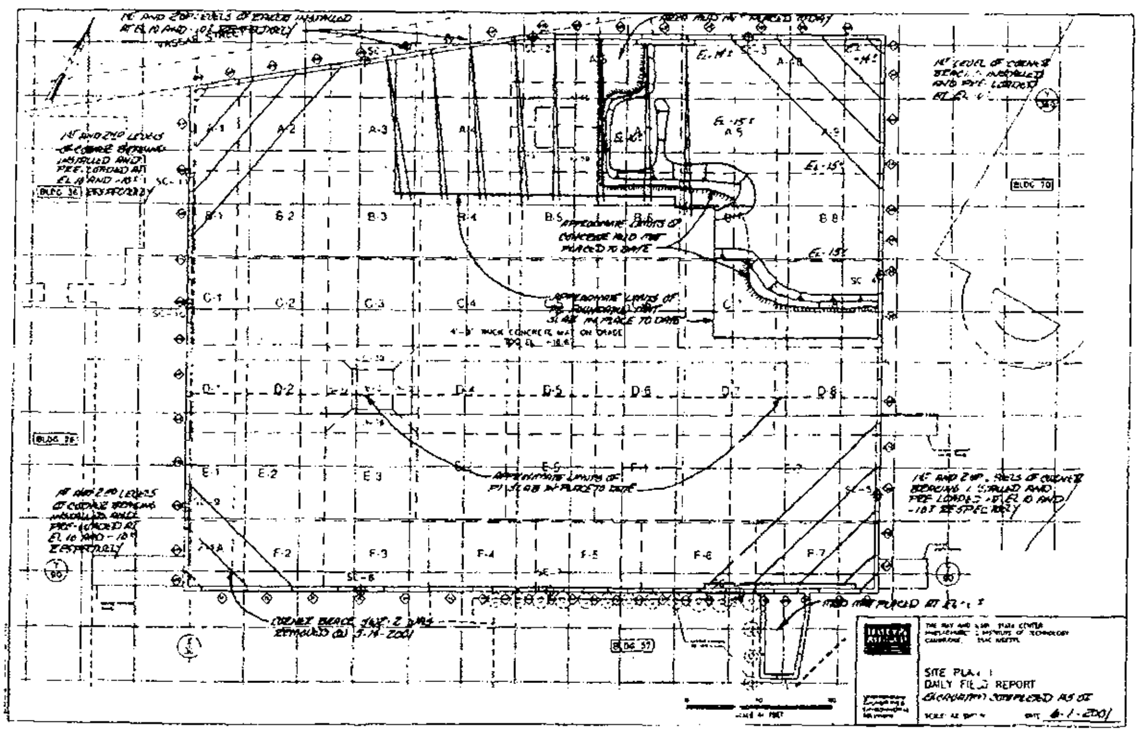
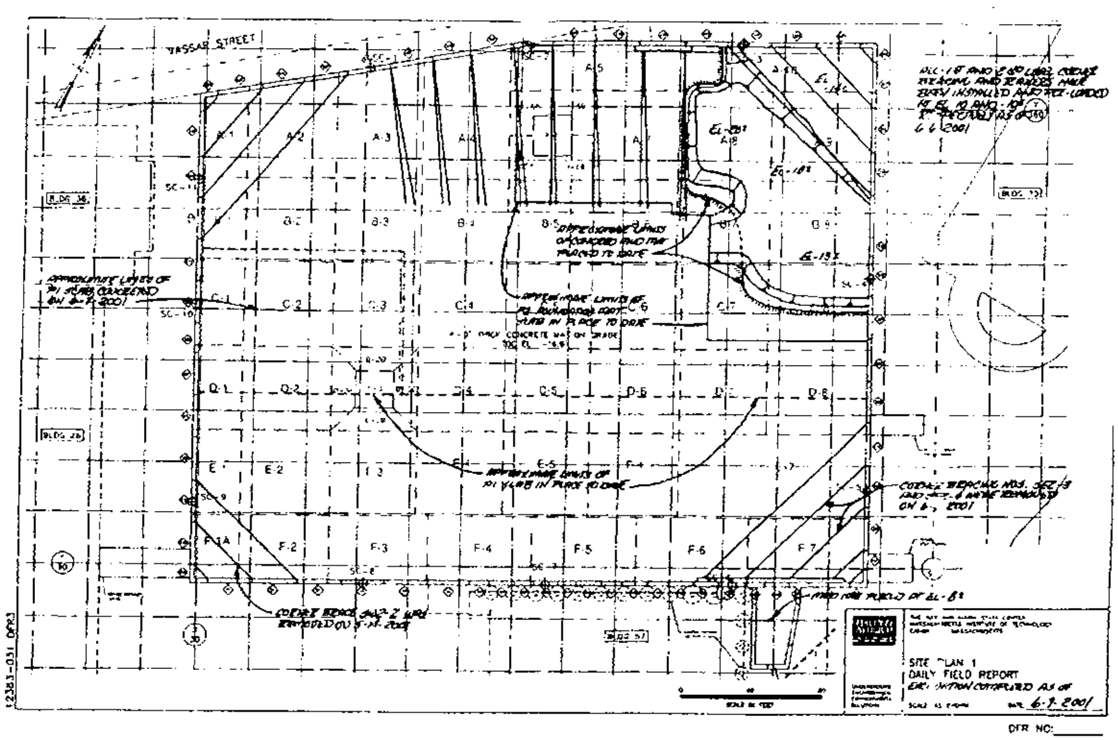


FIGURE B-14: Plan view of approximate excavation limits on May 19, 2001 (top) and June 1, 2001 (bottom). (Haley & Aldrich)



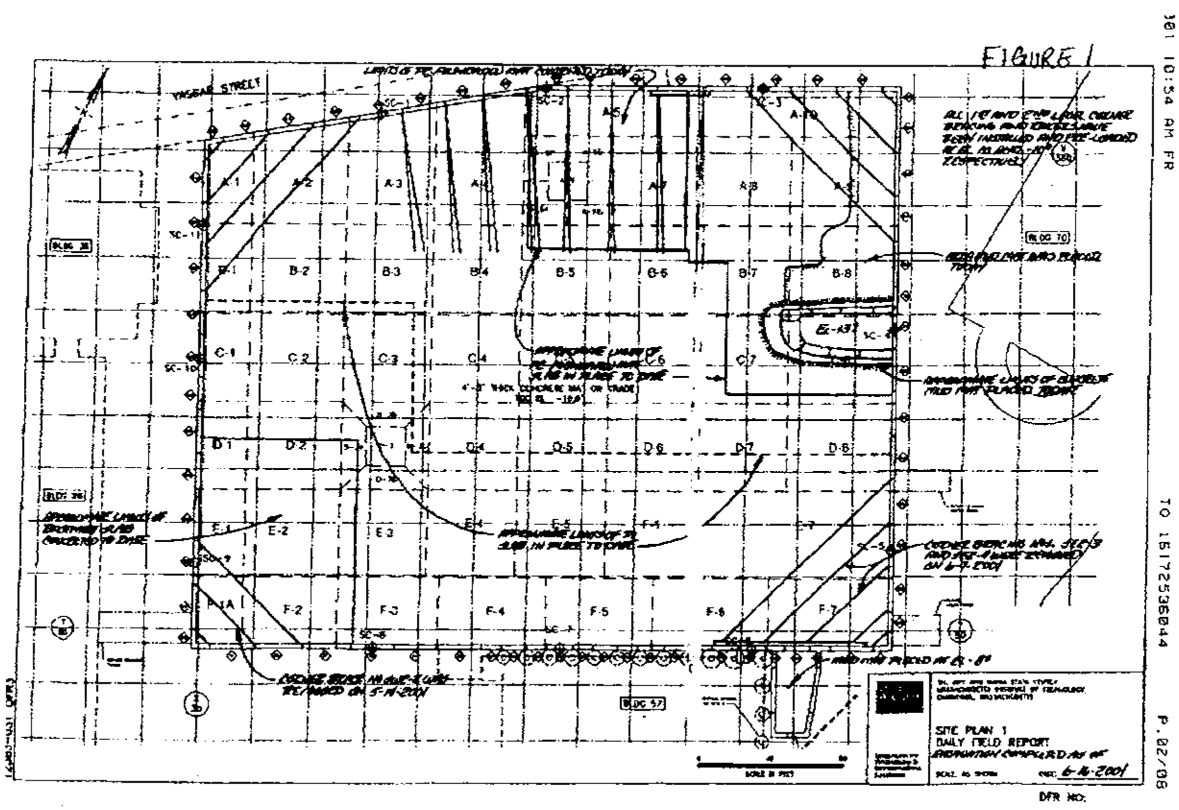


FIGURE B-15: Plan view of approximate excavation limits on June 9, 2001 (top) and June 16, 2001 (bottom). (Haley & Aldrich)

APPENDIX C: CONSTRUCTION PHOTOGRAPHS

Description of Figures C-1 to C-17

Figure C-1: Site on 9/15/01
State of site during slurry wall construction and before excavation viewed from the northwest corner.

Figure C-2: Excavation on 11/28/00
Excavation along the west, south and east sides.
Top Left: view of northwest portion of excavation from southwest corner.
Top Right: view of the middle of the site from southwest corner.
Bot. Right: view of south portion of the site from the southwest corner.
Bot. Left: view of southeast portion of site from the northeast corner.

Figure C-3: Excavation on 1/18/01
Excavation along the west, south and east sides.
Top Left: view of northwest and middle portions of site from southwest corner.
Right: view of south portion of the site from the southwest corner.
Bot. Left: view of south portion of site from middle of the east side.

Figure C-4: Excavation on 1/29/01
Top Left: view of middle portion of site from southwest corner.
Right: view of south portion of the site from the southwest corner.
Bot. Left: view of northwest portion of site from the southwest corner.

Figure C-5: Excavation on 2/5/01
Excavation and bracing along the west, south and east sides.
Top Left: view of northwest portion of excavation from southwest corner.
Top Middle: view of middle of the site from southwest corner.
Top Right: view of the south portion of the site from southwest corner.
Bot. Right: southeast corner and east side of site from the southeast corner.
Bot. Left: middle of the site showing the truck loading operation from the east side of the site.

Figure C-6: Excavation on 2/12/01
Excavation and bracing along the west, south and east sides.
Top Left: view of the northwest portion of site from southwest corner.
Right: view of south portion of the site from the southwest corner.
Bot. Left: view of middle portion of site from the southwest corner.

Figure C-7: Excavation on 2/20/01
Excavation and supports along the west, south and east sides.
Top Left: view of northwest portion of site from southwest corner.
Top Right: view of the middle of the site from southwest corner.
Bot. Right: view of south portion of the site from the southwest corner.

Bot. Left: view of southwest corner of site from the southwest corner showing installation of strip drains and the mudmat.

Figure C-8: Excavation on 2/28/01

Shows the berm on the north, excavation to grade on the west side, construction of reinforcing steel in the southwest and installation of supports in the northeast.

Top Left: view of northwest portion of site from southwest corner.

Top Right: view of the middle of the site from southwest corner.

Bot. Right: view of south portion of the site from the southwest corner.

Bot. Left: view of northeast corner of site from the east showing installation of corner bracing.

Figure C-9: Excavation on 3/7/01

View of the site from the southwest corner after a snow storm.

Figure C-10: Excavation on 3/14/01

Shows the state of excavation and construction from the north.

Top Left: view of east portion of site.

Top Right: view of the east middle portion of the site.

Bot. Right: view of west middle portion of the site.

Bot. Left: view of west portion of the site.

Figure C-11: Excavation on 3/19/01

Shows the state of excavation and construction from the southwest.

Top Left: the middle of the site with the remaining berm and soil to excavate.

Top Right: southern portion of the site with excavation moving east.

Bot. Right: southwest corner with the completed slab covered with plastic.

Bot. Left: northwest corner with the soil berm and preparation for slab construction.

Figure C-12: Excavation on 3/28/01

Shows the state of excavation and construction from the southwest.

Top Left: the middle of the site with a large portion of site at grade and berm remaining on the north.

Top Right: southern portion of the site with excavation almost completed along the south wall.

Bot. Right: soil berm on the north side and construction of a slab section with kicker blocks for the rakers.

Bot. Left: northwest corner with the soil berm and preparation for slab construction.

Figure C-13: Excavation on 4/4/01

Shows the state of excavation and construction from the southwest.

Top Left: the middle of the site with a large portion of site at grade and berm remaining on the north.

Top Right: southern portion of the site with excavation completed along the south wall and steel preparation for slab.

Bot. Right: soil berm on the north side and completed slab section with kicker blocks for the rakers with notch in berm for raker construction.

Bot. Left: northwest corner.

Figure C-14: Excavation on 4/10/01

Shows the state of excavation and construction from the southwest.

Top Left: the middle of the site with a large portion of site at grade and berm remaining on the north and forms being placed in southwest corner for the second level slab.

Top Right: southern portion of the site with excavation completed along the south wall and forms being placed in southwest corner for the second level slab.

Bot. Right: soil berm on the north side and the first three level one rakers.

Bot. Left: northwest corner.

Figure C-15: Excavation on 4/18/01

Shows the state of excavation and construction from the southwest.

Top Left: northwest corner excavation for installation of level two corner bracing.

Top Right: middle of the site showing rakers and excavation work along with the form for the second level slab in the southwest (black in squares on the bottom right of the picture).

Bot. Right: southern portion of the site with the form for the second level slab.

Bot. Left: northwest corner excavation for the second level corner bracing.

Figure C-16: Excavation on 5/4/01

Excavation progress and support construction.

Top Left: middle of the site showing rakers, excavation on the north, completed second level slab and continued work of second level slab.

Top Right: southern part of the wall showing the first section of the second level slab in place and the extension of the form for this layer along the south wall from the west to the east.

Bot. Right: view from the northwest showing the first and second level construction.

Bot. Left: northwest corner showing rakers covered to insulate them from temperature changes.

Figure C-17: Excavation on 5/30/01

Construction progress shown from the southwest and south. These pictures show that the rakers are completed and all that remains for the excavation work is the northeast corner and installation of the second level of corner bracing. It can be also be seen that the construction of the second level slab has progressed with the form along the west wall and all of the slab completed along the south wall.

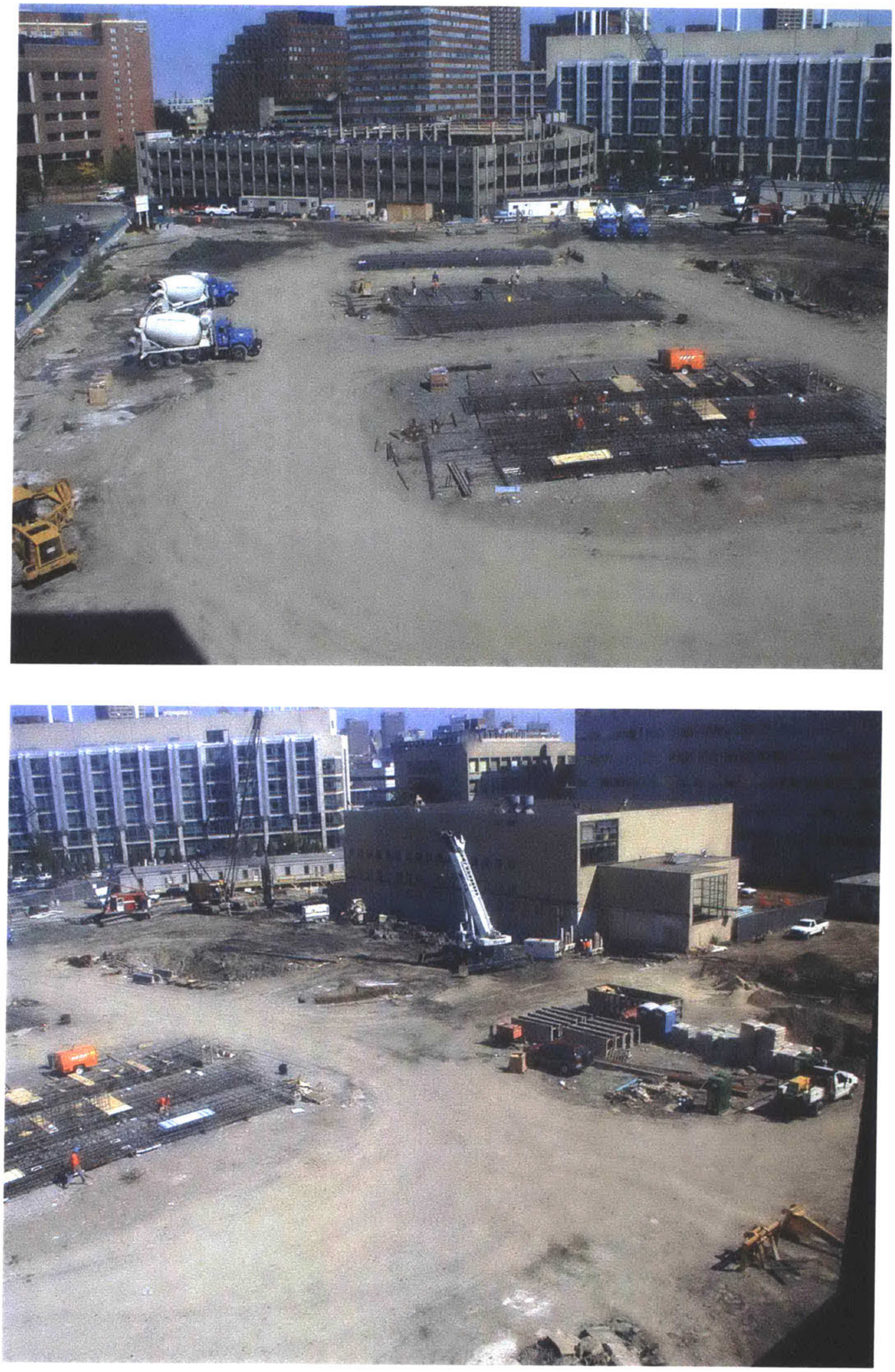


FIGURE C-1: Site on 9/15/00.

FIGURE C-2: Excavation on 11/28/00.

FIGURE C-3: Excavation on 1/18/01.

FIGURE C-4: Excavation on 1/29/01.

FIGURE C-5: Excavation on 2/5/01.

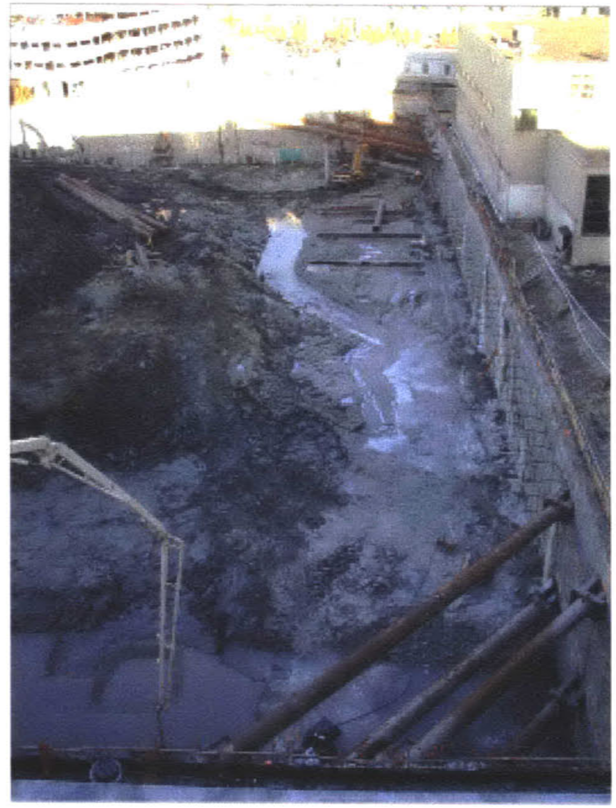


FIGURE C-6: Excavation on 2/12/01.

FIGURE C-7: Excavation on 2/20/01.

FIGURE C-8: Excavation on 2/28/01.

FIGURE C-9: Excavation on 3/7/01.

FIGURE C-10: Excavation on 3/14/01.

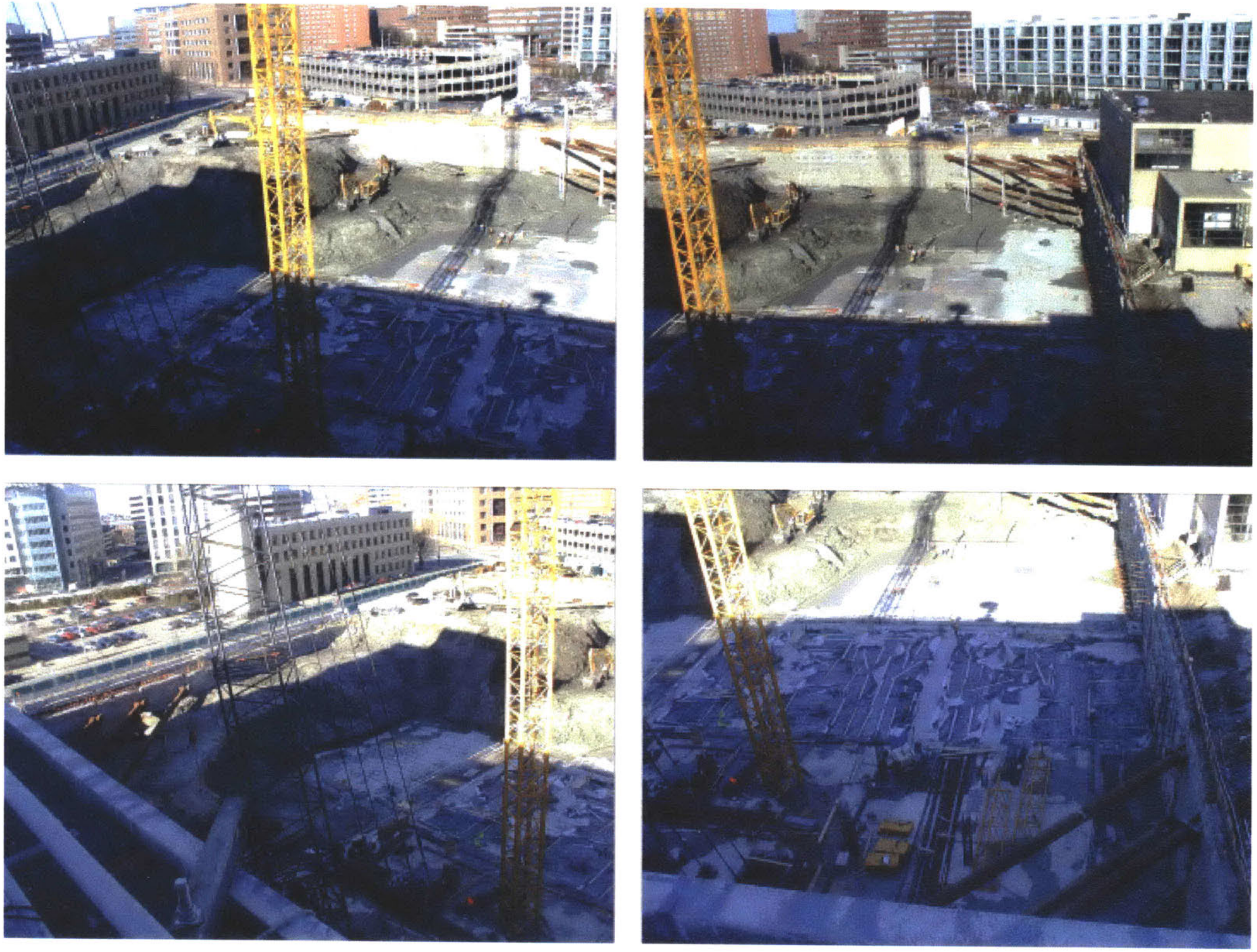


FIGURE C-11: Excavation on 3/19/01.

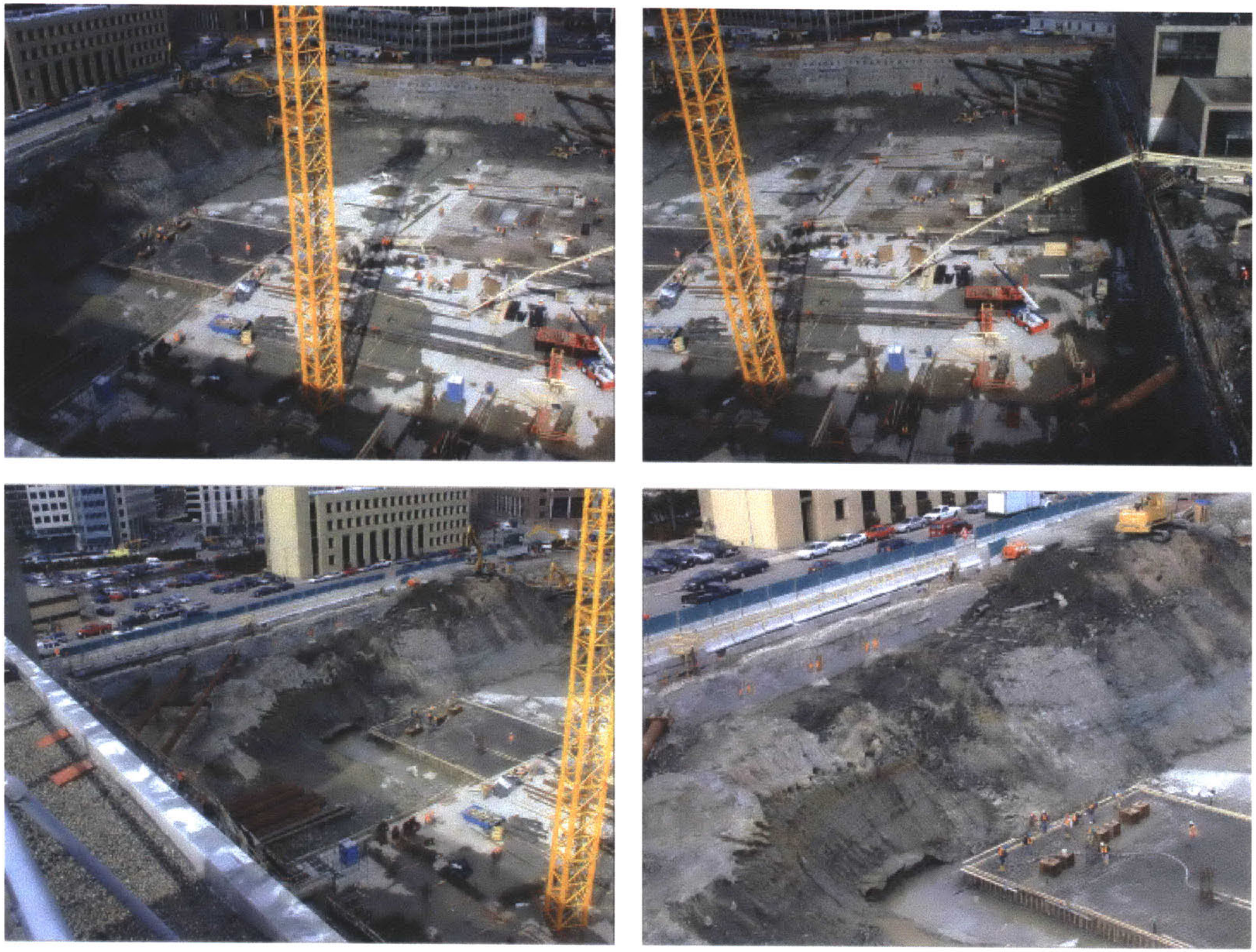


FIGURE C-12: Excavation on 3/28/01.

FIGURE C-13: Excavation on 4/4/01.

FIGURE C-14: Excavation on 4/10/01.

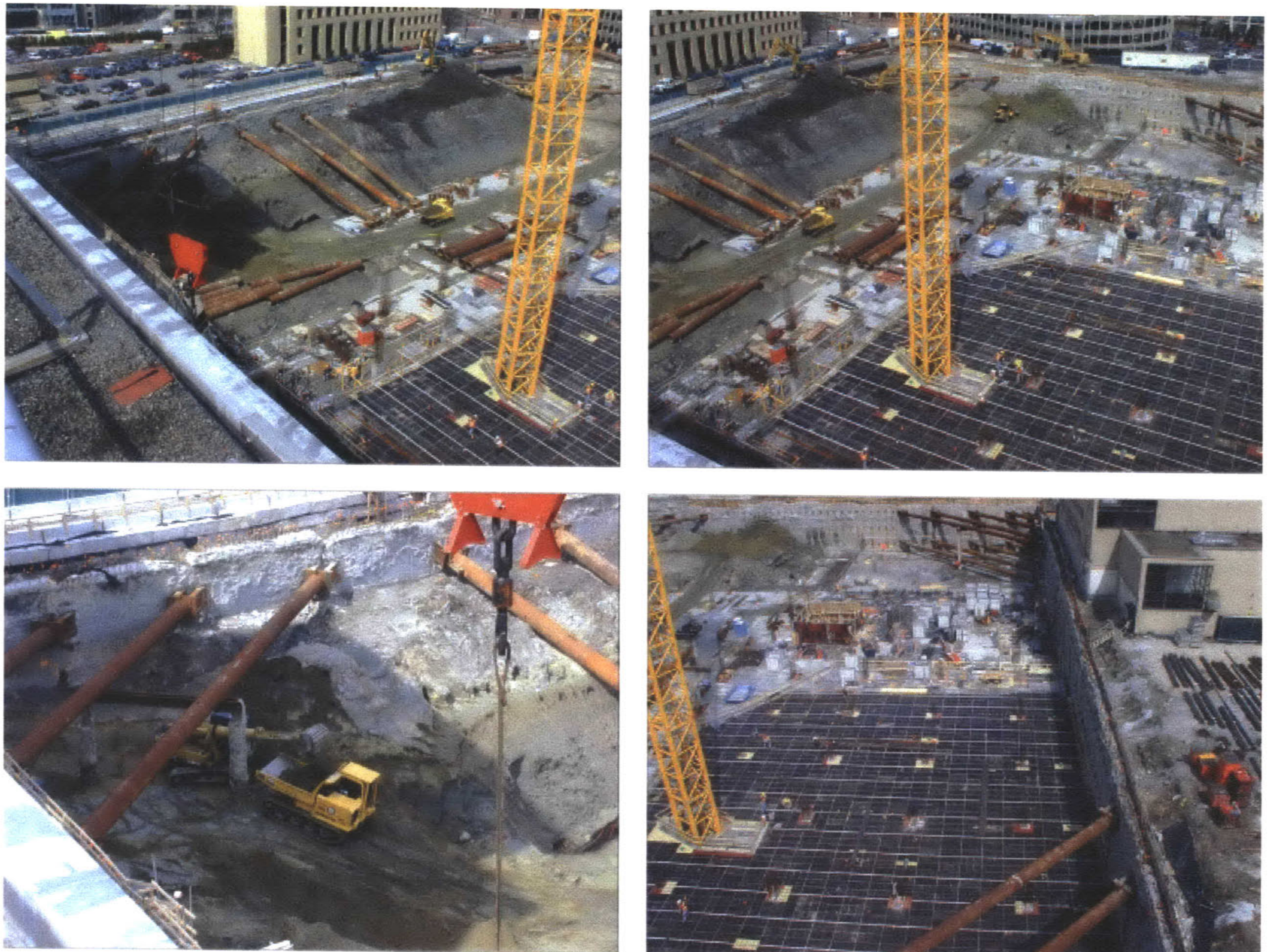


FIGURE C-15: Excavation on 4/18/01.

FIGURE C-16: Excavation on 5/4/01.

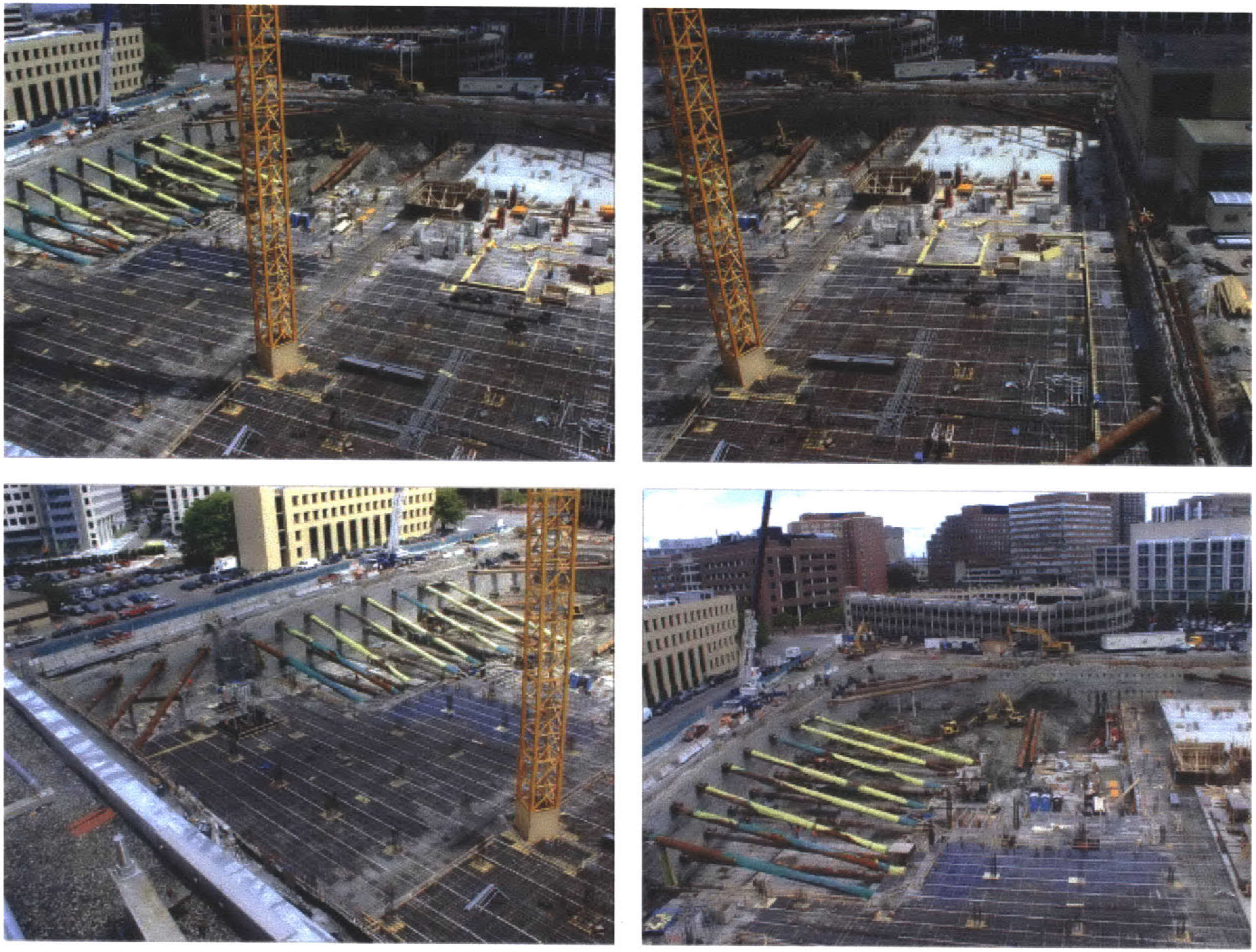


FIGURE C-17: Excavation on 5/30/01.

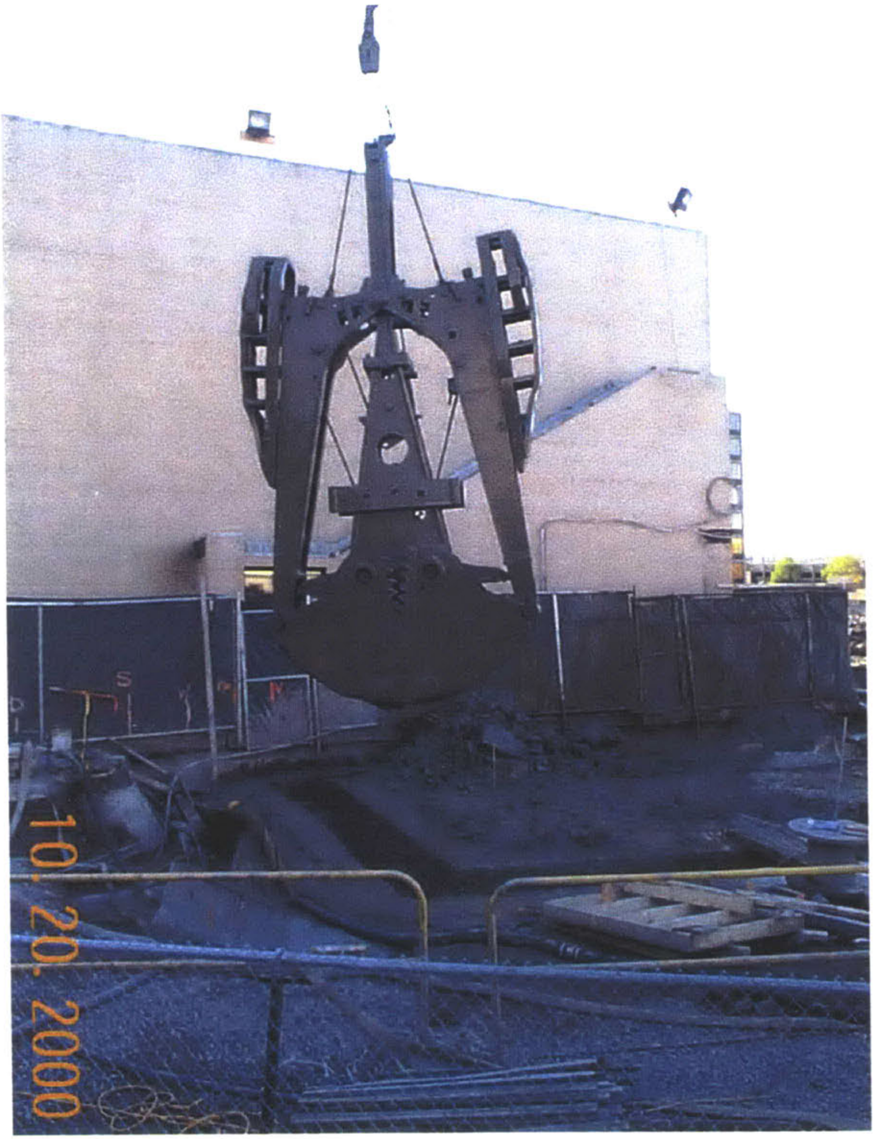


FIGURE C-18: Mechanical clamshell used to excavate wall panels.

FIGURE C-19: Placement of reinforcement steel for a wall panel.

FIGURE C-20: Tieback drilling and exposed tieback sleeve, tendon, and grout tube.

FIGURE C-21: Exposed Tieback Sleeves

FIGURE C-22: Exposed tieback sleeve with tieback grouting in progress.

FIGURE C-23: Southeast corner bracing showing piles used to support the long braces.

FIGURE C-24: Slurry wall opening in the SE corner for exhaust structure, and wale used to bridge the opening.

FIGURE C-25: Assembly of corner bracing in the Northeast corner.

FIGURE C-26: Construction of a slab section with kicker blocks for the rakers.

FIGURE C-27: Attachment of the rakers to the kicker blocks.

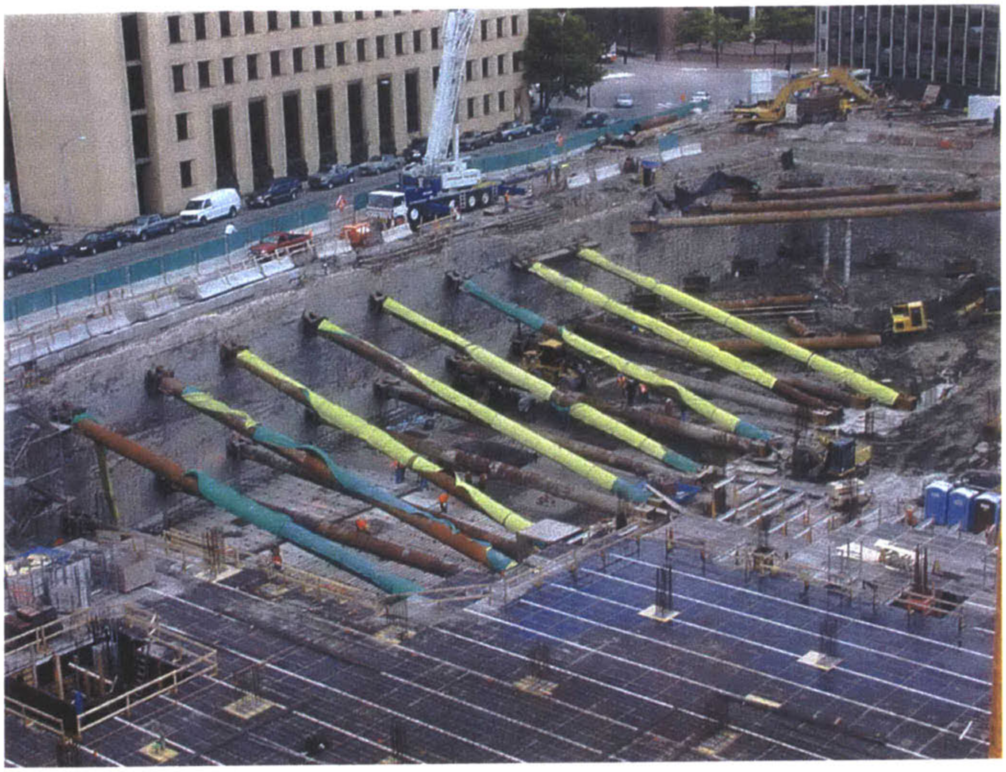


FIGURE C-28: Raker supports on the North wall with coverings to help protect struts from the heat of the sun.

FIGURE C-29: Installation of grade beams on the West side of the site.

FIGURE C-30: Placement of strip drains below the mudmat, and construction of the mudmat.

FIGURE C-31: Construction of the slab for the second floor starting in the Southwest corner.