ABSTRACT

Title of Dissertation: EARTH PRESSURE DETERMINATION IN TRENCH RESCUE SHORING SYSTEMS

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Despite Occupational Health and Safety Administration (OSHA) excavation safety requirements, victims continue to be trapped in collapsed excavations every year. The fire service has been tasked with developing technical rescue practices and procedures for efficient rescues and/or body recoveries in trenches or shallow excavation failures. Rescuers and victims depend on the performance of the standardized trench rescue shoring system developed by the technical rescue industry for their safety and welfare. The system has undergone little technical analysis. This dissertation presents a building block towards developing a technical understanding and analysis of the behavior of a shoring system that is used in trench rescues or as a bracing in shallow excavations.

The accepted engineering practice widely used for determining earth pressure in a braced excavation is based on soil type and deep excavations and does not account for strut loading, whereas, it has been shown that shallow braced excavations respond differently. This research evaluated the applicability of present engineering practice on the current standardized trench rescue shoring system. A new method was developed for calculating the earth pressure developed using the rescue shoring system. The method determines the earth pressure as a function of strut loading. The method can also be used

to determine the maximum strut loading that can be used without causing the soil to fail. The method can be used for any type of soil as field work validated the concept that soil type has little effect on the shape of the earth pressure distribution in shallow braced excavations.

EARTH PRESSURE DETERMINATION IN TRENCH RESCUE SHORING SYSTEMS

by

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DEDICATION

To my parents

Acknowledgments

As they say, it takes a village. I would be nowhere without the unflagging support of the village around me. First and foremost I must thank my advisor, Prof. M. Sherif Aggour. Without his dedication and support this simply would never have happened. I am also grateful to all the members of the dissertation committee for your time and consideration.

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Fire Chief John Senft supported my endeavors to the point he got his crisp white uniform shirt dirty lifting a panel out of the hole. Thank you!

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Chapter 1 Introduction

A trench is defined by the Occupational Safety and Health Administration (OSHA) in regulation 1926 Subpart P, section 1926.650(b) as “a narrow excavation (in relation to its length) made below the surface of the ground. In general, the depth is greater than the width, but the width of a trench (measured at the bottom) is no greater than 15 feet. If forms or other structures are installed or constructed in an excavation so as to reduce the dimension measured from the forms or structure to the side of the excavation to 15 feet or less (measured at the bottom of the excavation), the excavation is also considered to be a trench” (OSHA 1989). OSHA states that employees working in excavations, including trenches, shall be protected from cave-ins by either benching and sloping or a protective shielding system when the excavation is five or more feet deep or when soil examination by a “competent person” determines there is no indication of potential cave-in (OSHA 1989).

Shoring a trench can be time consuming and costly for contractors not set up to regularly perform those operations. Efficiency sometimes overshadows safety. Other new or small companies and do-it-yourselfers are simply ignorant of the risks. Whatever the reason, not all individuals are conscientious about safety around open excavations. The Bureau of Labor and Statistics reported that thirty eight construction workers died in excavations and trench cave-ins in 2000 (Bureau 2001) and there were twenty three fatalities related to trenches and excavations in 2007 (Bureau 2008). There are estimated to be hundreds of entrapments in failed trenches every year, many of which are never reported.

The average cubic foot of soil weighs 90-110 lbs. One foot to 18 inches of soil on an adult human chest, approximately 2 ft wide, applies 200-300 pounds of force, severely restricting expansion of the victim's chest. If a victim is not completely entombed and immediately at risk of suffocation, the greatest hazards are internal injuries from the initial blow and crush syndrome during and after extrication.

OSHA has enforceable safety requirements to prevent such accidents and yet every year workers are killed in shallow excavations. On October 29, 2008 a worker in Arlington, VA was killed when a shallow, unshored trench collapsed on him. The Fairfax County technical rescue team worked for hours to safely extricate his body. He was located only 4 feet below grade. The technical rescue industry, dominated by fire and rescue services, has been tasked with developing means to attempt rescues and inevitably recover bodies. This industry has used OSHA recommendations to develop a standardized shoring system to rescue or recover trapped individuals. There has been little technical analysis of the system. The objective of this dissertation is to perform a technical analysis of a typical trench rescue shoring system to verify its current design and provide recommendations for a design methodology that is applicable for any shallow braced excavation.

1.1 Dissertation Organization

The chapters of this dissertation are organized in a manner that presents information in the following order: Chapter 1 presents an introduction to the components and process of the trench rescue shoring system as well as a literature review of the research performed on shallow and deep braced excavations. Chapter 2 gives a verification of the finite element model developed to perform the parametric studies that are used to evaluate the variables that affect the trench rescue shoring system. Chapter 3 presents the results of the parametric studies that examine how the earth pressure changes with strut load, panel stiffness, thickness, width, and surcharge configuration and size. Chapter 4 presents the results of full scale testing of a trench rescue shoring system performed in two shallow trenches excavated in different soil types, a clayey type and a granular type, in order to gauge the effect of soil type on the shoring system performance. Chapter 5 presents a discussion and analysis of the results of the parametric studies and the field work as well as presents a recommended procedure to determine the earth pressure in a shallow braced excavation. Chapter 6 presents the conclusions and recommendations of this dissertation.

1.2 Description of Current Standardized Trench Rescue Shoring System

Trench rescue shoring was developed from OSHA shoring requirements found in Standard - 29 CFR Part 1926 Safety and Health Regulations for Construction Subpart P 1926.652 which was in turn developed from the National Bureau of Standards (NBS) reports on industry practice. The basic system consists of a 4 ft x 8 ft panel made from 14 plys of arctic white birch for a total thickness of 0.75 in. Some rescue teams use 1.00 in thick panels. Typically the panels are FinnForm or ShorForm brand and are bolted to a 2 in x 12 in x 12 ft long board called a strongback. The strongback is typically fir or pine. The bolted panel and strongback assembly are held against the trench wall by pressurized pneumatic or hydraulic struts placed no farther than 4 ft down from the top edge of the excavation, spaced no father than 4 ft apart, and no farther up than 4 ft from the bottom of the excavation. The struts' internal pistons are pressurized by air or hydraulic fluid at pressures ranging from 50 psi to 1500 psi depending on the manufacturer recommendations and specific circumstance of an incident. These pressures produce loads from 325 lbs to 4712 lbs distributed over 5 in x 5 in strut feet. The panel, strongback, and strut configuration is illustrated in Figures 1-1, 1-2, and 1-3. Figure 1-1 shows an elevation of the shoring system as placed against a trench wall. Figure 1-2 shows a cross section of the shoring system as placed in the trench. Figure 1-3 shows a plan view of the shoring system as placed in a trench. Figure 1-4 shows an actual incident with the system in use.

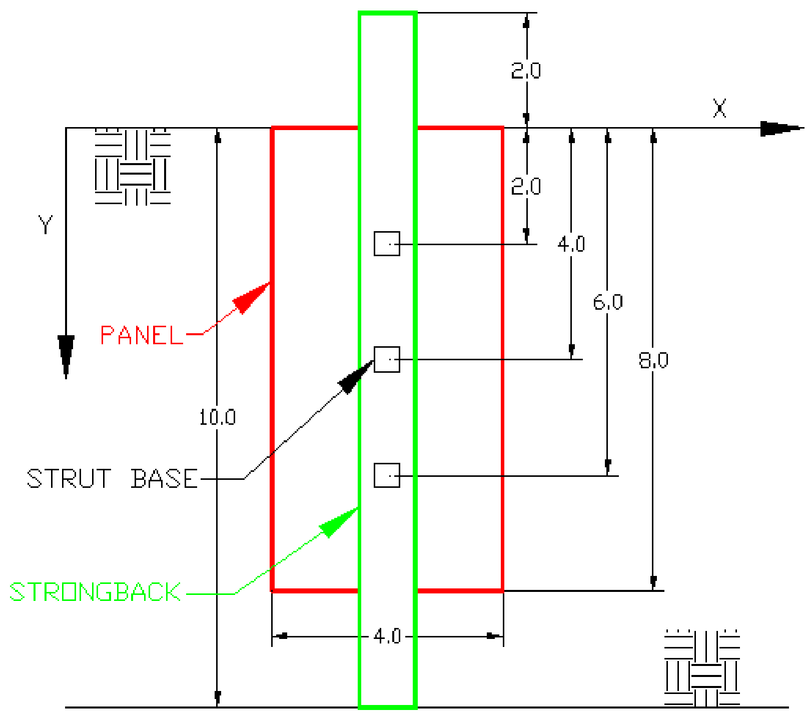


Figure 1-1 Trench elevation: Panel (red), strongback (green) and strut location

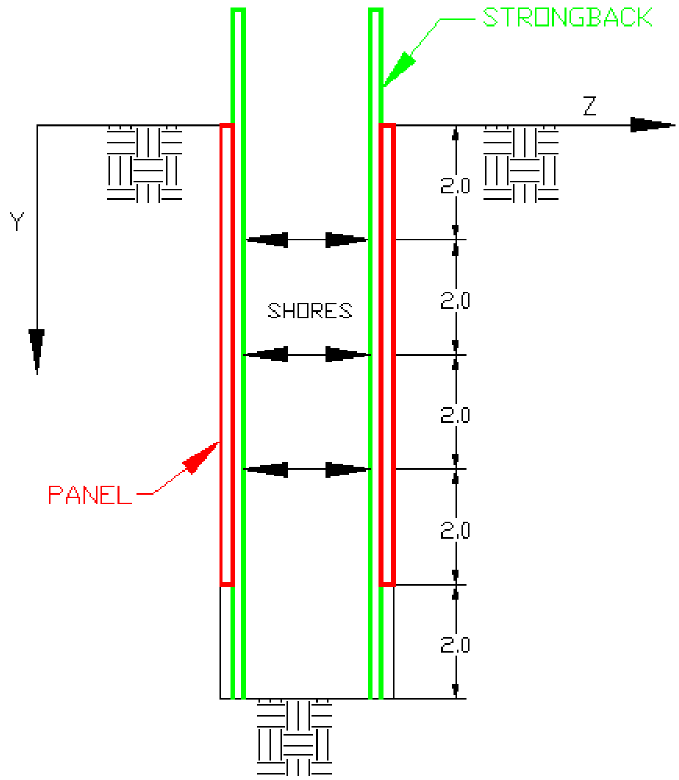


Figure 1-2 Trench section: Panel (red), strongback (green) and strut location

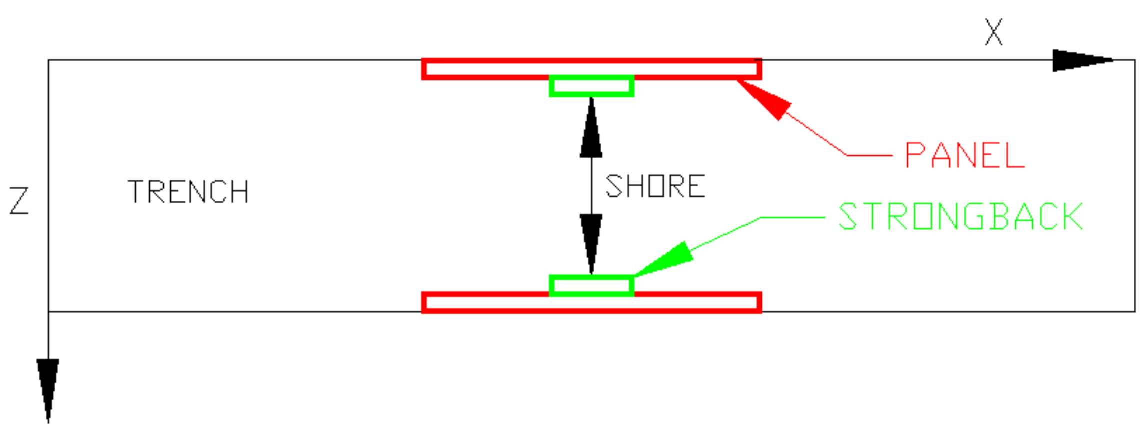


Figure 1-3 Trench plan view: Panel (red), strongback (green) and strut configuration


Figure 1-4 Typical trench rescue shoring operation

In the event of a trench collapse involving victims, emergency services are dispatched, effectively introducing additional personnel to an already compromised environment. Where available, jurisdictions dispatch a technical trench rescue team. Teams are equipped with their own trench shoring systems consisting of panels, strongbacks, and struts. According to industry practice, these systems can be used without consultation with a professional engineer in trenches up to 15 feet deep (Gargan 1996, Martinette 2006, 2008).

A 1980 NBS report by Felix Y. Yokel examined spaced sheeting, not dissimilar to rescue shoring systems, and concluded that the shoring system must still resist the same resultant force that would be resisted by a shoring system with tight sheeting.

> There are presently no data by which soil properties such as cohesive strength can be correlated with the ability of the soils to stand in the interval between the spaced supports without collapsing or spalling. The recommended provisions are based entirely on empirical practice and on field observations reported by

> experienced contractors and foremen. In essence NBS could find no evidence that the present OSHA requirements with respect to spaced sheeting are unsatisfactory. (Yokel 1980)

OSHA 1926 Subpart P Appendix D(g)(7) requires that plywood sheeting used in conjunction with aluminum hydraulic shores be either 1.125 in softwood or 0.75 in 14-ply arctic white birch. The technical rescue industry has adopted this requirement and uses both aluminum air pressured pneumatic shores and aluminum hydraulic shores. Airshore brand struts by Hurst and Paratech brand struts are popular pneumatic shores widely used by the industry. Speed Shore brand struts are popular hydraulic shores. In a rescue shoring system, the shores are pressurized against 2 in x 12 in x 12 ft boards bolted to 4 ft x 8 ft sheets of 0.75 inch thick 14 ply arctic white birth plywood. FinnForm and ShorForm are the most widely used brands of 14 ply arctic white birch in the technical rescue industry.

1.3 Current Design Methods and Finite Element Models

In 1969 Peck published "Deep excavations and tunneling in soft ground" which included apparent earth pressure diagrams for braced excavations in sand/gravel, soft-to-medium clay, and stiff clay that were originally published in Terzaghi and Peck's 1967 *Soil Mechanics in Engineering Practice*. The diagrams were obtained as the envelopes of maximum earth pressures found at several projects in Chicago, IL and Berlin, Germany during subway system construction. They are shown in Figures 1-5 (a), (b) and (c). Peck stated the "envelopes, or apparent pressure diagrams, were not intended to represent the real distribution of earth pressure at any vertical section in a cut, but instead constituted hypothetical pressures from which there could be calculated strut loads that might be approached but would not be exceeded in the actual cut" (Peck 1969).

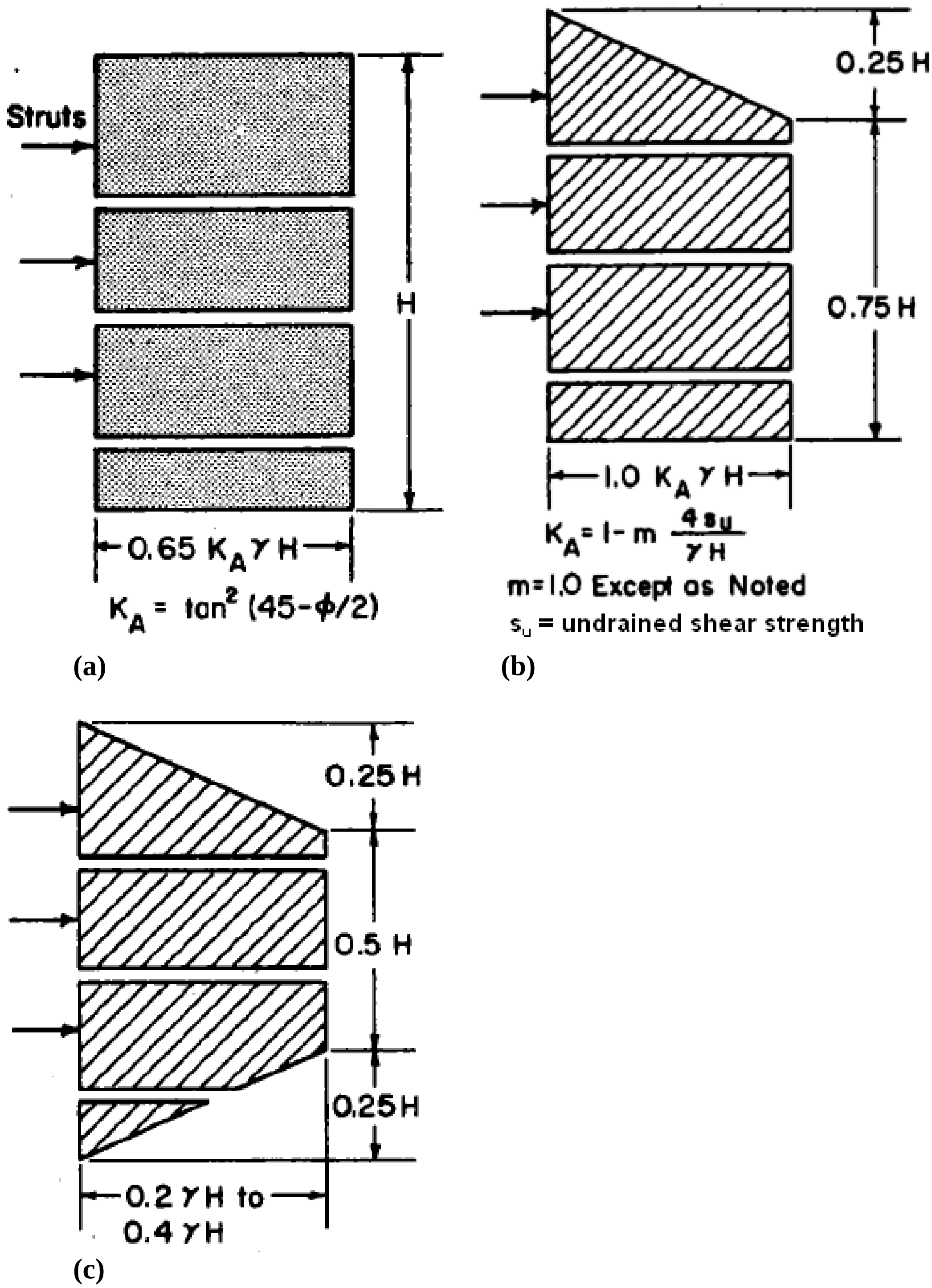


Figure 1-5 Terzaghi and Peck (1967) apparent earth pressure diagrams for (a) sand/gravel (b) soft to medium clay (c) stiff clay

Bowles (1996) summarized Terzaghi and Peck's design method using apparent earth pressure and produced a diagram of staged earth pressure developed during excavation and strut installation as shown in Figure 1-6.

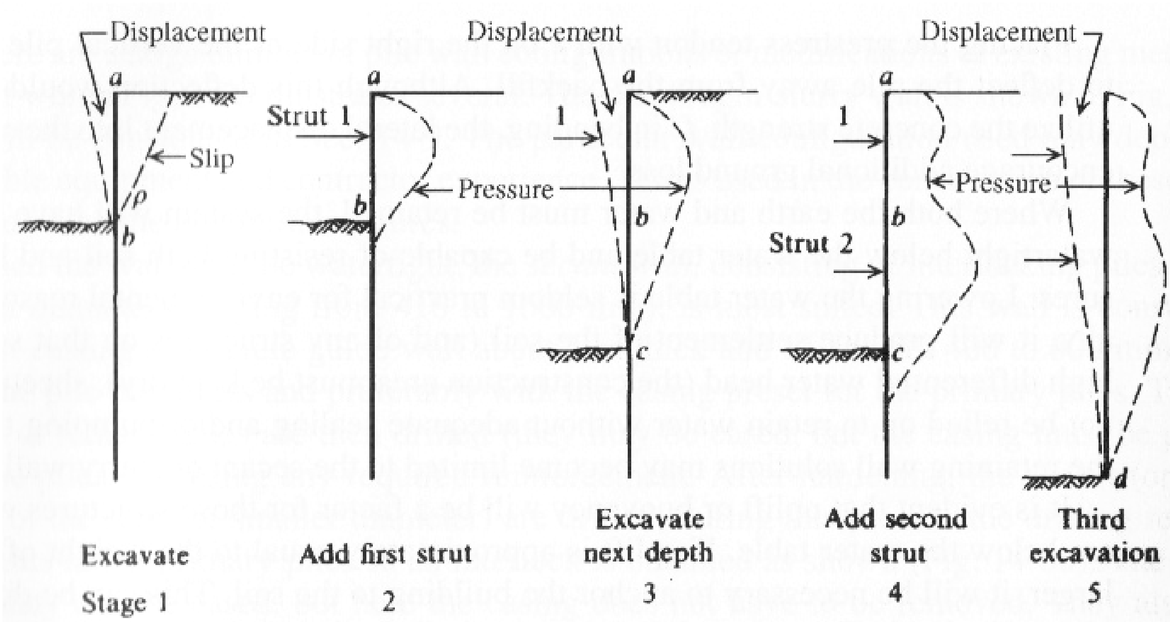


Figure 1-6 Staged development of earth pressure in a braced excavation (Bowles 1996)

Bowles found that the strut load produces larger pressure than the active earth pressure and consequently causes increased wall pressure. It is therefore "evident that if one measures pressures in back of this wall they will be directly related to the strut forces and have little relation to the actual soil pressures involved in moving the wall into the excavation" (Bowles 1996).

> If one designs a strut force based on the apparent pressure diagram *and* uses simply supported beams for sheeting as proposed by Terzaghi and Peck, the strut force will produce not more than the contributory area of that part of the apparent pressure diagram. The sheeting may be somewhat overdesigned because it is continuous and because simple beam analysis always gives larger bending moments; however, this overdesign was part of the intent of using these apparent pressure diagrams. (Bowles 1996)

Yokel (1979) proposed earth pressure envelopes specific to shallow excavations in soil types A, B and C. As defined by OSHA CFR 29 1926 Subpart P Appendix A (2)(b) (1989), soil type A is cohesive soil such as clay with an unconfined compressive strength of 1.5 ton per square foot (tsf) (144 kPa) or greater. Type B soil is cohesive soil with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa). It is generally characterized as soil that is a medium between type A and type C. Type C soils are non-cohesive soils and cohesive soil with an unconfined compressive strength of 0.5 tsf (48 kPa) or less. Examples of type C soil include granular soils, and submerged unstable rock. Yokel assigned lateral weight effects (w_e) to each soil type. He proposed that in type A soil w_e is 20 lb/ft^3, in type B soil w_e is 40 lb/ft^3 and in type C soil w_e is 80 lb/ft^3. Yokel theorized that the earth pressure envelope for shallow excavations was equivalent to $P = w_e(H+2)$ where H is the height of the supported excavation. Yokel added 2 ft to H to allow for surcharge. Figure 1-7 shows Yokel's earth pressure envelopes for soil types A, B, and C.

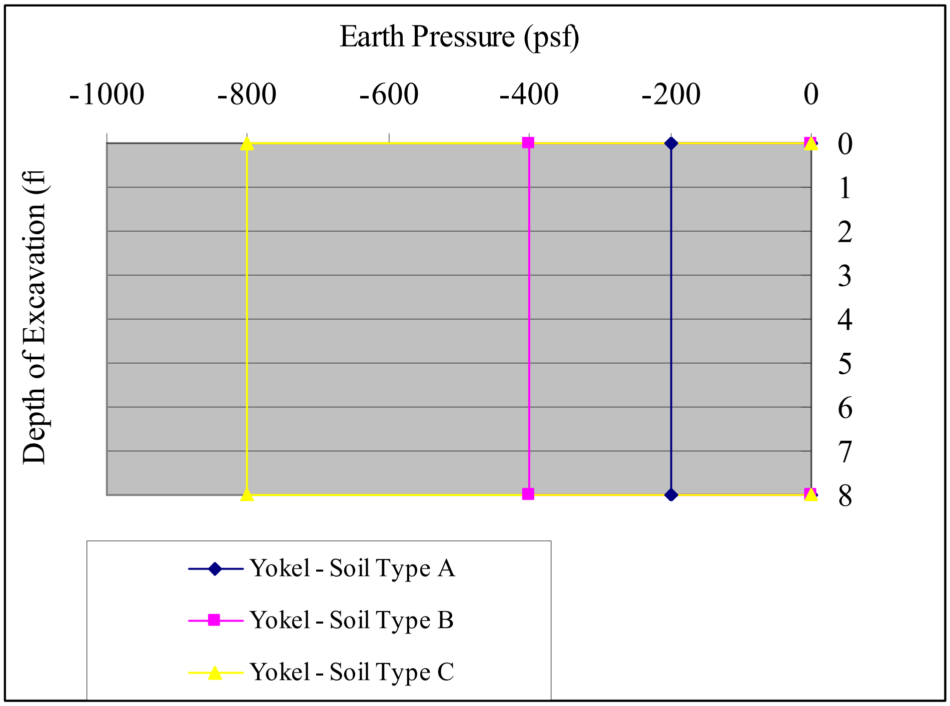


Figure 1-7 Yokel earth pressure envelopes for soil types A, B and C

Ou et al (1996) used a three-dimensional finite element technique to study the effect of corners on the deflection behavior of a deep excavation in soft to medium clayey subsoil stratum. The writers showed that the behavior of the braced wall during excavation is related to excavation sequence, method of excavation, method of wall support, excavation depth, excavation wall penetration depth, excavation geometry, wall stiffness, and soil strength. The writers also showed that three-dimensional finite element analysis can more accurately model deformation behavior of a braced wall during excavation and two-dimensional analysis overestimates wall deformation near the corner of an excavation.

Bose and Som (1998) analyzed an instrumented section of braced cut in soft clay 13.6 m (44.6 ft) deep. In their study they showed that wall length and width of excavation influence the soil-wall deformation as well as the lateral force. They also showed that strut load has a marked effect on the performance of the braced cut. By increasing the strut load, wall deflection and ground movement is reduced. The value of maximum ground settlement reduced linearly with increasing strut load.

Wang (2001) stated that available empirical methods for structural design of a braced excavation were basically developed from the observations of the conventional shoring systems, typically a sheet piling wall. The case studies from Wang's research imply that Peck's pressure envelopes possibly underestimate the strut loads for a heavily supported excavation, consisting of either a rigid diaphragm wall or a sheet pile wall of heavy sections toed into firm strata. He found that numerical methods provide a versatile tool for the complex ground excavation problems due to their ability to model construction stages and consider the soil-structure interactions. However, the results of a numerical analysis can vary significantly with the assumed soil and structure properties. "The strut loads are so variable, depending on the detailed construction sequence, shoring stiffness, ground and water conditions, temperature changes, and perhaps more importantly the workmanship of the contractor. Therefore is no method yet which can accurately predicate the structural forces in reality." (Wang 2001)

Karlsrud and Anderson (2005) performed a parametric finite element analysis for a strutted sheet pile wall in soft clay. The modeled excavation is 10 m (32.8 ft) deep and 16 m (52.5 ft) wide. The sheet pile wall is braced internally with four struts at depth 1 m (3.3 ft), 3.5 m (11.5 ft), 6 m (19.7 ft), and 8 m (26.2 ft). The excavation was performed sequentially in five steps to a depth of 0.5 m (1.6 ft) below the struts with successive installation of the struts. The variables were the shear strength of the clay and strut loading. The parametric study determined the earth pressure and bending moments in the sheet pile wall.

Karlsrud and Anderson found that using a simple isotropic linear elastic-plastic soil model gave fairly similar results to using an anisotropic non-linear soil model. However, using a beam-on-spring type finite element model as used by practicing engineers produced significant differences in comparison to the continuous finite element analysis. The writers argue that beam-on-spring elements cannot capture the significant effect of arching on earth pressure and strut loads. In comparing the earth pressure to the classical Rankine earth pressure, the earth pressure determined was almost twice the Rankine pressure along the top 4 m (13.1 ft) to 6 m (19.7 ft) and below that depth the pressure is lower then the classical Rankine pressure. The writers also found that the maximum strut loads are significantly higher than those given by existing empirical design rules, i.e. the apparent earth pressure by Peck in 1969. By back calculating the apparent earth pressure deduced from the maximum strut loads calculated, the apparent earth pressure is significantly larger than the apparent earth pressure based on Peck (1969).

Finno and Blackburn (2006) used a three-dimensional finite element analysis of a 10 m (32.8 ft) deep excavation sequence in an internally supported excavation through medium stiff clay. A comparison between the measured strut loads and the results of the three-dimensional finite element solution showed that for uniform excavation sequences, the loading on the lower level struts was under-predicted and the force on the upper struts was over-predicted.

Kung et al (2007) used finite element method on selected hypothetical excavation cases and using stress-strain behavior of soils at small strain levels in a study of braced excavations. The writers developed a simplified semi-empirical model for estimating maximum wall deflection, maximum surface settlement and surface-settlement profile due to excavation in soft to medium clays.

Blackburn and Finno (2007) collected data in an internally braced excavation in soft clay for the Ford Engineering Design Center in Evanston, Illinois. They found that "while the deformations that occurred at the site were within expected values, the forces in the internal braces were slightly larger than those expected based on Terzaghi and Peck (1967) apparent earth pressure diagram for soft clays."

In summary, it was found from the literature review that previous research has identified some limitations in using Terzaghi and Peck's 1967 apparent earth pressure diagrams to determine earth pressure behind externally loaded braced shoring. It also found that the maximum strut loads are significantly higher than those given by Terzaghi and Peck's apparent earth pressure diagrams. Previous work also identified that linear elastic finite element modeling is a valid method of analyzing deep excavations. However, such analysis has not been used for shallow excavations to date. Other than Yokel's proposed earth pressure envelopes in 1979, no research examined the earth pressure and strut loading in braced shallow excavations. Previous research does not present any technical analysis of a typical OSHA recommended trench rescue shoring system.

1.4 Differences between Trench Rescue Shoring and Braced Deep Excavations

There are significant differences in the typical braced excavation analysis using available empirical methods and the standard trench rescue shoring system that raise questions about the validity of using the same analysis methods to examine the standard trench rescue shoring system. Yokel states that pressure envelopes presented by Peck in

1969 “were developed on the basis of measured data which originated from deep excavations (deeper than 20 ft). Because of the time element usually associated with such excavations, the data are from excavations which were open for weeks or even several months. There were fundamental differences between such excavations and typical shallow utility trenches.” These difference include depth, time excavation is open, excavation and shoring methods (excavation and shoring in lifts), and trench discontinuities (short sections of shallow trench not continuously shored).

Bowles (1996) similarly states that conventional designs are specific to continuous walls of shoring. Trench rescue shoring is not a continuous system. It is placed in 4ft sections, often no wider than 12ft for a complete operation. It is also not installed during phased excavation. It is installed after excavation is complete and some portion has failed.

Trench rescue shoring is intended for trenches 15 ft (4.6 m) or less in depth. The literature review found that the shallowest excavation examined (aside from Yokel’s work) was 7.5 m (24.6 ft) which is 9.6 ft below the maximum depth intended for trench rescue shoring (15 ft).

Chapter 2 Model Construction and Verification

A three-dimensional linear elastic finite element model was created to examine the effects of different trench rescue shoring components on earth pressure. This chapter summarizes the construction of the model using Abaqus/Standard 6.7 and determines the model size used in the parametric studies. This chapter also examines the validity of using the finite element model to determine the earth pressure on a rigid retaining wall by comparing the model results with theoretical vertical and horizontal earth pressure. The accuracy of the model to determine the load transfer of the shoring system through multiple layers of materials with varying properties was also examined.

2.1 Model Construction Using Abaqus/Standard 6.7

The model was constructed with Abaqus/Standard 6.7. Half the symmetric problem was modeled with a three dimensional linear elastic model using 8-node linear 3D stress elements with reduced integration and hourglass controls (Abaqus 2007a). The model consisted of three parts: soil, panel, and strongback. All three parts were assumed to be solid and homogeneous with elastic, isotropic material properties. The soil was also assigned a density to account for gravity effects. Interaction between the strongback and panel was assumed to be rough with no tangential or normal movement. The interaction between the panel and soil did not allow any normal separation, but assumed frictionless tangential behavior.

The soil dimensions were 36 ft x 25 ft x 36 ft with a 1.5 ft x 13 ft cut centered at x = 18 ft, y = 0 ft and z = 0 ft as shown in Figure 2-1. The panel dimensions were 4 ft x 8 ft x 0.75 in and the strongback dimensions were 2 in x 12 in x 12 ft as shown in Figure 2-2. The struts were modeled by loading three 5 in x 5 in faces centered on the strongback at depths y = 2 ft, y = 4 ft, and y = 6 ft. Internal piston pressure was set at 200 psi, 750 psi, and 1500 psi, producing loads on the strut base of 1298 lbs (7479 psf), 2356 lbs (13572 psf), and 4712 lbs (27143 psf) respectively. Meshing in Abaqus assigned 820 elements to the panel, 300 elements to the strongback and 130000 to 140000 elements to the soil.

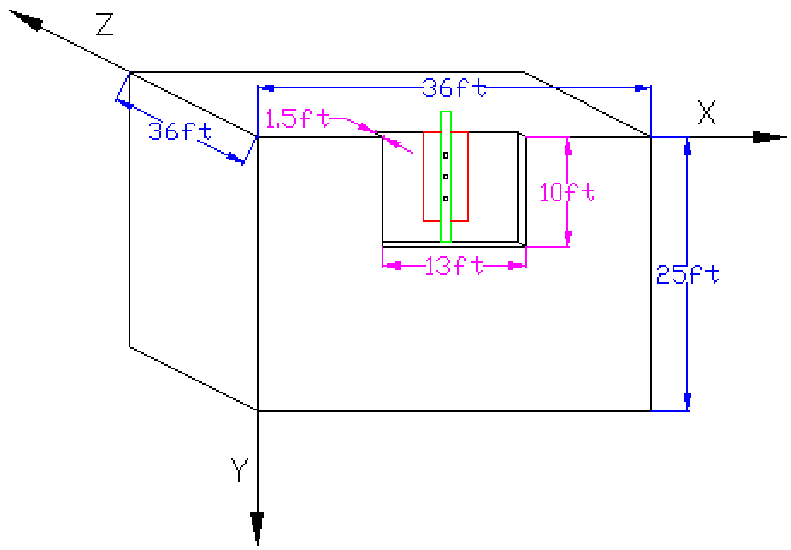


Figure 2-1 Model dimensions

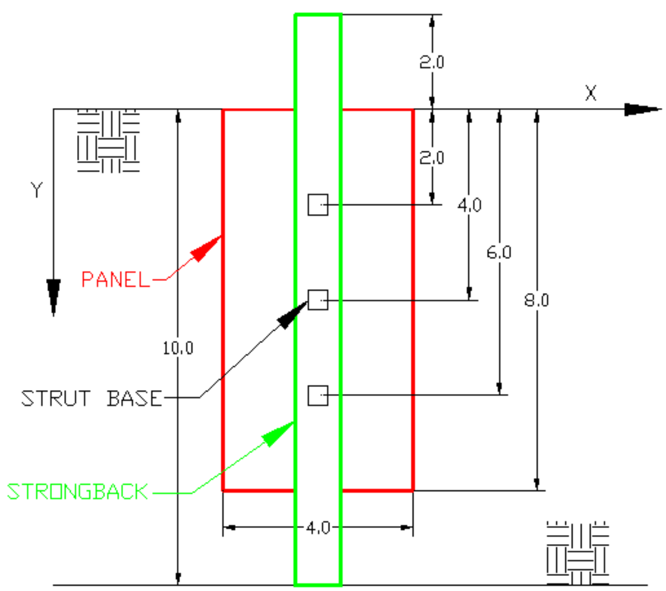


Figure 2-2 Panel (red), strongback (green) and strut configuration

Boundary conditions were only assigned to the soil. As shown in Figure 2-3 each face of the model is assigned a label that corresponds to a boundary condition. The walls of the cut in the FZ1 face are all free and do not have boundary conditions associated with them. The boundary condition for the FZ1 face itself is symmetric, meaning forces, displacements, and rotations were equal on both sides of the face. Faces FX1, FX2, FY2 and FZ2 are restrained from all rotation and any movement perpendicular to their plane. The FY1 face and all the exposed faces of the cut in the FZ1 plane were left free of restraint.

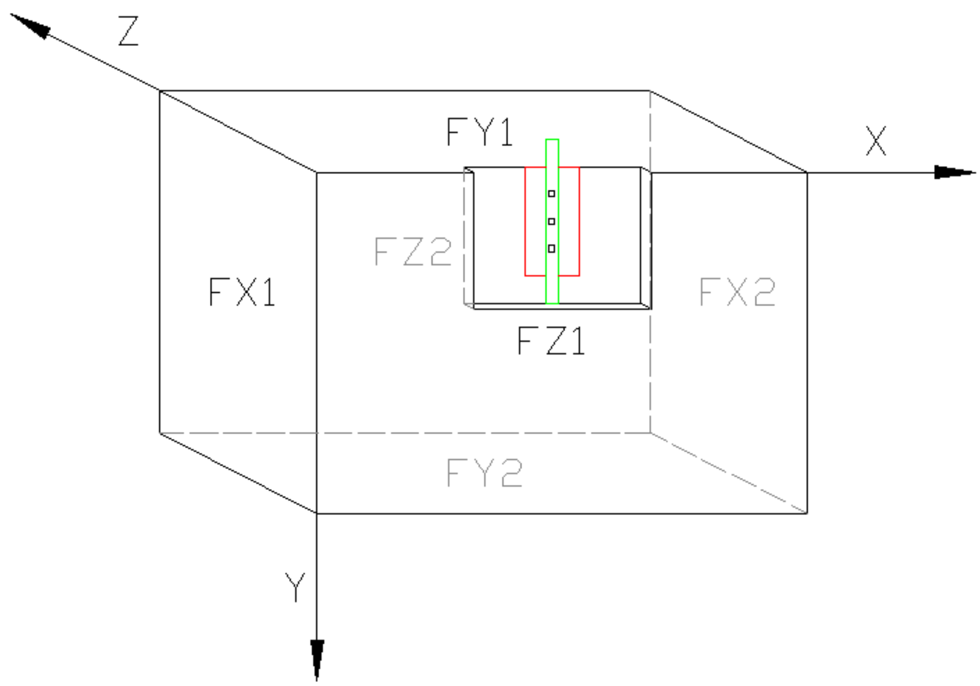


Figure 2-3 The model with its faces labeled

Section 2.2.1 of the *Abaqus Theory Manual* (Abaqus 2007e) describes the solution methods used by an Abaqus finite element model. Section 22.1.1 of the *Abaqus Analysis User's Manual* (Abaqus 2007a) describes the elements. Appendix A provides a detailed description of building a finite element model for analysis using Abaqus.

2.2 Model Verification

Multiple verifications were used to ensure the results obtained using the model agree with available theoretical methods. With the linear elastic modeling capability of Abaqus verified, the research focused on obtaining earth pressures behind trench rescue shoring with varying parameters such as strut load, surcharge size and location, and panel dimensions and properties.

2.2.1 Verification of Model Size

As would be expected, the locations of model boundaries affect earth pressure. To eliminate the effects of boundary conditions on earth pressure determination, the minimum size of the model was determined by varying the size of the model and examining the effects on earth pressure. Figure 2-4 shows the model orientation in space.

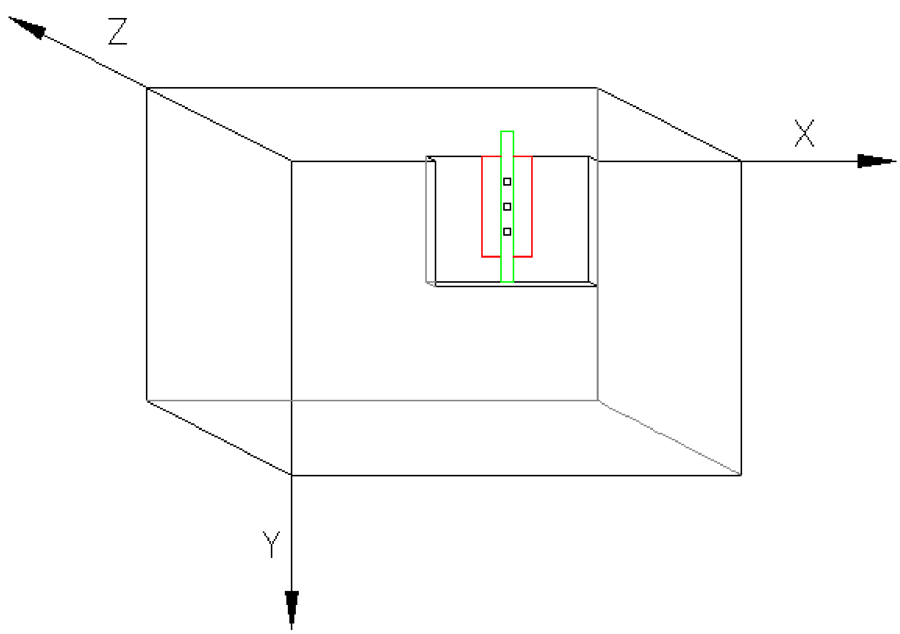


Figure 2-4 Model orientation

Models were run using Abaqus to evaluate effects of changing dimensions in the X (width), Y (height), and Z (depth) directions.

Two models were run with the X dimension (width) set at 36 ft and then 56 ft. Horizontal stress at a vertical depth of 4 ft changed 2.89% between the 36 ft model and the 56 ft model. The minimal change implied that 36 ft was an appropriate model width.

Three models were run with the Z dimension (depth) set at 14 ft, 24 ft, and 36 ft respectively. Horizontal stress at a vertical depth of 4 ft changed 8.9% between the 14 ft model and the 24 ft model and 1.1% between the 24 ft model and the 36 ft model. The reduction in percentage from 8.9% to 1.1% implied 36 ft was an appropriate model length.

In the Y direction, the height was changed from 15 ft, 20 ft, and then 25 ft. Vertical stress at a depth of 4 ft changed 0.64% between the 15 ft model and the 20 ft model and 0.16% between the 20 ft model and the 25 ft model. The reduction in percentage from 0.64% to 0.16% implied 25 ft was an appropriate model depth. Appendix B shows the tabulated data.

2.2.2 Verification of Determining Earth Pressure

The first verification model examined the determination of earth pressure against a rigid retaining wall. The linear elastic equations used to calculate earth pressure are vertical stress = γh and horizontal earth pressure = $k_o \gamma h$ where $k_0 = \nu/(1-\nu)$ and ν is Poisson's ratio. Using a unit weight of $\gamma 1$ = 124 pcf and $\nu = 0.3$, Figure 2-5 shows a comparison between the vertical stress results from the Abaqus model versus calculated stress values. Figure 2-6 shows the same comparison for the horizontal earth pressure. Figure 2-7 and Figure 2-8 show the same comparisons for a unit weight $\gamma 2$ = 119 psf. The model and calculated results differ an average of 1.94%. Appendix C shows the tabulated data.

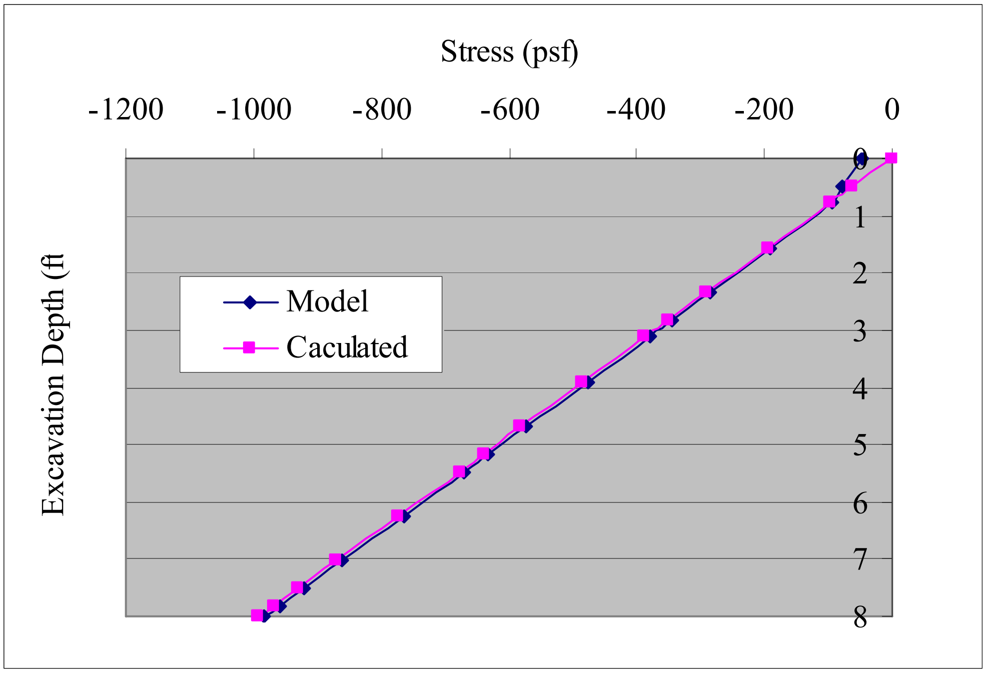


Figure 2-5 Vertical stress comparison for soil with a unit weight of γ1

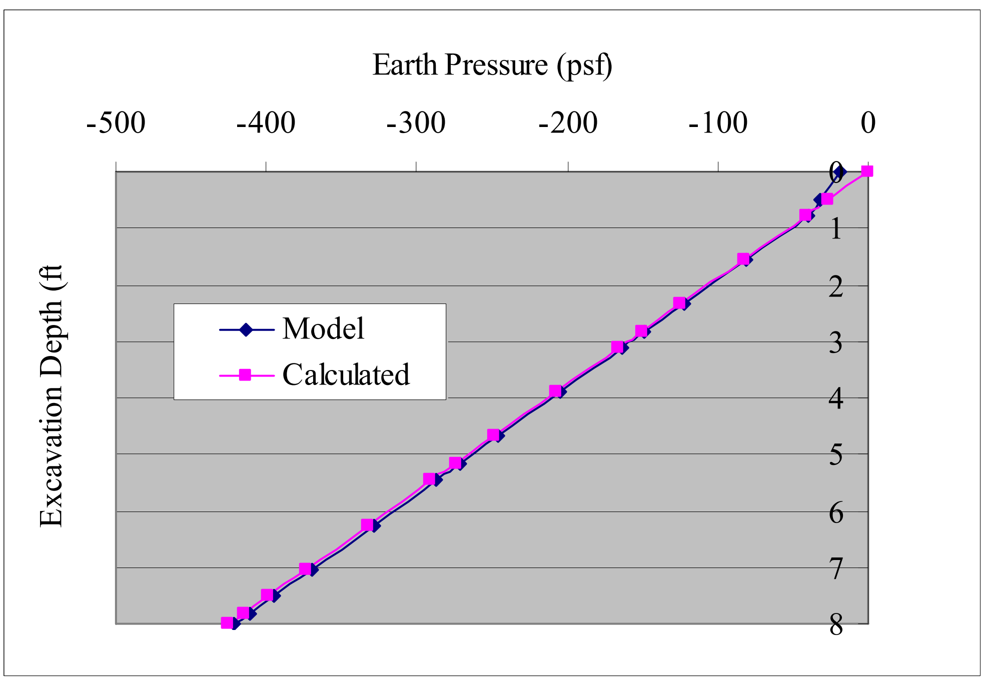


Figure 2-6 Horizontal earth pressure comparison for soil with a unit weight of γ1

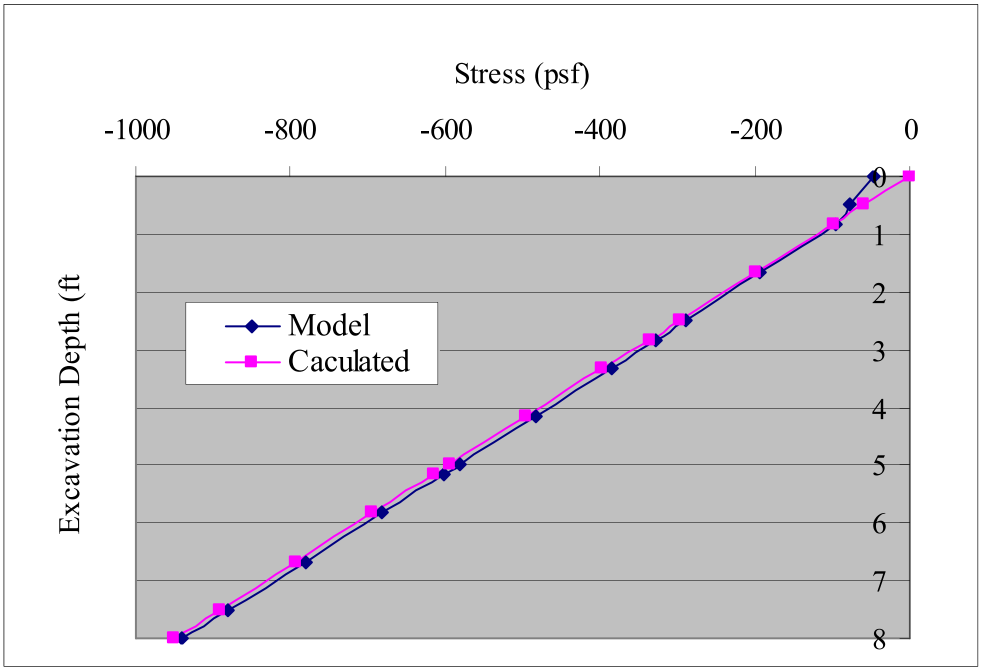


Figure 2-7 Vertical stress comparison for soil with a unit weight of γ2

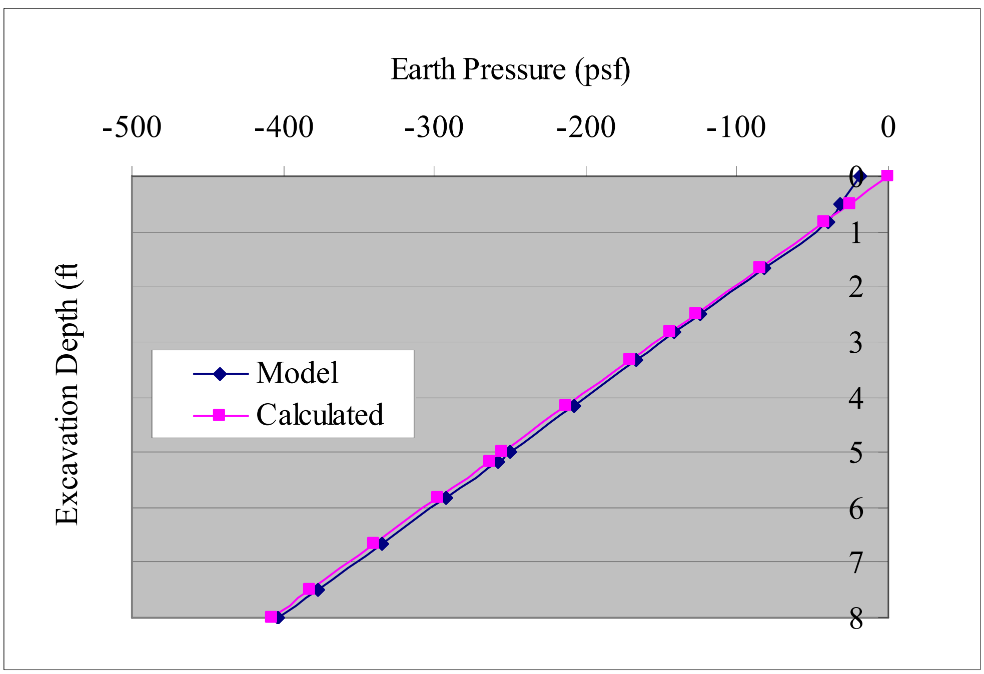


Figure 2-8 Horizontal earth pressure comparison for soil with a unit weight of γ2

2.2.3 Verification of Determining the Response to Strut Loads

The focus of this research is examining the soil response to a uniformly distributed strut load over a small area, 5 in x 5 in (0.17 ft^2), through multiple materials with varying properties. To verify the accuracy of an Abaqus model to perform this type of analysis, an example problem from Huang, 2004 (*Pavement Analysis and Design* section 2.2.2) was recreated. Using Burmister's 1945 layered theory, Huang determined the stress to be 2103.8 psf at a depth of 0.5 ft and 1025.3 psf at a depth of 1ft. Figure 2-9 shows a sketch of the problem solved by Huang for a load of 8686 lbs over 0.50 ft^2 (17280 psf).

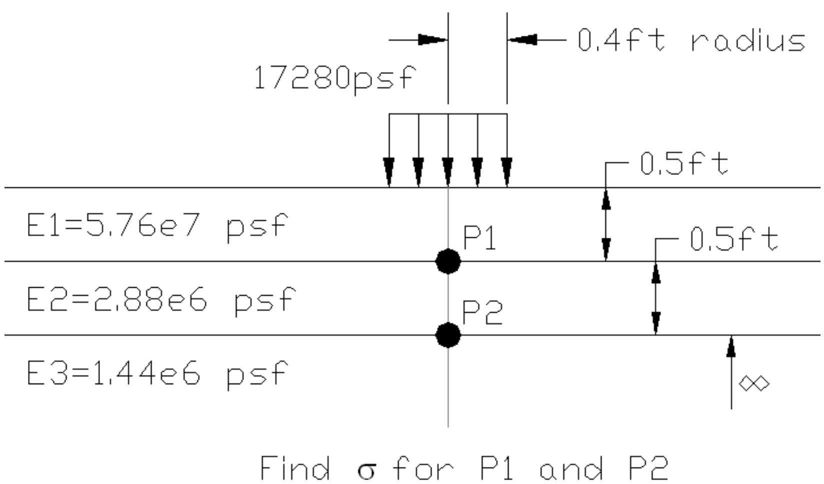


Figure 2-9 Sketch of example problem

An Abaqus model was constructed with 0.1ft x 0.1ft mesh size in the top layer of the material and 0.2 ft x 0.2 ft mesh size in the remaining layers as shown in Figure 2-10. The Abaqus model results determined the stress to be 2216.6 psf at a depth of 0.5 ft for a 5% difference from the analytical result and 1020.9 psf at 1 ft for a 0.4% difference from the analytical result as shown in Figure 2-11. Appendix D shows the tabulated data.

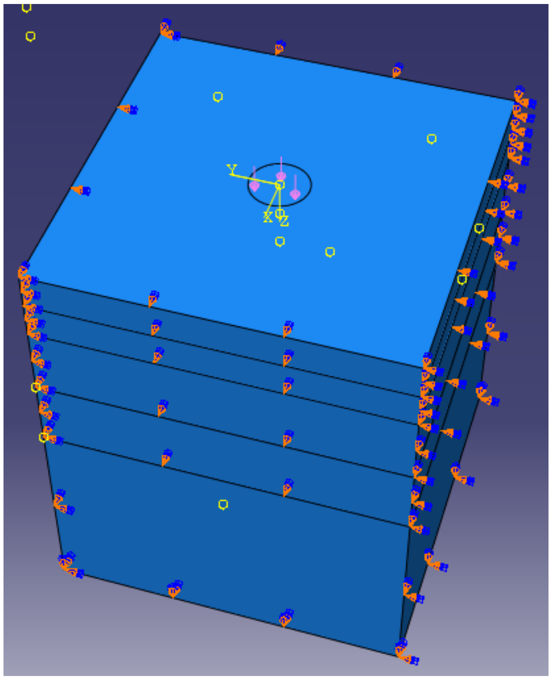


Figure 2-10 Abaqus rendering of example problem

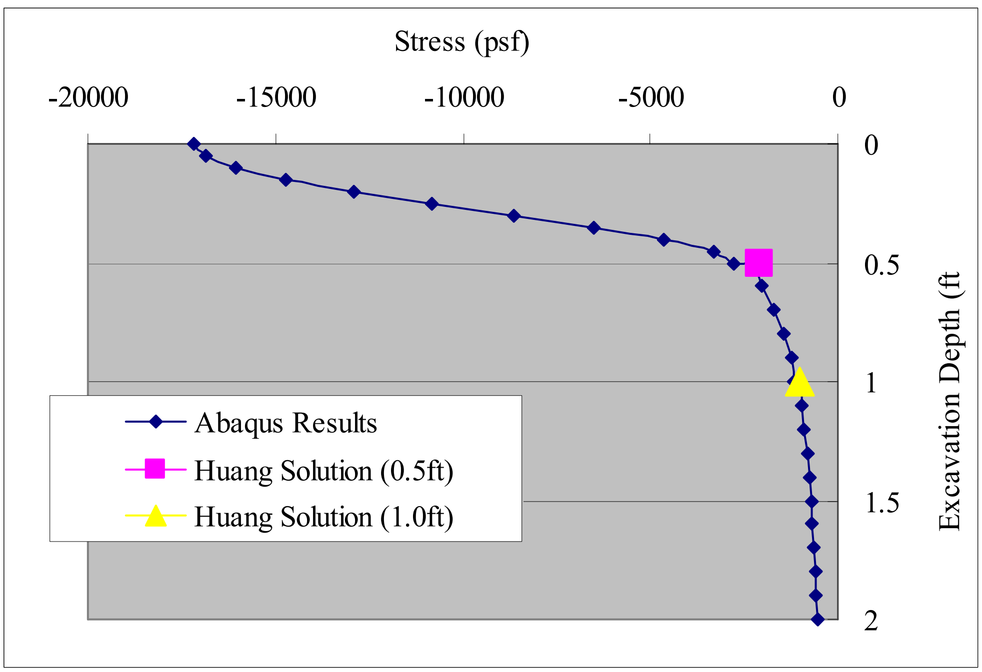


Figure 2-11 Comparison of calculated results using Huang's method vs. Abaqus model results

2.3 Summary

A linear elastic analysis was used in the modeling for this research due to the short term nature of trench rescue shoring operations and the anticipated small deformations. Models were run to verify the appropriate sized model to be used to minimize the effects of the boundary conditions on the results. The model size selected for use in the parametric studies was finalized at 36 ft (X dimension) x 25 ft (Y dimension) x 36 ft (Z dimension).

Verification models were run to validate the use of Abaqus/Standard 6.7 to determine solutions to problems involving vertical earth pressure, horizontal earth pressure, and load transfer by the shoring system of a uniformly distributed load over a small area through multiple layers of materials with varying properties. The vertical and horizontal earth pressure comparison of the model to the calculated results produced an average difference of 1.94%. The load transfer verification analysis for the shoring system produced less than a 5% difference between the model and the calculated results, i.e. the Abaqus solutions were within 5% overall of the analytical solutions.

Chapter 3 Parametric Studies

This chapter presents the parametric studies that were performed to determine the advantages, disadvantages, and limitations of the current standardized trench rescue shoring system. Linear elastic analysis is appropriate for these studies because of the short term loading and anticipated small deformations. Shoring during typical rescue operations is pressurized less than twelve hours. The studies examined strut load, panel stiffness, panel thickness, panel width, and the surcharge load, size and configuration.

3.1 Model Orientation

Figure 3-1 shows an elevation of the panel, strongback, and strut configuration used in a trench rescue shoring. Figure 3-2 shows the three-dimensional model orientation for the model used in the parametric studied.

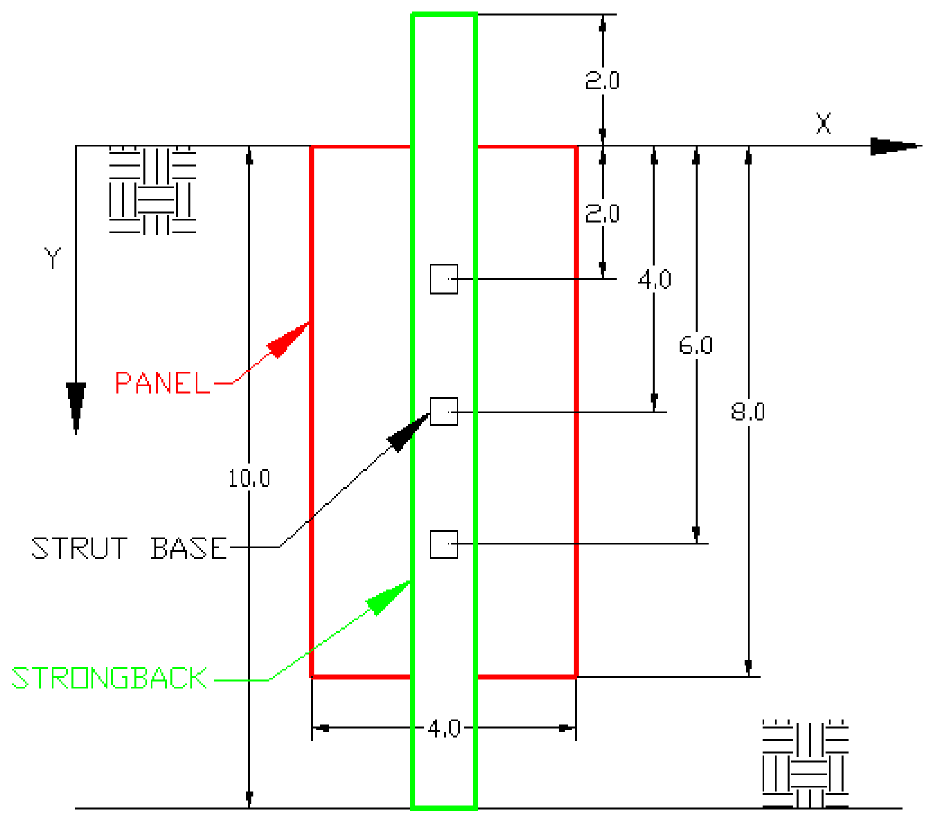


Figure 3-1 Elevation of shoring system

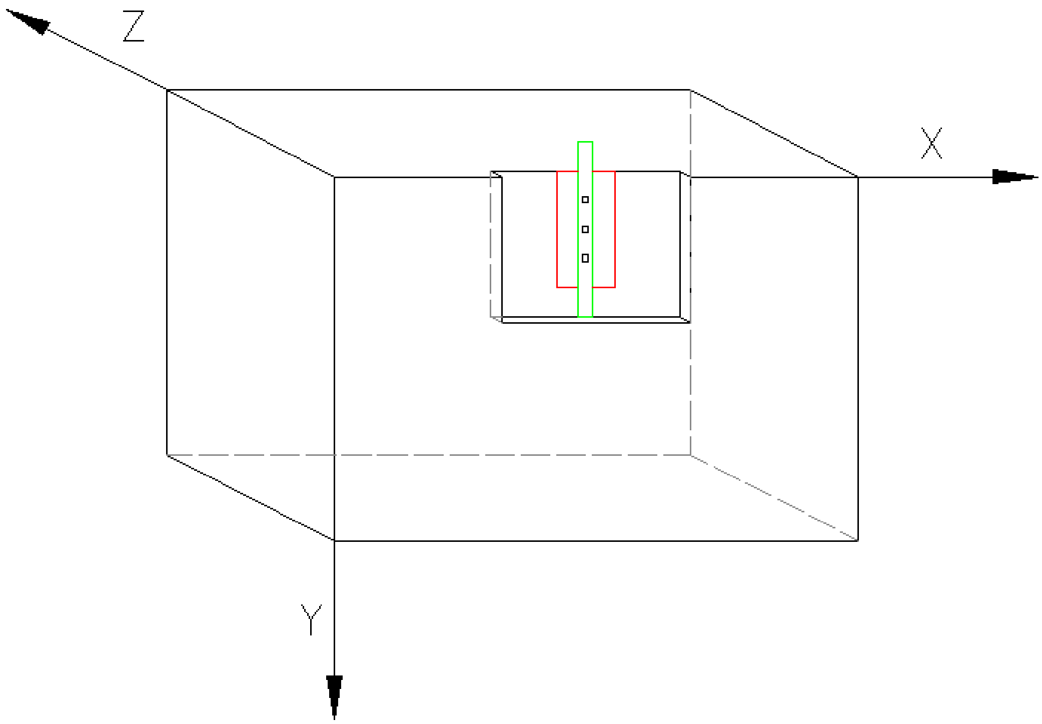


Figure 3-2 Model orientation

3.2 Material Properties Used in the Study

The model consisted of 3 parts: strongback, panel, and soil. The material properties as entered into Abaqus are summarized below for two soil types termed Soil 1 and Soil 2.

Table 3-1 Material properties summary

	Soil		Panel (manufacturer supplied)	Strongback (manufacturer supplied)
Young's Modulus	Soil 1	1.67×10^{6} psf (80 MPa)	2.92×10^{8} psf	1.80×10^{8} psf
	Soil 2	6.27×10^{5} psf (30 MPa)		
Poisson's Ratio	Soil 1 & 2	0.3	0.45	0.3
Density	Soil 1	124 lb-f/ft^3 (3.85 lb-m/ft^3)		
	Soil 2	119 lb-f/ft^3 (3.70 lb-m/ft^3)		

3.3 Effects of Varying Strut Load

The first parametric study examined the effects of varying strut load. Soil 2 was used for this analysis with no surcharge. Three strut loads were selected for evaluation. Typical pneumatic shoring operations begin with an initial internal pressure of 50 psi to 150 psi; however "the manual systems used to tighten and lock shores can exert far greater pressure than the air used to shoot the shore. In some cases, this pressure can exceed 200 pounds per square inch" (Martinette 2008). For that reason, in internal pressure of 200 psi was selected to represent the pneumatic struts, exerting a force of 1298 lbs onto a 5 in x 5 in (0.17 ft^2) base for a pressure of 7479 psf. The internal piston pressure operating range for hydraulic shores is 750 psi to 1500 psi, so the two remaining internal pressures examined by this study were 750 psi, exerting a force of 2356 lbs for a pressure of 13572 psf, and 1500 psi, exerting a force 4712 lbs for a pressure of 27413 psf. Table 3-2 shows the forces for each internal piston pressure selected for this analysis.

Table 3-2 Internal piston pressure relationship to strut load

Type of Shore	Internal Piston Pressure (psi)	Internal Piston Diameter (in)	Force (lbs)	Base/strut foot area (ft^2)	Pressure Exerted (psf)
Pneumatic	200	2.875	1298	0.17	7479
Hydraulic	750	2	2356	0.17	13572
Hydraulic	1500	2	4712	0.17	27143

Earth pressure behind the panel was determined for the three internal piston pressures of 200 psi, 750 psi, and 1500 psi. Figure 3-3 compares the earth pressure at a depth of y = 4 ft, which corresponds to the location of the middle strut. Maximum earth pressure is equal to 393 psf behind the panel at y = 4 ft when the strut force is 1298 lbs. Maximum earth pressure is equal to 723 psf behind the panel at y = 4 ft when the strut force is 2356 lbs. Maximum earth pressure is equal to 1473 psf behind the panel at y = 4 ft when the strut force is 4712 lbs. Figure 3-4 shows the earth pressure distribution comparison for the full height of the panel. The earth pressure is higher at the location of the struts and is much less in between the struts. As expected, earth pressure increases as strut load increases. Figure 3-5 shows the relationship between earth pressure at y = 4 ft and the strut load to be linear.

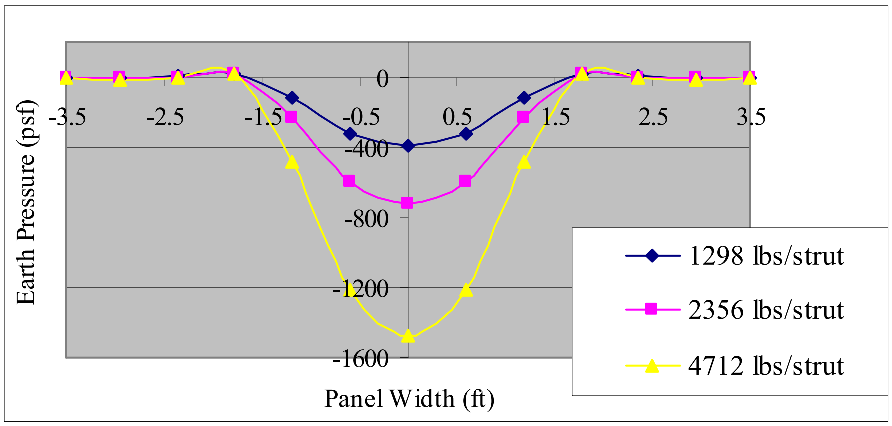


Figure 3-3 Comparison of earth pressure in the soil behind the center of the panel at a depth of y = 4 ft

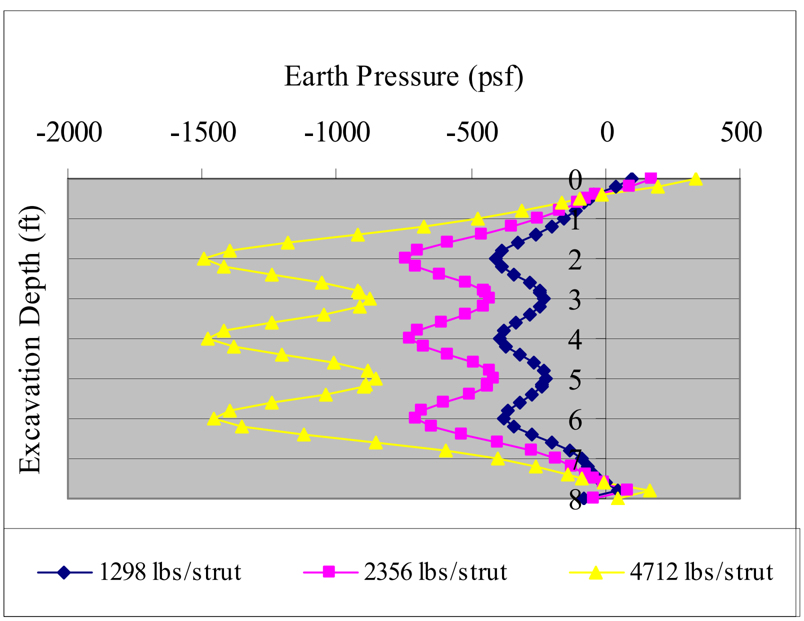


Figure 3-4 Comparison of earth pressure in the soil behind the center of the panel for the full height of the panel

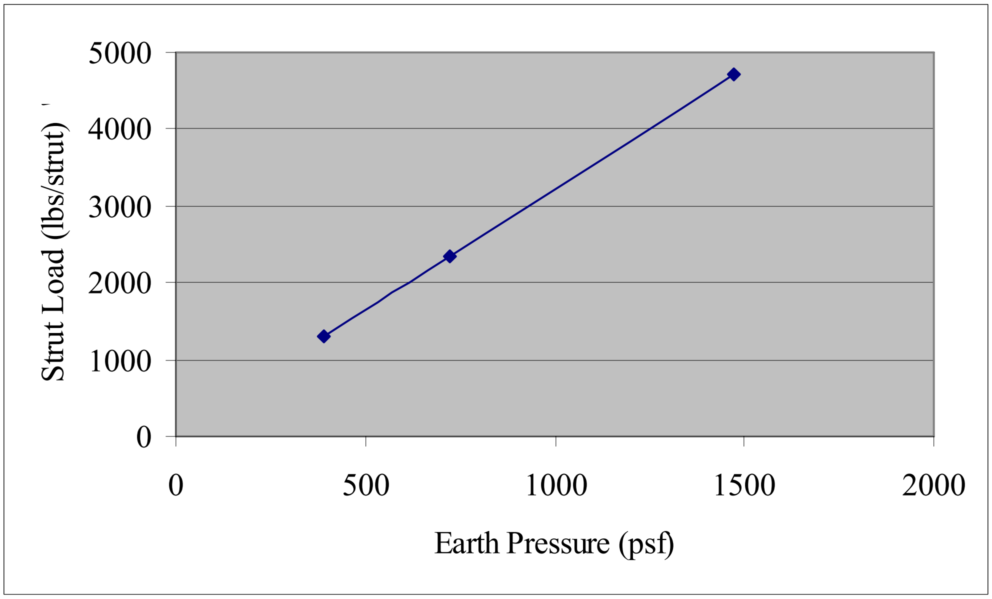


Figure 3-5 Earth pressure vs. strut load at a depth of y = 4 ft

Displacement of the strongback occurs due to a combination of soil pressure and strut loading. As shown in Figure 3-6, the inward displacement of the strongback and panel increases as strut load increases. The smallest strut load of 1298 lbs/strut is not sufficient to overcome the outward displacement due to horizontal earth pressure effects from soil pressure. As the strut force increases, the panel is pushed into the soil with an increasing inward displacement. Thus it appears that the displacement due to soil pressure is overcome by increased strut loading, potentially inducing passive failure in the soil. Appendix E shows the tabulated data.

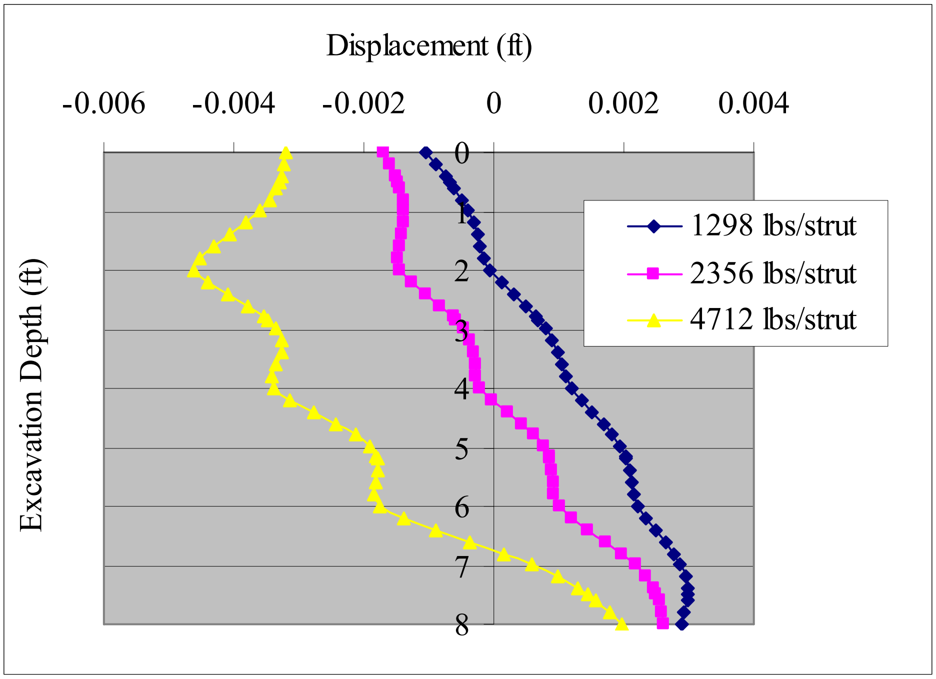


Figure 3-6 Comparison of displacement of the strongback down the full height of the panel

3.4 Effects of Varying Panel Stiffness

To study the effects of panel stiffness, seven models were run. One model simulated struts directly against the soil wall with no strongback or panel, the second examined effects of placing a 2 in x 12 in strongback between the struts and soil wall, and the remaining five models introduced a panel between the strongback and soil and varied the panel manufacturer supplied Young's Modulus, E = 292,320,000 psf. Panel dimensions are 4 ft x 8 ft x 0.75 in and stiffness varied as follows: 0.25E = 7.31×10^7 psf, 0.5E = 1.46×10^8 psf, 1E = 2.92×10^8 psf, 1.5E = 1.38×10^8 psf, and 2.0E = 5.85×10^8 psf. Soil 2 properties were used. Strut loading was assumed to be 7479 psf (200 psi internal piston pressure). The results of the study are summarized in Figure 3-7 and Figure 3-8. Appendix F shows the tabulated data.

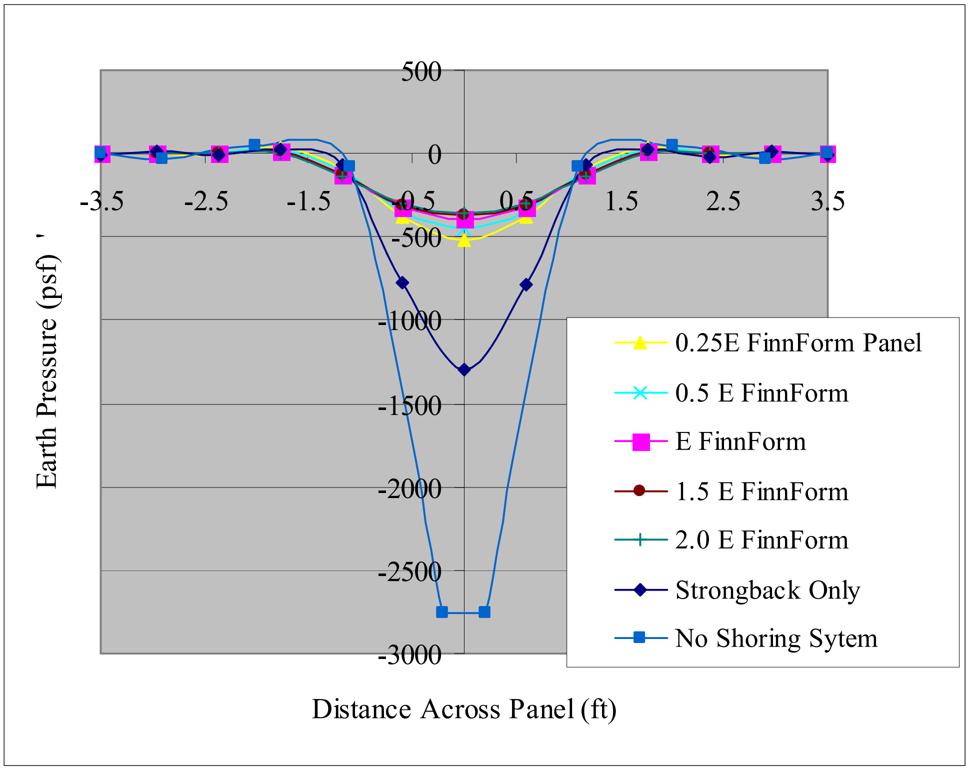


Figure 3-7 Earth pressure behind panel at a depth of y = 4 ft for varying panel stiffness; strut force is 1298 lbs/strut

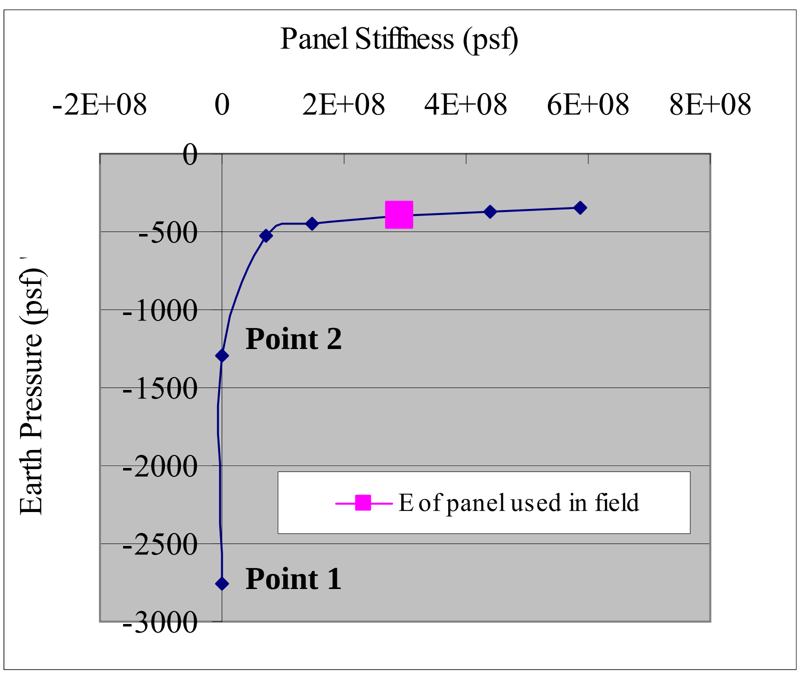


Figure 3-8 Maximum earth pressure at a depth of y = 4 ft vs. panel stiffness

When there are only struts and no panel or strongback in place, the earth pressure at y = 4 is 2760 psf (Figure 3-8 Point 1). When a strongback with manufacturer supplied material properties such as 1.0E is introduced between the struts and the soil, the earth pressure at y = 4 ft is 1293 psf (Figure 3-8 Point 2), a reduction of 53%. When a 0.75 in thick panel with a stiffness of 0.25E is introduced between the strongback and the soil to complete the standard shoring system, the earth pressure is reduced by 60% to 522 psf. As panel stiffness increases from 0.25E FinnForm (7.31×10^7 psf) to 2.0E FinnForm (5.85×10^8 psf), earth pressure drops from 522 psf to 352 psf, an additional reduction of 33%.

The results show the strongback and panel each have significant structural value. Once a panel is introduced, the stiffness also significantly affects load distribution in the soil.

3.5 Effects of Varying Panel Thickness

FinnForm panels are available in the following thicknesses: 0.25 in, 0.50 in, 0.75 in, 1.00 in, and 1.25 in. The typical thicknesses used by technical trench rescue teams are 0.75 in or 1.00 in. The minimum thickness allowed by OSHA for 14 ply arctic white birch or FinnForm brand panels is 0.75 in. However, in the same section OSHA also states that "plywood is not intended as a structural member, but only prevention of local raveling" (OSHA 1989). This study varied panel thickness from 0.25 in to 1.25 in. Strut force was held constant at 1298 lbs/strut. Soil 2 properties were used and the panel stiffness was held constant at $E = 2.92 \times 10^8$ psf. The results of the study are summarized in Figure 3-9 and Figure 3-10. Appendix G shows the tabulated data.

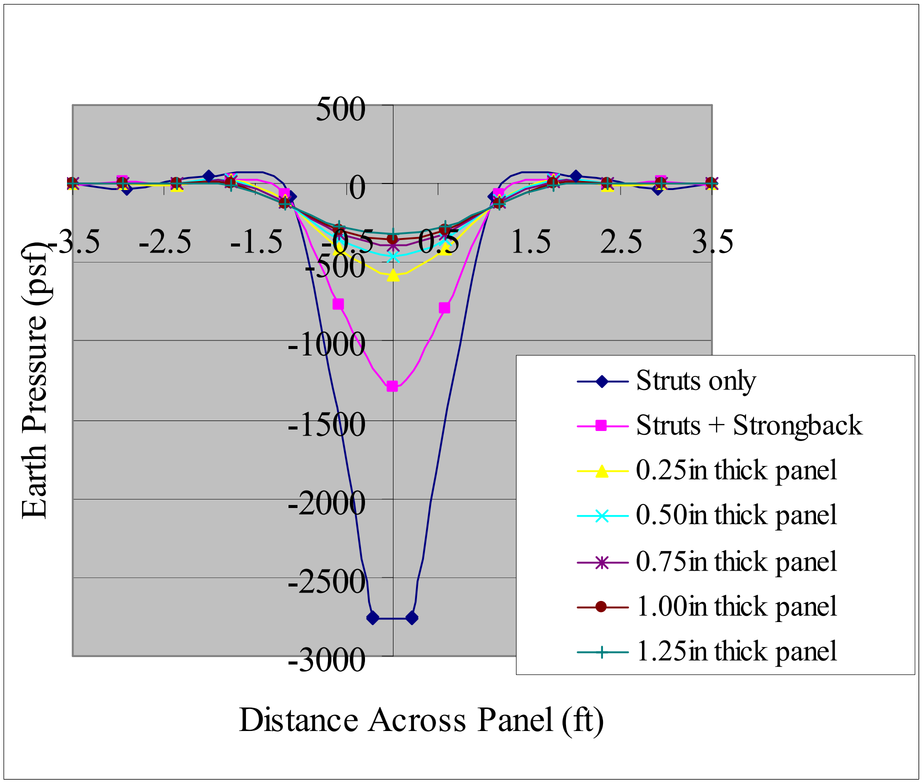


Figure 3-9 Comparison of earth pressure behind panel at y = 4 ft for varying panel thicknesses

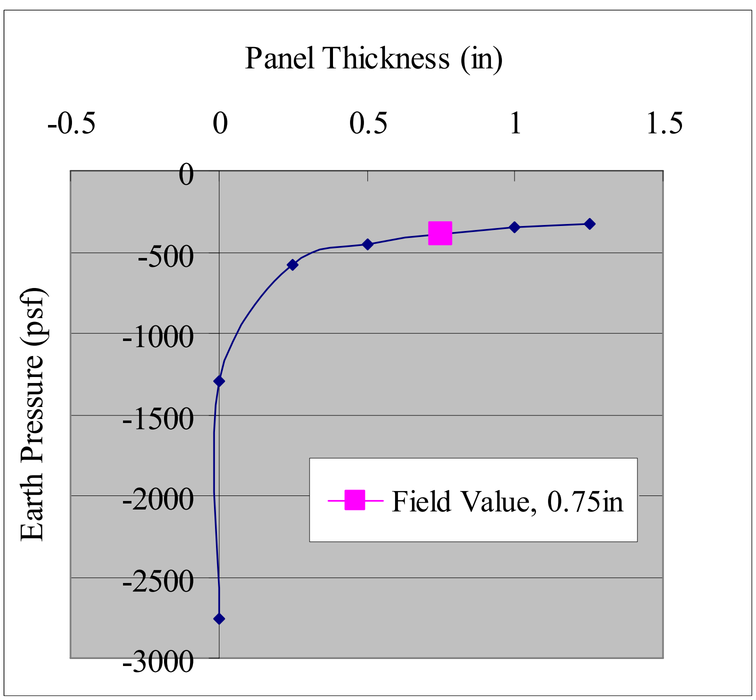


Figure 3-10 Maximum earth pressure behind panel at y = 4 ft vs. panel thickness

When there are only struts and no panel or strongback in place, the earth pressure at y = 4 ft is 2760 psf. When a strongback with manufacturer supplied material properties is introduced between the struts and the soil, the earth pressure at y = 4 ft is 1293 psf, a reduction of 53%. When a 0.25 in thick panel is introduced between the strongback and the soil to complete the standard shoring system, earth pressure drops to 575 psf, an additional 55% reduction. As panel thickness increases from 0.25 in to 1.25 in, earth pressure drops from 575 psf to 321 psf, a 44% reduction.

The results show the strongback and panel each have significant structural value. Once a panel is introduced, thickness also significantly affects load distribution in the soil.

3.6 Effects of Varying Panel Width

Although FinnForm is currently only available in 4 ft x 8 ft sheets, models were run to examine earth pressure behind panels of varying width in order to evaluate if using wider or narrower panels significantly affect load distribution. The first model analyzed the effects of using only a strongback between the struts and the soil. The rest of the models analyzed the effects of using a strongback and a 0.75 in thick FinnForm panel with the following widths: 2 ft, 3 ft, 4 ft, 6 ft and 8 ft. The models in this case were run with both Soil 1 and Soil 2 properties. Strut force was held constant at 1298 lbs/strut and the panel stiffness was set at the FinnForm manufacturer supplied Young's Modulus, E = 2.92×10^8 psf. The results of the study are summarized in Figure 3-11 and Figure 3-12. Figure 3-13 presents a comparison of earth pressures at a depth of y = 4 ft obtained from Figure 3-11 and Figure 3-12 versus panel width. There is minimal change in earth pressure once the panel is introduced between the soil and the strongback. Appendix H shows the tabulated data.

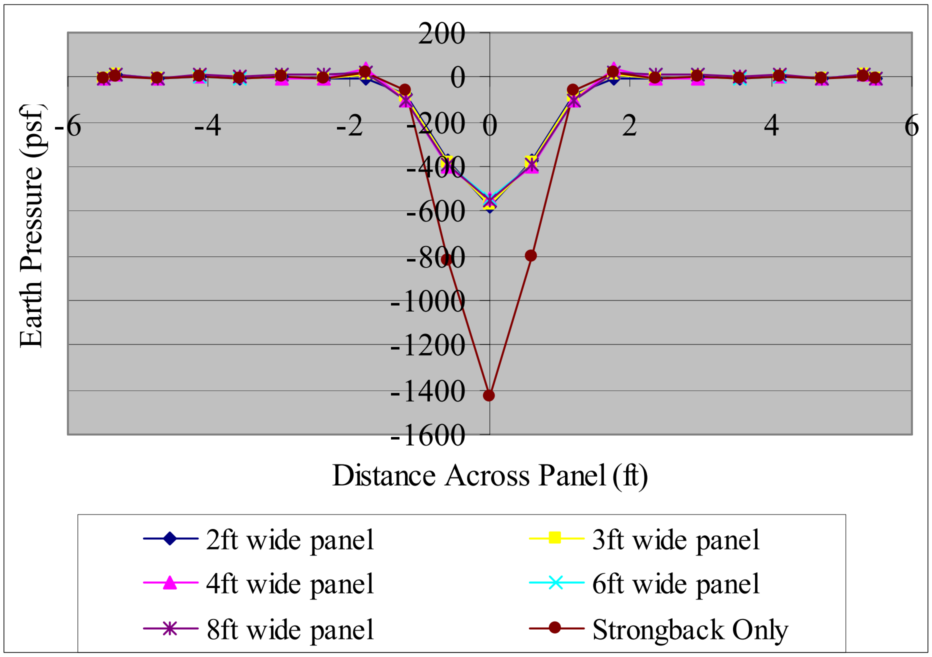


Figure 3-11 Comparing the effects of panel width on earth pressure at a depth of y = 4 ft in Soil 1, E = 1.67x10^6 psf

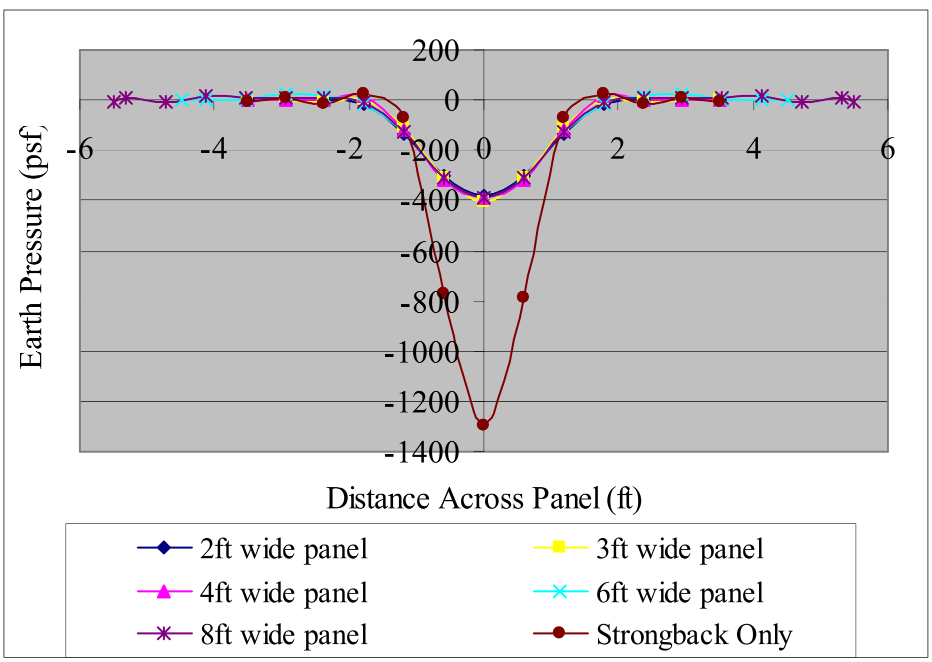


Figure 3-12 Comparing the effects of panel width on earth pressure at a depth of y = 4 ft in Soil 2, E = 6.27x10^5 psf

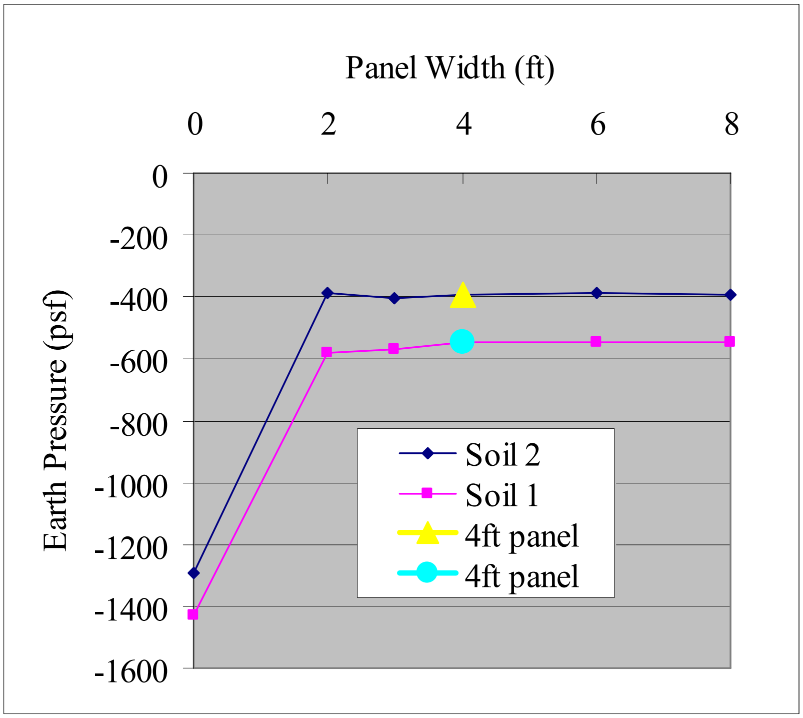


Figure 3-13 Maximum earth pressure at y = 4 ft vs. panel width

When only a strongback (no panel) with manufacturer supplied material properties is introduced between loaded struts and the soil, the maximum earth pressure at y = 4 ft in Soil 1 is 1432 psf. The earth pressure at the same location in Soil 2 is 1293 psf. When a 0.75 in thick FinnForm panel is introduced between the strongback and the soil to complete the standard shoring system, earth pressure is significantly reduced, but not greatly affected by panel width. Earth pressure ranged from 580 psf to 544 psf in Soil 1, equivalent to a 6.25% change, and 402 psf to 384 psf in Soil 2, equivalent to a 4.37% change. Thus, there was little change in pressures with panel width. Fluctuations may be attributable to altered panel/soil mesh alignment in the model as panel width changed.

The results show the panel has a significant structural value, but width does not affect the load distribution in the soil.

3.7 Surcharge Variation Study

A series of models were run to examine the effects of surcharge size and configuration. The first models examined effects of surcharge distance from the edge of the trench on the earth pressure by moving the surcharge incrementally away from the edge of the trench in the Z direction. The next models examined the combined effects of distance from the edge of the trench and size of the surcharge on earth pressure. The final models examined effects of changing surcharge placement in the X direction. Figure 3-14 shows the model orientation in space.

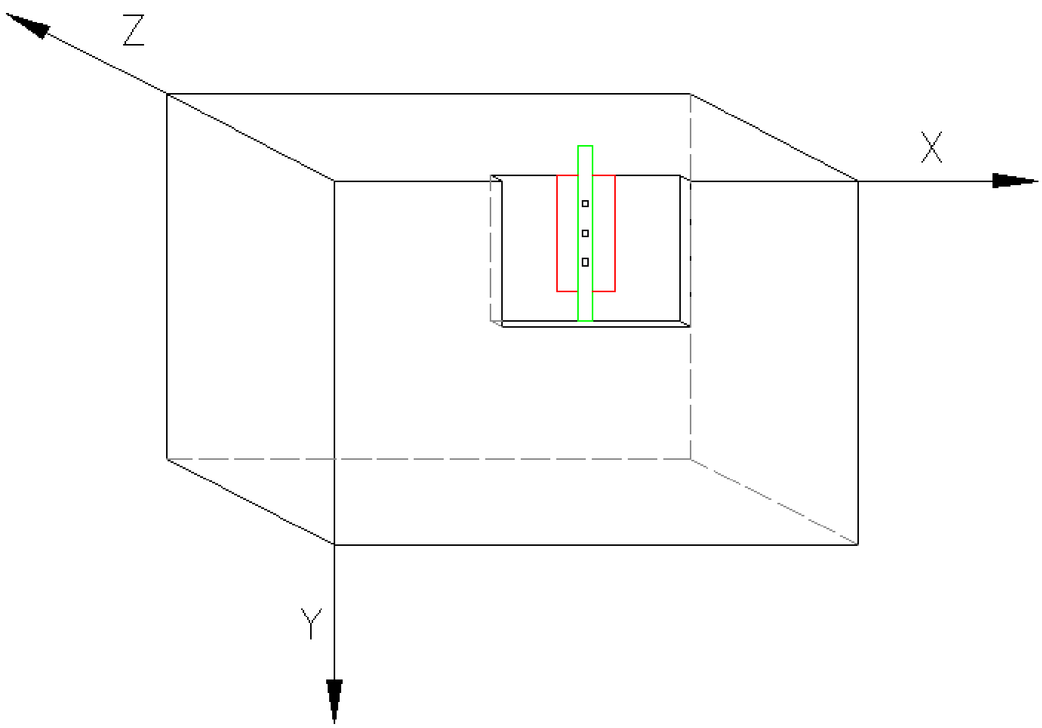
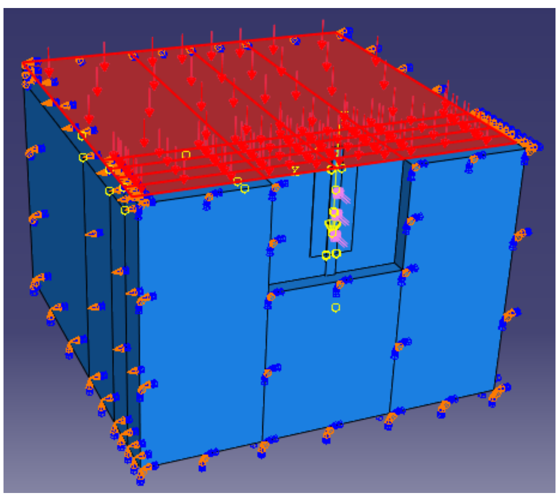


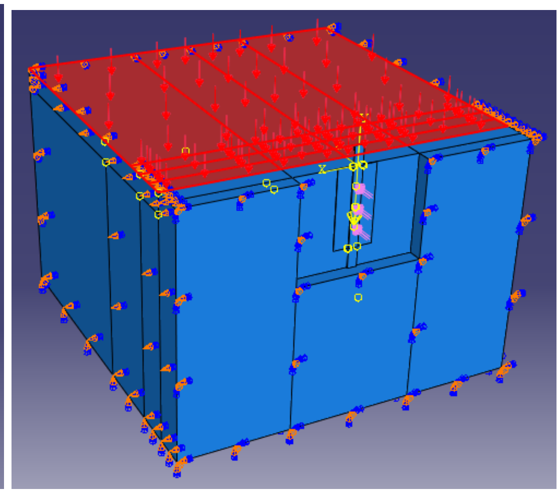
Figure 3-14 Model orientation

3.7.1 Evaluating Effects of Surcharge Distance from the Edge of the Trench

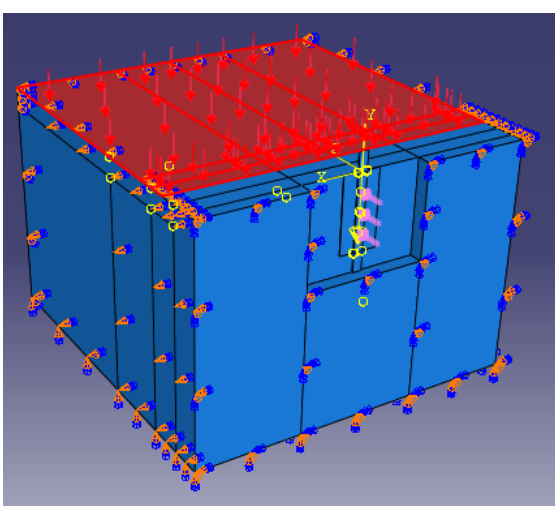
The first models looked at a 300 psf surcharge and its effects on earth pressure in relation to distance from the edge of the trench in the Z direction. Figure 3-15 (a) through (g) illustrates surcharge location in relation to the edge of the trench. Soil 2 properties were used. Unit weight of Soil 2 is 119 pcf, therefore 300 psf represents a spoil pile that is approximately 2.5 ft high. Manufacturer supplied material properties were used for the 2 in x 12 in x12 ft strongback and 4 ft x 8 ft x 0.75 in panel. Strut force was set to 1298 lbs representing an internal piston pressure of 200 psi. The first model placed the surcharge directly up to the edge of the trench (Z = 0 ft). The next four models moved the surcharge away from the trench edge in 2 ft increments (Z = 2 ft, 4 ft, 6 ft, 8 ft). The next model moved the surcharge an additional 8 ft away from the edge (Z = 16 ft) and the final model had no surcharge present at all, representative of Z = 36 ft. At Z = 36 ft, the boundary conditions govern.



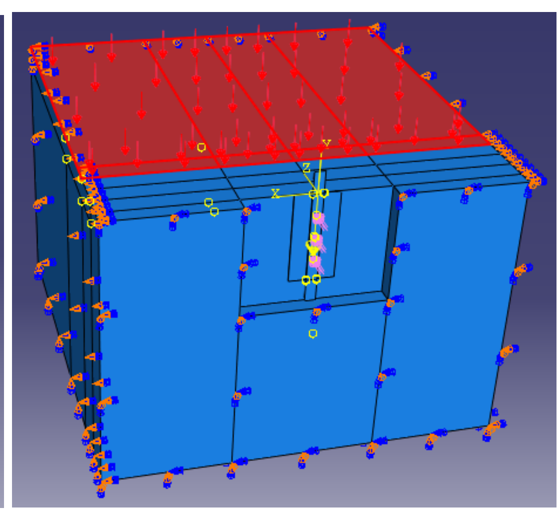
(a)



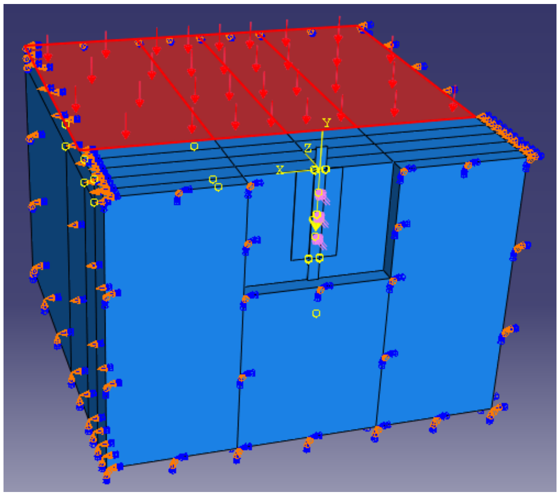
(b)



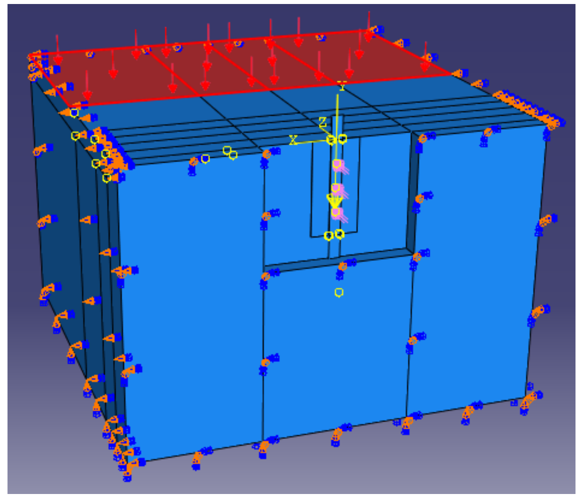
(c)



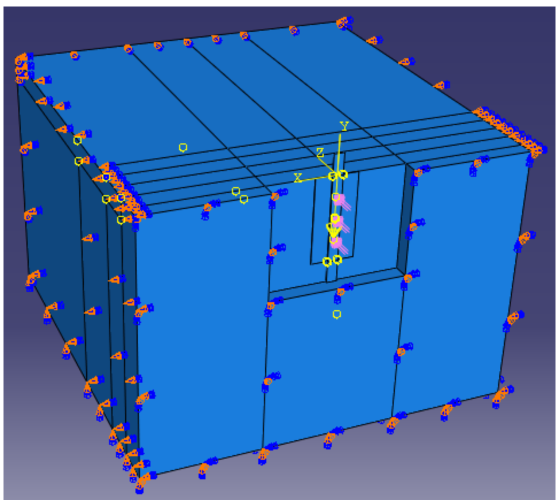
(d)



(e)



(f)



(g)

Fig 3-14 Surcharge shown in red up to (a) the edge of the trench, (b) 2 ft from the edge of the trench, (c) 4 ft from the edge of the trench, (d) 6 ft from the edge of the trench, (e) 8 ft from the edge of the trench, (f) 16 ft from the edge of the trench, (g) 36 ft from the edge of the trench (due to the overall model size, at 36 ft, there is no surcharge present at all)

The resultant maximum earth pressures behind the panel at a depth of y = 4 ft is as shown in Figure 3-16. When the surcharge is at Z = 0 ft, the pressure is 306 psf. When the surcharge is at Z = 4 ft, the pressure increases to 323 psf. The pressure then decreases as Z increases beyond 4 ft.

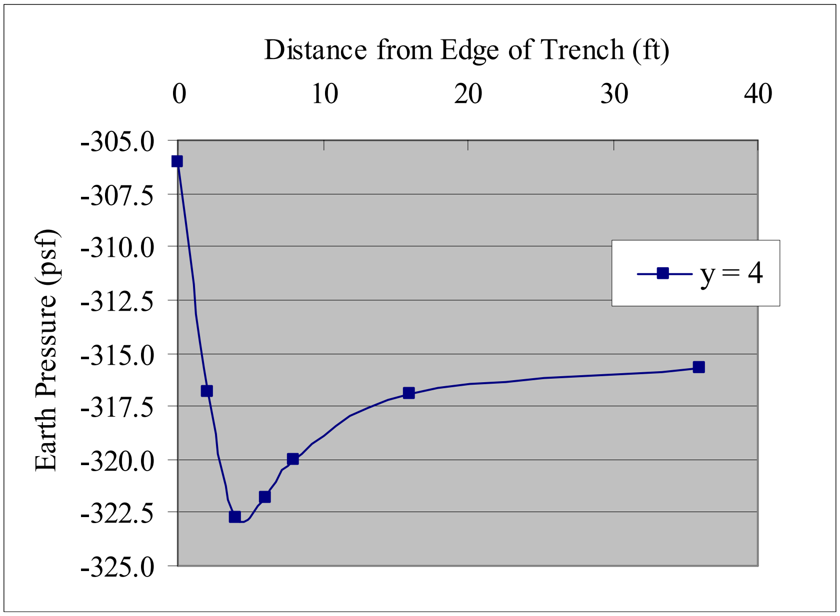


Figure 3-16 Maximum earth pressure at a depth of y = 4 ft vs. surcharge distance from the edge of the trench

The same models were then run for 600 psf and 900 psf. Unit weight of Soil 2 = 119 pcf, therefore 600 psf represents a 5 ft high spoil pile and 900 psf represents 7.6 ft high spoil pile. When the surcharge was set at 600 psf, maximum earth pressure behind the panel at y = 4 ft ranged from 296 psf to 330 psf, an 11% difference. When the surcharge was set at 900 psf, maximum earth pressure behind the panel at y = 4 ft ranged from 287 psf to 334 psf, a 17% difference. In all analyses, earth pressure was greatest when the surcharge was 4 ft away from the edge of the trench. The results for all three surcharge analyses are summarized in Figure 3-17. Appendix I shows the tabulated data.

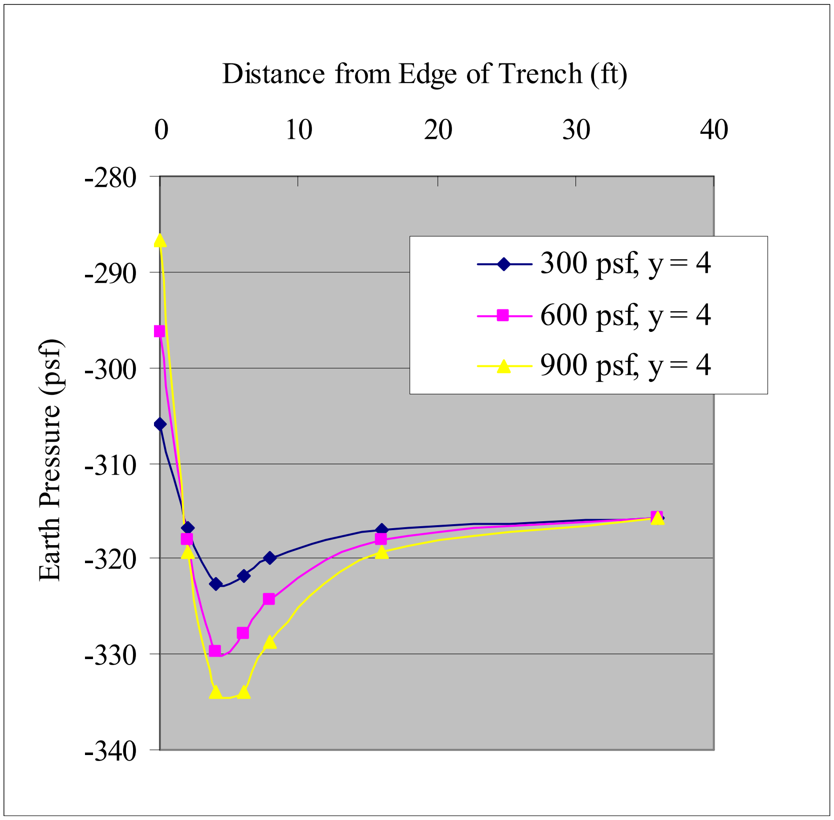
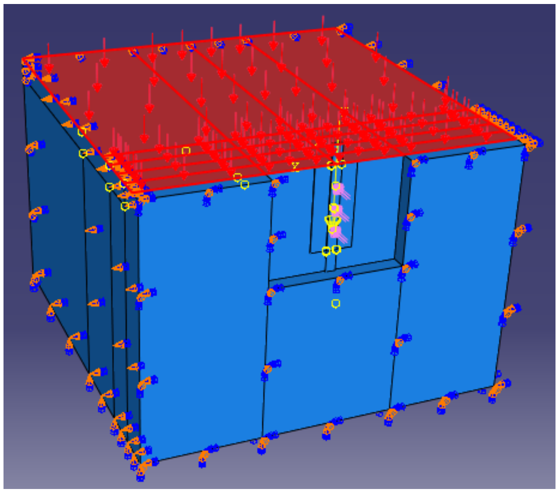


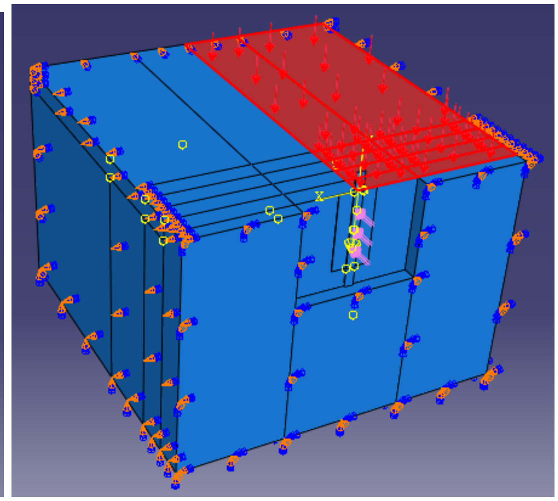
Figure 3-17 Comparison of maximum earth pressure behind the panel at y = 4 ft vs. distance from edge of trench to surcharge

3.7.2 Evaluating Effects of Lateral Surcharge Location in the X Direction

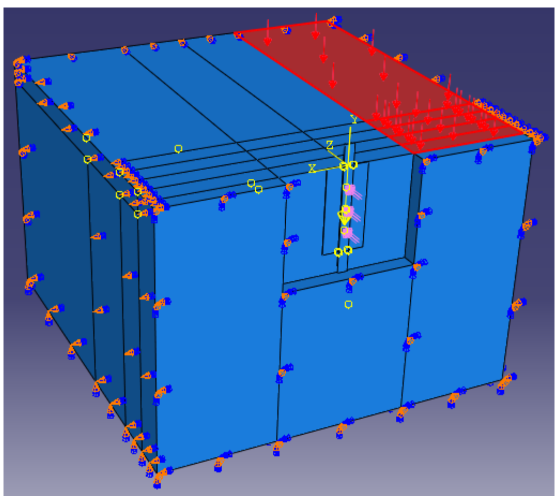
The last surcharge models examined the effects on earth pressure due to the location of a 300 psf surcharge changing in the location in the X direction. Figures 3-18 (a) through (c) illustrate the three surcharge configurations evaluated in this analysis.



(a)



(b)



(c)

Fig 3-18 Surcharge shown in red (a) across the entire model (X = 0) (b) across half the model (X = 18) and (c) from the end of the trench to the end of the model (X = 25.5)

The maximum earth pressure behind the panel at y = 4 ft was equal to 306 psf when the surcharge covered the entire model (X = 0). The maximum earth pressure behind the panel at y = 4 ft was equal to 312 psf when the surcharge covered half of the model (X = 18). The maximum earth pressure behind the panel at y = 4 ft was equal to 315 psf when the surcharge covered from the end of the trench to the end of the model (X = 25.5). The total change in earth pressure as the surcharge location varied in the X direction was 3%. The results for the three 300 psf surcharge models moving from X = 0 to X = 25.5 are summarized in Figure 3-19.

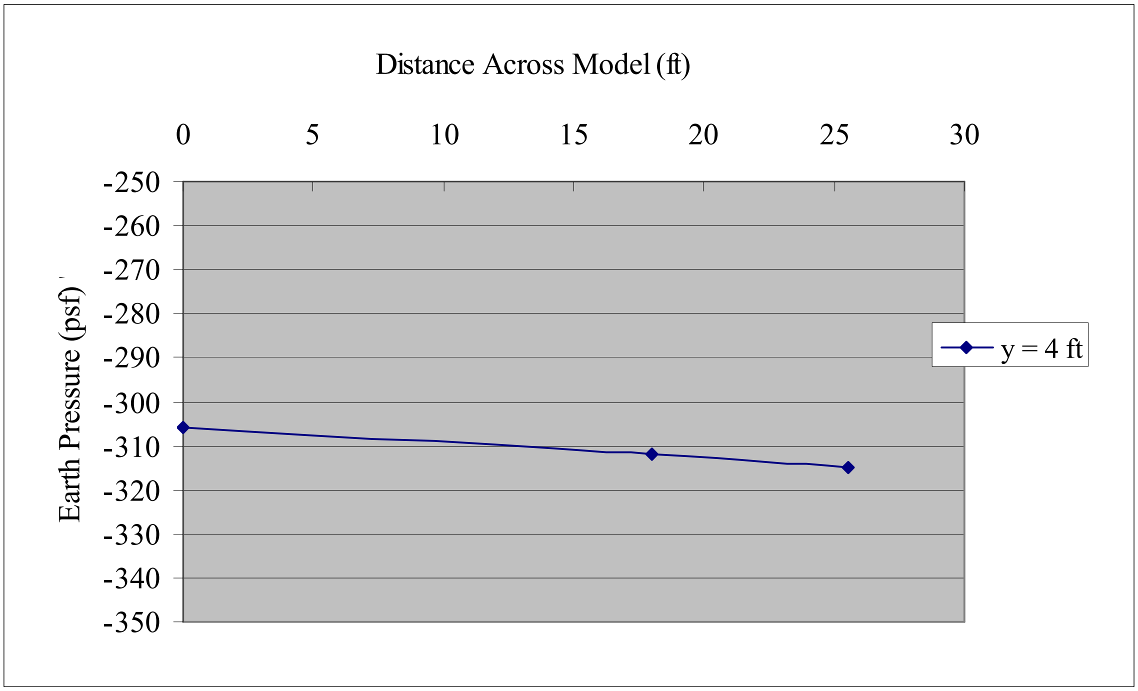


Figure 3-19 Earth pressure comparison behind the panel at y = 4 ft due to changing surcharge locations in the X direction

The analysis results show that the location of a 300psf surcharge (approximately equivalent to a 2.5 ft high spoil pile) does not have a significant effect on the earth pressure. Appendix J shows the tabulated data.

3.8 Summary

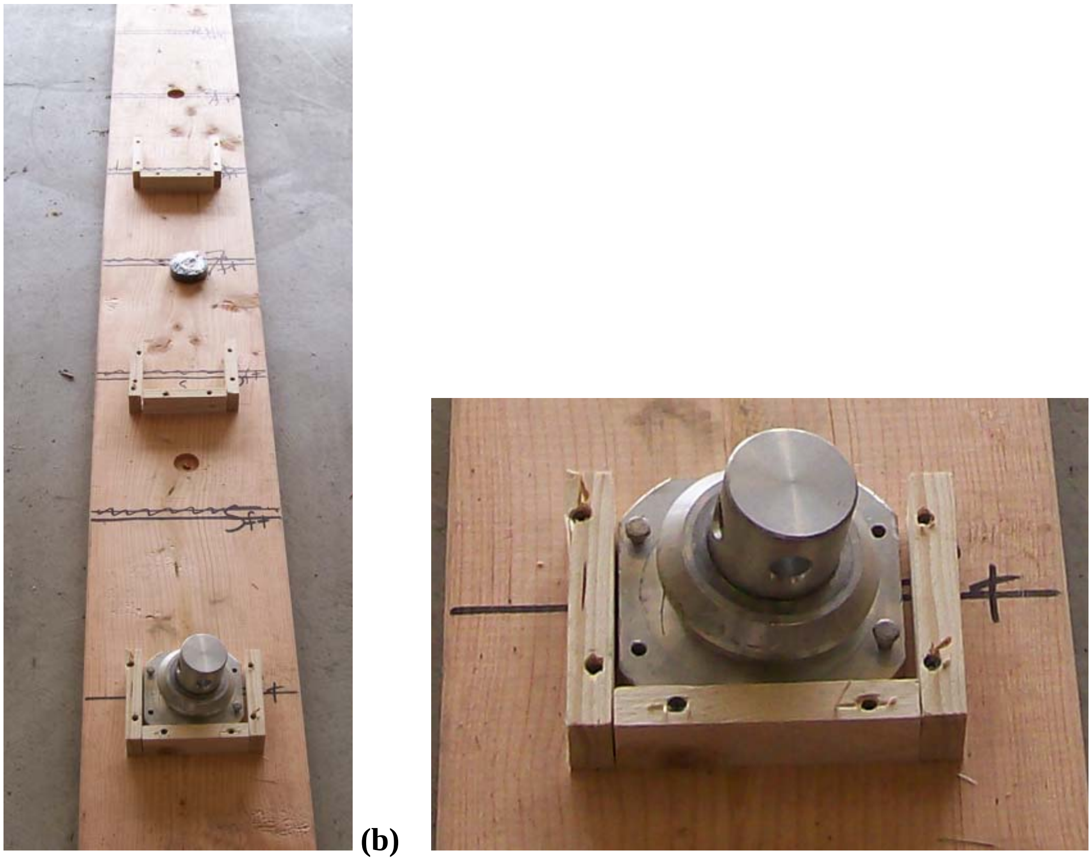
This chapter used multiple parametric studies of a linear elastic model to examine the effects of the current standardized trench rescue shoring system on earth pressure. The studies examined effects of varying strut load, panel stiffness, panel thickness, panel width, and surcharge size and configuration. It was found that varying strut load had a significant effect on earth pressure. It was also found that the strongback and panel each have significant structural value. A #2 kiln dried southern yellow pine strongback reduced earth pressure behind the panel an average of 53% and the typically used 0.75 in thick 4 ft x 8 ft FinnForm brand panel reduced the earth pressure an additional 32%. It was found that increasing panel stiffness from 0.25E to 2.0E also significantly affected earth pressure, reducing it from 522 psf to 352 psf, a 33% difference. Increasing panel thickness from 0.25 in to 1.25 in was also found to significantly affect earth pressure, reducing it from 575 psf to 321 psf, a 44% difference. The width of the panel was not found to have a significant effect on earth pressure. Surcharge configurations did not greatly affect earth pressure, but increasing surcharge size close to the trench edge produced increasing earth pressure. The parametric study examined a 10 ft deep by approximately 3 ft wide trench, so spoil piles of 2 ft to 3 ft high were a reasonable assumption. This size trench is typical for short term utility work. In deeper or wider excavations that produce larger spoil piles, surcharge may be of increasing concern.

Chapter 4 Laboratory and Field Work

An experimental program was undertaken to study the effects of soil type on the response of a typical trench rescue shoring system. Two full scale trenches were used. One was in Frederick, MD where the soil was mostly granular in nature and the second was in York, PA where the soil was clayey in nature. This chapter describes a typical trench rescue shoring system and presents results from laboratory tests performed on its components as well as the results of laboratory testing on the soils excavated from the trenches. The information presented includes the locations of the trenches, a description of the tests performed in the trenches, and the data collected.

4.1 Shoring System Assembly

The standard trench rescue shoring system, as described in section 1.4, consists of a panel, strongback, and struts. A full scale system was assembled and tests were run in two trenches with different soil properties: granular (Frederick, MD) and clay (York, PA). The strongback was prepared with slots for ease of strut foot placement during testing (Figure 4-1 a & b). The slots were centered 4 ft, 6 ft, and 8 ft from the top of the 12 ft long strongback, placing the strut feet at y = 2 ft, y = 4 ft, and y = 6 ft during testing (Figure 4-2). The strongback was then bolted to the panel (Figure 4-3).



(b) (a)

Figure 4-1 (a) & (b) Strongback setup with strut foot in place

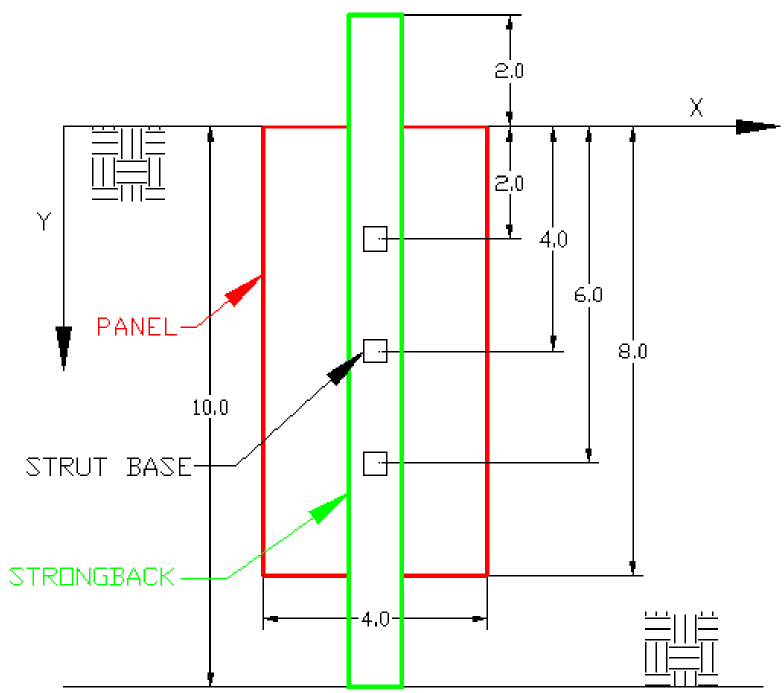


Figure 4-2 Panel (red), strongback (green) and strut placement in relation to top of trench

Figure 4-3 Assembled (bolted) strongback and panel

4.1.1 Panel: FinnForm Brand 0.75 in Thick 14 Ply Arctic White Birch

FinnForm, the panel manufacturer, provided the modulus of elasticity and Poisson's ratio for a 0.75 in thick panel as 2.9232×10^8 psf and 0.45 respectively. "The superior hardwood strength properties of Finnish White Birch combined with thin multiple veneer panel construction makes FinnForm the benchmark for quality in the plywood forming industry. The 200 g/m^2 phenolic surface film on both faces provides very high reusability" (Plywood and Door Manufacturers Company 1999a).

Four samples were prepared for testing at the University of Maryland Civil Engineering Materials Testing Lab. Samples 2 in wide by 30 in long were cut from a 0.75 in thick FinnForm panel. Standard Test Methods for Small Clear Specimens of Timber (ASTM D143-94) was followed except with regard to sample size. The sample was not 2 in thick as specified in the standard because the material was only available in a 0.75 in thickness. Testing of the samples gave an average Young's modulus of

1.83×10^8 psf at failure which is 62.5% of the manufacturer supplied value of 2.9232×10^8 psf and 2.1183×10^8 psf in the elastic range of the test, which is 72.2% of the manufacturer supplied value of 2.9232×10^8 psf. The test data obtained may be different from the manufacturer because the manufacturer supplied data was "derived from thousands of in-use tests", or plate testing (Plywood 1999a). The samples tested for this research in the Materials Testing Lab were tested as beams. The data obtained is shown in Appendix K. Figure 4-4 shows a sample during testing.

Figure 4-4 Testing FinnForm sample, 0.75 in thick

4.1.2 Strongback: #2 Kiln Dried Southern Yellow Pine

According to the 2003 *Wisconsin Building Products Evaluation* the modulus of elasticity at ultimate load for #2 kiln dried south yellow pine is 1.80×10^8 psf and Poisson's ratio is 0.30 (Wisconsin 2003). Three samples were prepared for testing at the

University of Maryland Civil Engineering Materials Testing Lab. Samples 2 in wide by 30 in long were cut from a 2 in thick strongback. ASTM D143-94 Standard Test Methods for Small Clear Specimens of Timber was followed. Material testing of the samples gave an average Young's modulus of 1.425×10^8 psf at failure, which is 78.6% of the manufacturer supplied value of 1.80×10^8 psf and 1.78×10^8 psf in the elastic range of the test which is 98.8% of the manufacturer supplied value of 1.80×10^8 psf. The data obtained is shown in Appendix L. Figure 4-5 shows a sample after failure.

Figure 4-5 Testing #2 kiln dried southern yellow pine sample, 1.5 in thick

4.1.3 Pneumatic Struts

Three Airshore brand pneumatic struts and air supply system were made available by the urban search rescue team in Montgomery County, MD for use in this research. Figure 4-6 illustrates the parts, internal construction and operation of a typical pneumatic strut. The air cylinder is connected to the pressure regulator in order to reduce the air pressure in the cylinders from as much as 4500 psi to the desired pressure for operating the struts (Airshore brand struts are rated for no more than 300 psi internal pressure). An air hose connects the pressure regulator to the control valve which is manned by a rescuer. When the valve is opened, pressurized air flows from the air cylinders shown in Figure 4-7 through the air fitting and fills the piston chamber, exerting a force on the piston which extends the strut. Once the strut is extended as far as possible, exerting force on the trench wall, a rescuer must descend part way into the trench to secure the collar with locking pins and T-handles. Once the collar is secured, the rescuer operating the control value can then release pressure. The locked collar maintains the force exerted on the trench wall.

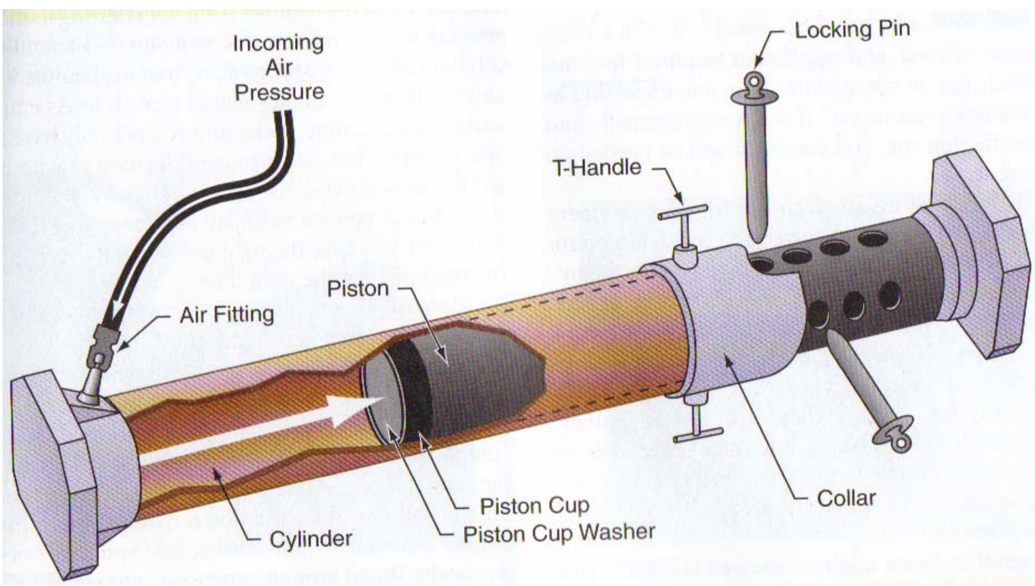


Figure 4-6 Internal diagram of a pneumatic shore (Martinette 2008)

Common practice in the field is for rescuers to manually tighten the collar beyond initial extension during pressurization rather than simply lock the collar in place, exerting additional forces beyond that induced by the strut’s internal piston. During testing specific to this dissertation care was taken to avoid exerting additional and unaccounted for forces beyond that induced by the initial strut pressurization.

Figure 4-7 Air supply system parts clockwise from bottom left: air supply hose, pressure regulator, control value, air cylinders

Figure 4-8 illustrates the 2.875 in internal diameter of the Airshore brand pneumatic strut used for testing. Figure 4-9 shows the internal piston head with rubber seal. Pressurized air supply forces the internal piston to extend the strut. Strut pressure for testing was set to 200 psi to represent a medium between typical air pressures used during operations and manually-induced unaccounted forces. Pressure of 200 psi on the internal piston head with an area of 6.49 in^2 translated to 1298 lbs distributed over a 0.17 ft^2 strut base.

Figure 4-8 Internal diameter of the Airshore brand pneumatic struts used in testing

Figure 4-9 Internal piston with rubber seal

Once pressurized, the strut is manually locked in position with pins and the pressure is released. In the locked position, Airshore brand pneumatic struts are rated by the manufacturer to resists 19000 lbs of force.

4.2 Trenches

Testing was done in two different soil types: granular soil in Frederick, MD and clayey soil in York, PA.

4.2.1 Frederick – Granular Soil

The first trench was excavated in Frederick County, Maryland. Figure 4-10 shows the trench location. The trench was 7 ft wide in a granular soil. The trench walls showed signs of instability during excavation (Figure 4-11), but remained stable after excavation and for the duration of the testing.

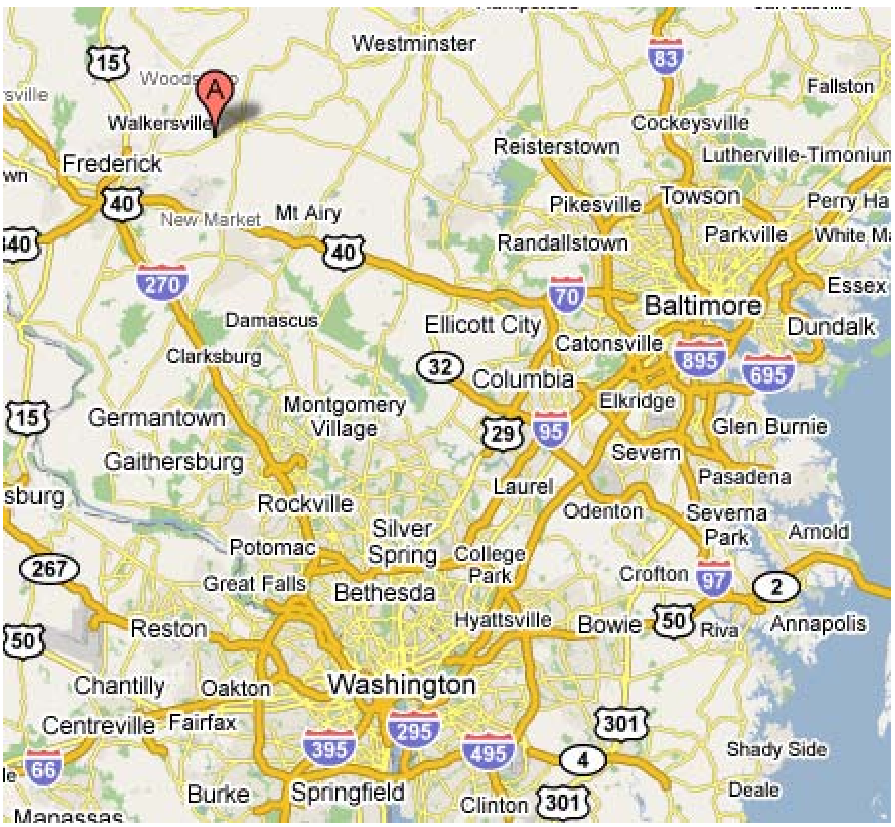


Figure 4-10 Google map of Frederick trench location (A) in relation to Washington, DC and Baltimore, MD (Google (a))

Figure 4-11 Frederick trench: partial collapse of trench wall during excavation

The excavator moved the spoils piles away from either side of the trench for safety concerns (Figure 4-12). The final location of the spoil pile was 7.5 ft right of panel center and six feet back from the edge of the trench. Figure 4-13 shows the testing location in relation to the spoil pile. The spoil was approximately 15 ft x 15 ft (225 sf), 2.5 ft high, and had a unit weight of 124 pcf. The total weight of the surcharge was 69,750 lbs, equivalent to a pressure of 310 psf.

Figure 4-12 Frederick trench: excavator moving soil pile away from trench edge after excavation

Figure 4-13 Frederick trench: testing setup between trench and spoil pile

4.2.1.1 Soil Properties

Standard Method for Particle-Size Analysis of Soils (ASTM D422-63) was performed at the University of Maryland Civil Engineering Materials Testing Lab on a soil sample from the trench to determine classification in order to estimate a Young's Modulus and Poisson's Ratio. Appendix M shows the results of the soil testing. Fifty four percent of the visible grains were larger than the No. 4 sieve (4.74 mm), therefore the sample was a gravel. However, visual inspection of the soil indicated that it consisted of gravel and larger rock that was held together with small amounts of clay. The larger rocks and clay fines were not captured in the test sample. Figure 4-14 shows a typical rock found in the spoil pile that could not be collected or accounted for by the test sample. Figure 4-15 shows that overall the soil is granular and rocky in nature. The modulus of elasticity for the granular soil was assumed to be 1.67×10^6 psf (80 MPa) and the Poisson's ratio was assumed to be 0.3 (Budhu 2007).

Figure 4-14 Sample rock pulled from Frederick trench spoil pile

Figure 4-15 Close up of spoil pile noting abundance of large rocks

4.2.1.2 Testing Equipment

The strain gauges were acquired from Vishay Micro-Measurements. They are General Purpose C2A-13-125LT-350: grid resistance = 350 ± 0.6% ohms, gauge factor = 2.13 ± 1.5%. The installation process is presented in the Vishay Instruction Bulletin B-127-14, provided in Appendix N. Figure 4-16 shows an installation in progress. Figure 4-17 shows the completed installation of a strain gauge. Vishay M-Coat A air drying polyurethane coating was added to protect the gauges from moisture interference in the trench. Figure 4-18 is a diagram of where the strain gauges were placed on the panel in relation to the trench wall and strut placement. Gauges 4 through 1 were considered the left side of the panel. Gauges 5 through 8 were considered the right side of the panel.

Figure 4-16 Gauges installation in progress

Figure 4-17 Complete gauge installation

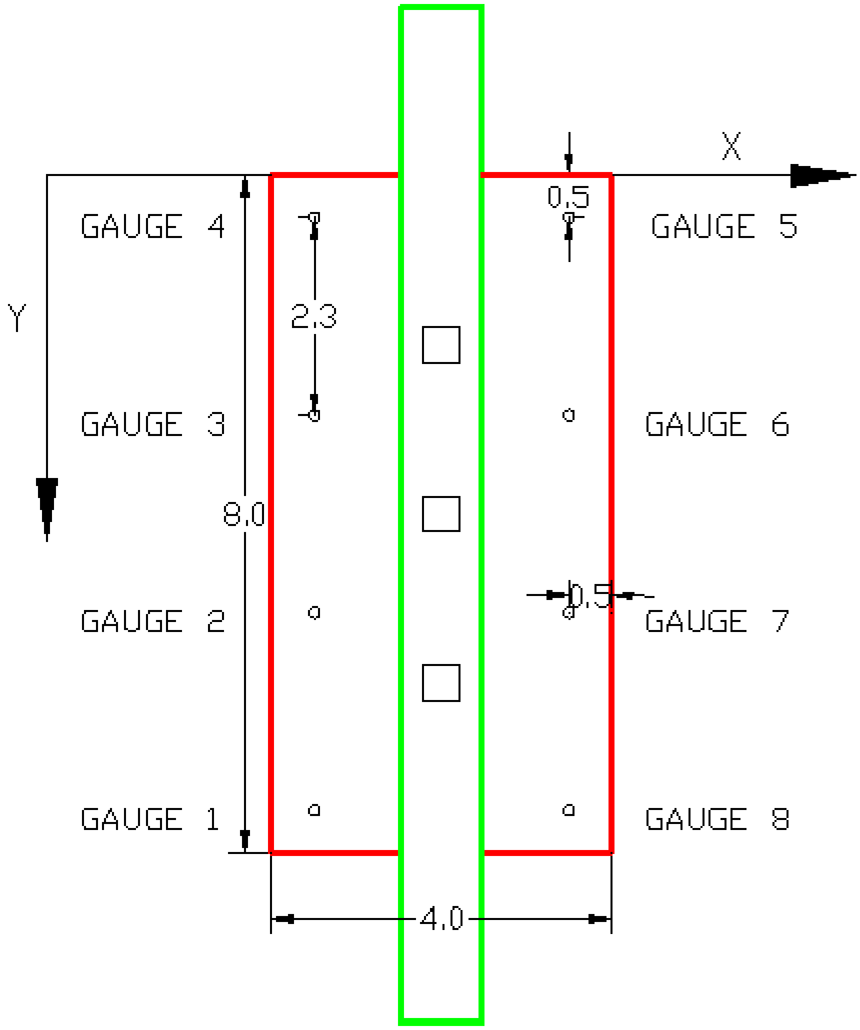


Figure 4-18 Gauge location diagram

The gauges were rosettes (Figure 4-19) with two separate 0.125 in gauges incorporated together, so one rosette had the capability of providing strain measurements in both the X and Y directions. The switch and balance unit, also by Vishay, had the capacity of reading ten measurements at a time. All of the X direction measurements were read while running one set of tests and the instrumentation hookup was changed to read all of the Y direction measurements during separate tests.

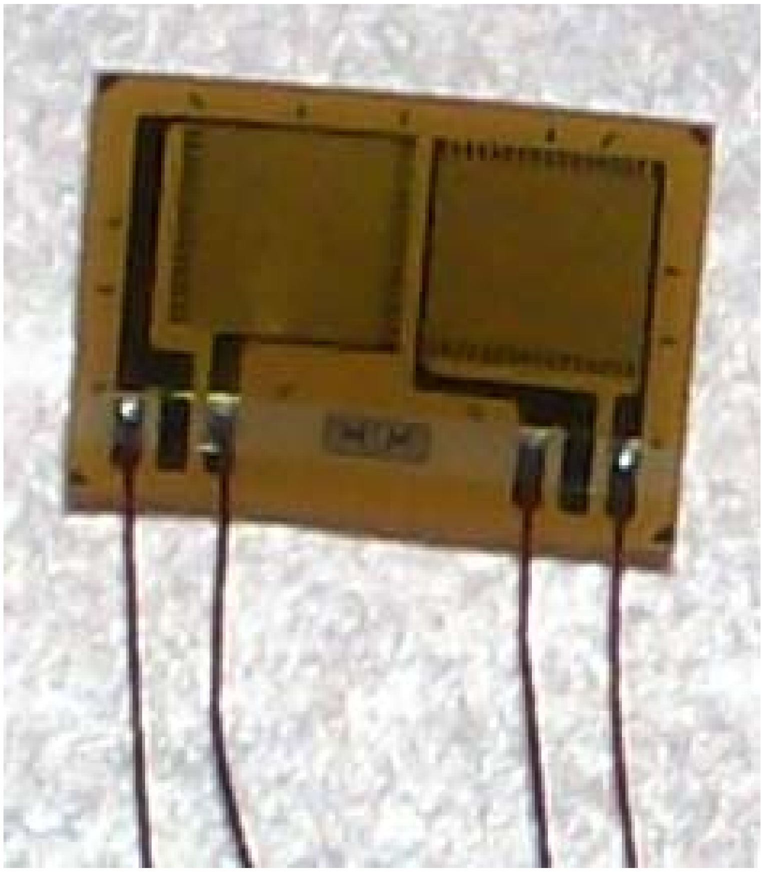


Figure 4-19 Rosette strain gauge: left gauge reads strain in the X direction; right gauge reads strain in the Y direction

The nominal gauge factor (2.13±1.5%) was used because gauges oriented in the X and Y directions were hooked up to the switch and balance unit (Figure 4-20) at the same time. The quarter bridge configuration was used in order to get separate readings from each direction on the strain gauge. If a full bridge connection had been used, the reading on the strain indicator would have been an average value over both directions. The values of strain in the X direction were expected to vary significantly from the values in the Y, so it was logical to measure the values separately rather than take an average. The switch and balance unit was then hooked up to the strain indicator (Figure 4-21).

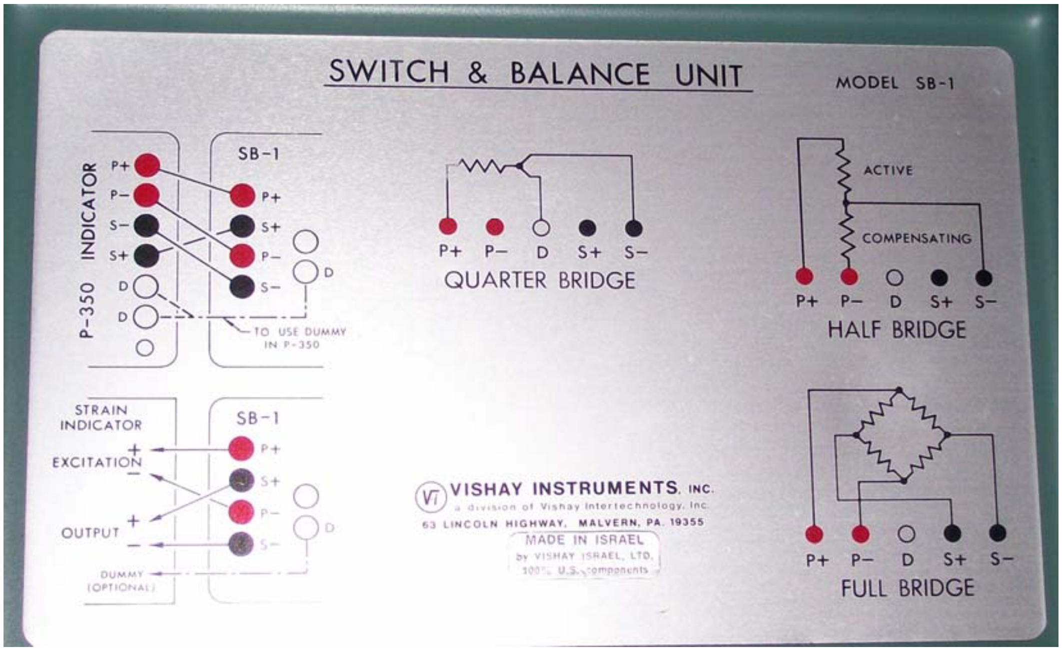


Figure 4-20 (a) Switch and balance unit by Vishay inside the top cover

Figure 4-20 (b) Switch and balance unit by Vishay gauge connections

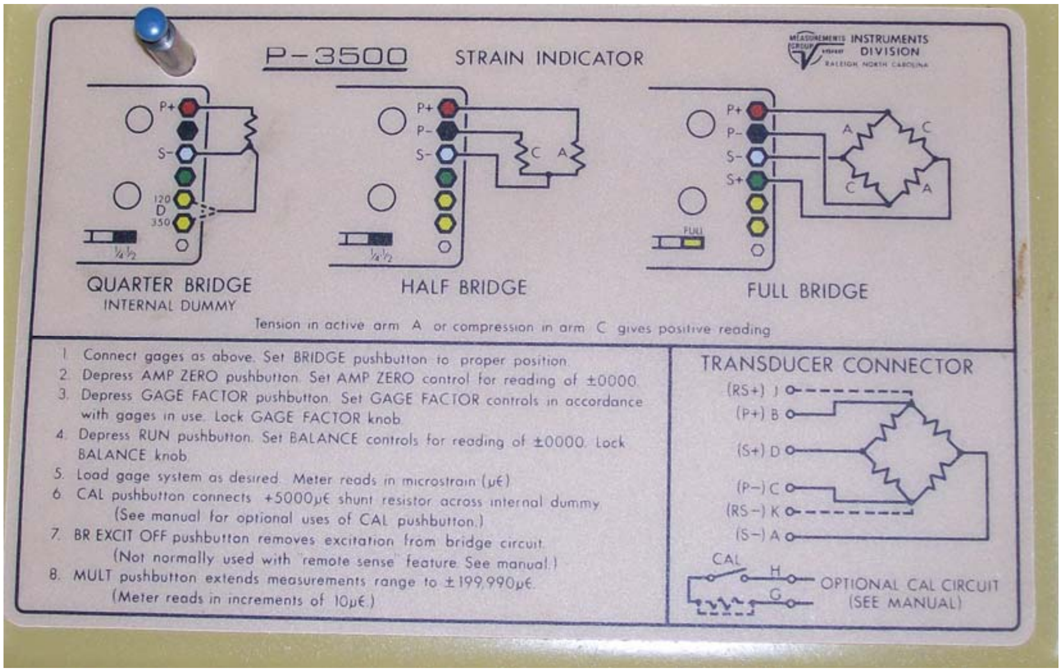


Figure 4-21 (a) Strain indicator by Vishay inside the top cover

Figure 4-21 (b) Strain indicator by Vishay output panel displaying nominal gauge factor

4.2.1.3 Test Equipment Setup at Test Site

The first piece of equipment placed in the trench was an ordinary plywood shoring panel. The instrumented panel was lowered into place opposite from the ordinary plywood (Figure 4-22).

Figure 4-22 The testing panel is set in the York (clay) trench opposite from an ordinary plywood panel

Once the panels were in place and tied back to stakes for stability, the selected connections were hooked up to the switch and balance unit as shown in Figure 4-23 (a). The switch and balance unit was then connected to the strain indicator as shown in Figure 23 (b).

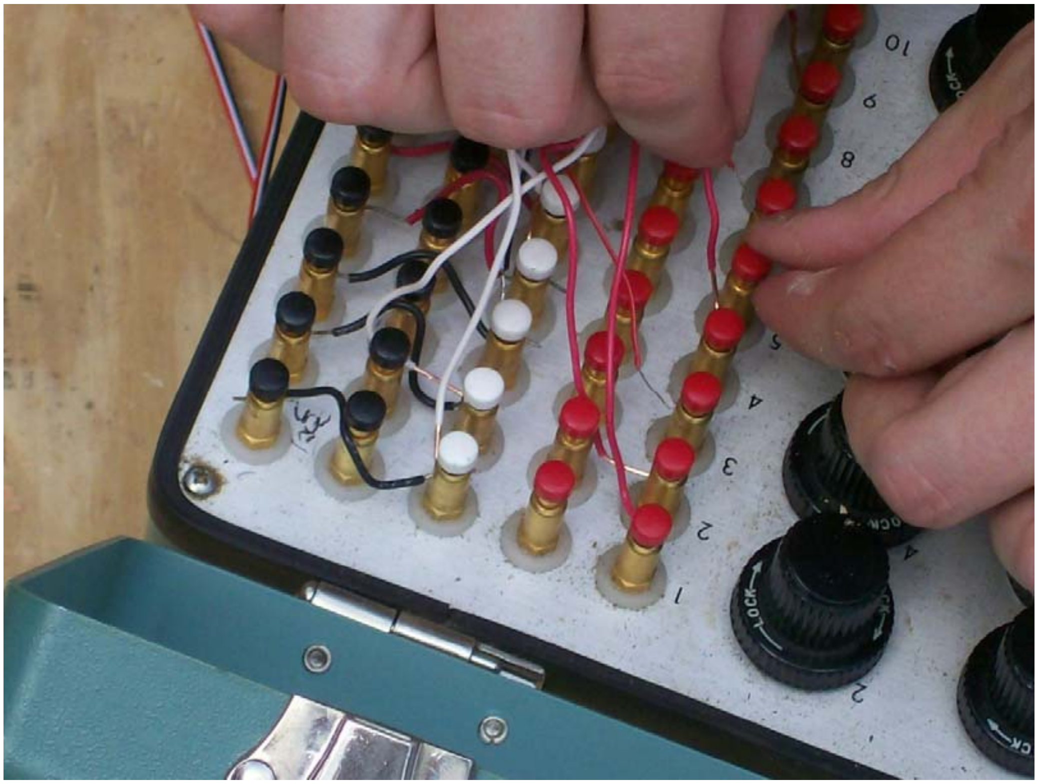


Figure 4-23 (a) Connecting gauges to the switch and balance unit

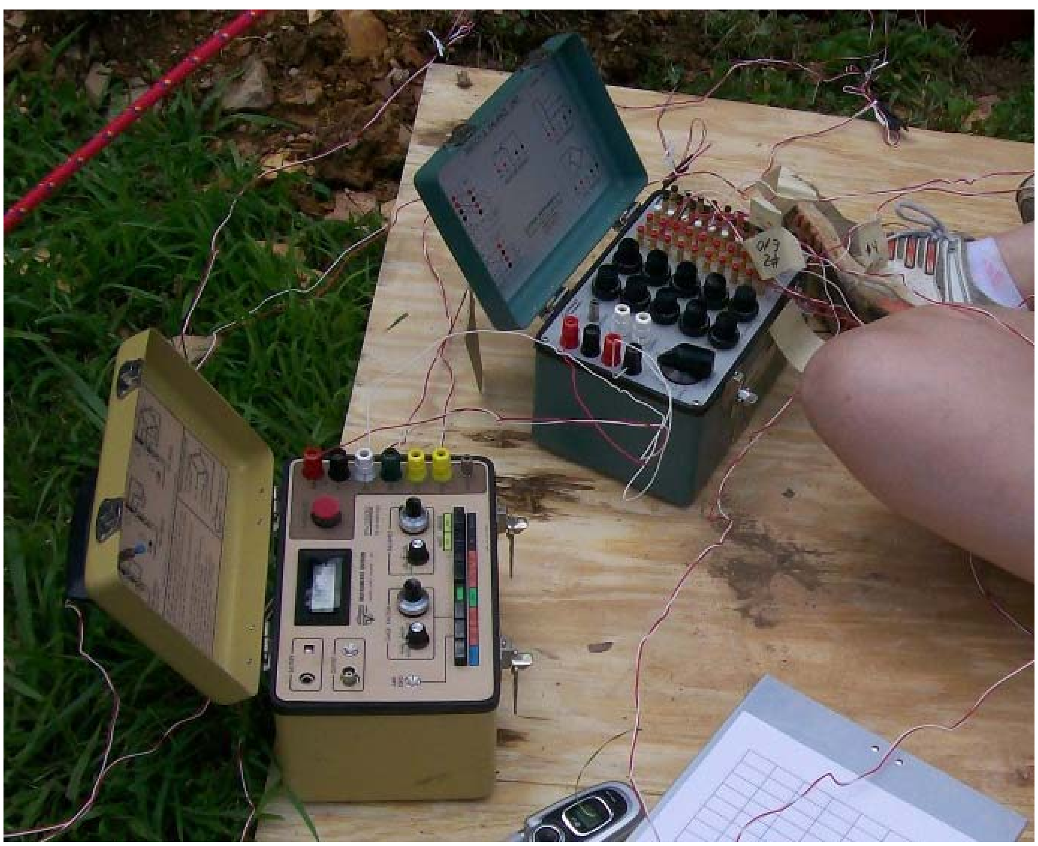


Figure 4-23 (b) Connecting switch and balance unit to the strain indicator

The Amp Zero was zeroed on the strain indicator and the gauge factor was set to 2.13. The MULT button was left black so reading were in με (microstrain). The indicator is shown reading a strain prior to being zeroed in Figure 4-24 (a). The BRIDGE button was also left black in order to read quarter bridge connections. Next the RUN key was depressed in order to read measurements. The knob on the bottom left of the switch and balance unit face could be turned to change the connection being read by the strain indicator. Each connection on the switch and balance unit was balanced to as close to zero as possible. The final setup on the switch and balance unit as seen during testing is shown in Figure 4-24 (b).

4-24 (a) Final setup of strain indicator and reading strain prior to being zeroed

Figure 4-24 (b) Final setup of switch and balance unit prior to first test

4.2.1.4 Test Procedure

During trench rescue shoring operations, struts are always installed from the top down. They can be no farther than 4 ft from the top edge of the trench and no farther than 4 ft from the bottom of the trench. In between they can be spaced no farther than 4 ft apart. These rules of shoring are taught in rescue shoring classes and can be found in trench rescue texts (Martinette 2006, Martinette 2008, Gargan 2006). The first shore, Strut #1, was lowered into place 2 ft below the edge of the trench and pressurized. A rescue technician toe-nailed the far strut foot to the non-instrumented panel strongback and assisted with placing the second shore, Strut #2. Strut #2 was pressurized and toe-nailed in place and the process was repeated for Strut #3. Figure 4-25 shows all the struts in place in the Frederick (granular) trench. Figure 4-26 shows all the struts in place in the York (clay) trench.

Figure 4-25 Frederick (granular) trench shoring setup

Figure 4-26 Strut placement in York (clay) trench

After taking the first set of readings, the struts were all unloaded and left in place. They were reloaded one at a time in order to record additional sets of measurements. This unloading and reloading process while leaving the system in place gave the panel an opportunity to settle and give consistent measurements as testing progressed.

Five sets of measurements were recorded from each gauge during a test. The first set was taken at the time of balancing because it was difficult to balance the gauges to a true zero. The second set was taken after loading Strut #1, the third after loading Strut

#2, fourth after loading Strut #3, and the fifth after unloading the system. A total of 14 test sequences were completed in the Frederick trench. A total of 12 test sequences were completed in the York trench.

4.2.2 York – Clay Soil

The second trench was in clay soil in York, PA. Figure 4-27 shows the trench location. The trench was stable for the duration of the test. The walls showed no signs of fissures or raveling. Figure 4-28 shows the uniform and stable nature of the trench wall.

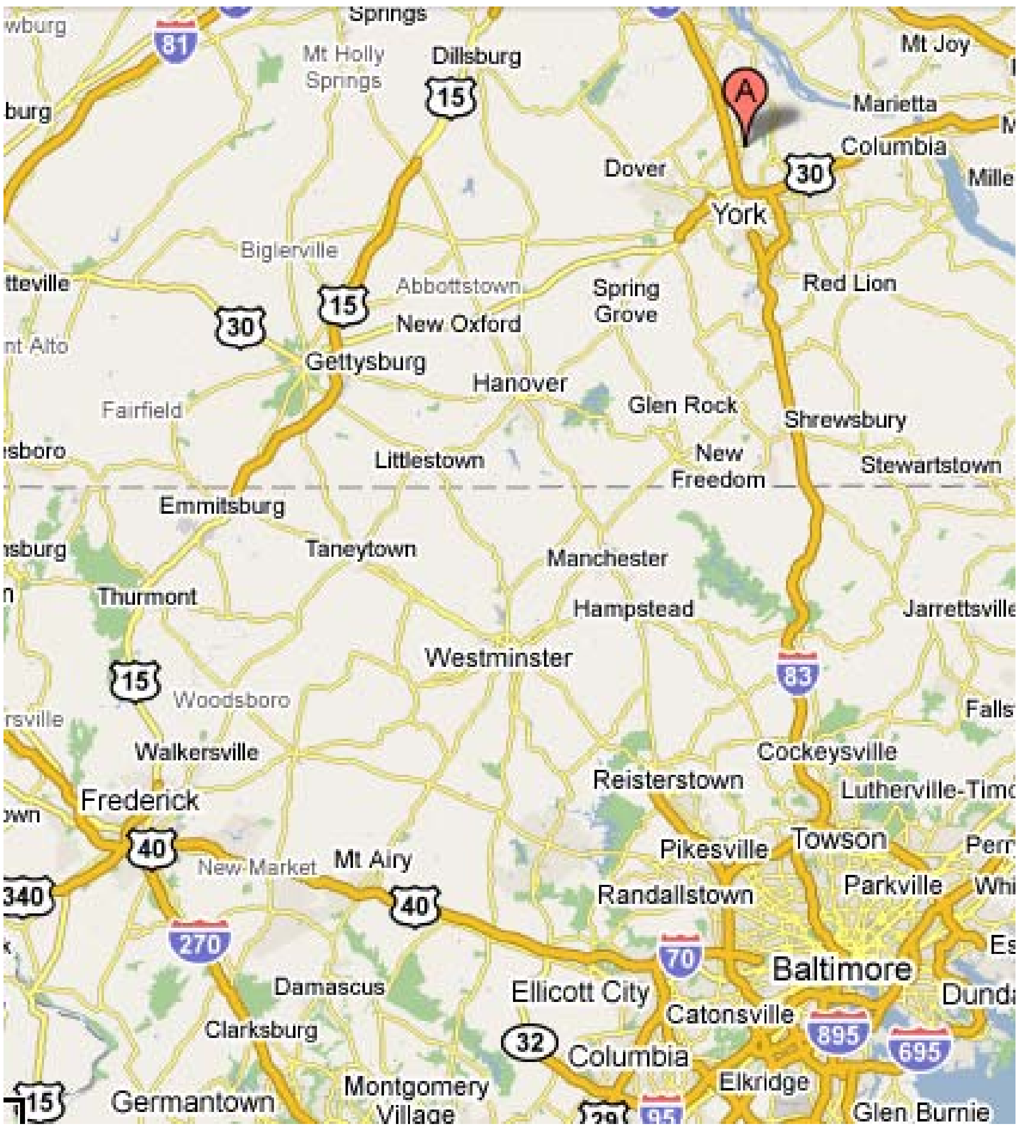


Figure 4-27 Google map of York trench location (A) in relation to Frederick, MD and Baltimore, MD (Google (b))

Figure 4-28 View into York trench: note clay appearance of soil and relatively uniform trench wall

The spoils pile was located 9.5 ft right of center and four feet back from the lip of the trench. The spoil pile was approximately 10 ft x 15 ft (150 sf), 3 ft high, and had a unit weight of 119 pcf. The total weight of the surcharge was 53,550 lbs, equivalent to a pressure of 360 psf. Figure 4-29 shows excavation in progress and Figure 4-30 shows the testing location in relation to the spoil pile.

Figure 4-29 Excavating the York trench


Figure 4-30 Spoil pile location shown in relation to the testing location

4.2.2.1 Soil Properties

Atterberg limit (Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soil ASTM D4318-05) and water content (Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass ASTM D2216) lab tests were performed on a soil sample from the trench to determine classification in order to estimate a Young's Modulus and Poisson's Ratio. The liquid limit and plasticity index determined by the testing indicated the soil is a borderline inorganic silt/inorganic with lower liquid limit (CL-ML). Appendix O shows test results and coefficient of plasticity index calculations. Additional test results for soil boring at the site done in April, 2007 by ECS LLC, Mid-Atlantic confirm the soil classification to be CL-ML (Appendix P). The standard penetration tests done at the time indicate the soil is medium stiff. The modulus of elasticity is assumed to be 30 Mpa (626,563 psf) and Poisson's ratio is assumed to be 0.3 (Budhu 2007).

4.2.2.2 Testing

Please see sections 4.2.1.2 through 4.2.1.4 for testing equipment, setup, and procedure.

4.3 Trench Test Results

The field tests measured strains on the exposed face of the panel in the X and Y directions at gauge locations 1 through 8. Gauges 4 through 1 are referred to as the left side of the panel while gauges 5 through 8 are referred to as the right side of the panel. Figure 4-31 illustrates the gauge locations. York testing results are summarized in Figure 4-32 and Figure 4-33**.** The complete results including data collection sheets are presented in Appendix Q.

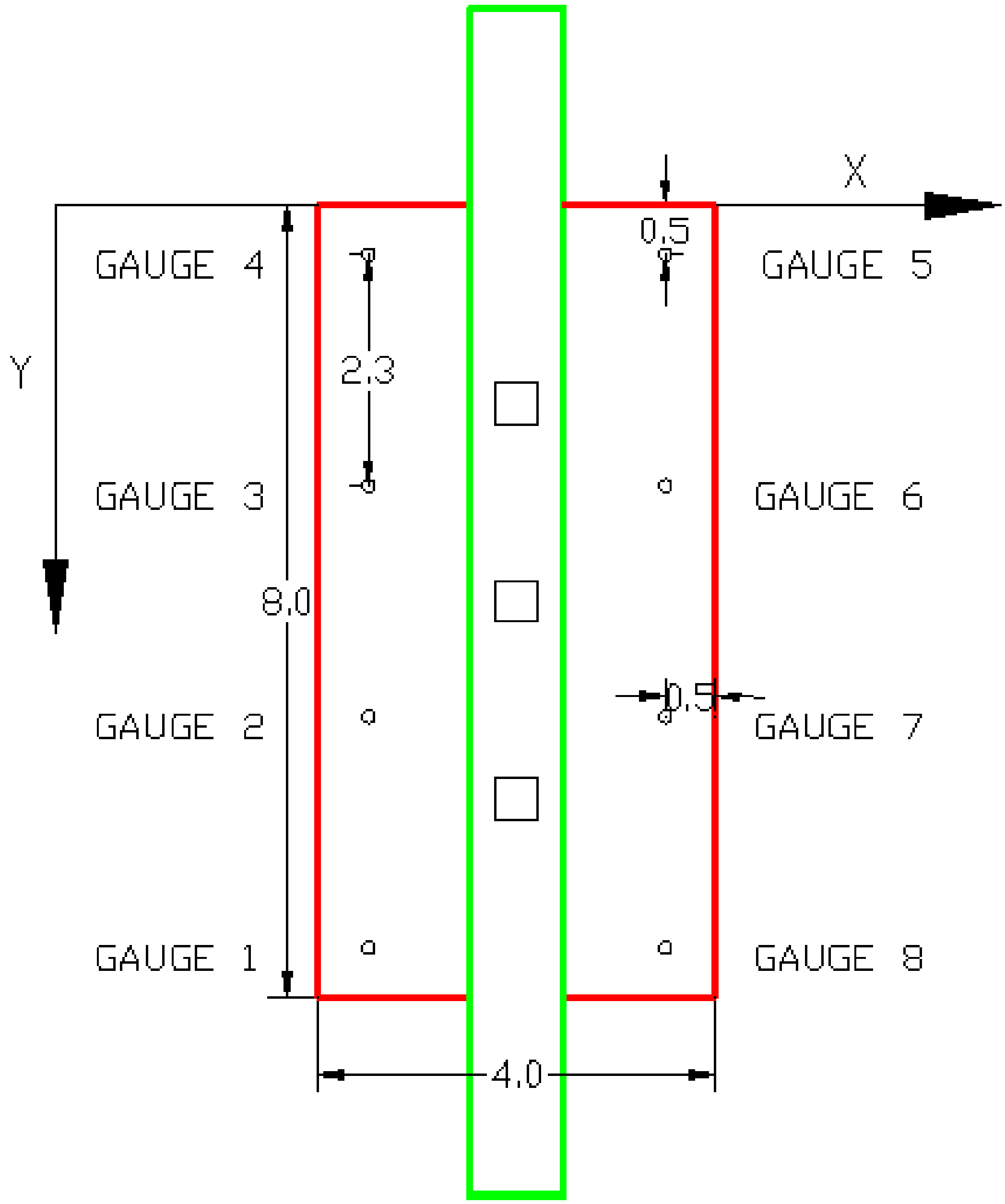


Figure 4-31 Gauge locations

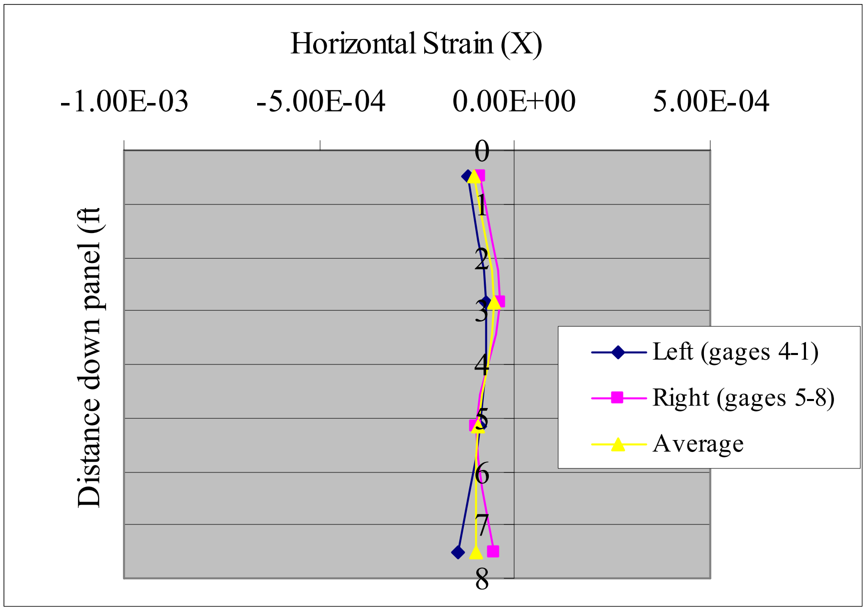


Figure 4-32 Strain in the X direction results from York (clay) trench testing

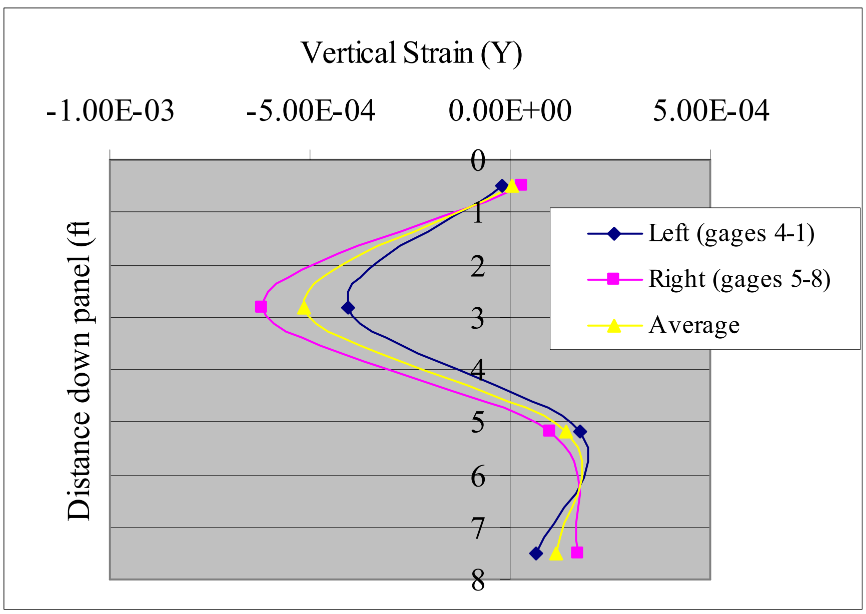


Figure 4-33 Strain in the Y direction results from York (clay) trench testing

The consistently small negative strains in the X direction indicate the exposed face of the panel was slightly in compression on both the right and left sides of the panel. The strains in the Y direction vary with trench depth. The negative values indicate that portion of the exposed face of the panel was in compression. The positive values indicate that portion of the exposed face of the panel was in tension. Figure 4-34 and Figure 4-35 illustrate the approximate shapes of the panel in the X and Y directions.

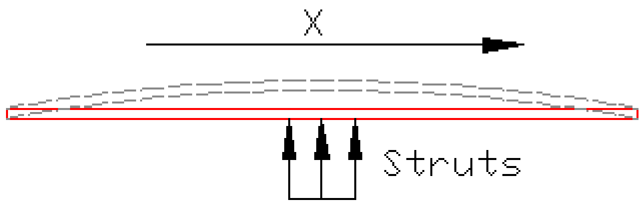


Figure 4-34 Plan view of panel: negative strain indicates exposed face of panel was in compression during testing in the York trench

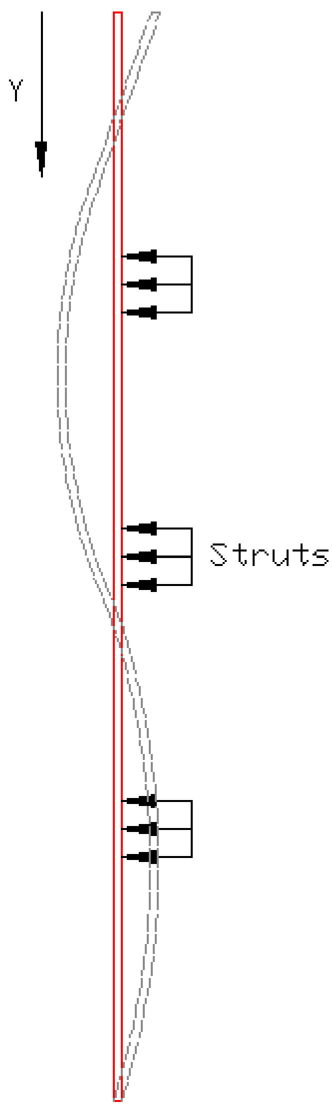


Figure 4-35 Section view of panel: positive and negative strain values indicate the panel shape varied in the Y direction during testing in the York trench

The Frederick (granular) trench test results are less consistent than the York (clay) trench test results. The inconsistent signs in both the X and Y directions indicate there was some irregularity in the trench wall. Frederick testing results are summarized in Figure 4-36 and Figure 4-37. The complete results including data collection sheets are presented in Appendix R.

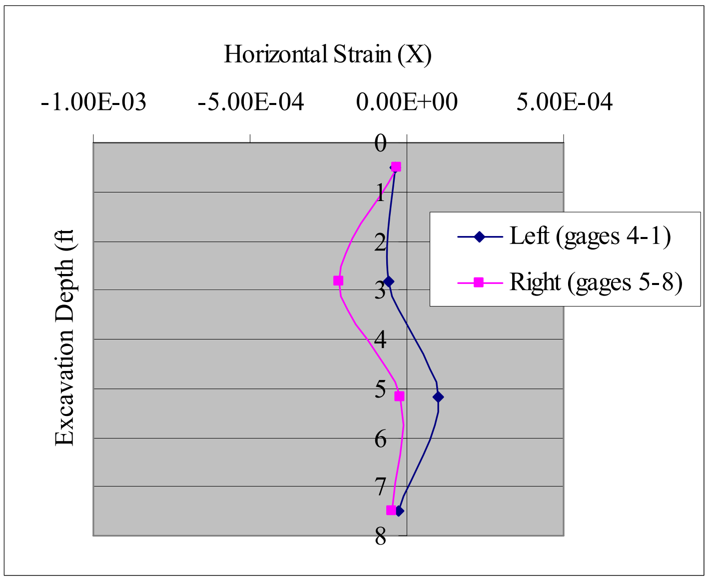


Figure 4-36 Strain in the X direction results from Frederick (granular) trench testing

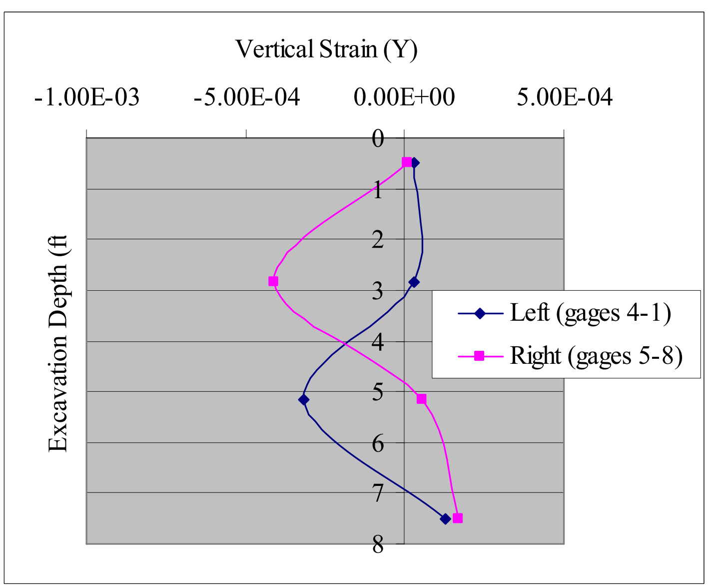


Figure 4-37 Strain in the Y direction results from Frederick (granular) trench testing

The difference in results between the two trenches is most pronounced in the vertical strains (Y direction). Figure 4-33 shows that the vertical strain results from the clay trench in York were consistent from the left side of the panel to the right side. As seen in Figure 4-37 the vertical strains from the granular soil are not consistent from the left side to the right side of the panel. Upon closer examination of the Frederick trench wall after testing, a small protrusion of rock and soil was located approximately 3 ft below the edge of the trench where the left side of the panel was located during testing. Figure 4-14 and Figure 4-15 exemplify the conditions found in the Frederick trench. The rocky conditions made it difficult to identify where irregularities would be significant enough to affect panel deformation. The shores were pressurized to the right of the protrusion where the trench wall was more uniform. The results from the right side of the panel are similar to the results from the clayey trench in York, implying that panel deformation is a factor of trench wall smoothness rather than soil type.

4.4 Summary

Twenty six tests were performed over two days in two trenches. One day was spent in York, PA to evaluate the effect of clay soil on the shoring system behavior. The second day was spent in Frederick, MD evaluating the effect of a granular soil on the shoring system behavior under similar loading conditions. The homogenous condition at the York site produced similar results from both sides of the panel. Thus, an average of results from both sides could be used as the final results of the testing. At the Frederick site, the results from the left side of the panel were not consistent with the results from the right side of the panel because of irregularities in the trench wall behind the left side of the panel. Therefore, the data from the left side of the panel was not used to determine the final results. The data from the right side of the panel alone was used to determine the final results of the field testing.

Chapter 5 Discussion and Analysis

This chapter examines the results from the parametric studies in Chapter 3 and the field validation described in Chapter 4. The results are discussed and analyzed in context with current technical rescue industry practice and current engineering practice in both shallow and deep excavation shoring system design.

5.1 Parametric Studies

This research used parametric studies to determine the variation of the earth pressure produced using the current standardized trench rescue shoring system. The studies examined variations in strut load, panel stiffness, panel thickness, panel width, and the surcharge load, size and configuration. The numerical results were presented in Chapter 3.

5.1.1 Effects of Varying Strut Load

The parametric studies indicate that strut loading has the most significant effect on the earth pressure as related to trench rescue shoring. This section presents the earth pressure determined as a result of varying strut load and how it compares to Rankine's active and passive pressures, at-rest earth pressure, Peck's earth pressure design envelope for sand and gravel in deep excavations, and Yokel's earth pressure design envelopes for shallow excavations. This section also presents an alternative method for determining the earth pressure that can then be used for shoring system design in shallow excavations as well as determining the maximum allowable strut load that can be used in a shoring operation.

5.1.1.1 Calculated Earth Pressure vs. Rankine

Figures 5-1 (a), (b), and (c) show the earth pressure determined for strut loads of 1298 lbs/strut, 2356 lbs/strut and 4712 lbs/strut versus Rankine's active and passive earth pressure and the at-rest earth pressure. As strut loading increases from 1298 lbs to 2356 lbs, the calculated earth pressure approaches Rankine's passive pressure as shown in Figure 5-1 (b). As shown in Figure 5-1 (c), when the strut loading is increased to 4712 lbs, the calculated earth pressure exceeds Rankine's passive pressure. This creates a potential passive failure in the top 2.5 ft of the excavation. Karlsrud and Anderson (2005) also found that the at-rest and Rankine's active pressures consistently underestimate earth pressure in the upper portions of braced excavations.

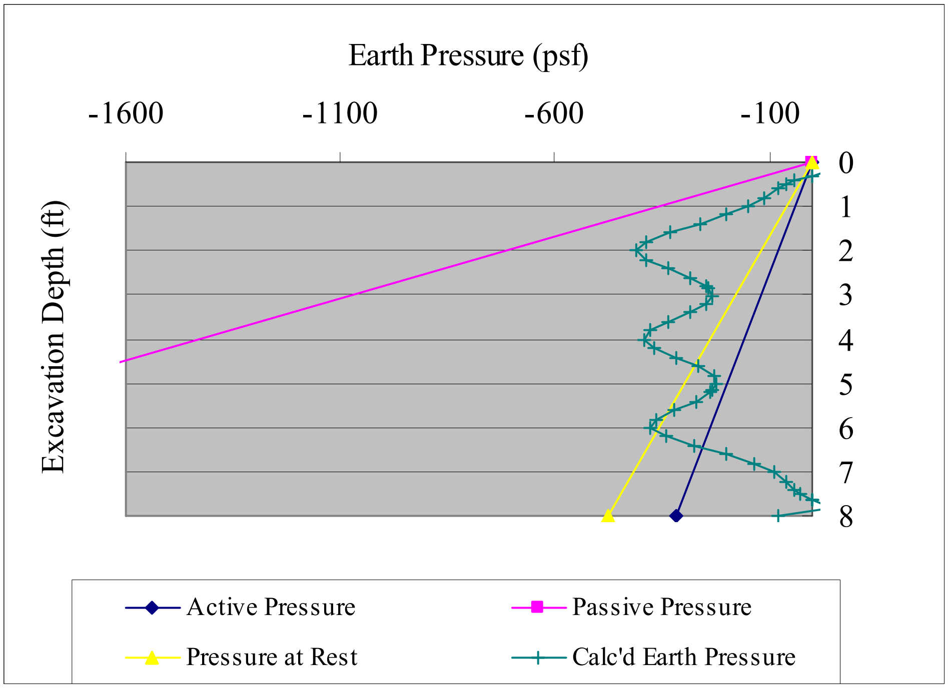


Figure 5-1 (a) Calculated earth pressure vs. Rankine pressures for 1298 lbs/strut

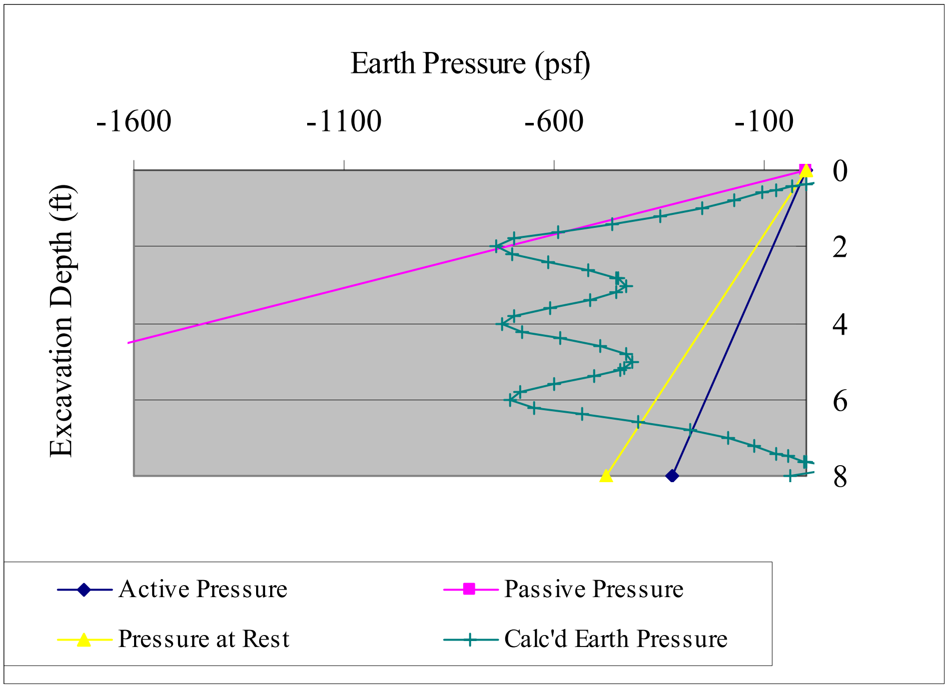


Figure 5-1 (b) Calculated earth pressure vs. Rankine pressures for 2356 lbs/strut

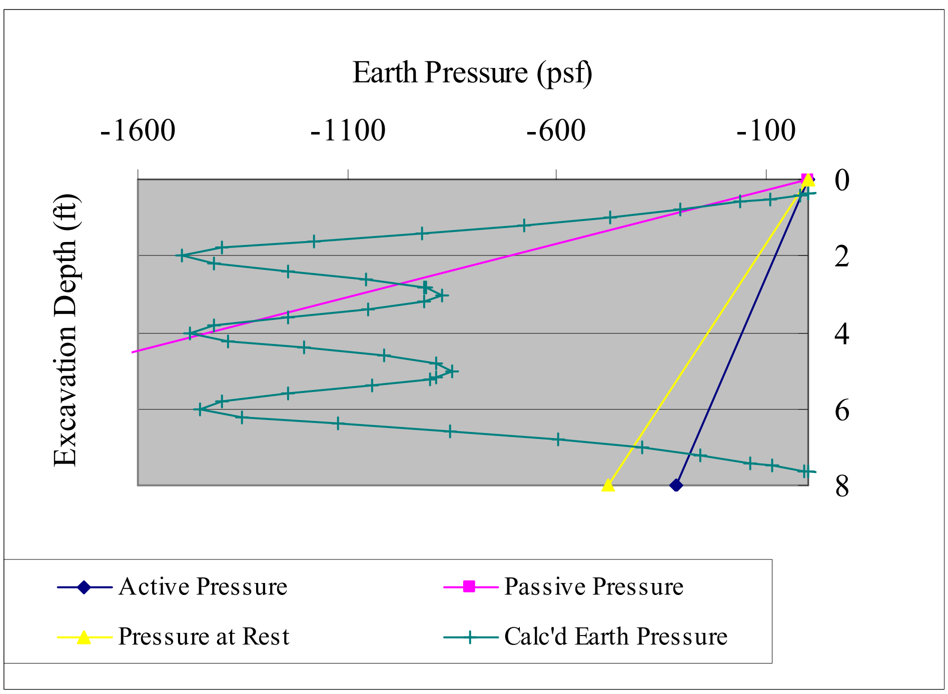


Figure 5-1 (c) Calculated earth pressure vs. Rankine pressures for 4712 lbs/strut

5.1.1.2 Calculated Earth Pressure vs. Peck's Apparent Earth Pressure Diagrams

Terzaghi and Peck (1967) presented apparent earth pressure diagrams as discussed in section 1.4. Figure 5-2 shows the diagrams for a granular soil with a unit weight of 119 pcf and an active earth pressure coefficient of 0.33, soft to medium clay with a unit weight of 119 psf and an undrained shear strength of 175 psf, and stiff clay with a unit weight of 119 pcf and the maximum earth pressure given by $0.3\gamma H$. Figures 5-3 (a), (b), and (c) plot Peck's apparent earth pressure diagram for a sand or gravel soil with unit weight $\gamma = 119$ pcf and active earth pressure coefficient $K_a = 0.33$ against the finite element model results for 1298 lb, 2356 lb and 4712 lb strut loads respectively.

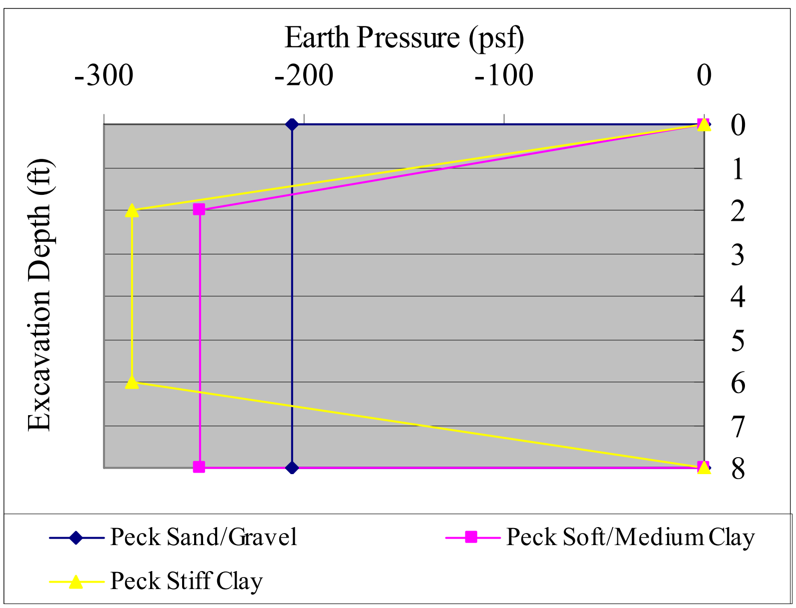


Figure 5-2 Peck's apparent earth pressure diagrams

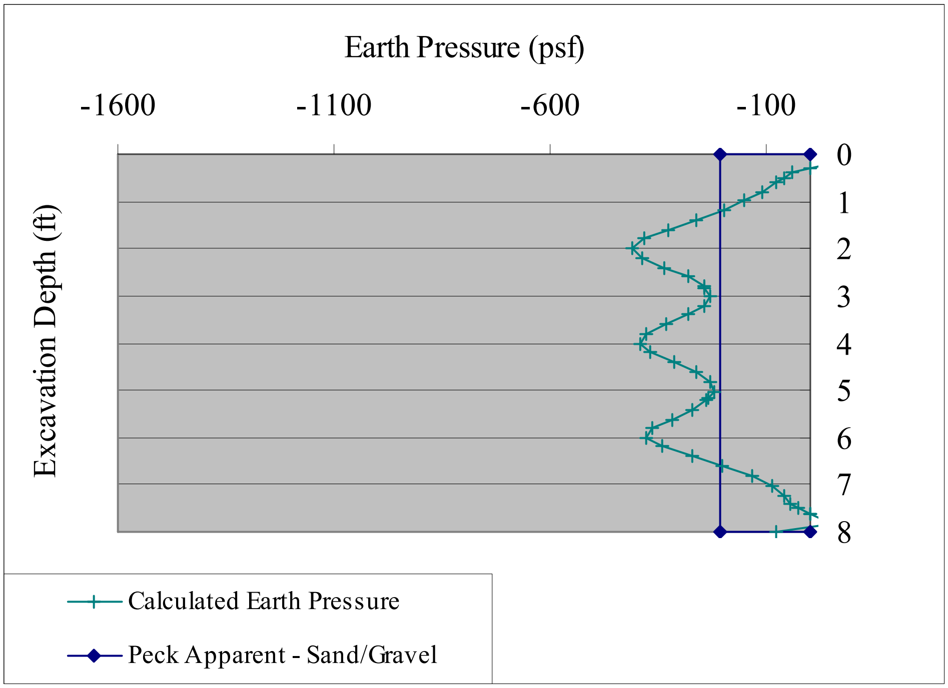


Figure 5-3 (a) Peck's apparent earth pressure diagram for a sand or gravel soil plotted against finite element model calculated earth pressures due to strut loads of 1298 lbs/strut

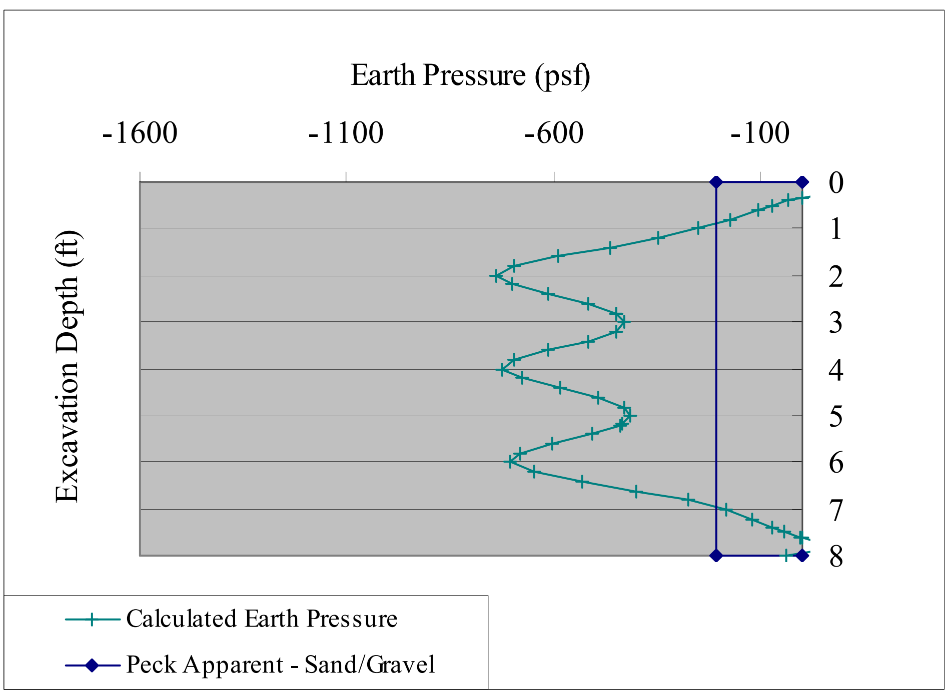


Figure 5-3 (b) Peck's apparent earth pressure diagram for a sand or gravel soil plotted against finite element model calculated earth pressures due to strut loads of 2356 lbs/strut

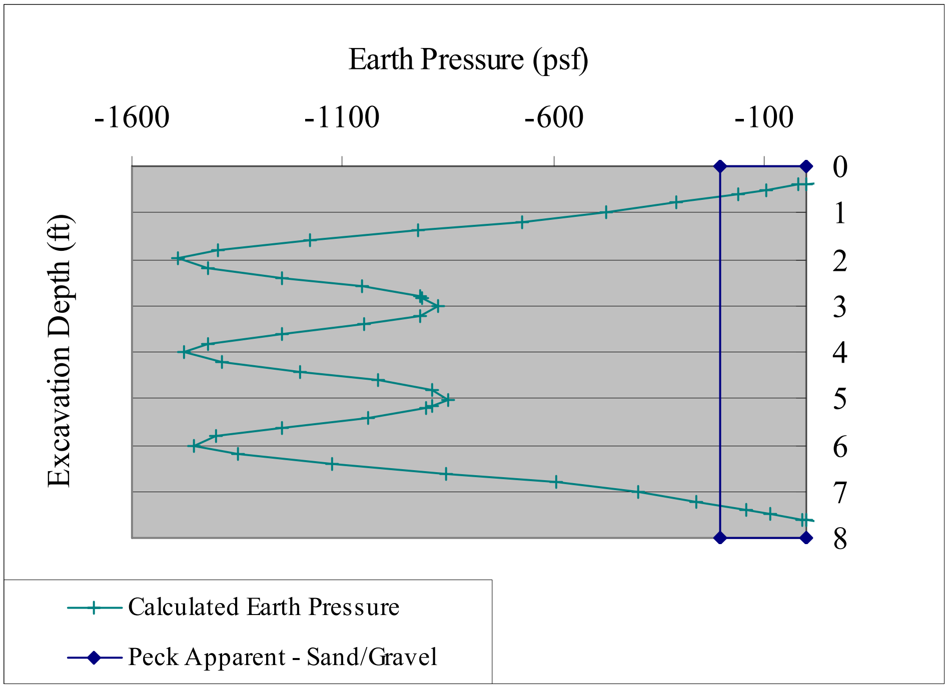


Figure 5-3 (c) Peck's apparent earth pressure diagram for a sand or gravel soil plotted against finite element model calculated earth pressures due to strut loads of 4712 lbs/strut

Peck's apparent earth pressure diagram is a uniform earth pressure that is theoretically "not intended to represent the real distribution of earth pressure at any vertical section in a cut, but instead constituted hypothetical pressures from which there could be calculated strut loads that might be approached but would not be exceeded in the actual cut" (Peck 1969). Bowles added that if "simply supported beams are used for sheeting, the strut force will produce not more than the contributory area of that part of [Peck's] apparent pressure diagram" and that earth pressure is "directly related to the strut forces" (Bowles 1969). As shown in Figures 5-3 (a), (b), and (c), Peck's pressure distribution is close to the calculated pressure for a strut force of 1298 lbs/strut, but is much smaller than the calculated earth pressure determined for the 4712 lbs/strut loading.

5.1.1.3 Calculated Earth Pressure vs Yokel's Shallow Excavation Earth Pressure Envelopes

Figures 5-4 (a), (b), and (c) show the earth pressure determined for varying strut loads vs. Yokel's minimum and maximum earth pressure envelopes (soil type A and soil type C respectively).

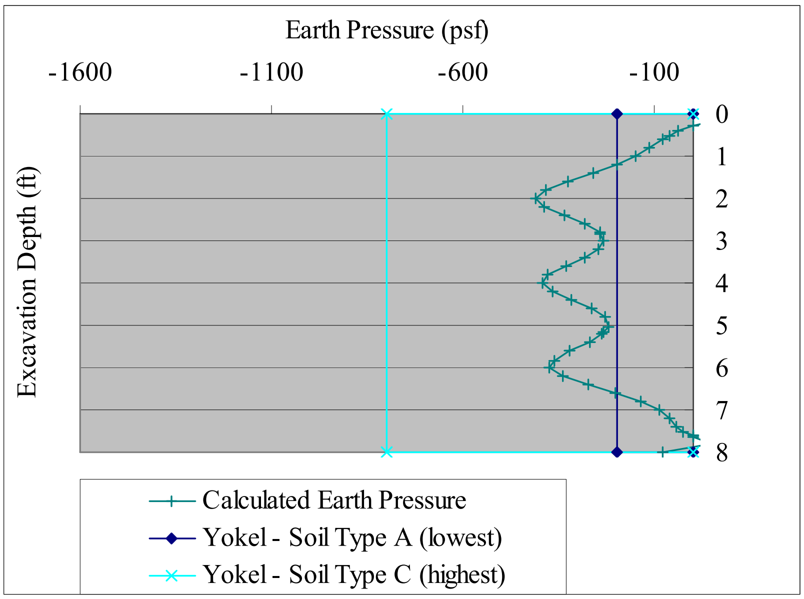


Figure 5-4 (a) Yokel maximum and minimum earth pressure envelopes vs. earth pressure determined for 1298 lbs/strut

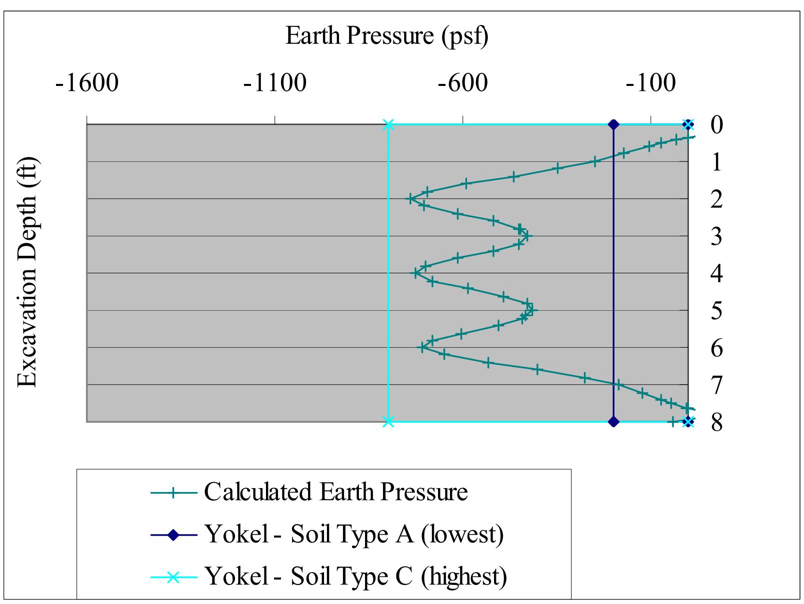


Figure 5-4 (b) Yokel maximum and minimum earth pressure envelopes vs. earth pressure determined for 2356 lbs/strut

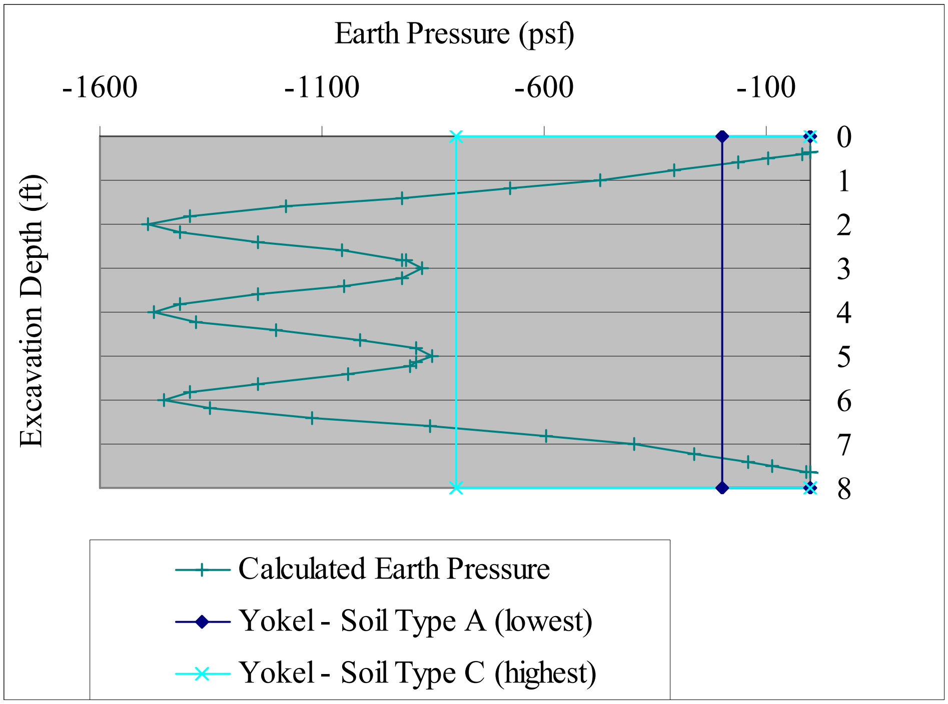


Figure 5-4 (c) Yokel maximum and minimum earth pressure envelopes vs. earth pressure determined for 4712 lb/strut

Figures 5-4 (a), (b), and (c) show that Yokel's shallow excavation earth pressure diagrams provide a wider range of possible earth pressures related to soil types that will conservatively estimate earth pressure when strut loading is approximately 2400 lbs or less. The range provided by Yokel does not, however, account for high strut loads as evidenced by Figure 5-4 (c). However, as previously stated, when strut loads are 4712 lbs/strut, the earth pressure exceeds Rankine's passive earth pressure in the upper 2.5 ft of the excavation. Once soil is loaded beyond the range of Rankine's passive earth pressure, it can be reasonably expected that the trench wall will fail.

5.1.1.4 Proposed Earth Pressure

A uniform earth pressure envelope can be calculated by distributing the known strut loads over the entire shoring area, i.e. 4 ft x 8 ft in the subject shoring system. For three 1298 lbs strut loads, the earth pressure would be 121.7 psf. For three 2356 lbs strut loads, the earth pressure would be 220.9 psf. For three 4712 lbs strut loads, the earth pressure would be 441.8 psf. Figures 5-5 (a), (b), and (c) show this uniform earth pressure envelope versus the earth pressure determined by the finite element model.

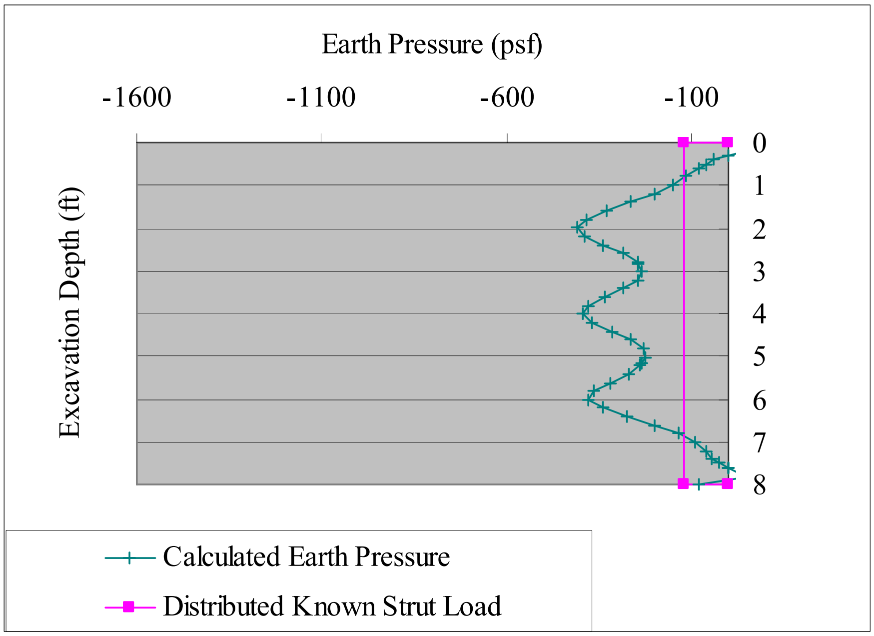


Figure 5-5 (a) Finite element model earth pressure vs. the earth pressure due to the known strut loads of 1298 lbs/strut distributed over the shoring area

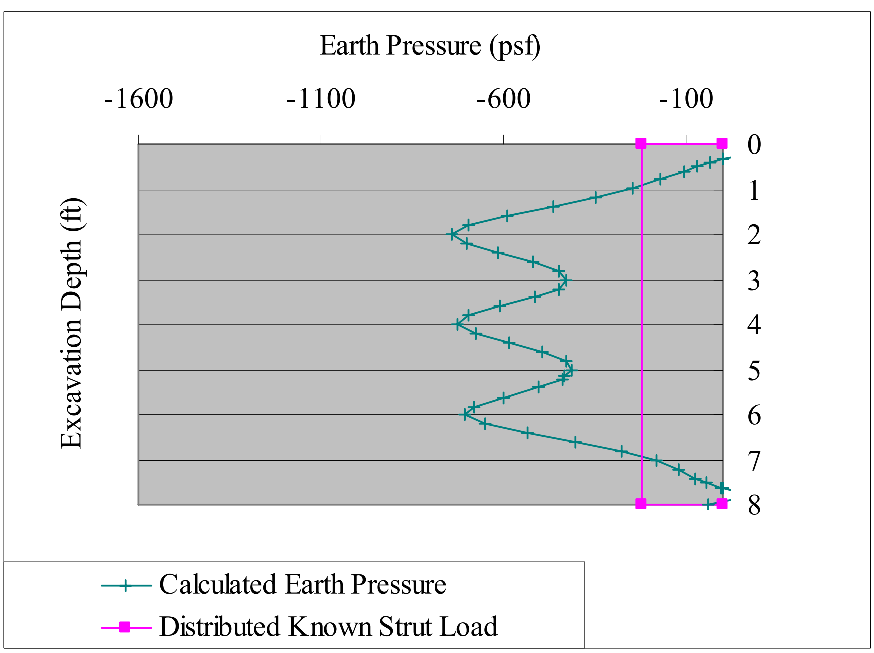


Figure 5-5 (b) Finite element model earth pressure vs. the earth pressure due to the known strut loads of 2356 lbs/strut distributed over the shoring area

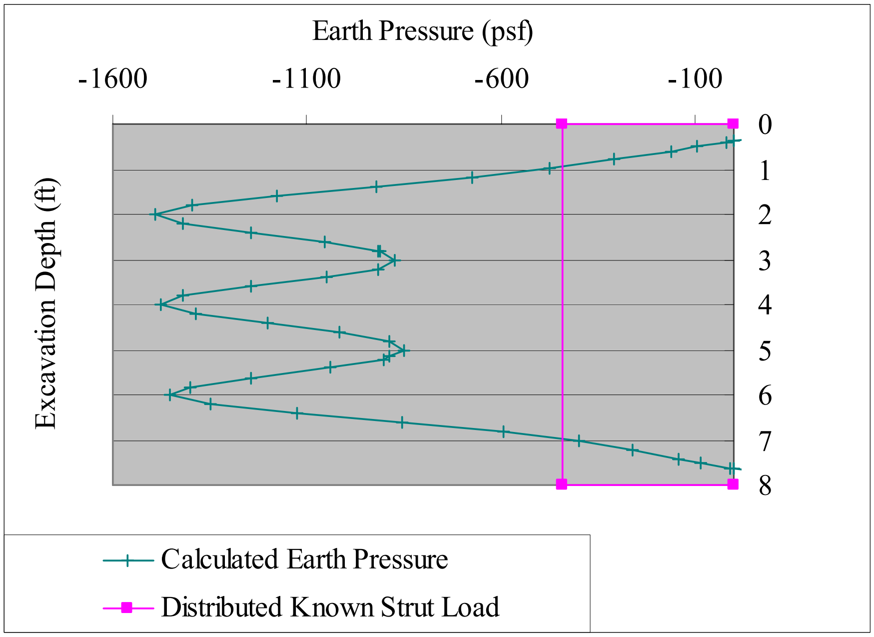


Figure 5-5 (c) Finite element model earth pressure vs. the earth pressure due to the known strut loads of 4712 lbs/strut distributed over the shoring area

The next step is to calculate a uniform earth pressure envelope while considering the fact that strut loads actually produce a bell shaped pressure distribution. An equivalent uniform pressure can be determined by finding the area under the finite element model pressure distribution curves and calculating an effective area. For all three strut loads examined in this research, 1298 lbs/strut, 2356 lbs/strut and 4712 lbs/strut, the effective panel width was found to be 1.92 ft and the effective height of the panel was determined to be 4.67 ft. Figures 5-6 (a) and (b) show an example of the effective area calculations by plotting the results from the finite element model against the effective width and height of the panel for a strut load of 2356 lbs/strut. Appendix S shows the tabulated data for 1298 lbs/strut, 2356 lbs/strut and 4712 lbs/strut.

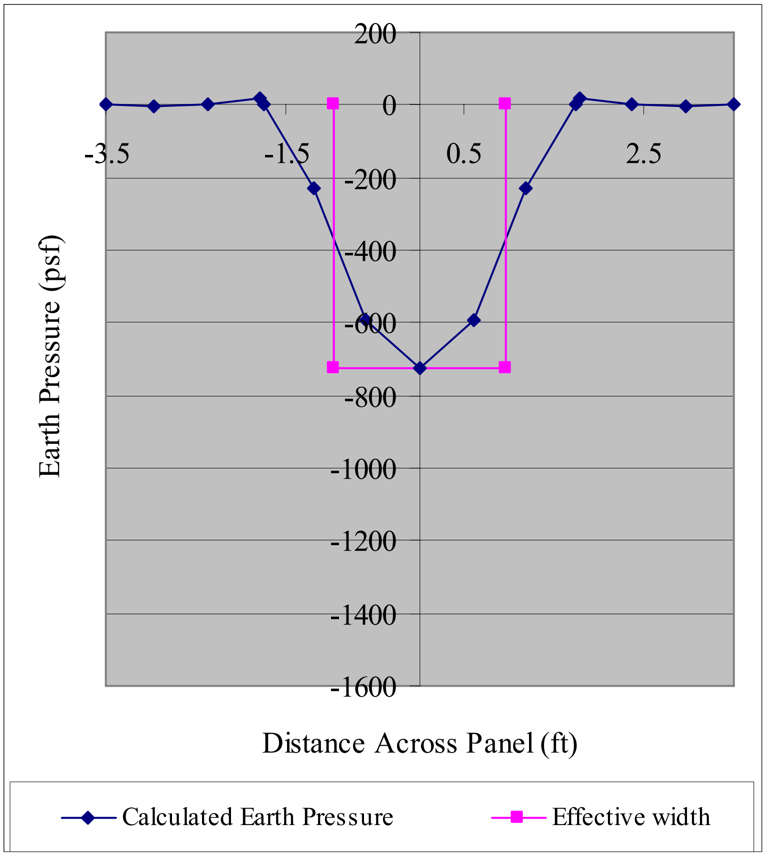


Figure 5-6 (a) Finite element calculated earth pressure plotted against effective width of the panel for a strut load of 2356 lbs/strut

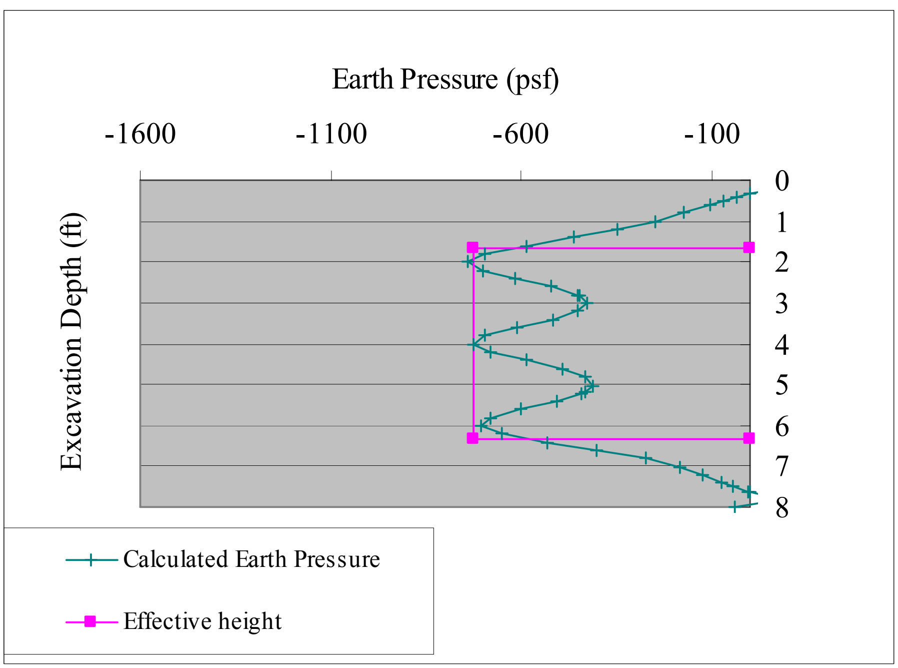


Figure 5-6 (b) Finite element calculated earth pressure plotted against effective height of the panel for a strut load of 2356 lbs/strut

Finding the total force from the area of the actual pressure distribution determined by the finite element model and redistributing it as a uniform earth pressure over the total area of the panel (4 ft x 8 ft) gives a uniform earth pressure over the entire panel area. Using the area under the curves for a strut loading of 1298 lbs/strut, the uniform earth pressure is 110.5 psf. For the area under the curves when strut loading is 2356 lbs/strut, the uniform earth pressure is 203.2 psf. For the area under the curves when strut loading is 4712 lbs/strut, the uniform earth pressure is 414.1 psf. Figures 5-7 (a), (b), and (c) show the redistributed earth pressure plotted against the earth pressure results obtained from the finite element models.

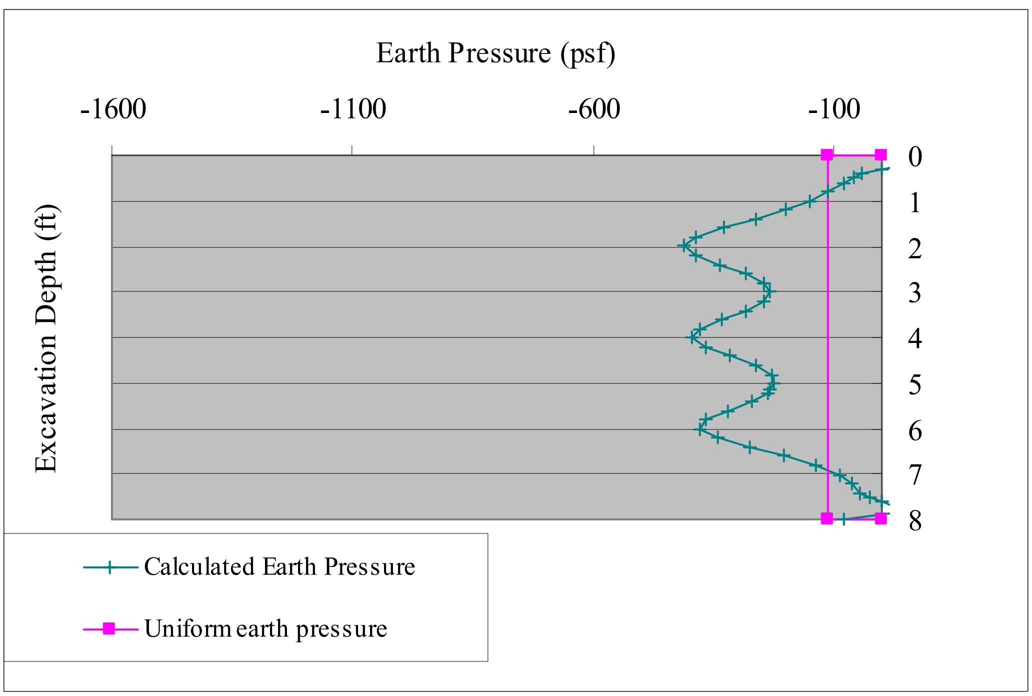


Figure 5-7 (a) Redistributed force as a uniform earth pressure over the total area of the panel (4 ft x 8 ft) for strut loads of 1298 lbs

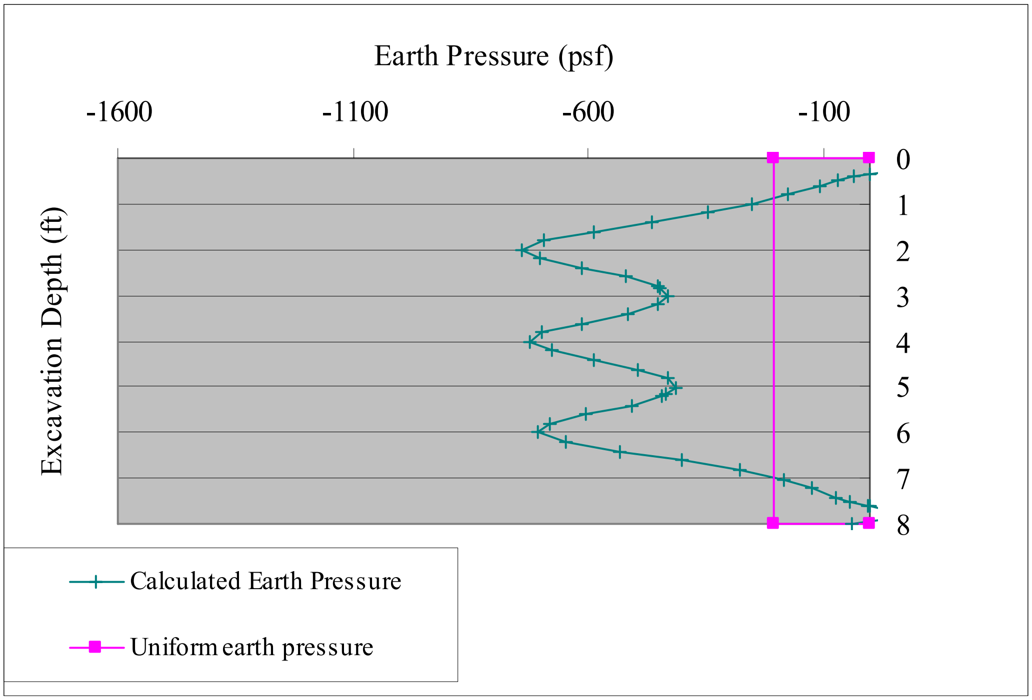


Figure 5-7 (b) Redistributed force as a uniform earth pressure over the total area of the panel (4 ft x 8 ft) for strut loads of 2356 lbs

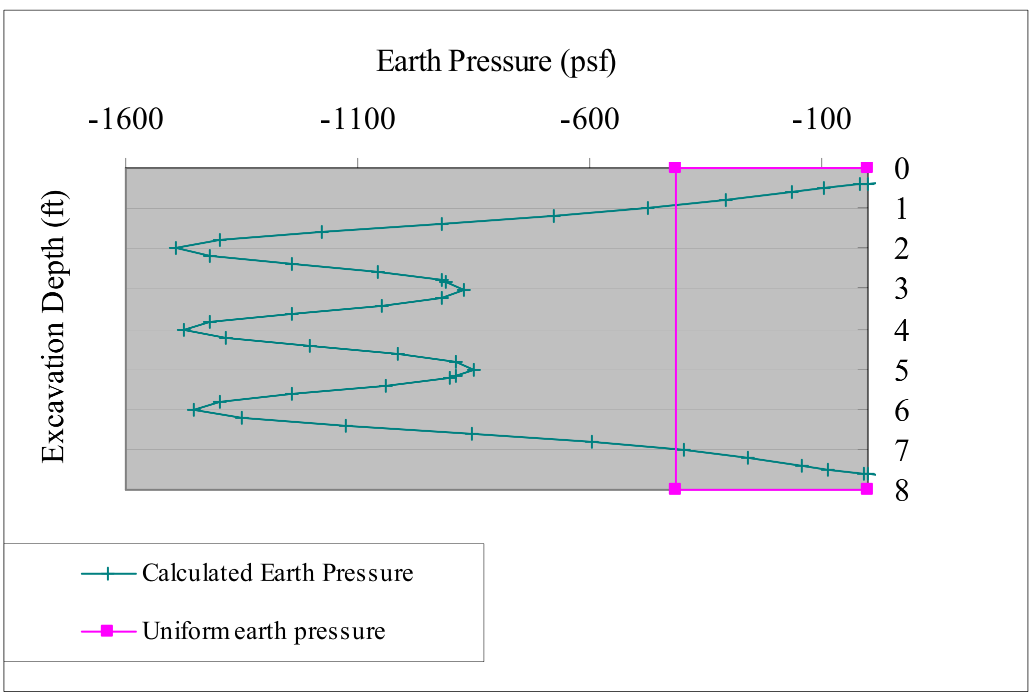


Figure 5-7 (c) Redistributed force as a uniform earth pressure over the total area of the panel (4 ft x 8 ft) for strut loads of 4712 lbs

In comparison, for the 1298 lbs/strut loading, the uniform earth pressure calculated by simply distributing the known strut loads over the 4 ft x 8 ft panel is 9% larger than the redistributed force from the finite element model results (121.7 psf compared to 110.5 psf). For the 2356 lbs/strut loading, the uniform earth pressure calculated by distributing the known strut loads over the 4 ft x 8 ft panel is 8% larger than the redistributed force from the finite element model results. For the 4717 lbs/strut loading, the uniform earth pressure calculated by distributing the known strut loads over the 4 ft x 8 ft panel is 6% larger than the redistributed force from the finite element model results. Since the uniform earth pressures calculated by distributing the known strut loads over the 4 ft x 8 ft panel are close in value, but consistently larger than the redistributed force from the finite element model results, it is reasonable to use the uniform earth pressure calculated by simply distributing the known strut loads over the total 4 ft x 8 ft panel area as the recommended earth pressure envelope that could be used in practice.

The finite element analysis results were examined for the models using the three different strut loads where all other variables were held constant and consistent with Soil 2 and manufacturer provided material properties for the shoring system components. It was determined from the finite element results that multiplying the uniform earth pressure found from distributing the known strut loads over the panel by a factor of approximately 3.3 produced the maximum earth pressure ordinate determined by the finite element model earth pressure distribution. The factor was found to be the same for all three strut loads examined. That factor is termed the “limit factor” in this study. Due to the elastic nature of the model, it is reasonable to assume that the limit factor will change as unit weight, Young's modulus, and Poisson's ratio of the soil and shoring components vary.

To determine the parameters that affect the limit factor a study was undertaken to determine the effects of differing material properties. When a soil with an increased density and Young's Modulus such as Soil 1 was examined while maintaining constant strut pressure at 1298 lbs/strut, the limit factor was found to be 4.5. The factor was also calculated for varying panel properties while maintaining constant strut pressure of 1298 lbs/strut and soil properties equivalent to Soil 2. The factor decreased from 4.3 to 2.9 as panel stiffness increased from 7.31 x 10^7 psf to 5.86 x 10^8 psf. The factor reduced from 4.7 to 2.6 as panel thickness increased from 0.25 in to 1.25 in. It can be assumed that changes in the strongback properties would also result in different factors. The limit factors determined in this study are summarized in Tables 5-1, 5-2, 5-3, and 5-4 according to the different parameters. The factor was found to vary from 2.9 to 4.7. An average limit factor of 3.8 can be used to determine the maximum earth pressure ordinate in a linear finite element solution. The tabulated data for each factor can be found in Appendix T.

Table 5-1 Limit factor related to varying strut load

Variable		**Limit**
Strut Load		**Factor**
1298	lbs/strut	3.2
2356	lbs/strut	3.3
4712	lbs/strut	3.3

Table 5-2 Limit factor related to varying panel stiffness

Variable		Limit
Panel Stiffness		Factor
7.31E+07	psf	4.3
1.46E+08	psf	3.7
2.92E+08	psf	3.2
1.38E+08	psf	3.0
5.85E+08	psf	2.9

Table 5-3 Limit factor related to varying panel thickness

Variable		Limit
Panel Thickness		Factor
0.25	in	4.7
0.50	in	3.8
0.75	in	3.2
1.00	in	2.9
1.25	in	2.6

Table 5-4 Limit factor related to varying soil properties

Variable				
Soil				Limit
Unit Weight		Young's Modulus		Factor
119	pcf	6.27E+05	psf	3.2
124	pcf	1.67E+06	psf	4.5

The limit factor can also be used to find the maximum recommended strut load for a shoring system. This can be determined from the uniform pressure calculated from the known strut loads, the limit factor described above, and the Rankine passive earth pressure. As previously indicated excessive strut loading can fail soil in a passive mode. For example, the Rankine passive earth pressure at y = 2 ft, the depth of the highest strut, was calculated to be 714 psf in the parametric studies that used Soil 2 and manufacturer supplied material properties for the shoring components. Dividing by the average limit factor of 3.8 as determined above, the maximum recommended uniform earth pressure is found to be 188 psf. The total strut loading is equal to 188 psf multiplied by the area of the shoring (8 ft x 4 ft), or 6012 lbs. The maximum allowable individual strut load before theoretically failing the soil in a passive mode is therefore 6012 lbs / 3 struts, or 2004 lbs/strut. That load corresponds to a strut with an internal pressure of 638 psi on a 2 in diameter internal piston and 308 psi on a 2.875 in diameter piston head.

The results of this method indicate that once the limit factor is determined based on the soil and shoring material properties, it can be multiplied by the uniform earth pressure calculated by distributing the known strut loads over the entire panel area. Comparing the result to Rankine's passive earth pressure at the depth of the first strut will indicate if the strut load should be reduced or can be increased.

5.1.2 Effect of the Panel Configuration

OSHA 1926 Subpart P Appendix D (g)(7) states that "0.75 in thick, 14 ply arctic white birch… plywood is not intended as a structural member, but only for preventing local raveling (sloughing of the trench face) between shores" (OSHA 1989). This research indicates that the plywood panel is structurally significant. At a depth of 4 ft the 0.75 in thick panel reduces earth pressure by 32% when added between the trench face and strongback. This research also indicates that increasing panel width beyond 2 ft does not improve structural performance, but in keeping with the OSHA statement about protecting the trench faces from raveling, the wider panel offers protection to rescuers and victims and stability to the trench wall preventing soil from failing that may be assisting in load transfer.

This research also indicates that increasing panel stiffness and thickness significantly reduce earth pressure. While a FinnForm panel with a higher stiffness is not currently commercially available, panels up to 1.25 inches thick are available from the manufacturer. Increasing the panel thickness from 0.75 in (currently recommended by OSHA) to 1.25 in reduces earth pressure at a depth of 4 ft by 18% from 394 psf to 321 psf. Increasing the panel stiffness in this study reduces earth pressure whereas increasing bracing stiffness in deep foundations increases earth pressure.

The tradeoff for rescuers is manageability. A 0.75 in thick panel weighs approximately 90 lbs. A 1.25 in thick panel weighs approximately 150 lbs. Rescuers must weigh the benefits of lower load transfer to the soil against increased manpower required to handle heavier panels. A 0.75 in thick panel may be adequate in a stable, cohesive soil while using lower strut loading in the range of 1300 lbs/strut or less. As strut loading increases to the range of 2356 lbs to 4712 lbs, the thicker panel may be a safer option.

5.1.3 Effect of Surcharge

The study of the effects of the surcharge found that the earth pressure was minimally affected by varying the configuration of a reasonably sized spoil pile from a shallow excavation. Earth pressure increased no more than 5% regardless of surcharge location when a 2.5 ft (300 psf) high spoil pile with a unit weight of 119 lb/ft^3 was examined. Earth pressure increased a maximum of 11% and 17% when spoil piles 5 ft (600 psf) and 7.6 ft (900 psf) high respectively with the same unit weight were examined. The effects of surcharge resulting from other sources such as excavation equipment were not examined and have the potential to increase the surface pressure markedly. Rescuers should take note of actual surcharge size during initial reconnaissance. As a matter of good safety practice, spoil piles and nearby equipment should be moved as far as possible from the trench to prevent spoil pile slides and failures of low cohesion soils at the trench edge. Effects of mechanical vibration were not examined in this study and also have the potential to detrimentally affect the shoring system.

5.2 Field Validation

An experimental program was undertaken to study the effects of different soil types on the response of a typical trench rescue shoring system. It is widely accepted in current engineering practice that the soil type has a strong influence on the earth pressure in deep excavations. However, in shallow trench excavations such arguments might not hold true. Soil near the surface has a layer of active plant life with organic materials and root structures that break up the soil structure (Yokel 1979). Oxidation, leaching, and volume change due to wetting and drying may act to reduce or eliminate the soil cohesion; hence, surface soils may act generally as granular soils. To verify Yokel's argument, testing was performed in two full scale trenches. One was in Frederick, MD where the soil was mostly granular in nature and the second was in York, PA where the soil was clayey in nature. This section discusses the results of the field testing presented in Chapter 4.

5.2.1 Field Testing Results

The Frederick and York trench vertical strain results are shown in Figure 5-8. The figure shows that the panel deformation was similar in both trenches. Because the two trenches were of different soil types, the results of the field testing indicate that the pressure distribution on the shoring system for both of the trenches is the same, i.e. trench rescue shoring performance for a given strut load is not dependent on soil type.

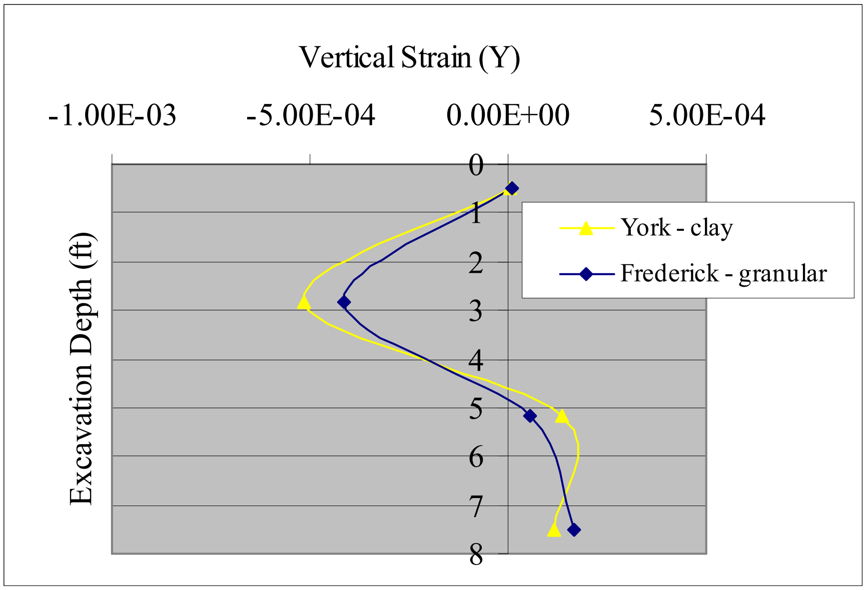


Figure 5-8 Comparison of vertical strain results from the York (clay) trench vs. the Frederick (granular) trench

The results of the field work imply that Yokel's argument is correctly applicable to shallow excavations regarding the type of the upper soil layers in addition to the argument that "the [pressure envelopes presented by Peck in 1969] were developed on the basis of measured data that originated from deep excavations (deeper than 20 ft). Because of the time element usually associated with such excavations, the data are from excavations that were open for weeks or even several months. There were fundamental differences between such excavations and typical shallow utility trenches" (Yokel 1979). These differences include depth, time an excavation is open, excavation and shoring methods (excavation in lifts), and trench discontinuities (short sections of shallow trench not continuously shored).

5.2.2 Field Testing Results in Relation to Finite Element Model Results

The field work indicates that the panel response to strut loading does not appear to be different between the Frederick and York trenches. The field work recreated actual trench rescue shoring operations where the system is installed after excavation is complete and struts are loaded one at a time. There is no boundary condition that prevents the bottom of the panel from bending away from the trench wall when the top strut is loaded. Loading of the bottom two struts appears to compensate for that initial outward movement.

The vertical strain results indicate that when the first strut was loaded the panel deformed into the trench wall at the top and deformed away from the trench wall at the bottom. The earth pressure in the finite element model when only the top strut is loaded supports the field results. Figure 5-9 shows that the earth pressure is inward in the top 3.5 ft and the earth pressure is outward in the lower part of the panel. The outward pressure implies the panel is pulling the soil. This occurs because the boundary conditions in the model do not allow the panel to separate from the soil. In actual conditions, the reverse pressure means that the panel has moved away from the trench wall and into the open space. Figure 5-10 shows the displacement of the panel when only the top strut is loaded to 1298 lbs. The outward deformation at the bottom is due to the pressure of the soil weight. Appendix U shows the tabulated data from the finite element model.

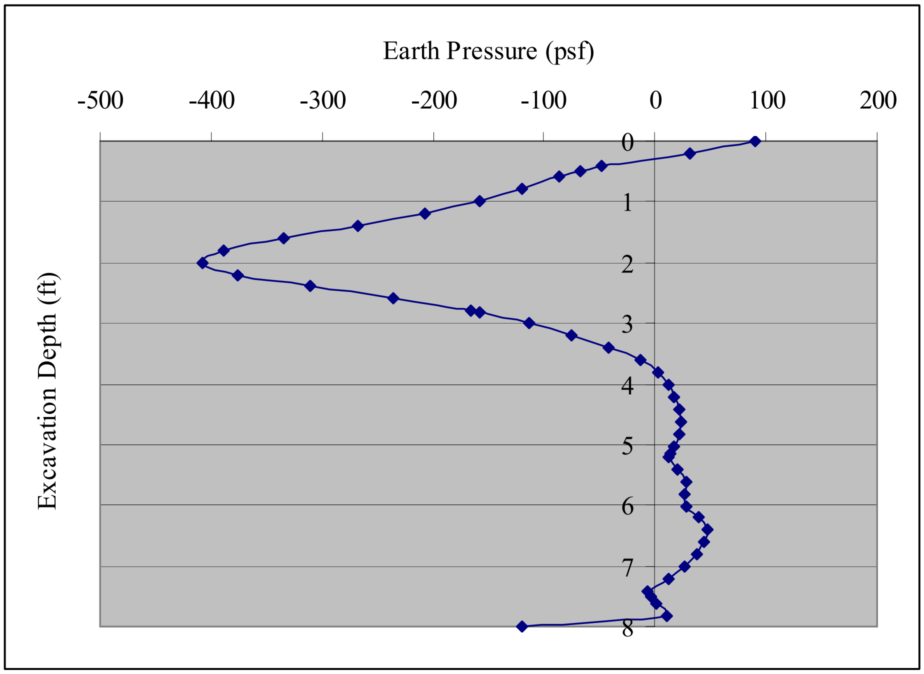


Figure 5-9 Earth pressure in soil with only the top strut loaded 1298 lbs/strut

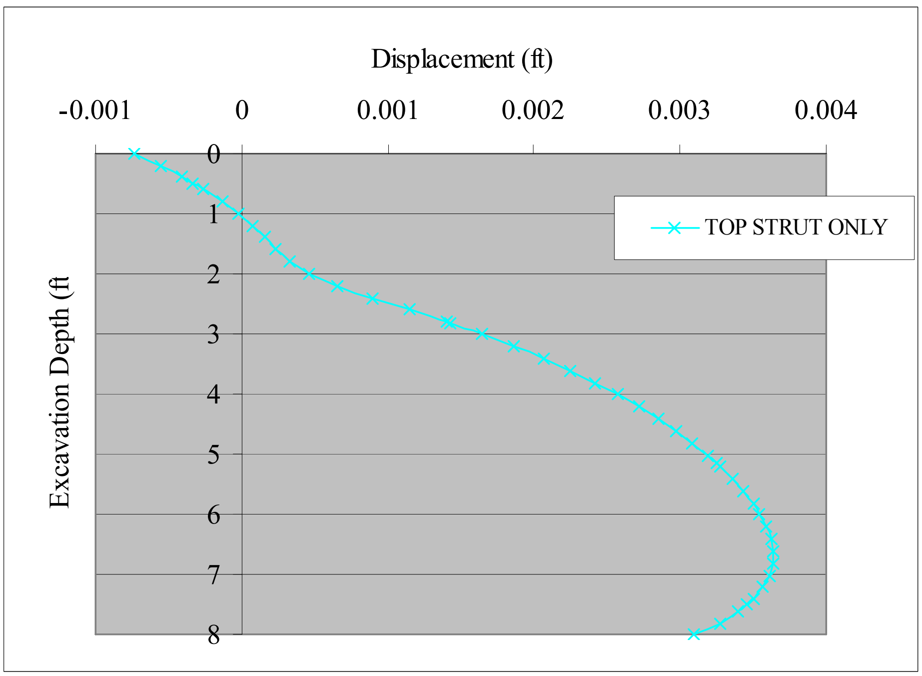


Figure 5-10 Displacement of the panel in finite element model when a single strut at y = 2 ft is loaded to 1298 lbs

Similar panel deformation implies there is a similar pressure distribution. However, earth pressure magnitude will differ as a function of the soil unit weight. The trenches in both the Frederick and York sites were analyzed using finite element models. The models simulated actual field conditions including location of the panel in the trench, surcharge size and surcharge orientation. The soil input parameters for the models were unit weight, Young's modulus and Poisson's ratio. Struts were loaded to 1298 lbs. The parameters were described in Chapter 4. Figure 5-11 shows a comparison of the earth pressure in both models. The comparison shows that unit weight and physical configuration differences do not significantly affect earth pressure in the elastic finite element model and the 16% difference between the results are due to the different physical properties of the soils. Appendix V shows the tabulated data.

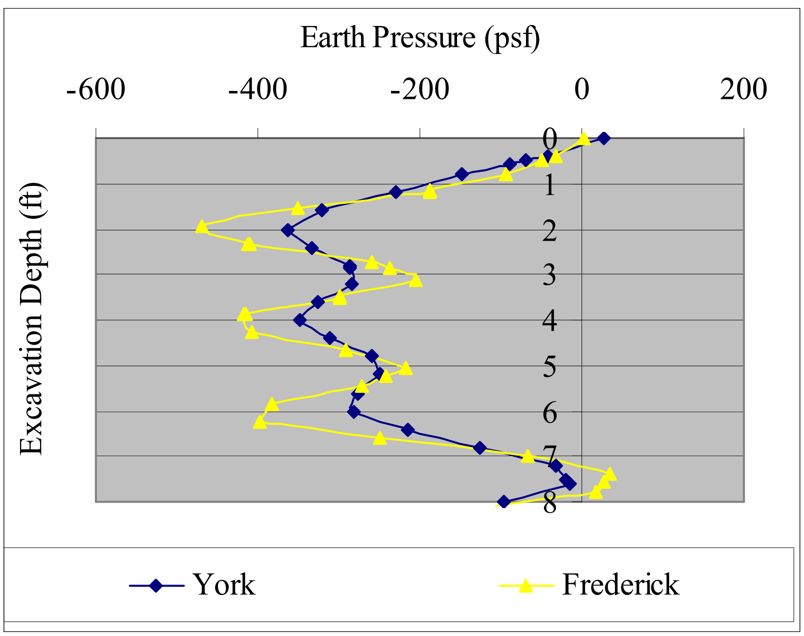


Figure 5-11 Comparison of Frederick and York finite element model stresses

5.2.3 Discussion of Field Work Significance

Trench rescue operations are narrowly focused on the end goal of victim rescue or body recovery. The field work in this research is significant because it recreated actual trench rescue field conditions in a controlled environment and allowed for analysis of quantifiable results previously not documented in shallow excavations. Results of this research imply that panel reaction to loading is affected more by trench wall conditions and sequence of loading rather than by soil type. This is important for rescuers to note. The more uniform a trench wall is, the more predictable the panel performance will be.

5.3 Summary

This chapter evaluated the performance of the existing standardized trench rescue shoring system used by many technical rescue teams and identified the factors having the greatest effect on the earth pressure produced and the actual panel deformation in the field. The research indicates that the most significant effect on earth pressure is strut loading. Earth pressure at a depth of 4 ft increased 375% from 394 psf to 1476 psf as strut loading increased from 1298 lbs to 4712 psf, equivalent to 200 psi internal pressure in an Airshore brand pneumatic strut and 1500 psi internal pressure in a Speed Shore brand hydraulic strut respectively. Potential for failure of the soil in the passive mode indicated that strut loads in the range of 4712 lbs are excessive. Figure 5-12 shows the comparison of Peck's sand/gravel apparent earth pressure diagram, Yokel's worst case earth pressure envelope for Type C soil, and the finite element results for 4712 lbs/strut loading and 1298 lbs/strut loading on soil with properties of Soil 2 used in the study. The comparison shows that actual earth pressure is highly dependent on the strut loading.

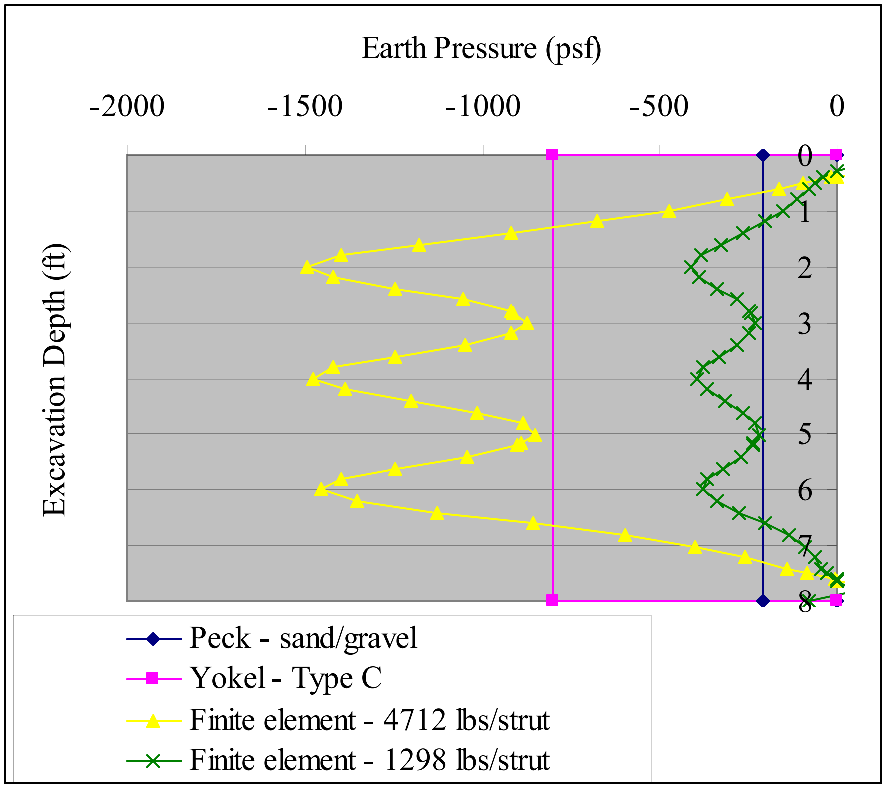


Figure 5-12 Range comparison for Peck, Yokel, and the finite element model results

Because it was found that earth pressure is highly dependent on strut loading, this research developed an alternative approach to determining earth pressure and maximum recommended strut loading for shoring systems. By distributing known strut loads over the total shoring area and multiplying by the average limit factor, a maximum earth pressure ordinate can be determined and compared against the Rankine passive pressure at the depth of the highest strut. Alternatively, by dividing the Rankine passive pressure at the depth of the highest strut by the limit factor, a maximum recommended strut load can be back-calculated from the uniform earth pressure. The field work validated using the same earth pressure distribution for analysis with any soil type because the soil type did not affect the earth pressure distribution shape.

Chapter 6 Conclusions and Recommendations

6.1 Conclusions

The Occupational Safety and Health Administration (OSHA) developed *Safety and Health Regulations for Construction* that include 1926 Subpart P – Excavations (OSHA 1989). These are enforceable safety requirements intended to prevent accidents and yet every year workers are killed in shallow excavations. The Bureau of Labor and Statistics reported that during 2000 thirty eight construction workers died in excavation and trench cave-ins (Bureau 2001) and there were twenty three fatalities related to trenches and excavations in 2007 (Bureau 2008). There are estimated to be hundreds of entrapments in failed trenches every year, many of which are never reported.

The technical rescue industry, dominated by fire and rescue services, has been tasked with developing a safe and efficient means of attempting rescues and inevitably body recoveries. While a standard shoring system was developed based on OSHA requirements, the OSHA requirements were developed from industry practice, not technical analysis. The objective of this dissertation was to perform a technical analysis of a typical trench rescue shoring system to verify its current design and provide recommendations for a design methodology that is applicable for any shallow braced excavation.

The following seven conclusions are drawn from the parametric studies using linear elastic finite element analysis and field validation of the shoring system in actual field conditions.

1) The pressure distributions determined by the finite element model are consistently greater than the Rankine active earth pressure and, in cases of excessive strut loading, can exceed the Rankine passive earth pressure.

2) Peck's apparent earth pressure envelopes are dependent on soil type and are not related to strut loading. This research shows that soil type does not have a significant effect on earth pressure distribution in shallow excavations, so Peck's envelopes are not generally applicable even though they are widely accepted in current engineering practice.

3) Yokel's earth pressure envelopes give a wider range of earth pressure than Peck, but are also mainly dependent on soil type. The envelopes are also not applicable because they do not depend on strut loading.

4) This research recommends a new earth pressure calculation method that is dependent on the strut load used. The method can be used in either of two scenarios: determining the earth pressure or determining the maximum recommended strut load.

    a. Determining the earth pressure: Distribute the known strut loads over the shoring system face to determine a uniform earth pressure behind the shoring system. The maximum ordinate of the true earth pressure distribution can be determined by multiplying the uniform earth pressure by a "limit factor" that was introduced in this study and is dependent on soil and shoring component material properties.

    b. Determining the maximum recommended strut load: In order to avoid failing the soil in a passive mode, the maximum strut load could be determined by back-calculation from the Rankine passive earth pressure. The strut load should be set such that the maximum ordinate of the true earth pressure distribution does not exceed the Rankine passive earth pressure as calculated at the depth of the highest strut.

5) This study showed that increasing the panel stiffness and thickness reduces earth pressure whereas in deep foundation bracing, increasing the stiffness increases earth pressure. While reducing earth pressure is desirable, the tradeoff for rescuers is manageability. A 0.75 in thick panel weighs approximately 90 lbs. A 1.25 in thick panel weighs approximately 150 lbs. Rescuers must weigh the benefits of lower load transfer to the soil against increased manpower required to handle heavier panels. A 0.75 in thick panel may be adequate in a stable, cohesive soil while using lower strut loading in the range of 1300 lbs/strut or less. As strut loading increases to the range of 2356 lbs to 4712 lbs, the thicker panel may be a safer option.

6) The field testing found that soil type did not appear to affect the earth pressure distribution shape because the panel deformation was similar in both the trench in the granular soil and the trench in the clayey soil. The finite element studies also show that soil type does not affect the earth pressure distribution shape in shallow excavations.

7) This research found that the panel typically used in trench rescue shoring does improve the structural value to the system. OSHA 1926 Subpart P Appendix D (g)(7) states that "0.75 in thick, 14 ply arctic white birch… plywood is not intended as a structural member, but only for preventing local raveling

(sloughing of the trench face) between shores" (OSHA 1989). This research indicates that at a depth of 4 ft the 0.75 in thick panel reduces earth pressure by 32% when added between the trench face and the strongback.

6.2 Recommendations

This research is intended to create a building block toward a comprehensive evaluation of the existing trench rescue shoring system. It provided the lateral loads that the shoring system should be designed to resist if external forces such as loaded struts are introduced. Further research should examine use of a non-linear model that may better capture the effects of loading one strut at a time. It is also recommended that further finite element modeling be performed varying the Young's modulus and Poisson's ratio of the soil and shoring system components as well as the soil unit weight to determine the effect of those parameters on the limit factor.

The goal of the field validation was to examine actual field conditions encountered by a technical rescue team rather than ideal laboratory conditions. It is recommended that further research include building a true homogeneous trench and installing load cells in order to measure changes in earth pressure during trench rescue shoring installation.

Trench rescue operations are performed in unstable soils that have already shown symptoms of low cohesion. Continued research should focus on effects of increased strut load on load transfer in soils of varying cohesion. Given the results of this research, it is reasonable to conclude that some combination of strut load and soil properties will produce passive failure behind the panel and/or active failure on either side of the panel.

There has traditionally been a lack of engineering analysis of technical rescue systems including trench rescue shoring. Being a derivative of the fire service, the technical rescue industry has trained most personnel to make do with available tools. Prior to current value engineering trends in most construction fields, this tactic was typically effective. Engineering analysis is becoming increasingly important to evaluate existing rescue system effectiveness in the face of less redundant systems utilized by the construction industry at large. As education and technology increasingly pervade the technical rescue industry and the fire service, the potential to research, evaluate and recommend changes to existing rescue systems is becoming a reality.

Appendix A: Building the Abaqus Model

This appendix describes the process of building the research models using Abaqus/CAE Version 6.7-1.

Parts/Materials:

¾ in x 4 ft x 8 ft FinnForm Panel (0.0625 x 4 x 8)

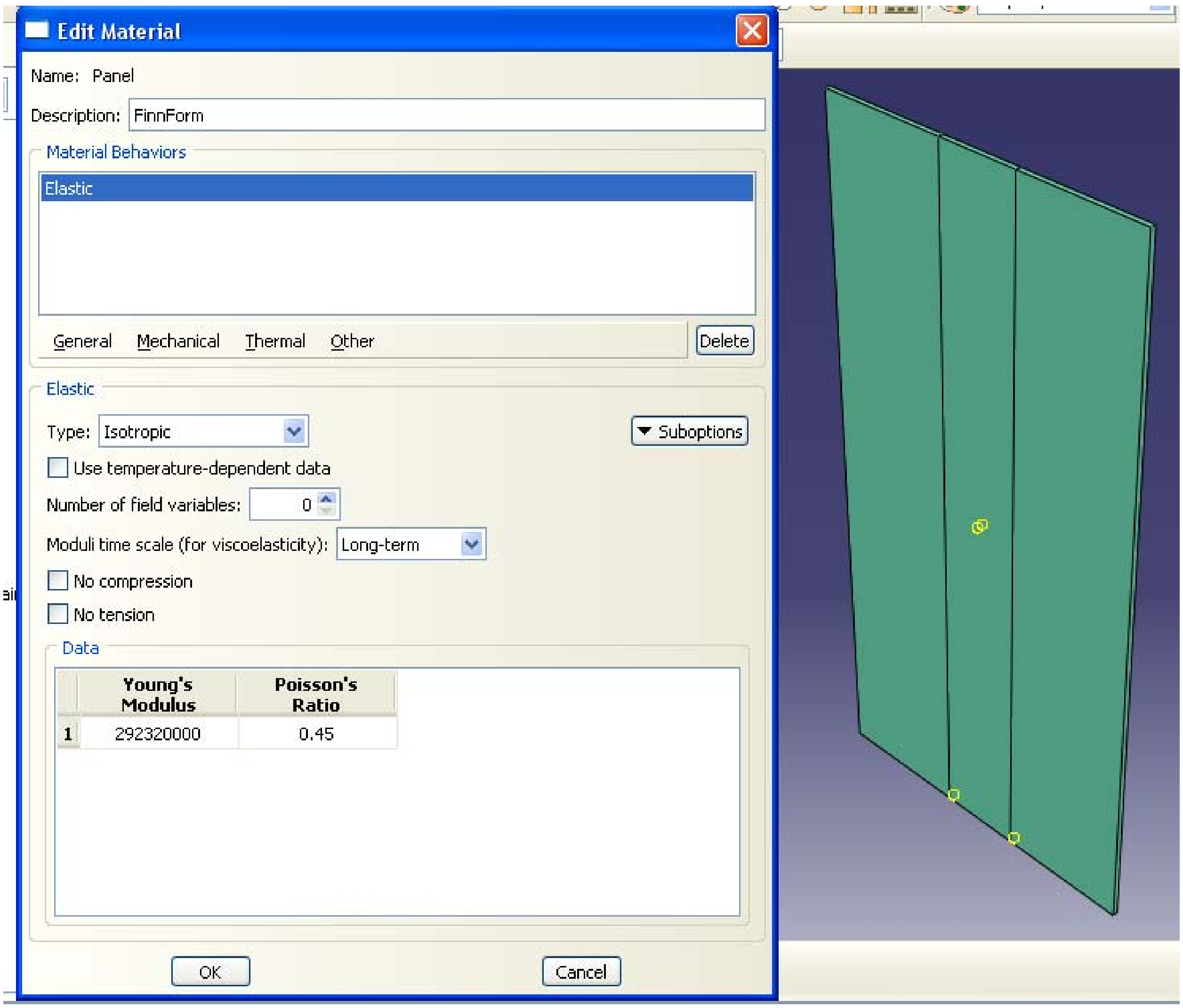


Figure A-1: Assigning material properties to the panel part

2 x 12 (1.5 in x 11.25 in) x 12 ft Kiln Dried #2 Southern Yellow Pine (0.125 x 0.958 x 12):

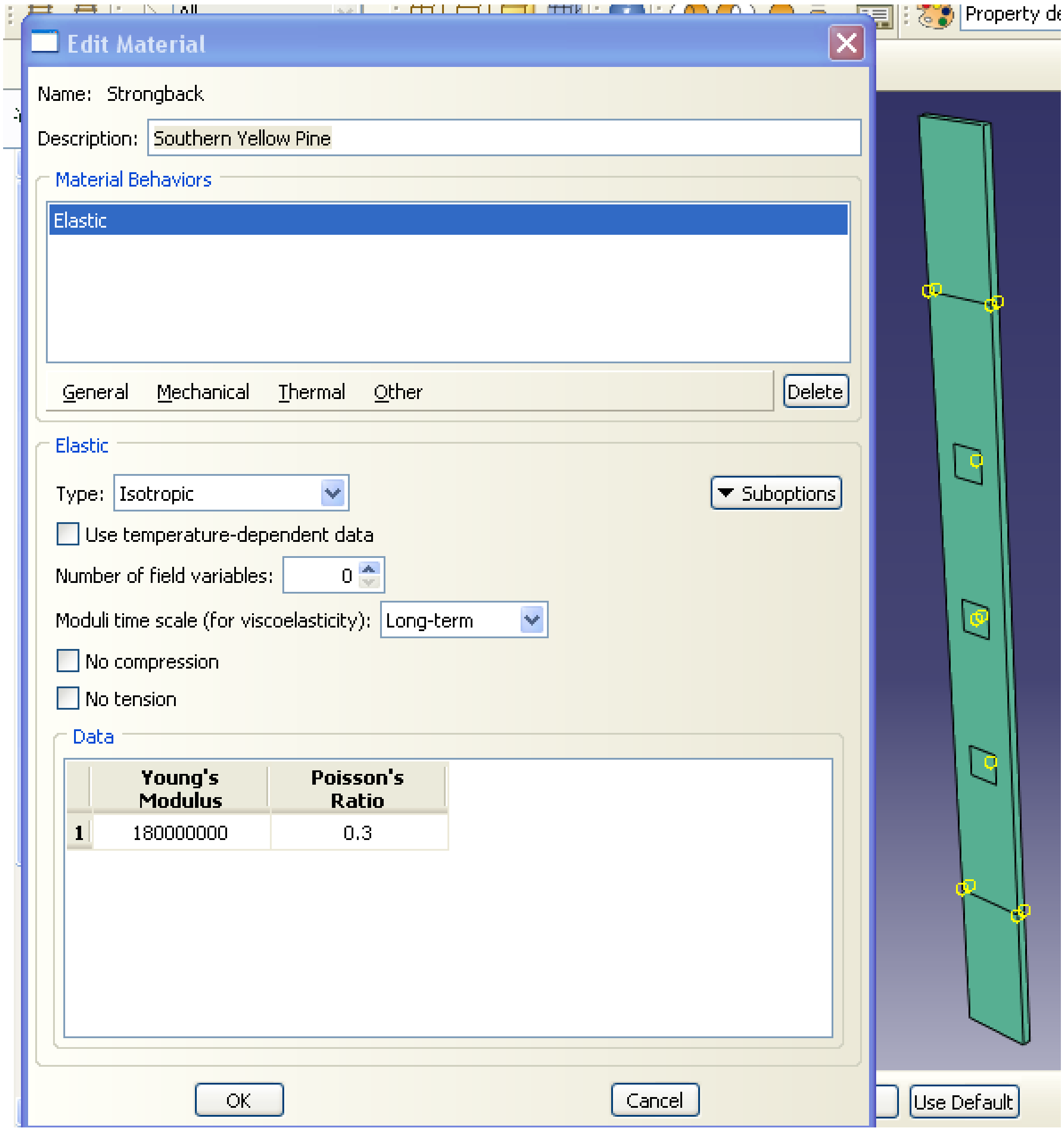


Figure A-2: Assigning material properties to the strongback part

Sample Soil Material Properties:

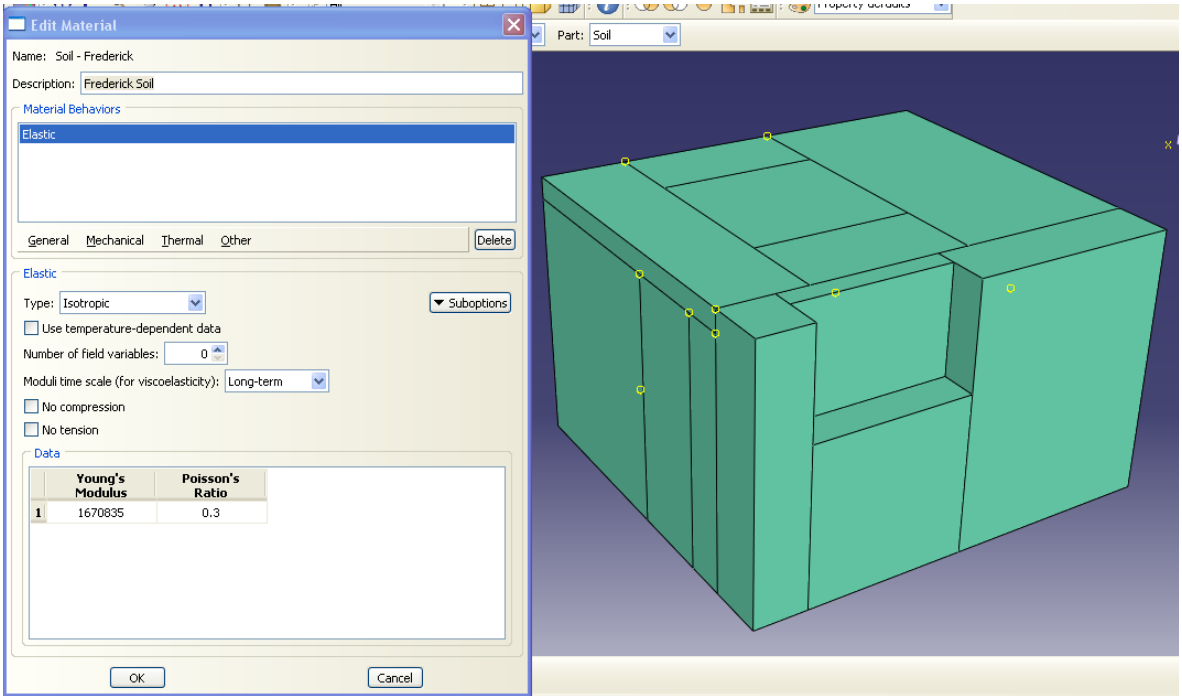


Figure A-3: Assigning material properties to the soil part

Sample Section Assignments – a solid, homogeneous section was assigned to all parts:

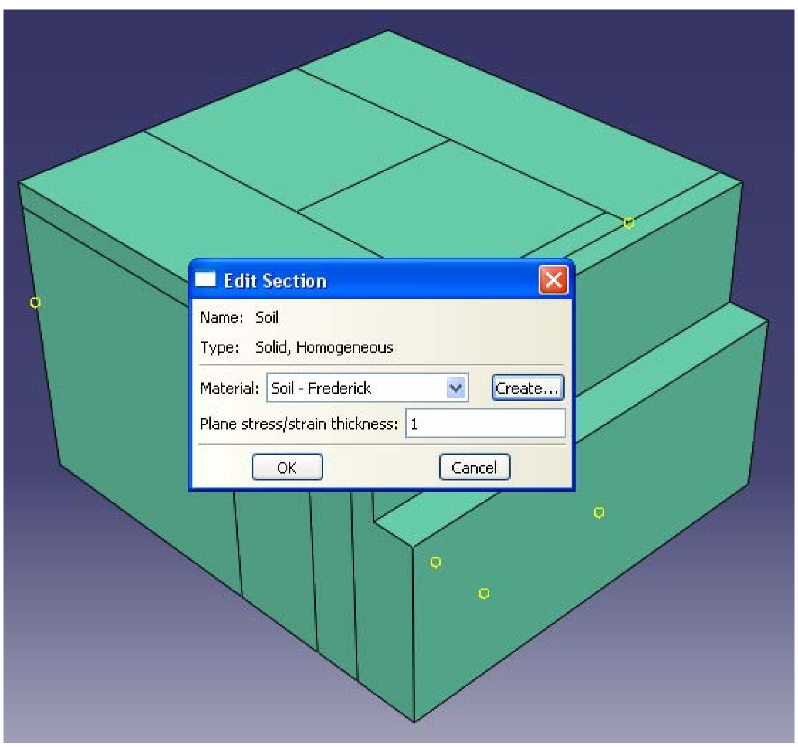


Figure A-4: Assigning a section type to the soil part

Sample Mesh Controls:

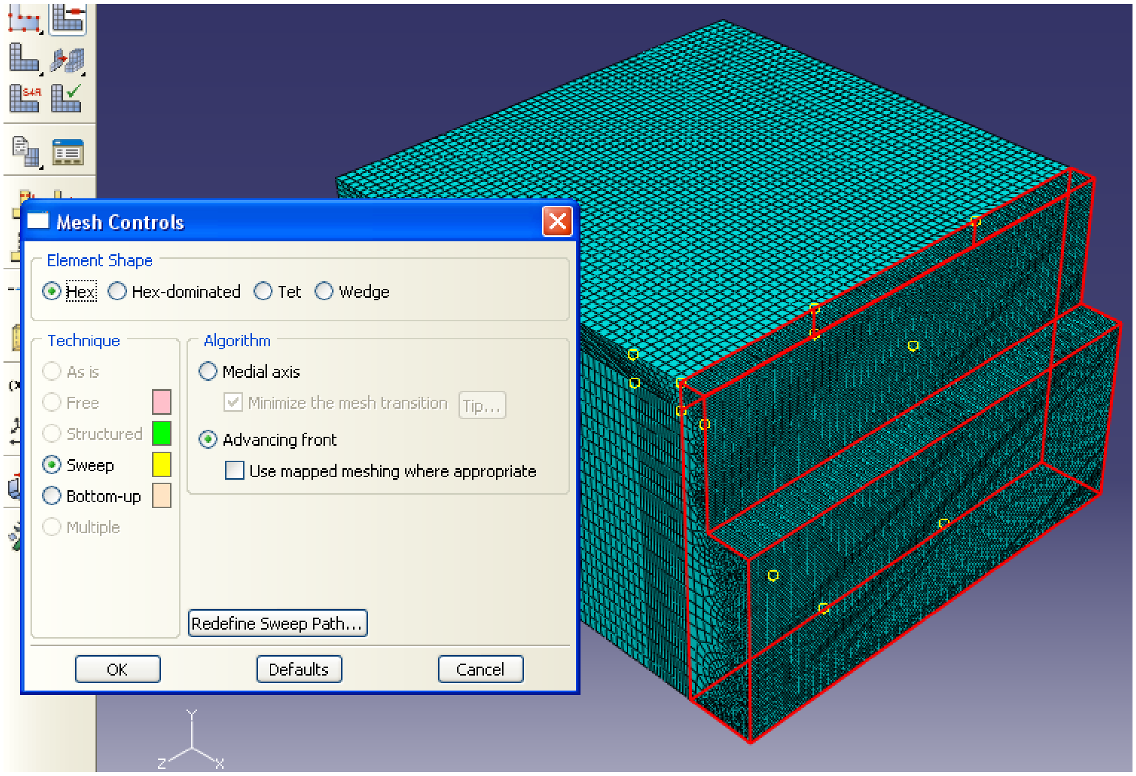


Figure A-5: Assigning mesh controls to the soil part

Assigning the Element Type:

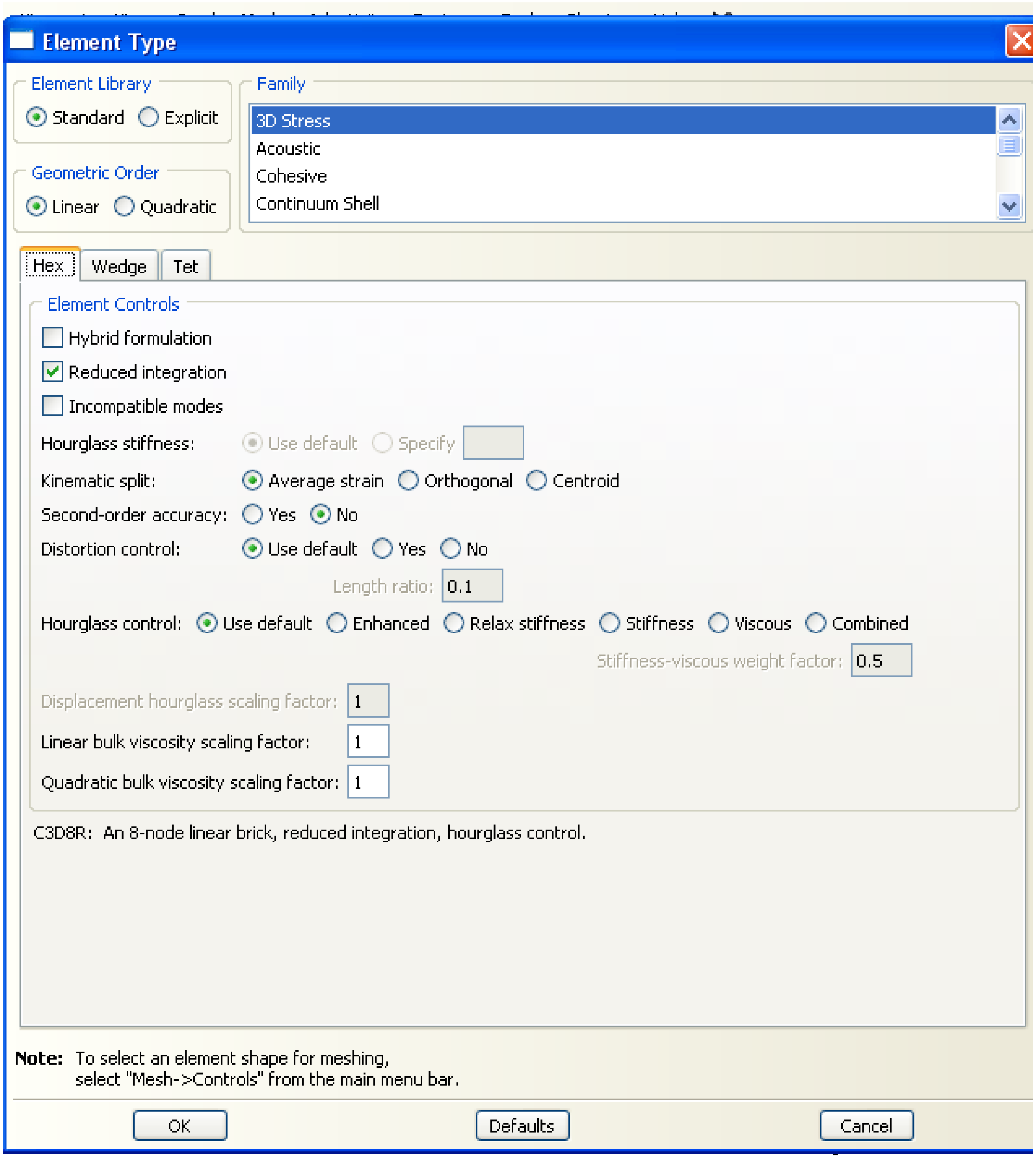


Figure A-6: Assigning the element type to the soil part

Sample Mesh:

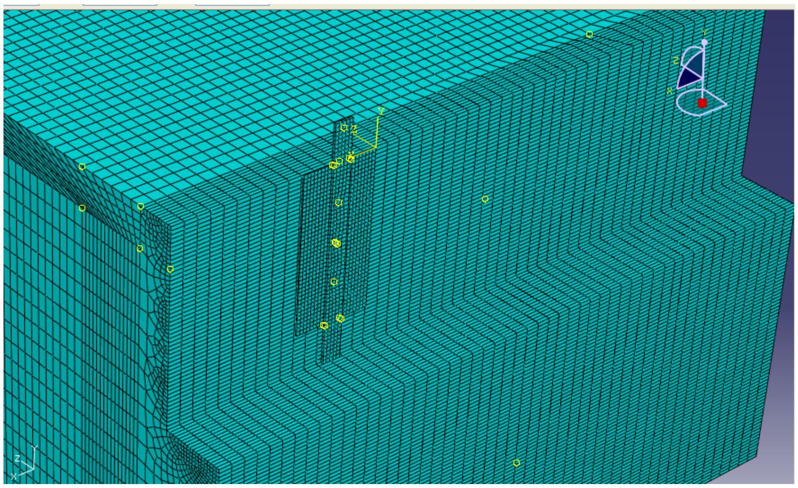


Figure A-7: Sample of what the parts look like once assembled and meshed

Sample Surface Interactions:

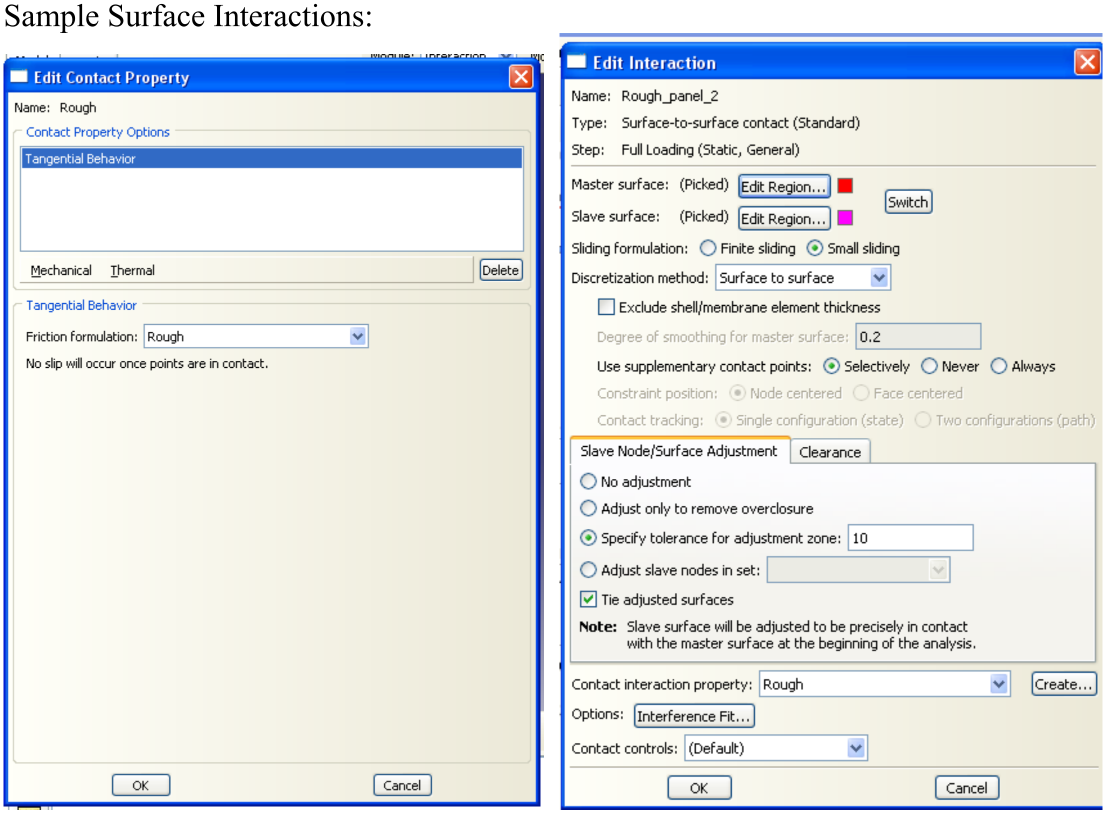


Figure A-8: Defining surface interaction properties

Boundary Conditions:

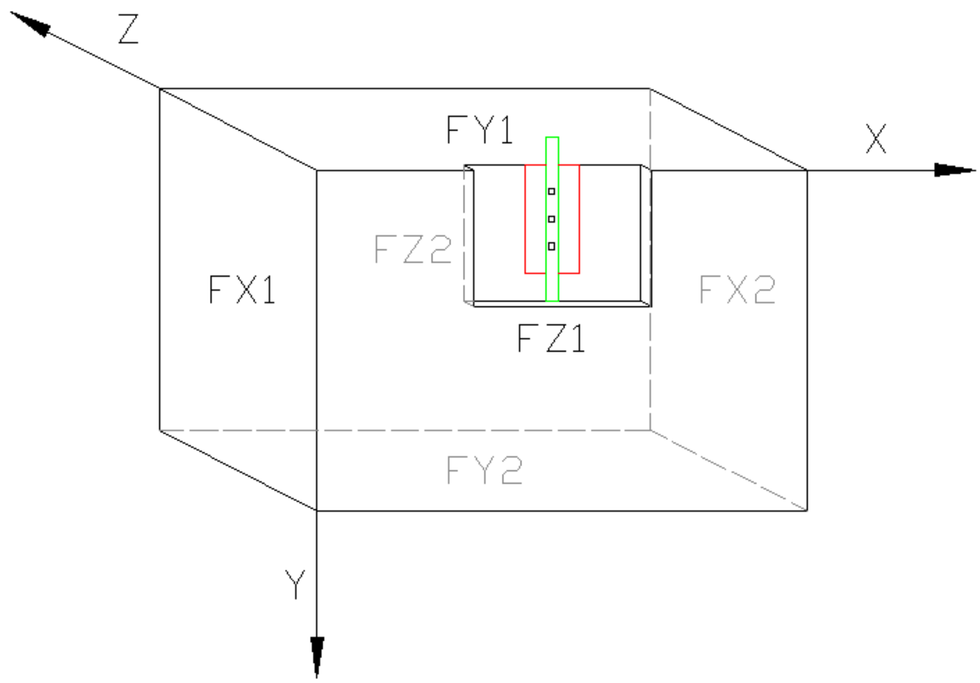


Figure A-9: The model with faces labeled

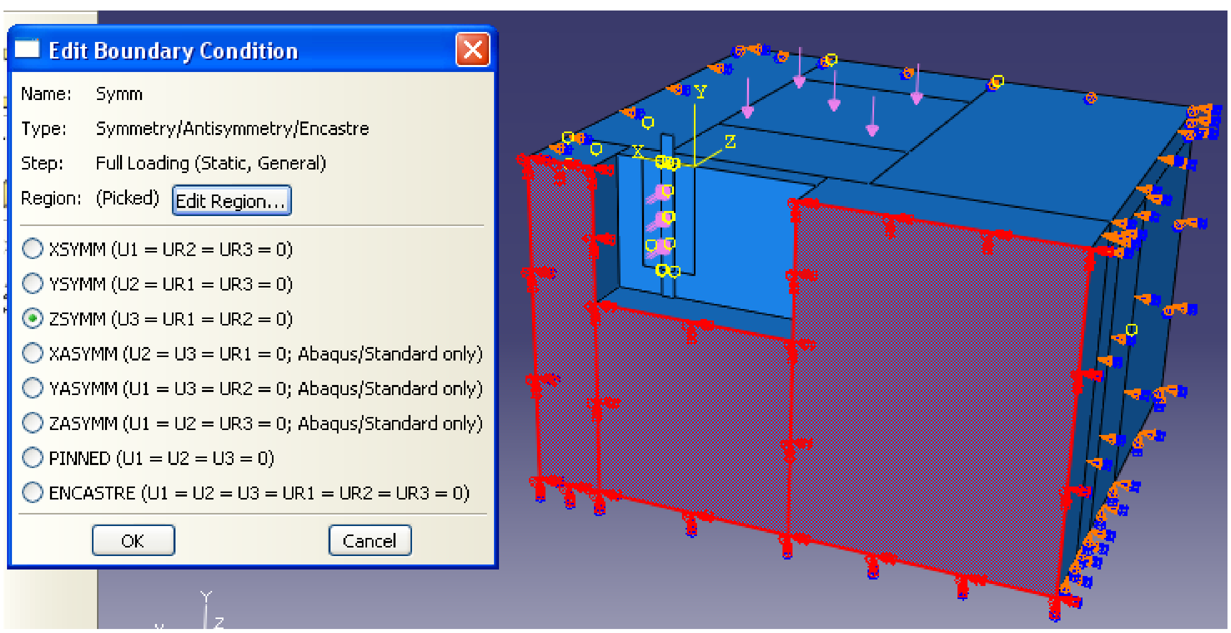


Figure A-10: Boundary conditions for the FZ1 face

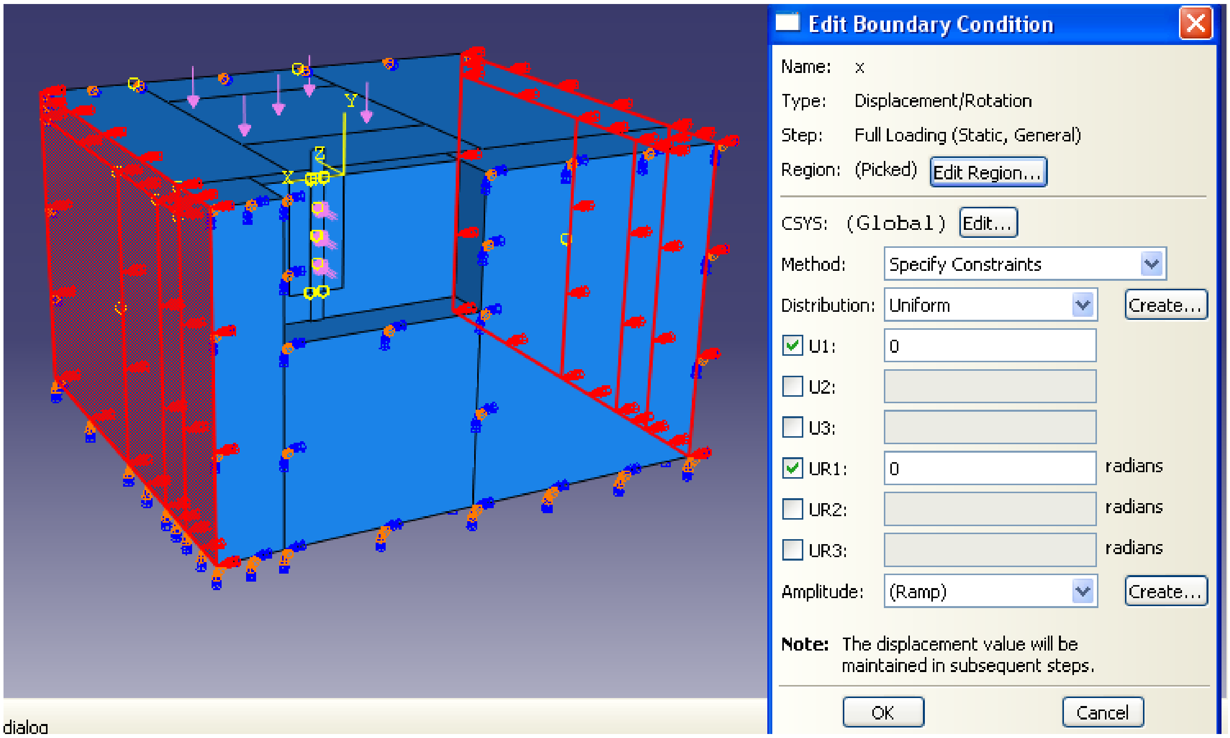


Figure A-11: Boundary conditions for the FX1 and FX2 faces

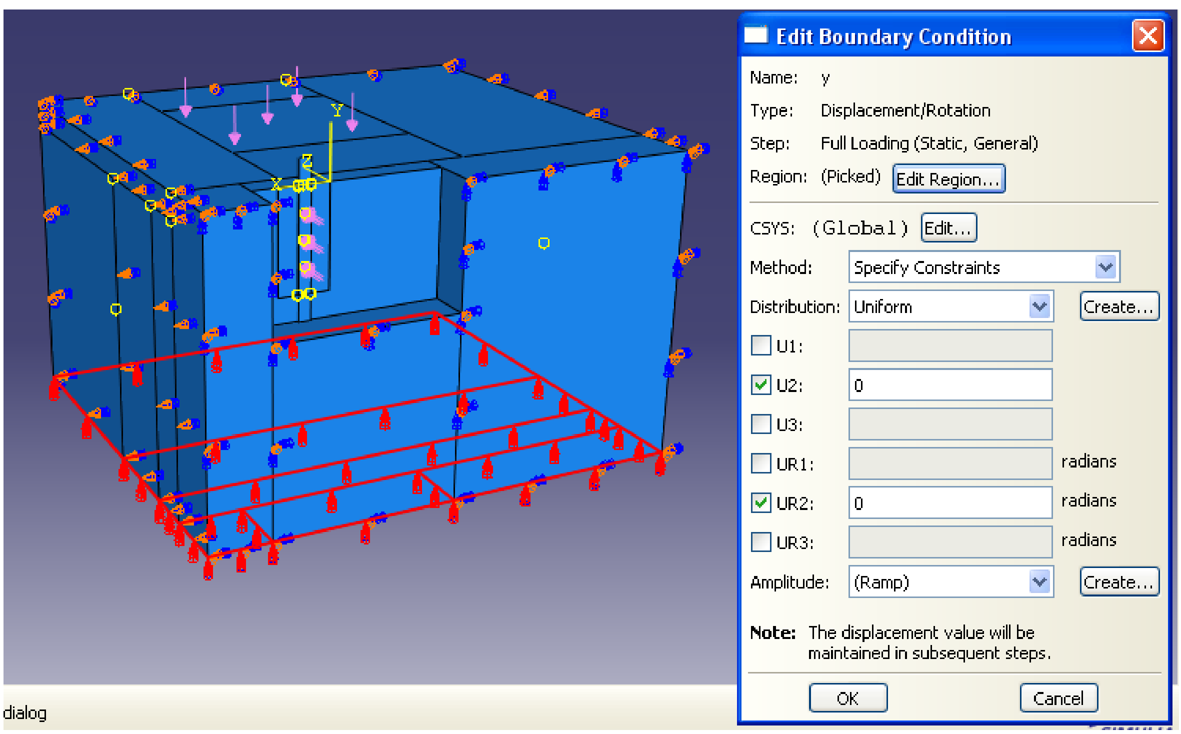


Figure A-12: Boundary conditions for the FY2 face

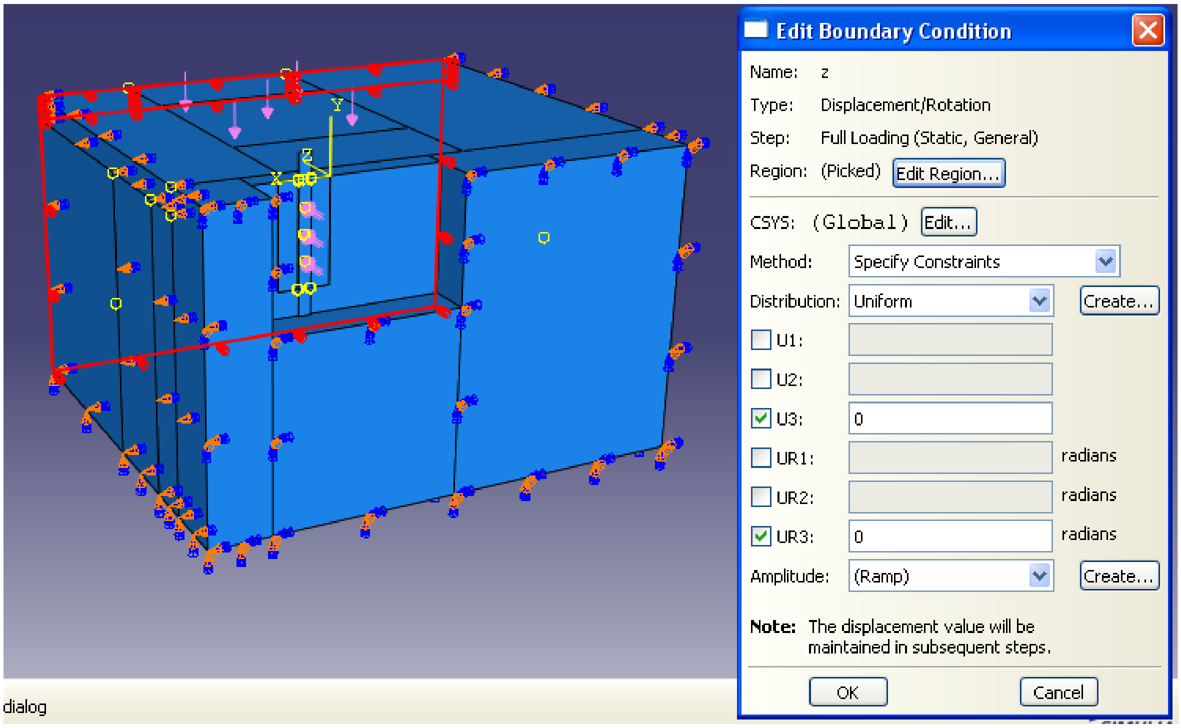


Figure A-13: Boundary conditions for the FZ2 face

Output Request:

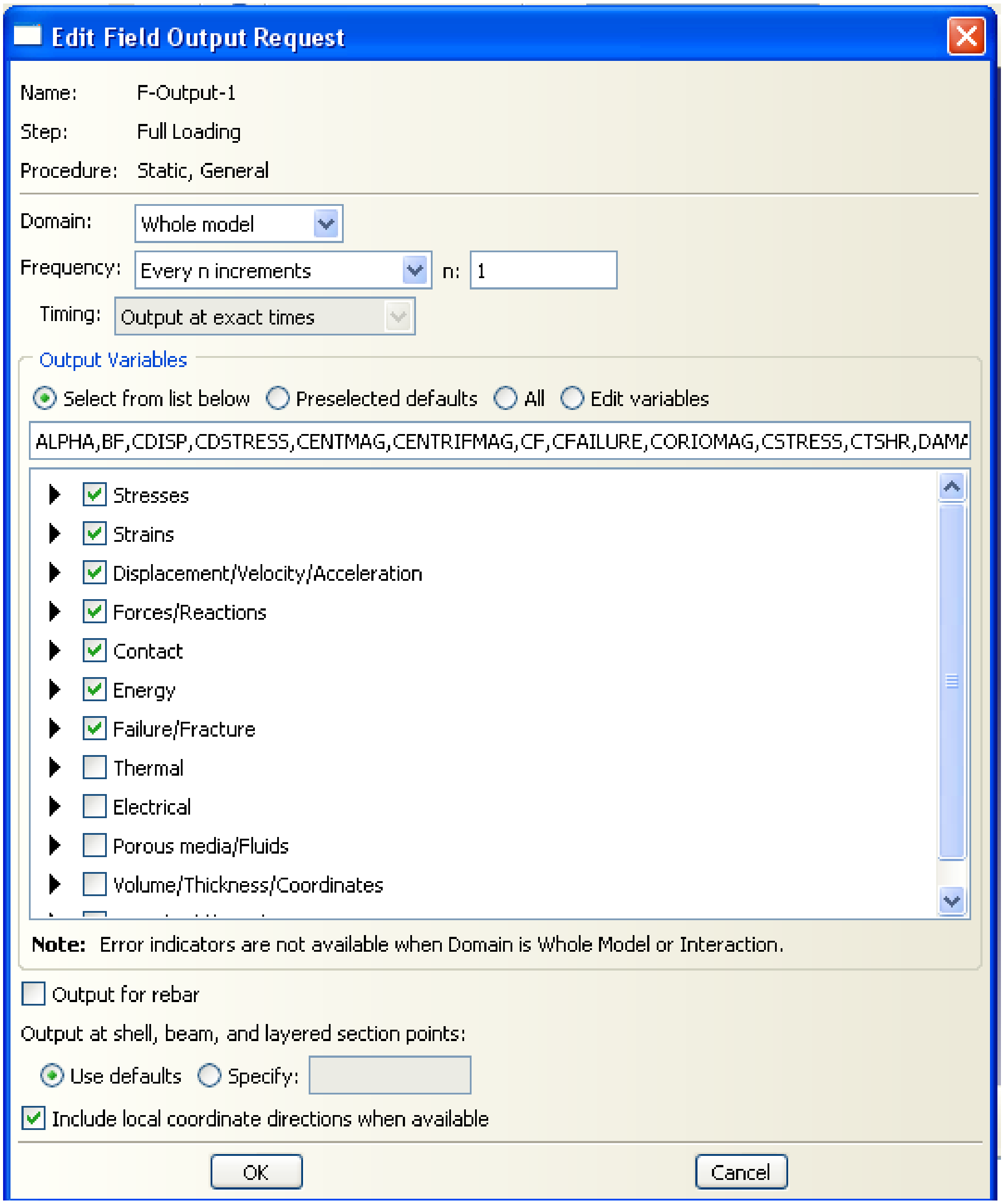


Figure A-14: Output request setup

Analysis Step Setup: Static Stress Analysis (see *Abaqus/CAE User's Manual* section 14.11.1 "Configuring general analysis procedures" for more information)

Basic Tab:

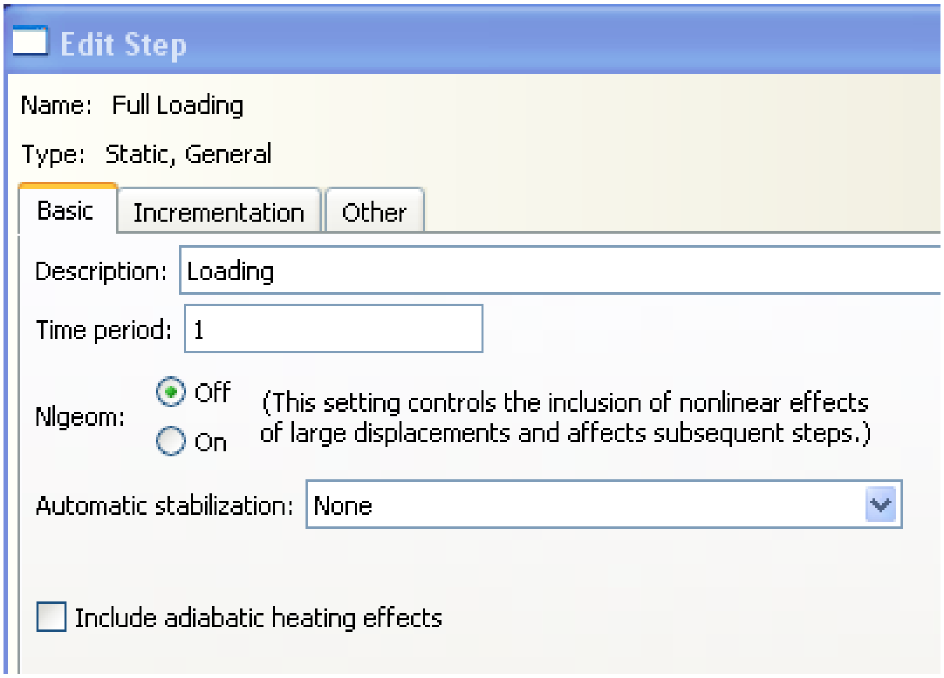


Figure A-15: Analysis step setup on the Basic tab

Nlgeom "Off": performing a geometrically linear analysis because displacements are expected to be relatively small (14.3.2 Linear and nonlinear procedures)

Automatic stabilization "None": no local instabilities such as surface wrinkling, material instability, or local buckling are expected (14.11.1 Configuring general analysis procedures)

Not performing an adiabatic stress analysis.

Incrementation Tab:

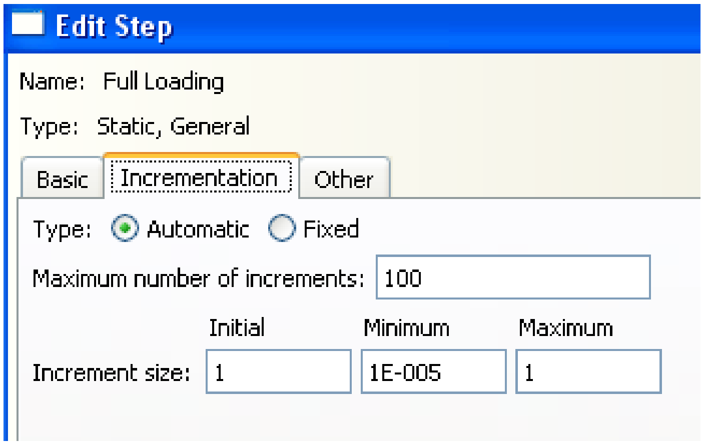


Figure A-16: Analysis step setup on the Incrementation tab

Use default values.

Type “Automatic”: allows Abaqus to choose the size of the time increments based on computational efficiency.

Maximum number of increments “100”: the number of increments in the step. The analysis stops if this maximum is exceeded before Abaqus/Standard arrives at the complete solution for the step.

Increment size: Abaqus creates default increment sizes based on computational efficiency. Abaqus will terminate the analysis if different values are needed.

Other Tab:

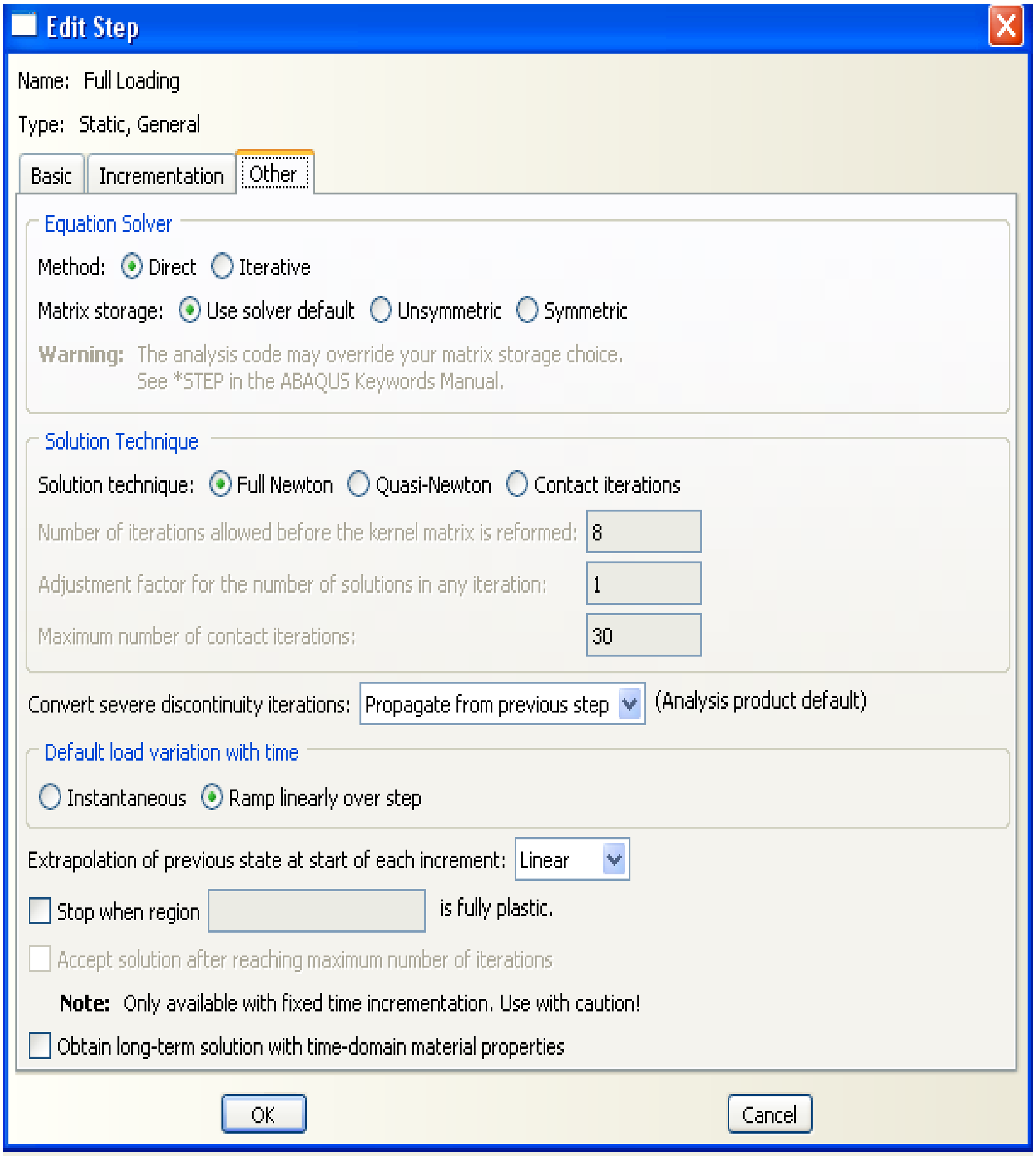


Figure A-16: Analysis step setup on the Other tab

Equation Solver:

- Method "Direct": choose "Direct" to use the default direct sparse solver. The panel and strongback were small enough in relation to the soil part that the direct sparse solver reduced computational time. (Abaqus Analysis User's Manual 6.1.4 Direct linear equation solver) When I attempted to use the iterative linear equation solver, computational demands for the same size model exceeded the abilities of my hardware.
- Matrix Storage "Use solver default": allows Abaqus to decide whether a symmetric or unsymmetric matrix storage and solution scheme is needed.

Solution Technique “Full Newton”: See *Abaqus Theory Manual* Section 2.2.1 Nonlinear solution methods in Abaqus for description of full Newton solution technique:

Convert severe discontinuity iterations “Propagate from previous step”: option for dealing with severe discontinuities during nonlinear analysis (Abaqus 2007e):

- Select Off to force a new iteration if severe discontinuities occur during an iteration, regardless of the magnitude of the penetration and force errors. This option also changes some time incrementation parameters and uses different criteria to determine whether to do another iteration or to make a new attempt with a smaller increment size.
- Select On to use local convergence criteria to determine whether a new iteration is needed. Abaqus/Standard will determine the maximum penetration and estimated force errors associated with severe discontinuities and check whether these errors are within the tolerances. Hence, a solution may converge if the severe discontinuities are small.
- Select Propagate from previous step to use the value specified in the previous general analysis step. This value appears in parentheses to the right of the field.

Default load variation with time “Ramp linearly over time”: in the purely elastic model, either option produces the same results at the end of the step (Abaqus 2007e).

- Choose Instantaneous if you want loads to be applied instantaneously at the start of the step and remain constant throughout the step.
- Choose Ramp linearly over step if the load magnitude is to vary linearly over the step, from the value at the end of the previous step to the full magnitude of the load.

Extrapolation of previous state at start of each increment “Linear”: method for determining the first guess to the incremental solution (Abaqus 2007e):

- Select Linear to indicate that the process is essentially monotonic and Abaqus/Standard should use a 100% linear extrapolation, in time, of the previous incremental solution to begin the nonlinear equation solution for the current increment.
- Select Parabolic to indicate that the process should use a quadratic extrapolation, in time, of the previous two incremental solutions to begin the nonlinear equation solution for the current increment.
- Select None to suppress any extrapolation.

Stop when region “region name” is fully plastic: In these cases plastic behavior is not expected.

Obtain long-term solution with time-domain material properties: obtains the fully relaxed long-term elastic solution with time-domain viscoelasticity or the long-term elastic-plastic solution for two-layer viscoplasticity. This parameter is relevant only for time-domain viscoelastic and two-layer viscoplastic materials. It does not apply to a fully elastic model. (Abaqus 2007e)

Appendix B: Verification of Model Size Tabulated Data

The following pages provide the tabulated data from the models run to determine appropriate overall model dimensions to minimize computational expense while minimizing boundary condition effects on earth pressure. Material properties were held constant and strut loading was set to 1298 lbs/strut for all iterations. While the dimensions of the model varied in the X (width), Z (length), and Y (depth) directions, the earth pressures were read at the same depths in the Y direction for each iteration. Figure B-1 shows the model orientation.

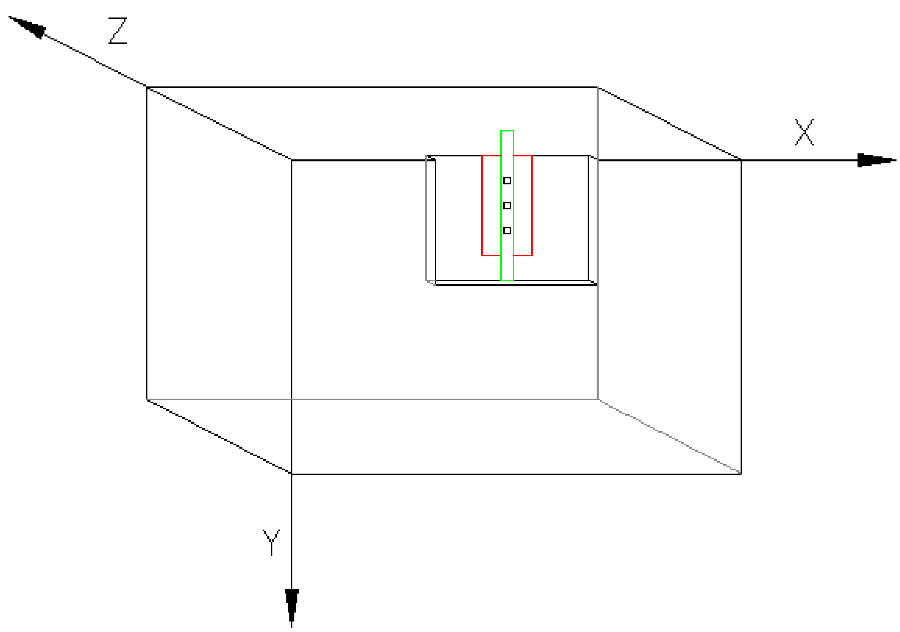
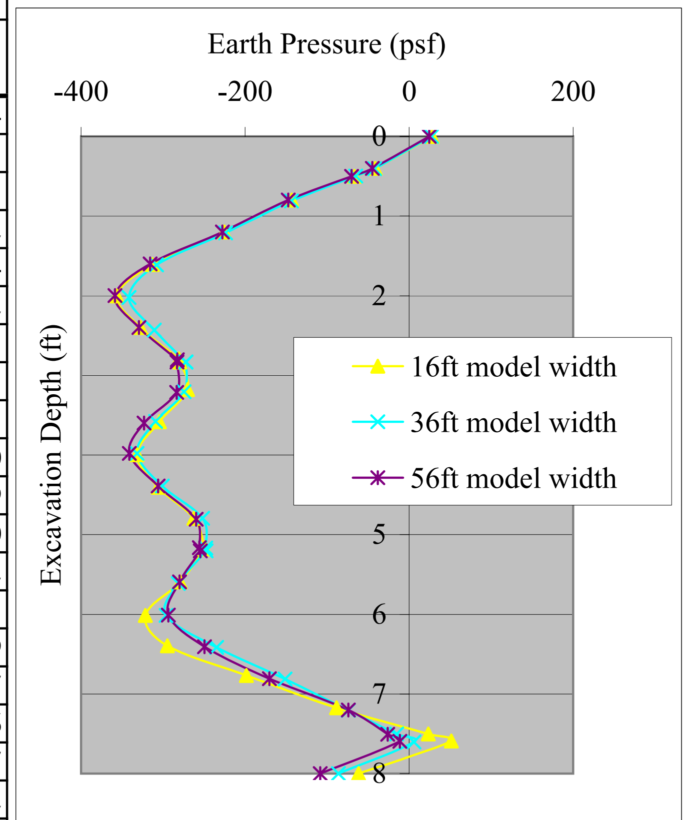


Figure B-1 Model orientation

Table B-1 Comparison of earth pressure as X (width) varied

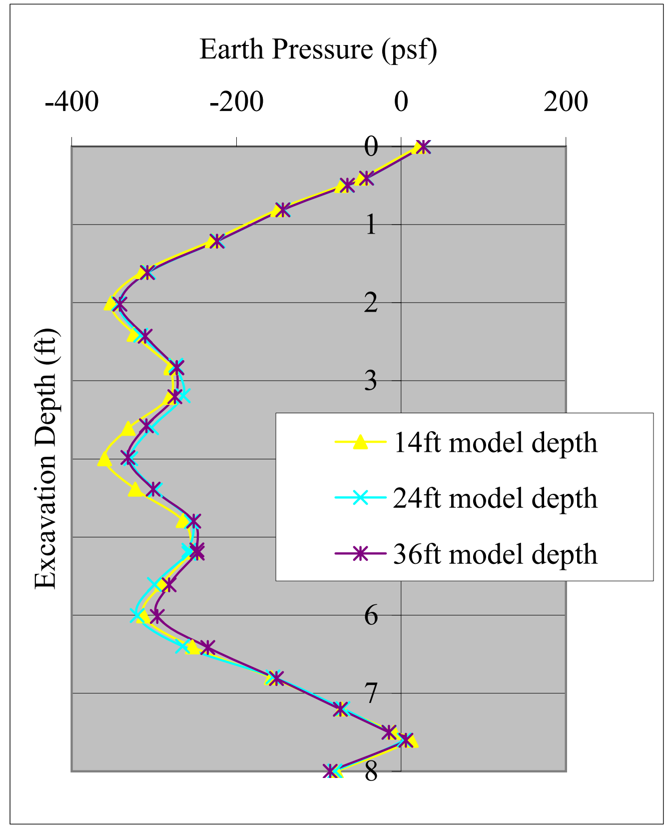
X width (ft)	16		36		56	
	y (ft)	EP (psf)	y (ft)	EP (psf)	y (ft)	EP (psf)
	0.00	28.00	0.00	27.14	0.00	24.64
	0.40	-41.15	0.41	-42.00	0.40	-44.71
	0.50	-66.81	0.50	-65.22	0.50	-70.31
	0.80	-143.12	0.81	-143.45	0.80	-147.22
	1.20	-225.17	1.21	-223.62	1.20	-227.54
	1.61	-312.87	1.61	-307.92	1.60	-315.82
	2.00	-356.41	2.02	-341.45	2.00	-358.71
	2.40	-327.36	2.43	-310.64	2.40	-329.53
	2.79	-280.66	2.83	-272.26	2.81	-282.83
	2.83	-279.47	2.83	-272.25	2.83	-282.90
	3.18	-270.07	3.20	-274.56	3.21	-283.39
	3.58	-304.64	3.58	-309.14	3.60	-323.30
	4.00	-331.86	3.98	-331.63	3.98	-341.22
	4.40	-306.20	4.39	-300.91	4.39	-306.01
	4.79	-262.42	4.80	-252.01	4.81	-259.49
	5.16	-249.55	5.17	-247.88	5.16	-255.77
	5.18	-249.35	5.20	-247.45	5.20	-254.66
	5.59	-279.31	5.61	-281.54	5.59	-279.77
	6.01	-321.64	6.02	-296.10	6.01	-293.72
	6.40	-294.77	6.42	-234.62	6.41	-249.42
	6.77	-198.33	6.81	-151.30	6.81	-170.26
	7.17	-88.38	7.21	-73.94	7.20	-73.95
	7.50	23.58	7.50	-15.04	7.50	-26.11
	7.59	51.36	7.60	5.75	7.60	-11.12
	8.00	-61.78	8.00	-86.13	8.00	-108.57
	8.01	-63.60			8.00	-108.37



Depth	EP (psf) y = 4	Difference
16	-331.86	-0.07%
36	-331.63	2.89%
56	-341.22	

Table B-2 Comparison of earth pressure as Z (depth) varied

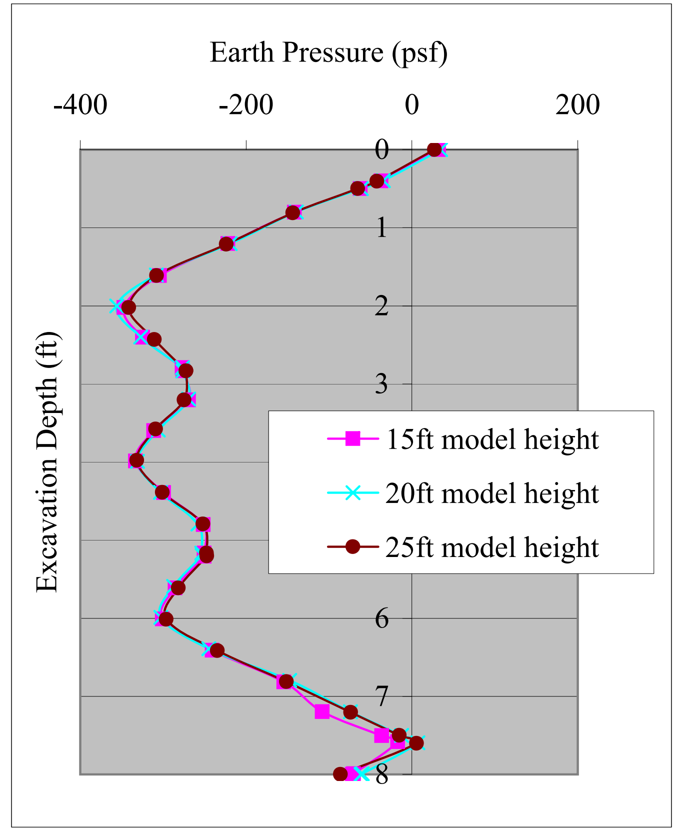
Z length (ft)	14		24		36	
	y (ft)	EP (psf)	y (ft)	EP (psf)	y (ft)	EP (psf)
	0.00	21.33	0.00	27.22	0.00	27.14
	0.40	-48.14	0.41	-41.61	0.41	-42.00
	0.50	-72.43	0.50	-65.60	0.50	-65.22
	0.81	-149.69	0.81	-142.84	0.81	-143.45
	1.21	-229.20	1.21	-222.49	1.21	-223.62
	1.60	-313.93	1.61	-306.46	1.61	-307.92
	2.00	-352.11	2.01	-342.82	2.02	-341.45
	2.40	-324.52	2.42	-316.33	2.43	-310.64
	2.81	-279.26	2.80	-273.44	2.83	-272.26
	2.83	-279.40	2.83	-272.59	2.83	-272.25
	3.21	-281.35	3.19	-264.54	3.20	-274.56
	3.61	-331.27	3.60	-302.97	3.58	-309.14
	4.00	-360.17	4.00	-328.06	3.98	-331.63
	4.39	-322.80	4.40	-298.55	4.39	-300.91
	4.79	-264.66	4.81	-254.74	4.80	-252.01
	5.17	-253.75	5.17	-257.54	5.17	-247.88
	5.19	-253.41	5.21	-257.52	5.20	-247.45
	5.60	-290.67	5.61	-299.65	5.61	-281.54
	6.01	-312.59	6.01	-320.60	6.02	-296.10
	6.41	-253.39	6.40	-265.53	6.42	-234.62
	6.80	-157.72	6.79	-155.70	6.81	-151.30
	7.20	-72.11	7.20	-70.67	7.21	-73.94
	7.50	-8.69	7.50	-14.40	7.50	-15.04
	7.60	12.45	7.60	3.64	7.60	5.75
	8.00	-78.84	8.00	-80.36	8.00	-86.13
	8.00	-78.86	8.00	-80.47		



Depth	EP (psf) y = 4	Difference
14	-360.17	-8.92%
24	-328.06	0.01
36	-331.63	

Table B-3 Comparison of earth pressure as Y (height) varied

Y depth (ft)	15		20		25	
	y (ft)	EP (psf)	y (ft)	EP (psf)	y (ft)	EP (psf)
	0.00	32.13	0.00	34.71	0.00	27.14
	0.40	-37.23	0.40	-34.51	0.41	-42.00
	0.50	-62.19	0.50	-61.08	0.50	-65.22
	0.80	-141.80	0.80	-139.37	0.81	-143.45
	1.20	-221.84	1.20	-219.01	1.21	-223.62
	1.61	-304.46	1.61	-308.37	1.61	-307.92
	2.02	-347.18	2.00	-355.90	2.02	-341.45
	2.40	-324.62	2.40	-326.98	2.43	-310.64
	2.79	-276.95	2.79	-276.79	2.83	-272.26
	2.83	-276.11	2.83	-276.05	2.83	-272.25
	3.21	-269.41	3.19	-269.42	3.20	-274.56
	3.60	-311.50	3.59	-305.84	3.58	-309.14
	3.99	-333.21	3.99	-331.10	3.98	-331.63
	4.39	-299.23	4.39	-303.13	4.39	-300.91
	4.80	-251.42	4.79	-257.93	4.80	-252.01
	5.17	-250.56	5.17	-253.63	5.17	-247.88
	5.21	-250.47	5.20	-253.14	5.20	-247.45
	5.61	-285.31	5.59	-287.45	5.61	-281.54
	6.01	-300.97	5.99	-302.98	6.02	-296.10
	6.41	-240.71	6.39	-244.81	6.42	-234.62
	6.82	-154.54	6.79	-147.35	6.81	-151.30
	7.20	-108.00	7.20	-74.17	7.21	-73.94
	7.50	-36.47	7.50	-12.54	7.50	-15.04
	7.58	-16.90	7.59	7.11	7.60	5.75
	7.99	-70.38	7.99	-60.00	8.00	-86.13
	8.00	-71.88	8.00	-61.29		



Depth	EP (psf) y = 4	Difference
15	-333.21	-0.64%
20	-331.10	0.00
25	-331.63	

Appendix C: Verification in Determining Earth Pressure Tabulated Data

The following pages provide the tabulated data from the verification models run to examine the determination of earth pressure against a rigid retaining wall as compared to linear elastic finite element results.

Table C-1 Vertical stress against a rigid retaining wall for γ1

γ1 = 124 pcf

h (ft)	Model (psf)	γh (psf)
0.00	-47.27	0.00
0.50	-77.66	-62.00
0.78	-94.75	-96.88
1.56	-189.81	-193.75
2.34	-284.97	-290.63
2.83	-344.61	-351.29
3.13	-380.21	-387.50
3.91	-475.93	-484.38
4.69	-572.39	-581.25
5.17	-631.79	-640.71
5.47	-669.18	-678.13
6.25	-766.05	-775.00
7.03	-862.96	-871.88
7.50	-921.12	-930.00
7.81	-959.89	-968.75
8.00	-983.15	-992.00

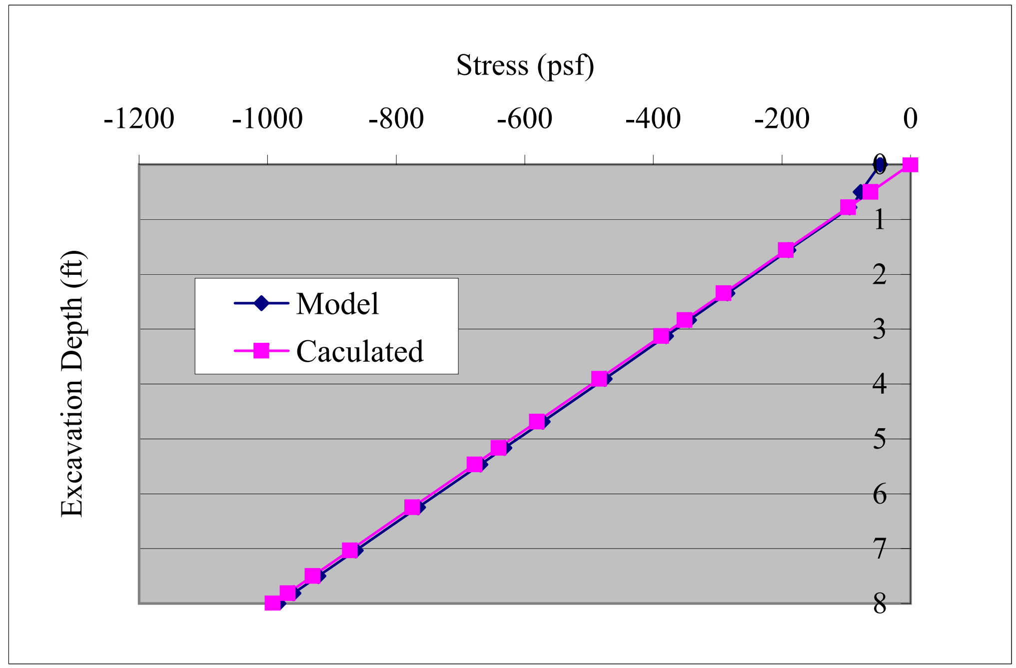


Table C-2 Horizontal earth pressure against a rigid retaining wall for $\gamma 1$

$\gamma 1$ = 124 pcf

ν = 0.3

$k_o = \nu/(1-\nu)$ = 0.428571429

h (ft)	Model (psf)	$k_o\gamma h$ (psf)
0.00	-18.51	0.00
0.50	-31.97	-26.57
0.78	-39.55	-41.52
1.56	-81.21	-83.04
2.34	-122.43	-124.55
2.83	-148.37	-150.55
3.13	-163.86	-166.07
3.91	-204.97	-207.59
4.69	-245.66	-249.11
5.17	-270.94	-274.59
5.47	-286.84	-290.63
6.25	-328.33	-332.14
7.03	-369.82	-373.66
7.50	-394.73	-398.57
7.81	-411.34	-415.18
8.00	-421.31	-425.14

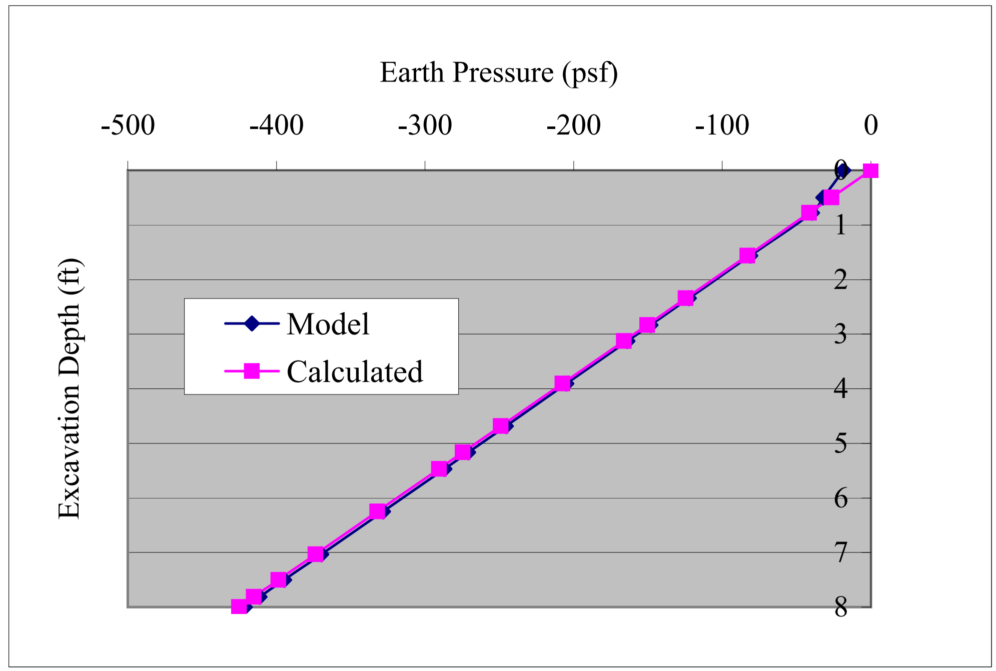


Table C-3 Vertical stress against a rigid retaining wall for γ2

γ2 = 119 pcf

h (ft)	Model (psf)	γh (psf)
0.00	-47.78	0.00
0.50	-76.69	-59.50
0.83	-95.97	-99.17
1.67	-192.53	-198.33
2.50	-289.23	-297.50
2.83	-327.94	-337.13
3.33	-386.09	-396.67
4.17	-483.71	-495.83
5.00	-582.40	-595.00
5.17	-602.31	-614.87
5.83	-681.76	-694.17
6.67	-781.34	-793.33
7.50	-881.01	-892.50
8.00	-940.83	-952.00

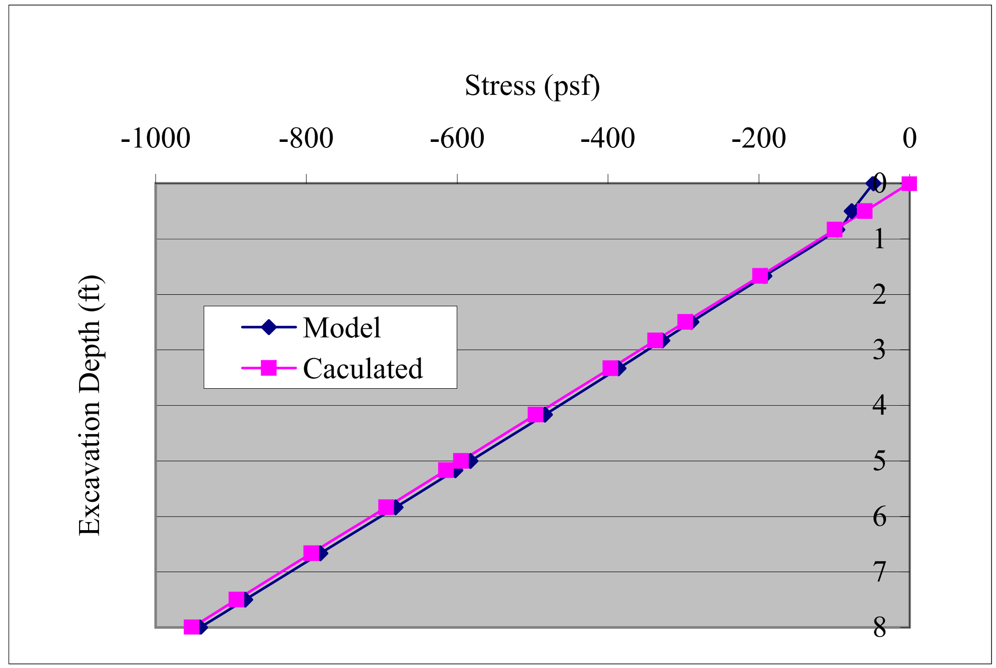


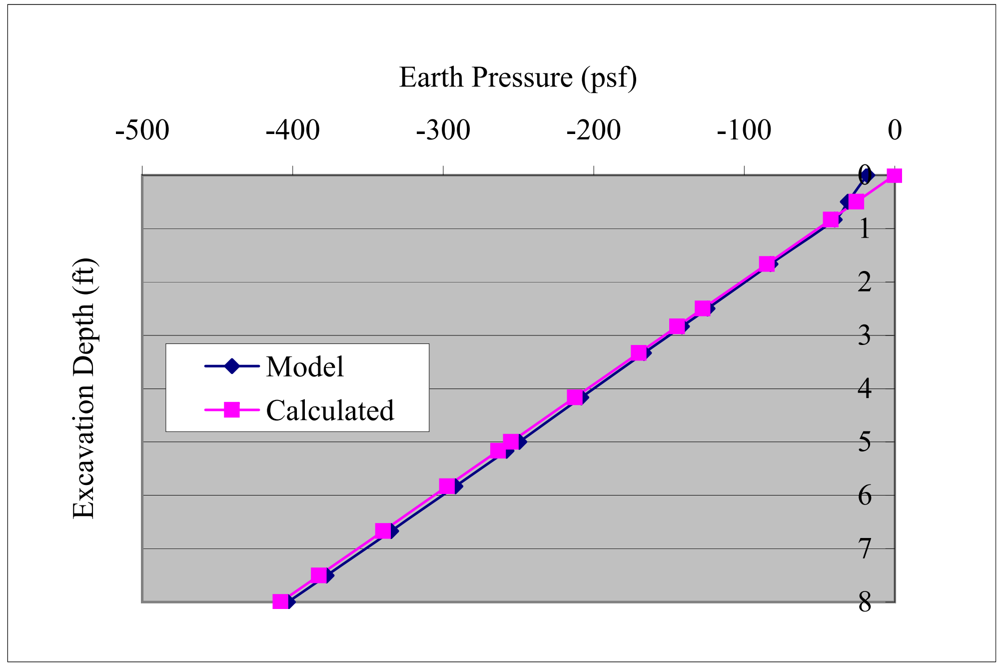
Table C-4 Horizontal earth pressure against a rigid retaining wall for $\gamma 2$

$\gamma 2 =$ 119 pcf

$\nu =$ 0.3

$k_o = \nu/(1-\nu) =$ 0.43

h (ft)	Model (psf)	$k_o \gamma h$ (psf)
0.00	-18.17	0.00
0.50	-31.20	-25.50
0.83	-39.89	-42.50
1.67	-82.57	-85.00
2.50	-124.48	-127.50
2.83	-141.36	-144.48
3.33	-166.73	-170.00
4.17	-208.27	-212.50
5.00	-249.56	-255.00
5.17	-258.06	-263.52
5.83	-291.97	-297.50
6.67	-334.71	-340.00
7.50	-377.46	-382.50
8.00	-403.11	-408.00



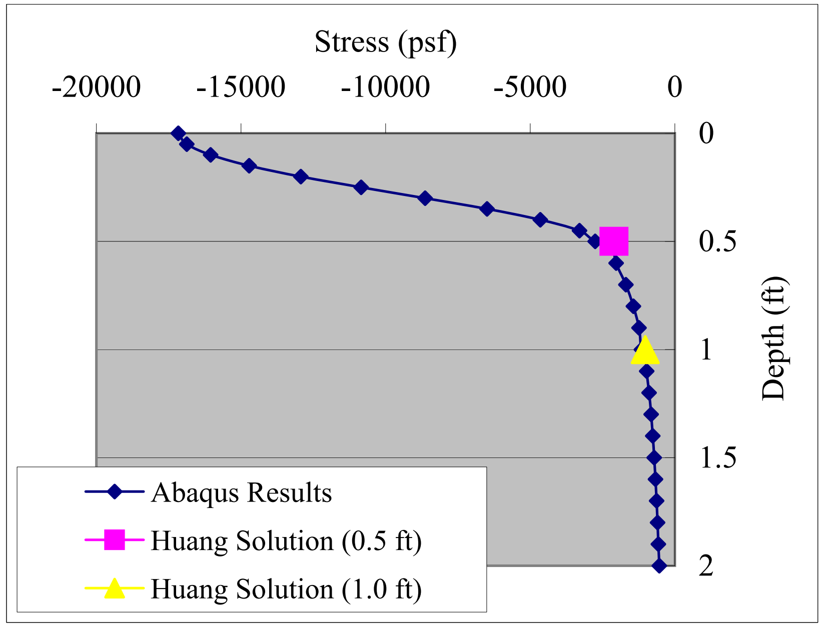
Appendix D: Verification in Determining the Response to Strut Loads

The following pages provide the tabulated data from the model run to verify the accuracy of an Abaqus model when determining the soil response to a uniformly distributed strut load over a small area, 5 in x 5 in (0.17 ft^2), through multiple materials with varying properties. The results are compared to the results from an example problem found in section 2.2.2 of Huang's 2004 *Pavement Analysis and Design*.

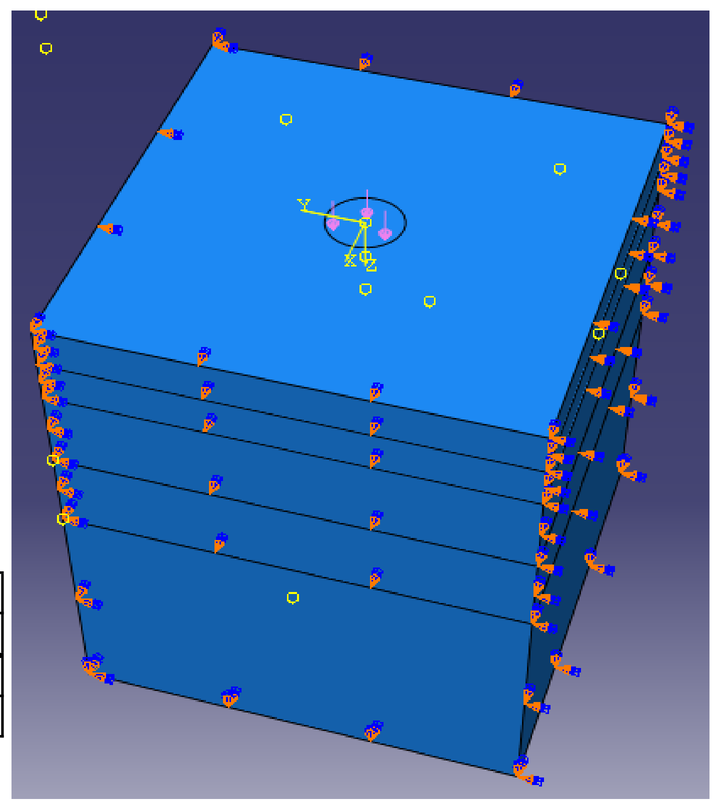
Table D-1 Verification of Abaqus model ability to determine the response to strut loads

Abaqus Results

h (ft)	Model (psf)
0.00	-17169.20
0.05	-16874.70
0.10	-16051.90
0.15	-14720.50
0.20	-12931.90
0.25	-10846.20
0.30	-8634.83
0.35	-6495.75
0.40	-4652.52
0.45	-3296.73
0.50	-2756.23
0.50	-2216.63
0.60	-2028.30
0.70	-1693.45
0.80	-1434.15
0.90	-1238.87
1.00	-1156.36
1.00	-1020.93
1.10	-975.24
1.20	-891.42
1.30	-821.15
1.40	-761.64
1.50	-710.81
1.60	-667.09
1.70	-629.27
1.80	-596.40
1.90	-567.70
2.00	-537.69



	Huang	
Depth (ft)	psi	psf
0.5	-14.61	-2103.84
1	-7.12	-1025.28



See Example 2.11 from Huang's *Pavement Design and Anaylsis, Second Edition*

Appendix E: Parametric Study Tabulated Data – Varying Strut Pressure

The following pages provide the tabulated data from the models run to evaluate the effects of varying strut load. Material properties were held constant and strut loading was evaluated at 1298 lbs/strut, 2356 lbs/strut and 4712 lbs/strut.

Table E-1 Earth pressure behind the horizontal width of the panel at depths y = 2 ft, y = 4 ft, and y = 6 ft when strut loading is 1298 lbs/strut

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.92	-3.50	-1.18	-3.50	-2.98
-2.95	-1.49	-2.95	-0.88	-2.95	2.40
-2.36	0.88	-2.36	2.25	-2.36	11.71
-1.77	4.16	-1.77	18.29	-1.77	37.45
-1.18	-134.70	-1.18	-120.37	-1.18	-104.16
-0.59	-334.98	-0.59	-321.28	-0.59	-304.48
0.00	-409.05	0.00	-393.03	0.00	-376.93
0.59	-334.99	0.59	-321.18	0.59	-304.50
1.18	-135.06	1.18	-120.67	1.18	-104.48
1.77	3.98	1.77	18.11	1.77	37.29
2.36	0.93	2.36	2.30	2.36	11.76
2.95	-1.49	2.95	-0.88	2.95	2.40
3.50	-0.91	3.50	-1.18	3.50	-2.98

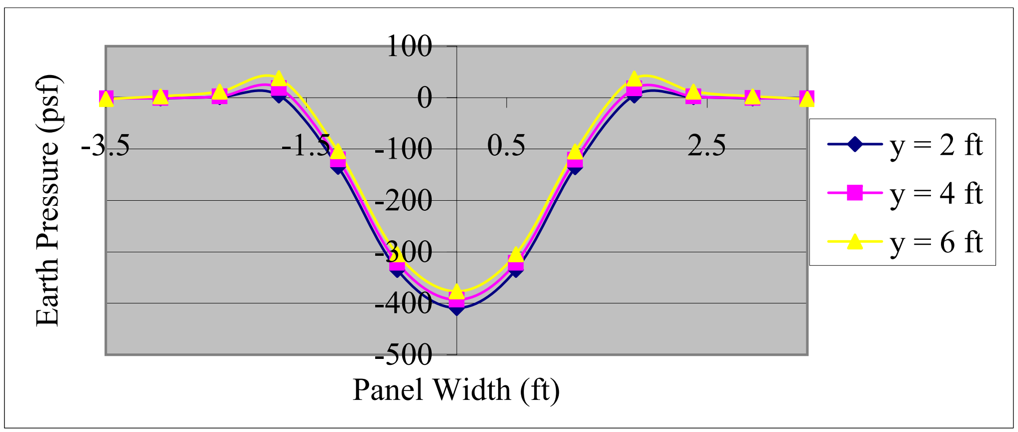


Table E-2 Earth pressure behind the vertical height of the panel for 1298 lbs/strut loading

Depth (ft)	EP (psf)	Depth (ft)	EP (psf)
0.00	99.41	4.62	-263.78
0.20	40.55	4.82	-229.11
0.40	-39.94	5.02	-222.35
0.50	-60.08	5.16	-234.72
0.60	-78.72	5.21	-238.64
0.80	-112.81	5.40	-271.17
0.99	-151.36	5.62	-320.78
1.19	-200.79	5.82	-364.52
1.40	-262.06	6.01	-377.09
1.60	-329.10	6.20	-340.54
1.80	-385.70	6.40	-273.26
2.00	-409.48	6.61	-201.83
2.20	-387.06	6.82	-135.25
2.40	-337.41	7.02	-89.20
2.60	-284.26	7.22	-61.82
2.80	-245.28	7.42	-44.28
2.83	-243.54	7.50	-26.26
3.01	-232.60	7.62	-1.92
3.21	-246.05	7.82	48.77
3.41	-282.24	8.00	-78.90
3.61	-333.63		
3.81	-379.37		
4.01	-393.79		
4.21	-367.66		
4.41	-316.32		

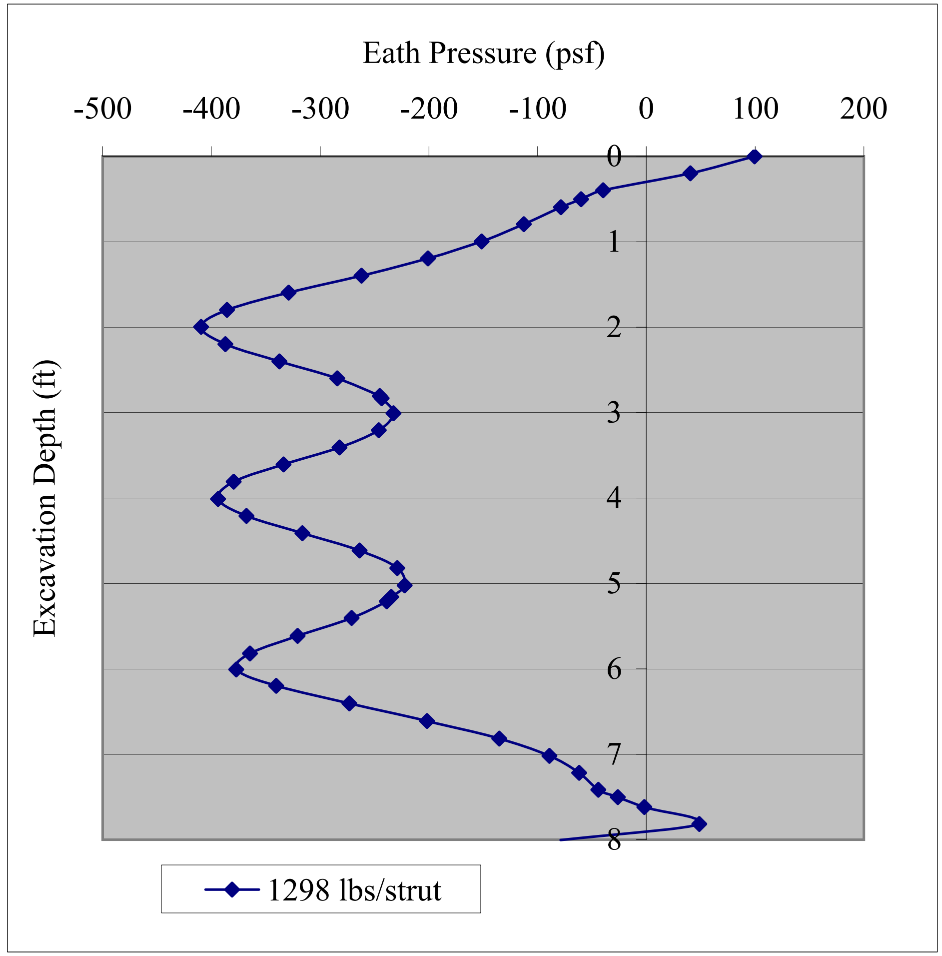


Table E-3 Earth pressure behind the horizontal width of the panel at depths y = 2 ft, y = 4 ft, and y = 6 ft when strut loading is 2356 lbs/strut

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.46	-3.50	-0.71	-3.50	-2.53
-2.95	-4.36	-2.95	-3.92	-2.95	0.57
-2.36	-1.91	-2.36	-0.81	-2.36	11.09
-1.77	4.06	-1.77	17.84	-1.77	39.51
-1.18	-244.76	-1.18	-230.60	-1.18	-212.15
-0.59	-607.78	-0.59	-593.39	-0.59	-575.30
0.00	-739.63	0.00	-722.83	0.00	-705.37
0.59	-607.73	0.59	-593.52	0.59	-575.21
1.18	-244.11	1.18	-230.01	1.18	-211.50
1.77	4.39	1.77	18.18	1.77	39.83
2.36	-1.99	2.36	-0.90	2.36	11.01
2.95	-4.36	2.95	-3.92	2.95	0.56
3.50	-0.46	3.50	-0.71	3.50	-2.53

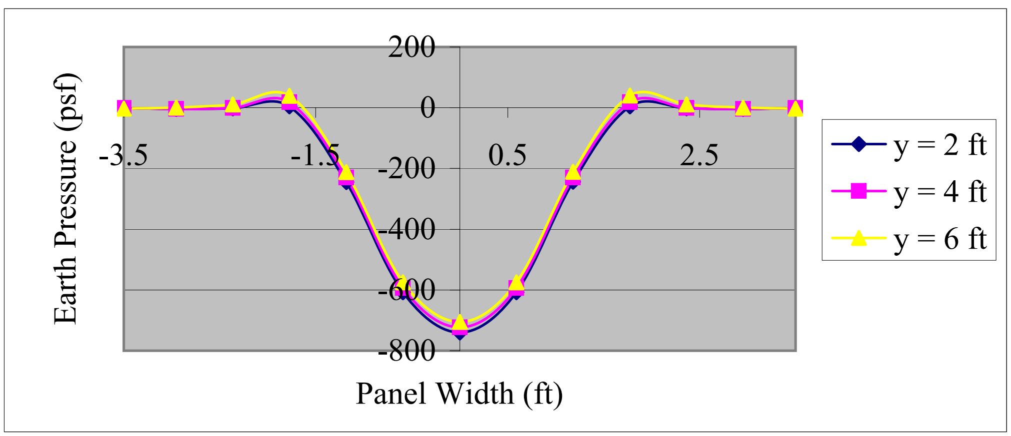


Table E -4 Earth pressure behind the vertical height of the panel for 2356 lbs/strut loading

Depth (ft)	EP (psf)	Depth (ft)	EP (psf)
0.00	170.61	4.41	-586.92
0.20	88.02	4.62	-492.80
0.40	-32.90	4.82	-429.89
0.50	-70.13	5.02	-414.47
0.60	-104.55	5.16	-434.22
0.80	-172.35	5.21	-440.88
0.99	-249.46	5.40	-506.32
1.19	-346.01	5.62	-602.27
1.40	-462.93	5.82	-680.42
1.60	-588.53	6.01	-705.88
1.80	-694.54	6.20	-648.79
2.00	-740.35	6.40	-533.13
2.20	-701.69	6.61	-401.49
2.40	-613.97	6.82	-275.70
2.60	-519.61	7.02	-183.35
2.80	-450.64	7.22	-122.11
2.83	-447.56	7.42	-73.75
3.01	-428.09	7.50	-43.93
3.21	-451.02	7.62	-3.79
3.41	-516.36	7.82	83.19
3.61	-611.64	8.00	-39.92
3.81	-696.58		
4.01	-724.31		
4.21	-678.34		

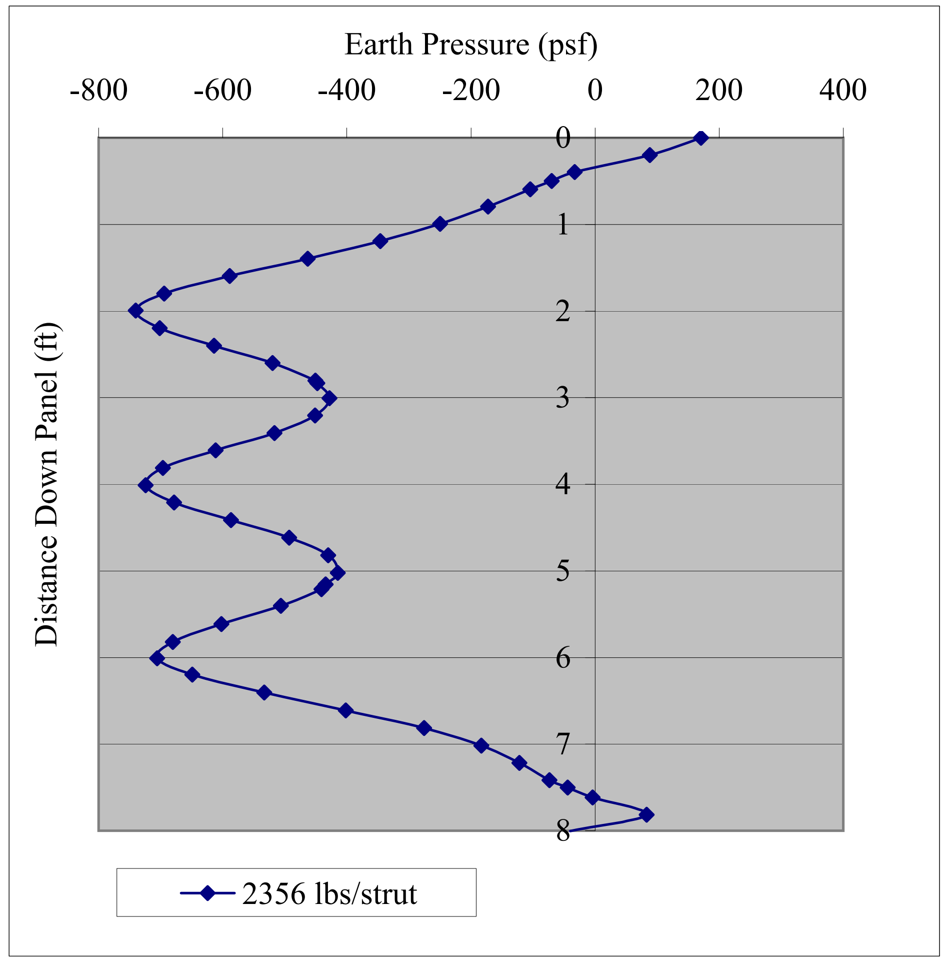


Table E-5 Earth pressure behind the horizontal width of the panel at depths y = 2 ft, y = 4 ft, and y = 6 ft when strut loading is 4712 lbs/strut

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	0.57	-3.50	0.37	-3.50	-1.51
-2.95	-10.89	-2.95	-10.83	-2.95	-3.60
-2.36	-8.36	-2.36	-7.89	-2.36	9.57
-1.77	4.26	-1.77	17.21	-1.77	44.57
-1.18	-494.33	-1.18	-480.68	-1.18	-457.10
-0.59	-1228.40	-0.59	-1212.64	-0.59	-1191.38
0.00	-1491.67	0.00	-1473.11	0.00	-1452.57
0.59	-1228.24	0.59	-1212.84	0.59	-1191.11
1.18	-493.02	1.18	-479.42	1.18	-455.71
1.77	4.92	1.77	17.92	1.77	45.25
2.36	-8.52	2.36	-8.07	2.36	9.40
2.95	-10.89	2.95	-10.84	2.95	-3.61
3.50	0.56	3.50	0.37	3.50	-1.50

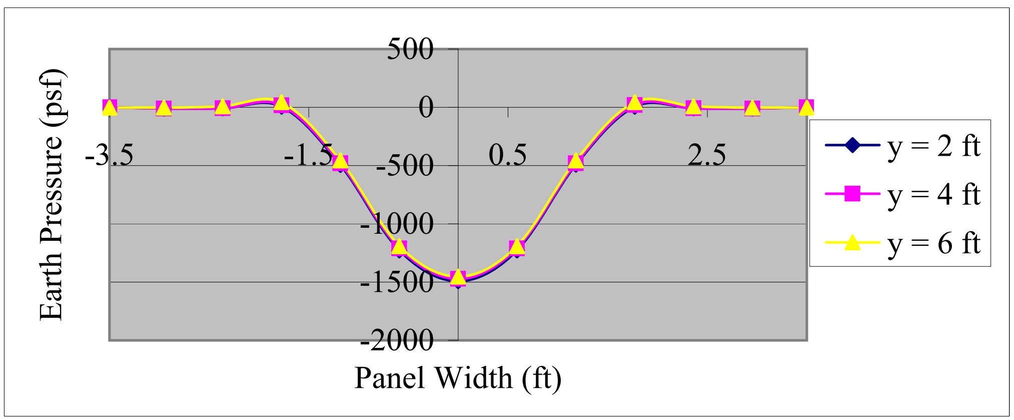


Table E -6 Earth pressure behind the vertical height of the panel for 4712 lbs/strut loading

Depth (ft)	EP (psf)	Depth (ft)	EP (psf)
0.00	332.59	4.21	-1385.14
0.20	196.02	4.41	-1202.51
0.40	-16.88	4.62	-1013.80
0.50	-93.01	4.82	-886.67
0.60	-163.31	5.02	-851.55
0.80	-307.79	5.16	-888.08
0.99	-472.66	5.21	-900.98
1.19	-676.38	5.40	-1041.29
1.40	-919.92	5.62	-1242.65
1.60	-1178.74	5.82	-1399.09
1.80	-1397.16	6.01	-1453.85
2.00	-1493.09	6.20	-1350.08
2.20	-1417.47	6.40	-1124.34
2.40	-1243.13	6.61	-855.73
2.60	-1055.01	6.82	-595.22
2.80	-917.83	7.02	-397.55
2.83	-911.71	7.22	-259.29
3.01	-872.83	7.42	-140.80
3.21	-917.32	7.50	-84.13
3.41	-1048.99	7.62	-8.06
3.61	-1244.10	7.82	161.50
3.81	-1418.24	8.00	48.76
4.01	-1476.23		

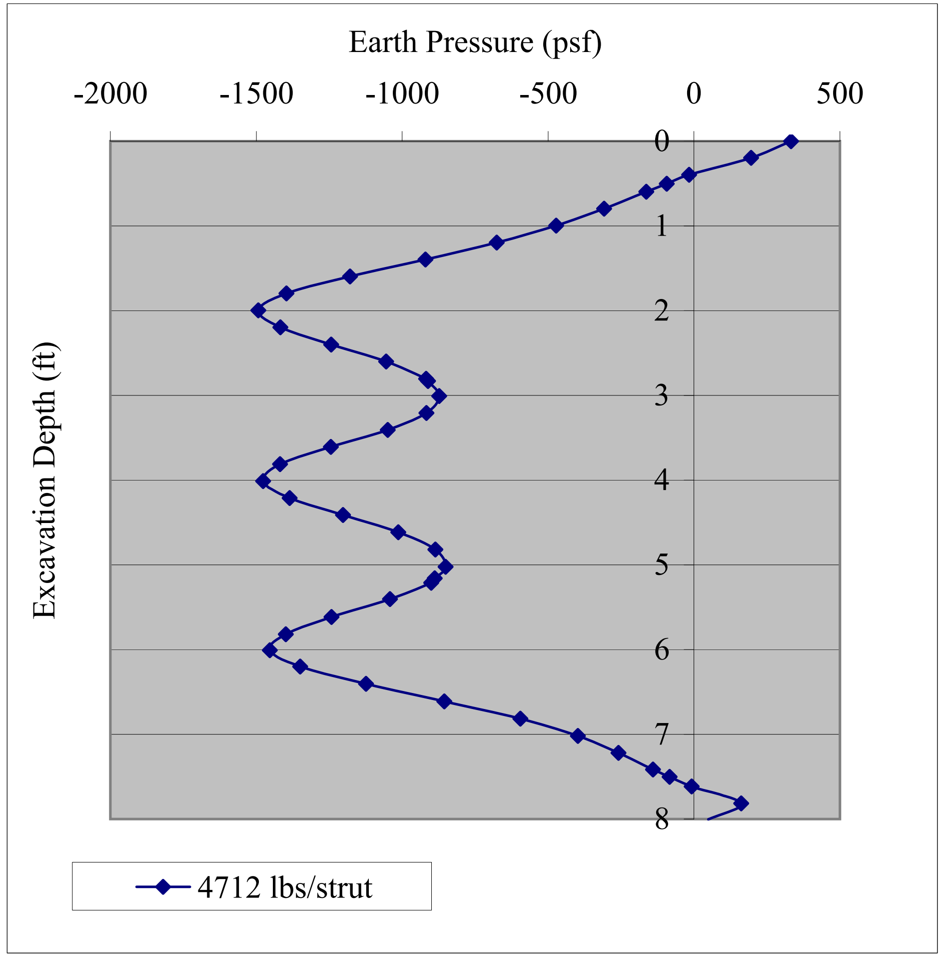


Table E-7 Strut load vs earth pressure at a depth of y = 4 ft

Strut Load (lbs/strut)	EP (psf)
1298	393
2356	723
4712	1473

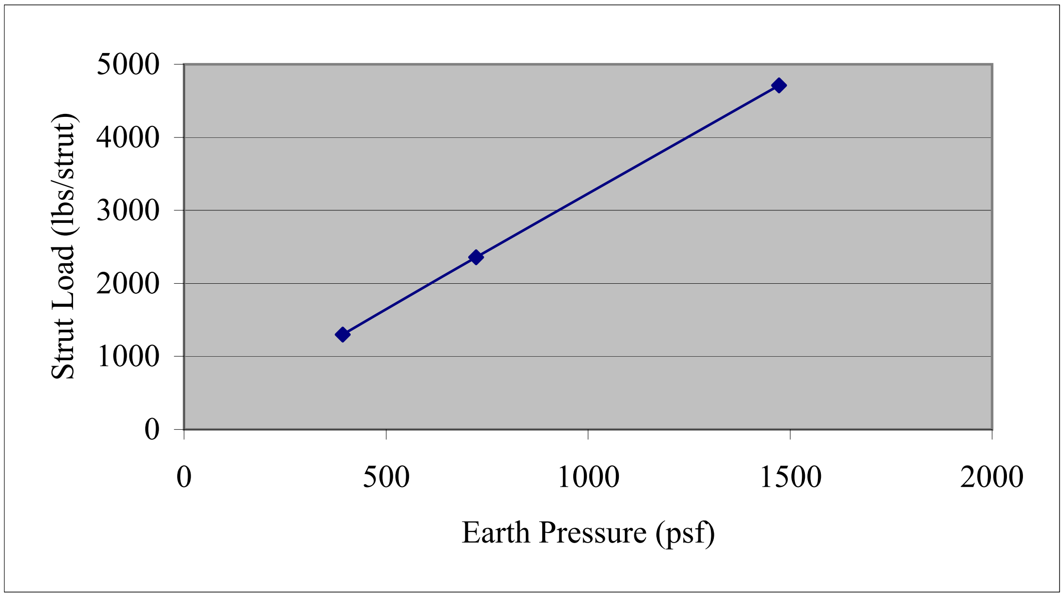


Figure E-1 Combined earth pressure behind the horizontal width of the panel at depth y = 4 ft for variable strut loads

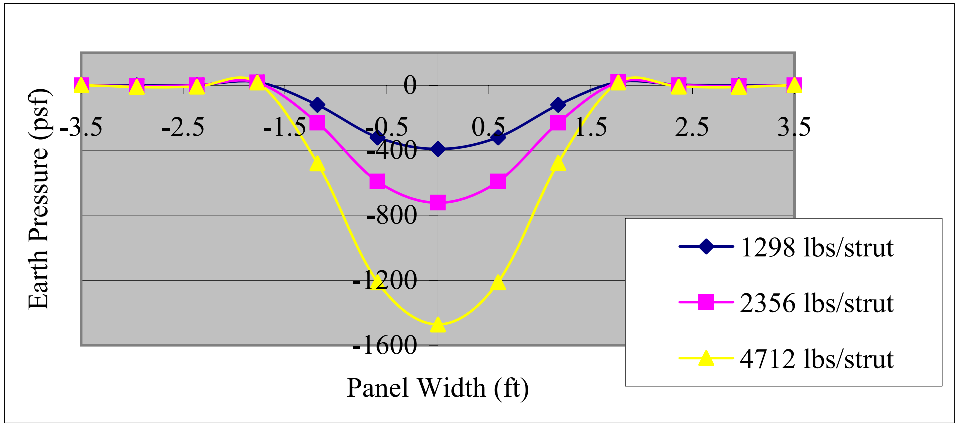
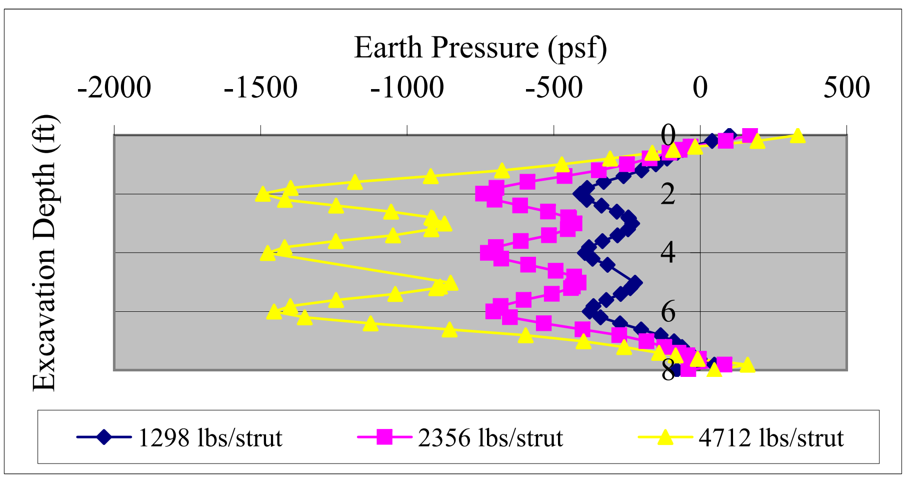


Figure E-2 Combined earth pressure behind the vertical height of the center of the panel for variable strut loads



Appendix F: Parametric Study Tabulated Data – Varying Panel Stiffness

The following pages provide the tabulated data from the models run to evaluate the effects of varying panel stiffness. Soil and strongback properties were held constant. Panel thickness was held constant while stiffness was set to 7.31 x 10^7 psf, 1.46 x 10^8 psf, 2.92 x 10^8 psf, 4.38 x 10^8 psf and then 5.86 x 10^8 psf. Strut loading was set to 1298 lbs/strut for all iterations.

Table F-1 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6ft with no shoring in place between the struts and trench wall

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	1.86	-3.50	1.15	-3.50	-0.22
-2.90	-32.29	-2.90	-28.72	-2.90	-29.21
-2.01	50.53	-2.01	46.28	-2.01	47.55
-1.11	-84.38	-1.11	-76.33	-1.11	-78.56
-0.21	-2665.58	-0.21	-2751.04	-0.21	-2824.78
-0.21	-2690.29	-0.21	-2757.14	-0.21	-2829.55
0.21	-2692.51	0.21	-2759.54	0.21	-2832.01
0.21	-2667.82	0.21	-2753.45	0.21	-2827.25
1.11	-83.76	1.11	-75.55	1.11	-77.51
2.01	51.52	2.01	47.32	2.01	48.65
2.90	-32.91	2.90	-29.37	2.90	-29.89
3.50	1.88	3.50	1.17	3.50	-0.21

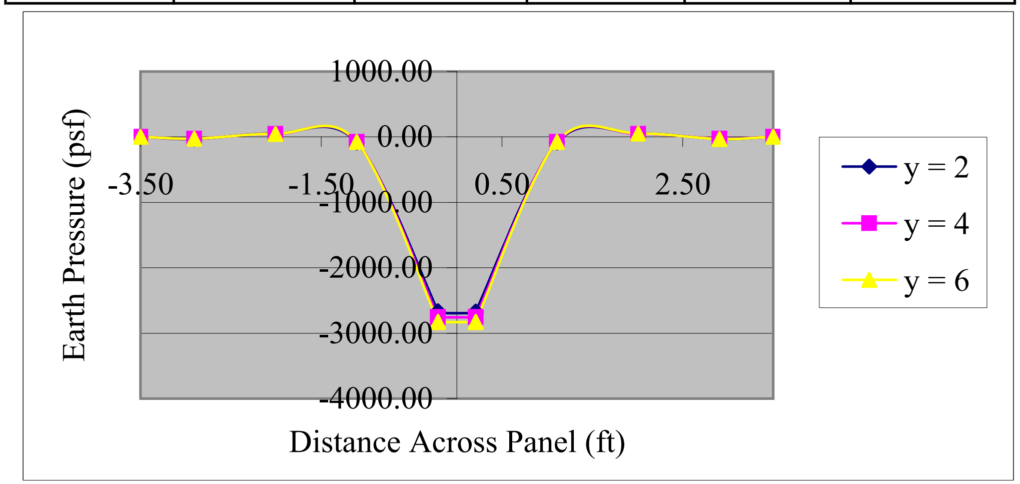


Table F-2 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6ft with a strongback only (no panel) between the struts and trench wall

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-5.53	-3.50	-6.42	-3.50	-9.19
-2.95	9.13	-2.95	9.25	-2.95	10.40
-2.36	-14.86	-2.36	-14.95	-2.36	-16.83
-1.77	25.24	-1.77	24.97	-1.77	23.93
-1.18	-68.29	-1.18	-67.71	-1.18	-53.75
-0.59	-797.78	-0.59	-770.65	-0.59	-807.88
0.00	-1338.63	0.00	-1293.48	0.00	-1395.52
0.59	-818.76	0.59	-790.75	0.59	-831.26
1.18	-71.67	1.18	-71.04	1.18	-56.66
1.77	26.87	1.77	26.55	1.77	25.50
2.36	-15.78	2.36	-15.82	2.36	-17.78
2.95	9.68	2.95	9.77	2.95	11.01
3.50	-5.82	3.50	-6.68	3.50	-9.52

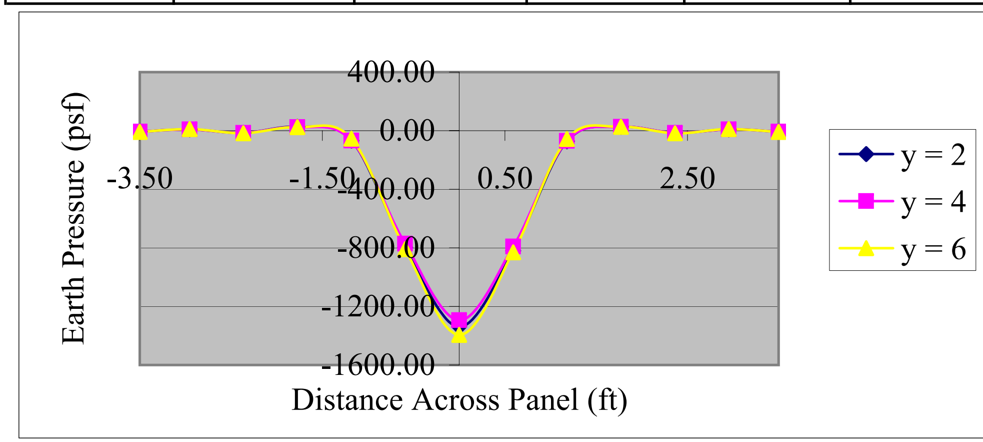


Table F-3 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6ft with panel stiffness = 7.31 x 10^7 psf (0.25 x E FinnForm)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.12	-3.50	-1.46	-3.50	-3.32
-2.95	-2.10	-2.95	-2.56	-2.95	0.12
-2.36	-3.08	-2.36	-4.52	-2.36	1.58
-1.77	24.72	-1.77	36.13	-1.77	53.41
-1.18	-102.94	-1.18	-89.14	-1.18	-72.89
-0.59	-400.25	-0.59	-378.96	-0.59	-366.93
0.00	-551.39	0.00	-521.72	0.00	-513.03
0.59	-400.40	0.59	-379.22	0.59	-367.21
1.18	-102.62	1.18	-88.94	1.18	-72.78
1.77	24.74	1.77	36.16	1.77	53.48
2.36	-3.17	2.36	-4.59	2.36	1.53
2.95	-2.08	2.95	-2.55	2.95	0.12
3.50	-1.14	3.50	-1.47	3.50	-3.33

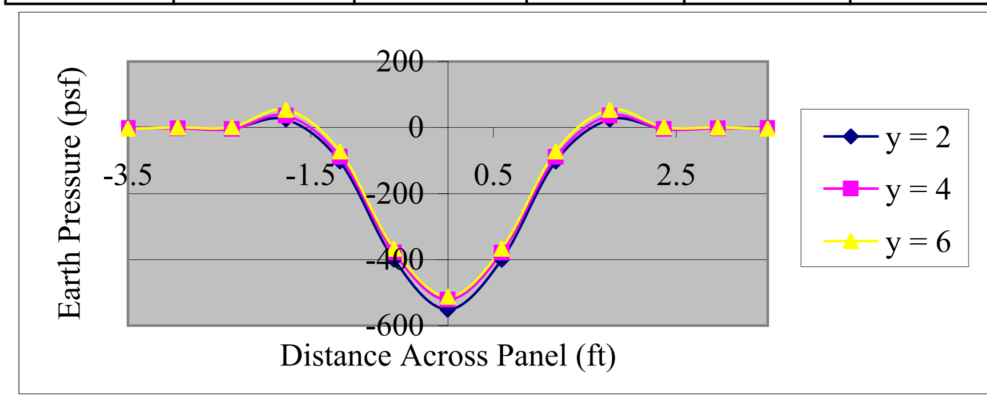


Table F-4 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6ft with panel stiffness = 1.46 x 10^8 psf (0.5 x E FinnForm)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.91	-3.50	-1.20	-3.50	-3.04
-2.95	-1.78	-2.95	-1.68	-2.95	1.36
-2.36	-0.02	-2.36	-0.02	-2.36	8.11
-1.77	17.15	-1.77	29.87	-1.77	48.20
-1.18	-120.52	-1.18	-105.97	-1.18	-90.05
-0.59	-365.05	-0.59	-347.04	-0.59	-332.12
0.00	-469.35	0.00	-446.53	0.00	-432.99
0.59	-364.95	0.59	-346.84	0.59	-331.99
1.18	-120.88	1.18	-106.25	1.18	-90.30
1.77	17.03	1.77	29.75	1.77	48.08
2.36	0.05	2.36	0.04	2.36	8.16
2.95	-1.78	2.95	-1.68	2.95	1.36
3.50	-0.90	3.50	-1.19	3.50	-3.04

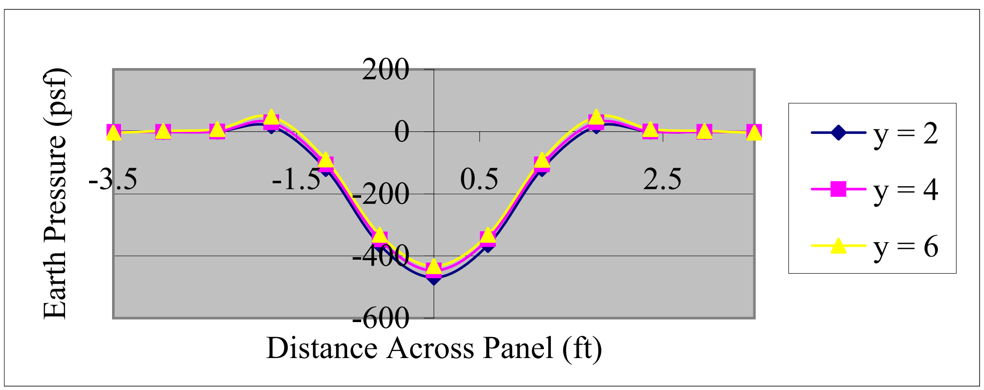


Table F-5 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6ft with panel stiffness = 2.92 x 10^8 psf (1.0 x E FinnForm)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.92	-3.50	-1.18	-3.50	-2.98
-2.95	-1.49	-2.95	-0.88	-2.95	2.40
-2.36	0.88	-2.36	2.25	-2.36	11.71
-1.77	4.16	-1.77	18.29	-1.77	37.45
-1.18	-134.70	-1.18	-120.37	-1.18	-104.16
-0.59	-334.98	-0.59	-321.28	-0.59	-304.48
0.00	-409.05	0.00	-393.03	0.00	-376.93
0.59	-334.99	0.59	-321.18	0.59	-304.50
1.18	-135.06	1.18	-120.67	1.18	-104.48
1.77	3.98	1.77	18.11	1.77	37.29
2.36	0.93	2.36	2.30	2.36	11.76
2.95	-1.49	2.95	-0.88	2.95	2.40
3.50	-0.91	3.50	-1.18	3.50	-2.98

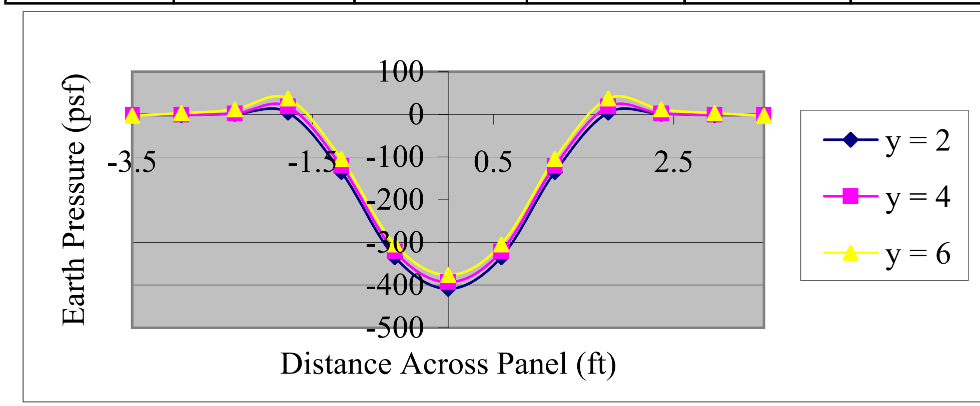


Table F-6 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6ft with panel stiffness = 4.38 x 10^8 psf (1.50 x E FinnForm)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.02	-3.50	-1.28	-3.50	-3.04
-2.95	-1.42	-2.95	-0.57	-2.95	2.85
-2.36	0.07	-2.36	2.21	-2.36	12.23
-1.77	-6.38	-1.77	8.63	-1.77	28.30
-1.18	-141.67	-1.18	-127.71	-1.18	-111.27
-0.59	-318.45	-0.59	-307.19	-0.59	-289.98
0.00	-380.80	0.00	-368.09	0.00	-351.51
0.59	-318.53	0.59	-307.17	0.59	-290.08
1.18	-142.04	1.18	-128.01	1.18	-111.61
1.77	-6.59	1.77	8.43	1.77	28.11
2.36	0.10	2.36	2.24	2.36	12.26
2.95	-1.42	2.95	-0.57	2.95	2.85
3.50	-1.01	3.50	-1.28	3.50	-3.05

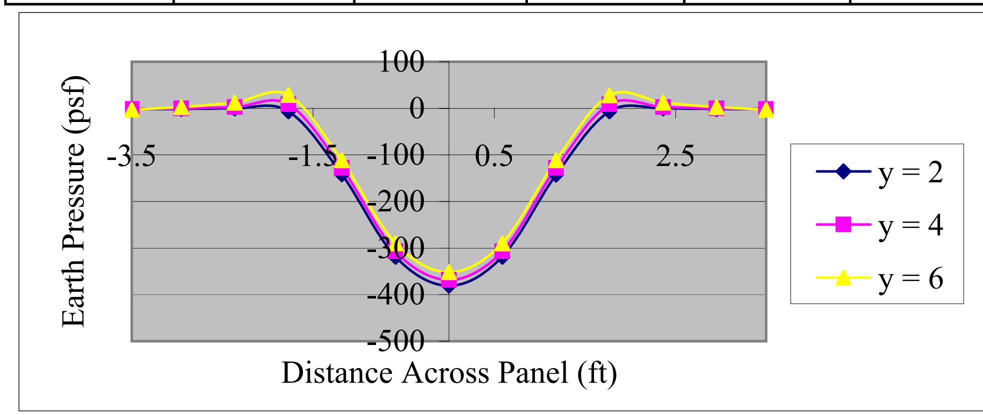


Table F-7 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6ft with panel stiffness = 5.86 x 10^8 psf (2.0 x E FinnForm)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.12	-3.50	-1.40	-3.50	-3.12
-2.95	-1.44	-2.95	-0.43	-2.95	3.09
-2.36	-1.15	-2.36	1.53	-2.36	11.89
-1.77	-15.47	-1.77	0.19	-1.77	20.25
-1.18	-146.65	-1.18	-132.91	-1.18	-116.37
-0.59	-306.92	-0.59	-297.12	-0.59	-280.04
0.00	-362.75	0.00	-352.00	0.00	-335.48
0.59	-306.79	0.59	-297.08	0.59	-279.89
1.18	-146.27	1.18	-132.60	1.18	-116.01
1.77	-15.26	1.77	0.39	1.77	20.45
2.36	-1.18	2.36	1.51	2.36	11.87
2.95	-1.44	2.95	-0.43	2.95	3.08
3.50	-1.12	3.50	-1.40	3.50	-3.12

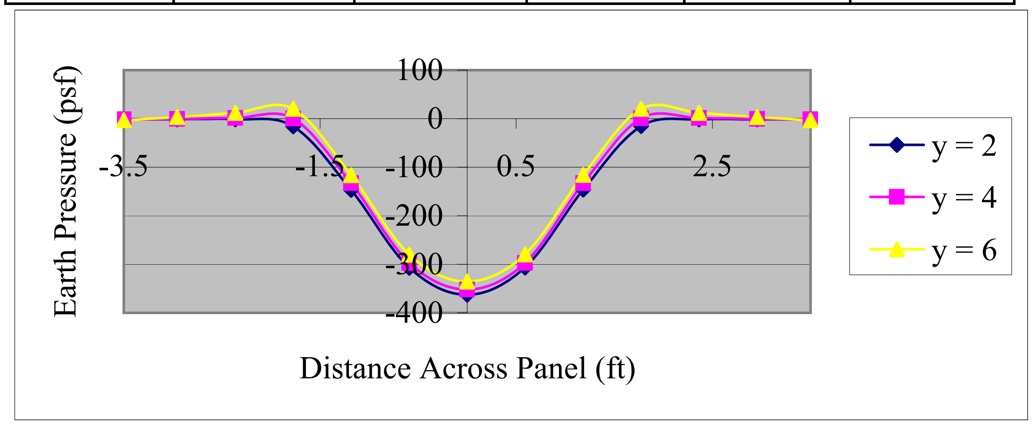


Figure F-1 Combined earth pressure behind the horizontal width of the panel at depth y = 4 ft for variable panel stiffness

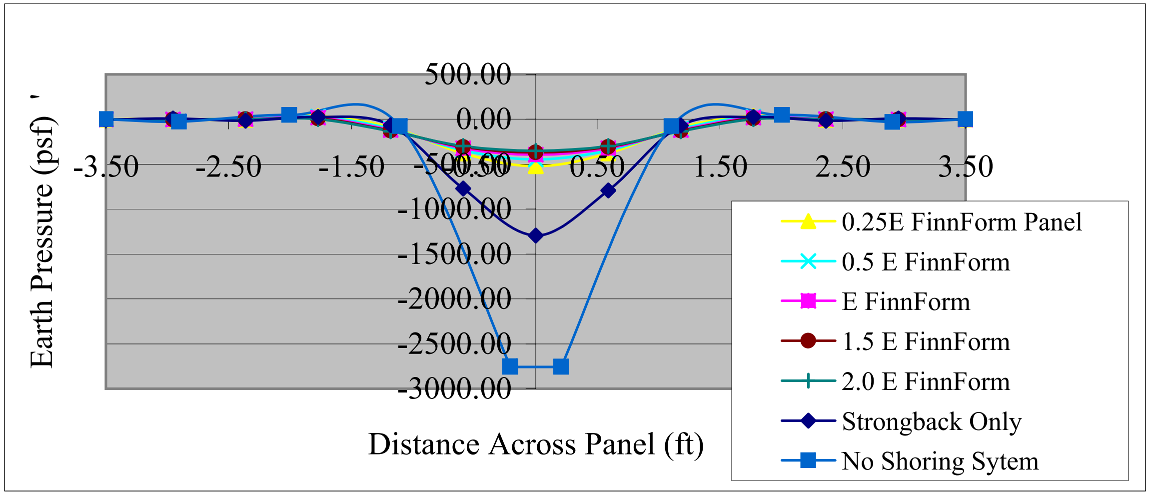
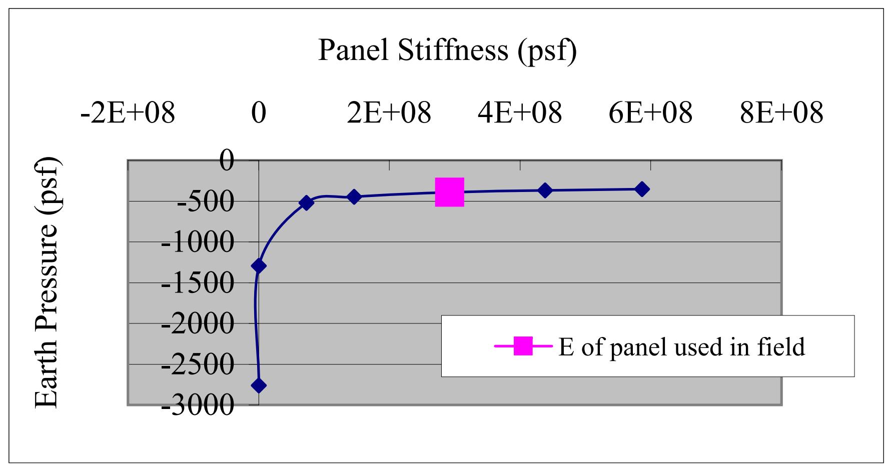


Table F-8 Panel stiffness vs earth pressure at a depth of y = 4 ft

* E FinnForm	Stiffness (psf)	EP (psf)
0	0	-2759.54
0	0	-1293.48
0.25	7.31E+07	-521.72
0.50	1.46E+08	-446.53
1.00	2.92E+08	-393.03
1.50	4.38E+08	-368.09
2.00	5.86E+08	-352.00



Appendix G: Parametric Study Tabulated Data – Varying Panel Thickness

The following pages provide the tabulated data from the models run to evaluate the effects of varying panel thickness. Soil and strongback material properties were held constant and strut loading was set to 1298 lbs/strut for all iterations. The panel stiffness was held at 2.92 x 10^8 psf while the panel thickness was set to 0.25 in, 0.50 in, 0.75 in, 1.00 in and 1.25 in.

Table G-1 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with struts only (no panel or strongback)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	1.86	-3.50	1.15	-3.50	-0.22
-2.90	-32.29	-2.90	-28.72	-2.90	-29.21
-2.01	50.53	-2.01	46.28	-2.01	47.55
-1.11	-84.38	-1.11	-76.33	-1.11	-78.56
-0.21	-2665.58	-0.21	-2751.04	-0.21	-2824.78
-0.21	-2690.29	-0.21	-2757.14	-0.21	-2829.55
0.21	-2692.51	0.21	-2759.54	0.21	-2832.01
0.21	-2667.82	0.21	-2753.45	0.21	-2827.25
1.11	-83.76	1.11	-75.55	1.11	-77.51
2.01	51.52	2.01	47.32	2.01	48.65
2.90	-32.91	2.90	-29.37	2.90	-29.89
3.50	1.88	3.50	1.17	3.50	-0.21

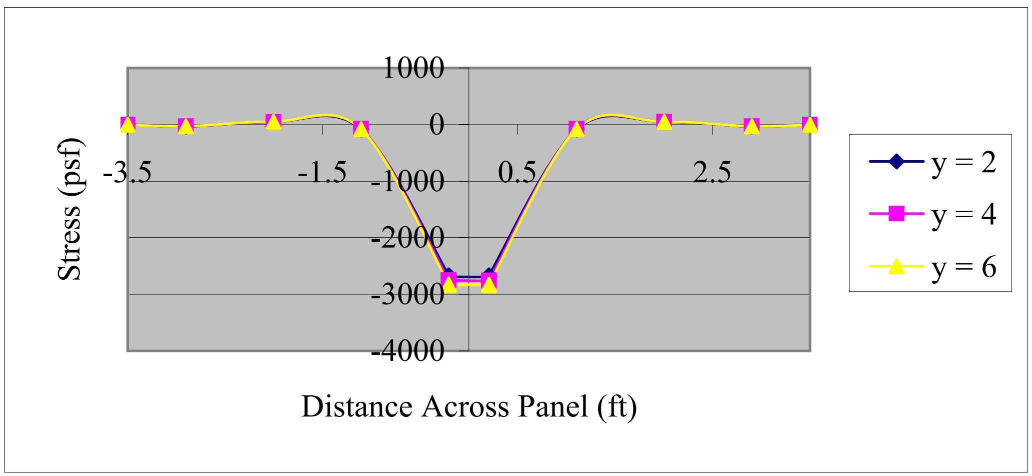


Table G-2 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with a strongback only (no panel) between the struts and trench wall

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-5.53	-3.50	-6.42	-3.50	-9.19
-2.95	9.13	-2.95	9.25	-2.95	10.40
-2.36	-14.86	-2.36	-14.95	-2.36	-16.83
-1.77	25.24	-1.77	24.97	-1.77	23.93
-1.18	-68.29	-1.18	-67.71	-1.18	-53.75
-0.59	-797.78	-0.59	-770.65	-0.59	-807.88
0.00	-1338.63	0.00	-1293.48	0.00	-1395.52
0.59	-818.76	0.59	-790.75	0.59	-831.26
1.18	-71.67	1.18	-71.04	1.18	-56.66
1.77	26.87	1.77	26.55	1.77	25.50
2.36	-15.78	2.36	-15.82	2.36	-17.78
2.95	9.68	2.95	9.77	2.95	11.01
3.50	-5.82	3.50	-6.68	3.50	-9.52

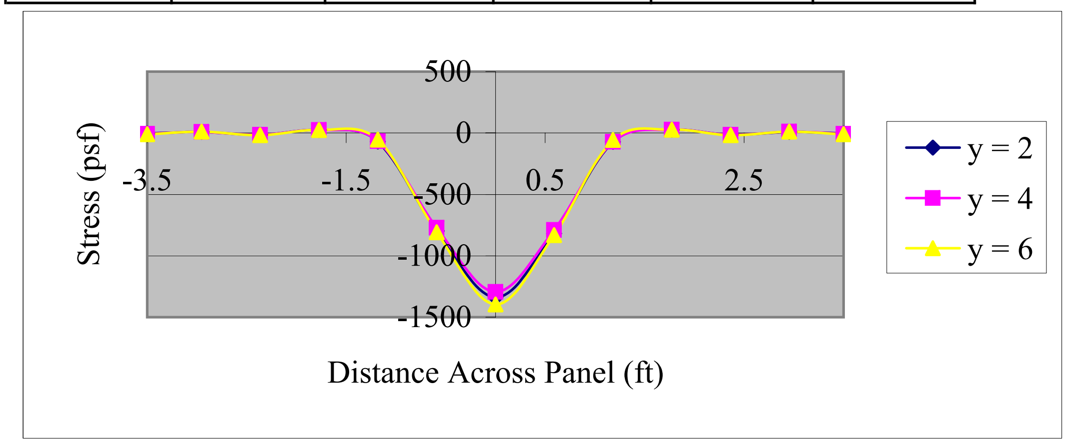


Table G-3 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel thickness = 0.25 in

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.61	-3.50	-2.07	-3.50	-3.76
-2.95	-2.60	-2.95	-3.06	-2.95	0.04
-2.36	-5.67	-2.36	-5.59	-2.36	-1.20
-1.77	10.25	-1.77	22.40	-1.77	33.53
-1.18	-116.37	-1.18	-102.78	-1.18	-93.68
-0.59	-426.36	-0.59	-408.59	-0.59	-400.90
0.00	-598.00	0.00	-575.34	0.00	-569.13
0.59	-428.15	0.59	-410.51	0.59	-402.43
1.18	-114.61	1.18	-101.23	1.18	-92.12
1.77	10.83	1.77	22.91	1.77	34.08
2.36	-5.67	2.36	-5.62	2.36	-1.18
2.95	-2.44	2.95	-2.92	2.95	0.14
3.50	-1.69	3.50	-2.15	3.50	-3.82

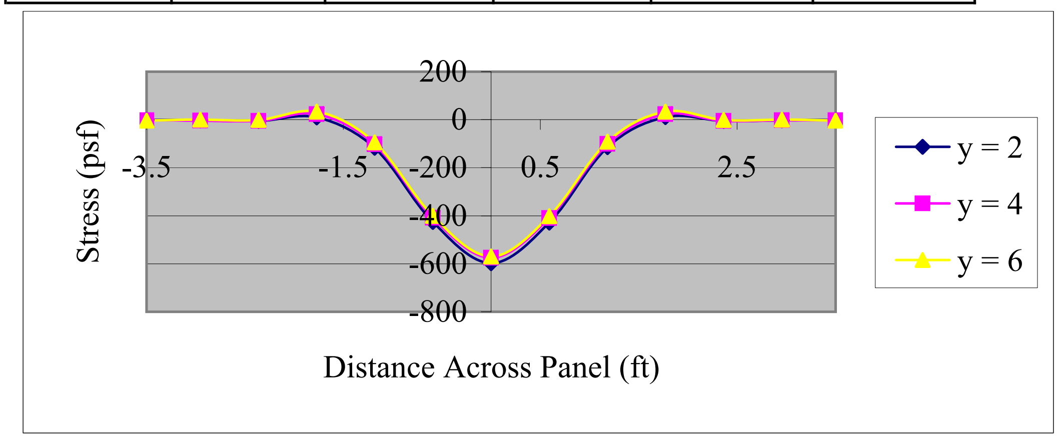


Table G-4 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel thickness = 0.50 in

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.89	-3.50	-1.20	-3.50	-2.98
-2.95	-2.50	-2.95	-2.37	-2.95	0.93
-2.36	-0.36	-2.36	0.30	-2.36	7.60
-1.77	15.27	-1.77	28.32	-1.77	44.34
-1.18	-125.27	-1.18	-111.47	-1.18	-97.80
-0.59	-371.68	-0.59	-354.99	-0.59	-342.62
0.00	-478.37	0.00	-457.27	0.00	-446.62
0.59	-372.25	0.59	-355.71	0.59	-343.09
1.18	-124.23	1.18	-110.56	1.18	-96.87
1.77	15.77	1.77	28.80	1.77	44.80
2.36	-0.46	2.36	0.21	2.36	7.53
2.95	-2.47	2.95	-2.34	2.95	0.95
3.50	-0.92	3.50	-1.21	3.50	-2.99

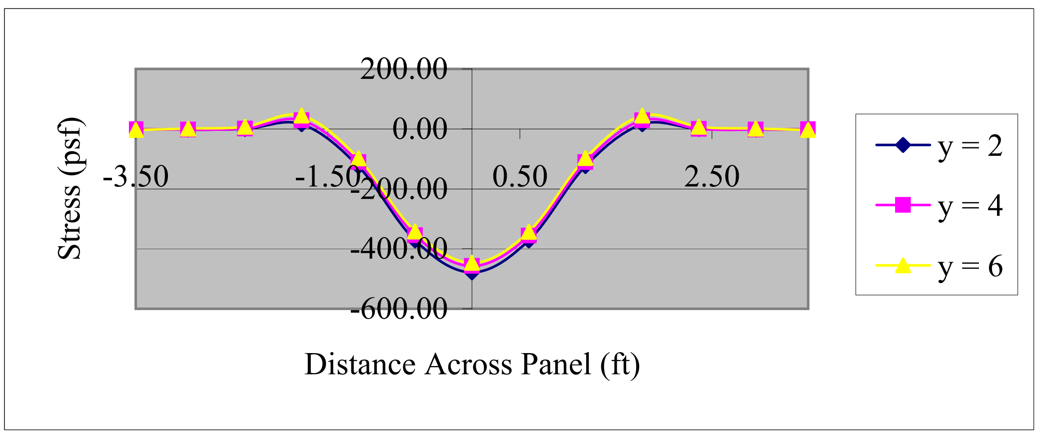


Table G-5 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel thickness = 0.75 in

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.92	-3.50	-1.18	-3.50	-2.98
-2.95	-1.49	-2.95	-0.88	-2.95	2.40
-2.36	0.88	-2.36	2.25	-2.36	11.71
-1.77	4.16	-1.77	18.29	-1.77	37.45
-1.18	-134.70	-1.18	-120.37	-1.18	-104.16
-0.59	-334.98	-0.59	-321.28	-0.59	-304.48
0.00	-409.05	0.00	-393.03	0.00	-376.93
0.59	-334.99	0.59	-321.18	0.59	-304.50
1.18	-135.06	1.18	-120.67	1.18	-104.48
1.77	3.98	1.77	18.11	1.77	37.29
2.36	0.93	2.36	2.30	2.36	11.76
2.95	-1.49	2.95	-0.88	2.95	2.40
3.50	-0.91	3.50	-1.18	3.50	-2.98

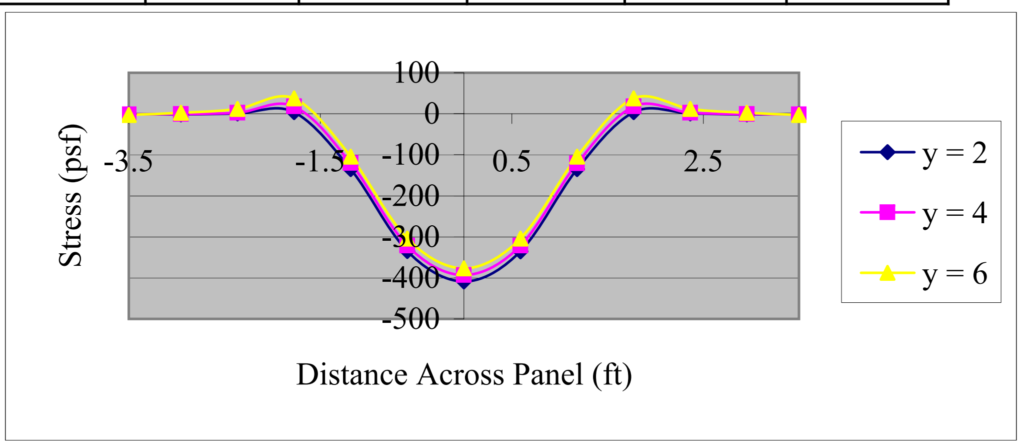


Table G-6 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel thickness = 1.00 in

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.17	-3.50	-1.45	-3.50	-3.27
-2.95	-0.61	-2.95	0.44	-2.95	3.78
-2.36	0.83	-2.36	3.22	-2.36	14.60
-1.77	-9.76	-1.77	5.41	-1.77	27.02
-1.18	-141.29	-1.18	-127.02	-1.18	-108.45
-0.59	-306.48	-0.59	-295.39	-0.59	-275.15
0.00	-363.28	0.00	-351.24	0.00	-331.27
0.59	-306.25	0.59	-295.22	0.59	-274.95
1.18	-141.18	1.18	-126.95	1.18	-108.38
1.77	-9.81	1.77	5.36	1.77	26.97
2.36	0.78	2.36	3.18	2.36	14.56
2.95	-0.61	2.95	0.44	2.95	3.78
3.50	-1.17	3.50	-1.45	3.50	-3.26

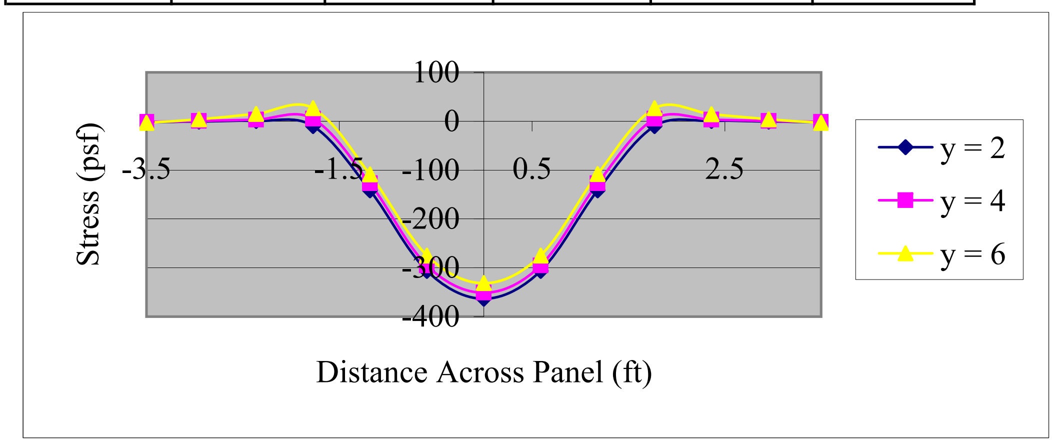


Table G-7 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel thickness = 1.25 in

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.43	-3.50	-1.74	-3.50	-3.58
-2.95	-0.10	-2.95	1.39	-2.95	4.85
-2.36	-0.03	-2.36	3.64	-2.36	16.84
-1.77	-21.88	-1.77	-5.92	-1.77	17.76
-1.18	-145.08	-1.18	-131.31	-1.18	-110.69
-0.59	-284.09	-0.59	-275.08	-0.59	-252.05
0.00	-330.40	0.00	-321.22	0.00	-298.50
0.59	-283.79	0.59	-274.81	0.59	-251.80
1.18	-145.06	1.18	-131.32	1.18	-110.72
1.77	-22.04	1.77	-6.08	1.77	17.59
2.36	-0.09	2.36	3.59	2.36	16.78
2.95	-0.10	2.95	1.38	2.95	4.84
3.50	-1.43	3.50	-1.74	3.50	-3.58

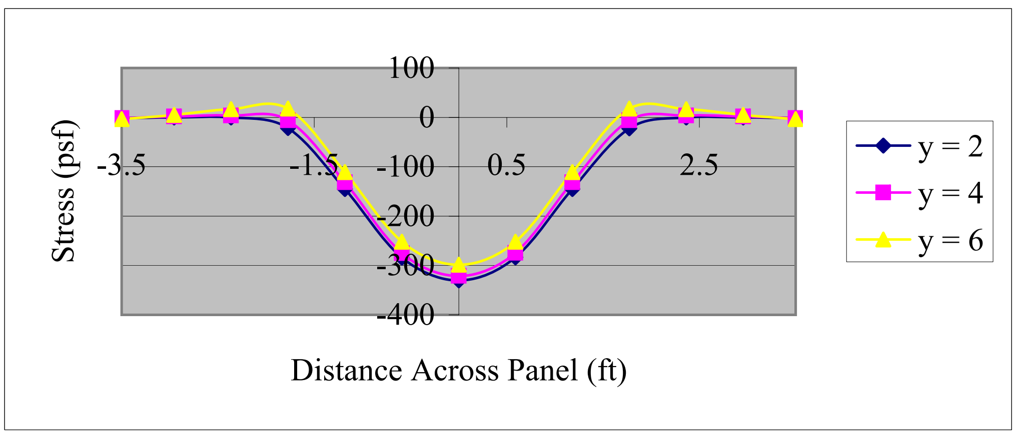


Figure G-1 Combined earth pressure behind the horizontal width of the panel at depth y = 4 ft for variable panel thickness

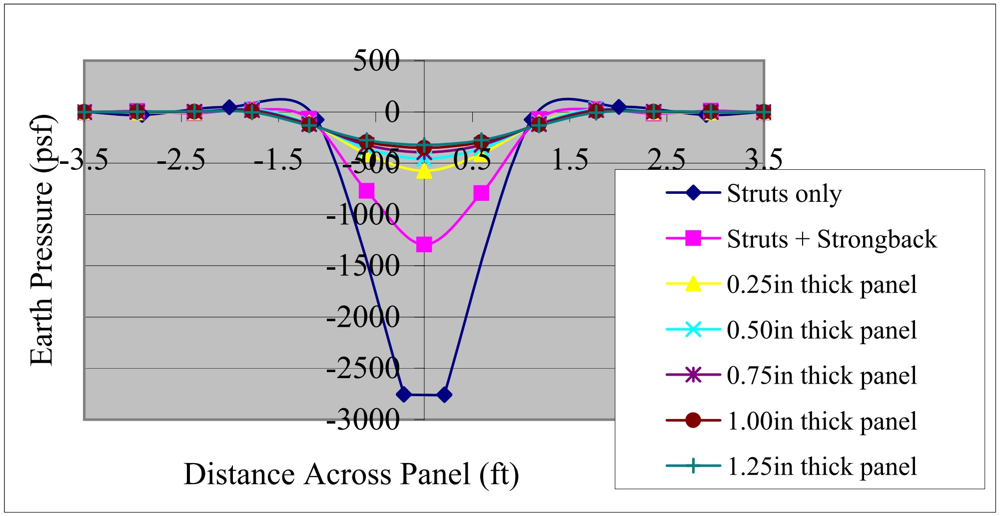
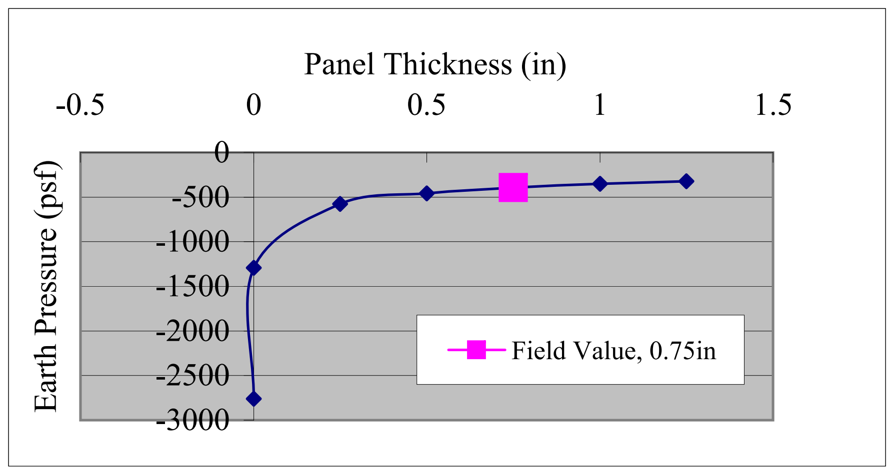


Table G-8: Panel thickness vs earth pressure at a depth of y = 4 ft

Panel Thickness (in)	Stiffness (psf)	EP (psf)
0	0.00	-2759.54
0	0.00	-1293.48
0.25	0.25	-575.34
0.50	0.50	-457.27
0.75	0.75	-393.03
1.00	1.00	-351.24
1.25	1.25	-321.22



Appendix H: Parametric Study Tabulated Data – Varying Panel Width

The following pages provide the tabulated data from the models run to evaluate the effects of varying panel thickness. Strongback material properties were held constant and strut loading was set to 1298 lbs/strut for all iterations. The panel stiffness was held at 2.92 x 10^8 psf while the width was set to 2 ft, 3 ft, 4 ft, 6 ft, and 8 ft. Models were run for both Soil 1 and Soil 2 with all panel widths.

Table H-1 (Soil 1) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with struts only (no panel or strongback)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-5.50	-6.85	-5.50	-15.93	-5.50	-22.53
-4.70	-9.77	-4.70	-7.07	-4.70	-3.29
-3.80	19.85	-3.80	18.26	-3.80	16.52
-2.90	-34.39	-2.90	-34.31	-2.90	-34.81
-2.01	56.92	-2.01	56.86	-2.01	58.45
-1.11	-106.03	-1.11	-105.46	-1.11	-106.61
-0.21	-639.48	-0.21	-606.54	-0.21	-618.77
-0.21	-5117.61	-0.21	-5014.82	-0.21	-5123.86
0.21	-5117.61	0.21	-5014.82	0.21	-5123.86
0.21	-644.79	0.21	-611.74	0.21	-623.93
1.11	-105.56	1.11	-104.98	1.11	-105.91
2.01	57.57	2.01	57.70	2.01	59.29
2.90	-34.77	2.90	-34.79	2.90	-35.27
3.80	20.06	3.80	18.54	3.80	16.81
4.70	-9.89	4.70	-7.21	4.70	-3.39
5.50	-6.69	5.50	-15.75	5.50	-22.32

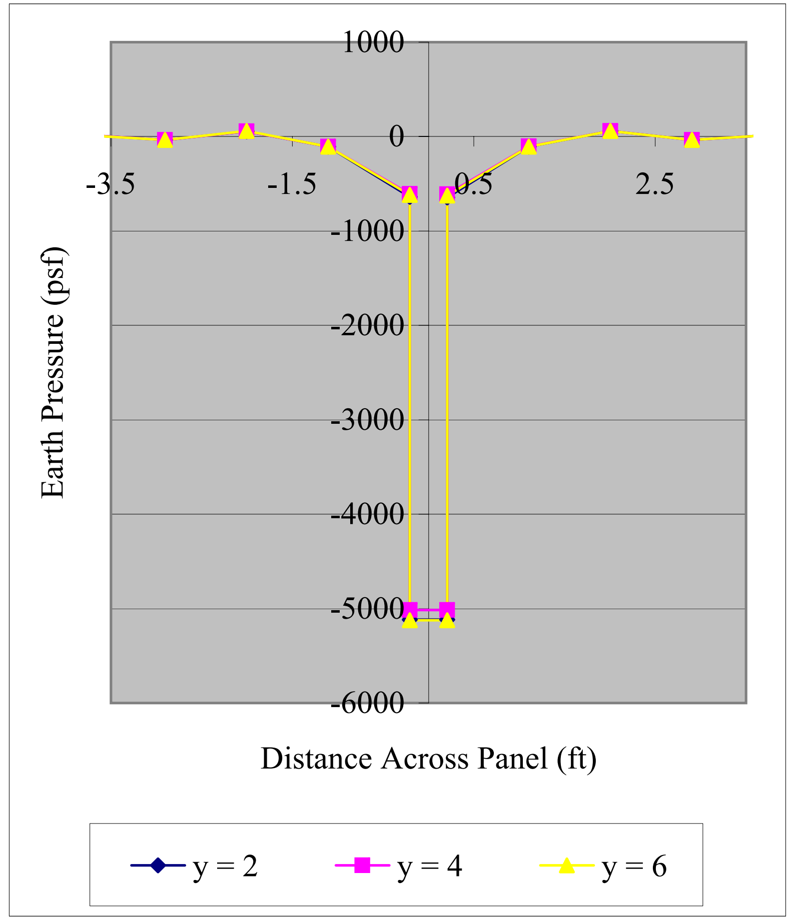


Table H-2 (Soil 1) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with a strongback only (no panel) between the struts and trench wall

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-5.50	-2.76	-5.50	-2.76	-5.50	-2.76
-5.32	6.48	-5.32	6.48	-5.32	6.48
-4.73	-4.70	-4.73	-4.70	-4.73	-4.70
-4.14	3.37	-4.14	3.37	-4.14	3.37
-3.55	-3.79	-3.55	-3.79	-3.55	-3.79
-2.95	5.23	-2.95	5.23	-2.95	5.23
-2.36	-9.92	-2.36	-9.92	-2.36	-9.92
-1.77	19.05	-1.77	19.05	-1.77	19.05
-1.18	-63.69	-1.18	-63.69	-1.18	-63.69
-0.59	-820.77	-0.59	-820.77	-0.59	-820.77
0.00	-1431.96	0.00	-1431.96	0.00	-1431.96
0.59	-799.03	0.59	-799.03	0.59	-799.03
1.18	-58.35	1.18	-58.35	1.18	-58.35
1.77	16.69	1.77	16.69	1.77	16.69
2.36	-8.65	2.36	-8.65	2.36	-8.65
2.95	4.50	2.95	4.50	2.95	4.50
3.55	-3.34	3.55	-3.34	3.55	-3.34
4.14	3.08	4.14	3.08	4.14	3.08
4.73	-4.49	4.73	-4.49	4.73	-4.49
5.32	6.30	5.32	6.30	5.32	6.30
5.50	-2.82	5.50	-2.82	5.50	-2.82

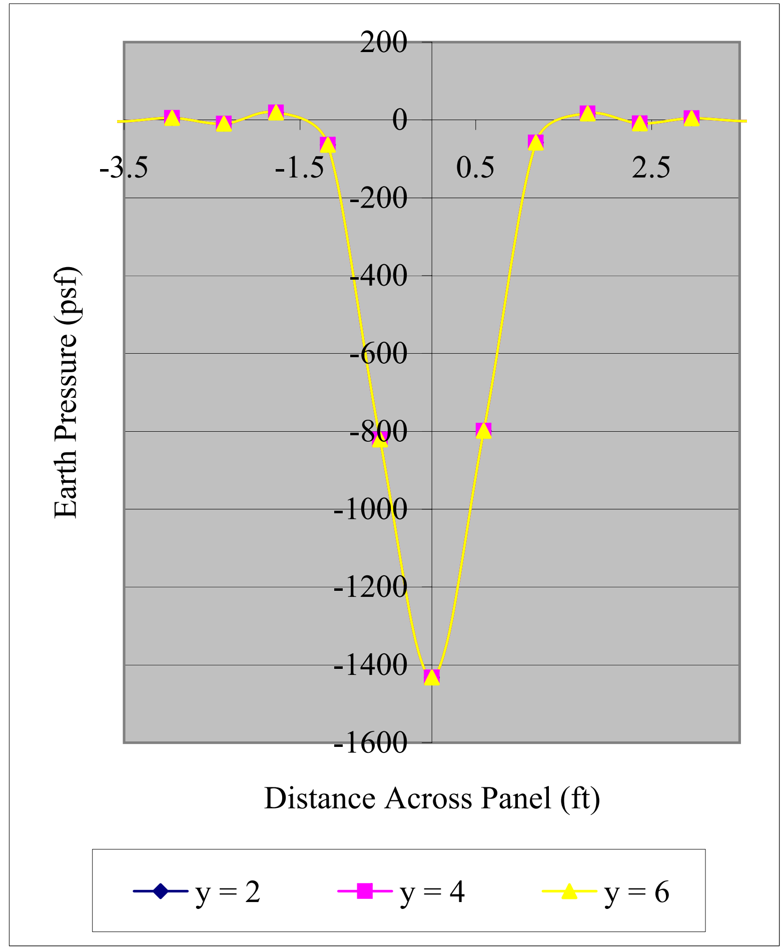


Table H-3 (Soil 1) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 2 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-5.50	-2.63	-5.50	-6.06	-5.50	-6.67
-5.32	6.39	-5.32	10.51	-5.32	12.99
-4.73	-4.65	-4.73	-7.82	-4.73	-11.50
-4.14	3.05	-4.14	4.06	-4.14	5.54
-3.55	-2.93	-3.55	-4.13	-3.55	-6.66
-2.95	2.47	-2.95	2.94	-2.95	3.71
-2.36	-3.72	-2.36	-4.10	-2.36	-5.70
-1.77	-5.20	-1.77	-5.07	-1.77	0.44
-1.18	-85.43	-1.18	-78.23	-1.18	-59.41
-0.59	-393.02	-0.59	-371.60	-0.59	-354.24
0.00	-603.76	0.00	-580.03	0.00	-574.80
0.59	-392.75	0.59	-371.20	0.59	-354.11
1.18	-86.27	1.18	-79.04	1.18	-60.19
1.77	-5.31	1.77	-5.20	1.77	0.36
2.36	-3.63	2.36	-4.02	2.36	-5.63
2.95	2.42	2.95	2.89	2.95	3.67
3.55	-2.89	3.55	-4.10	3.55	-6.63
4.14	3.02	4.14	4.04	4.14	5.52
4.73	-4.63	4.73	-7.81	4.73	-11.49
5.32	6.35	5.32	10.50	5.32	12.99
5.50	-2.64	5.50	-6.07	5.50	-6.68

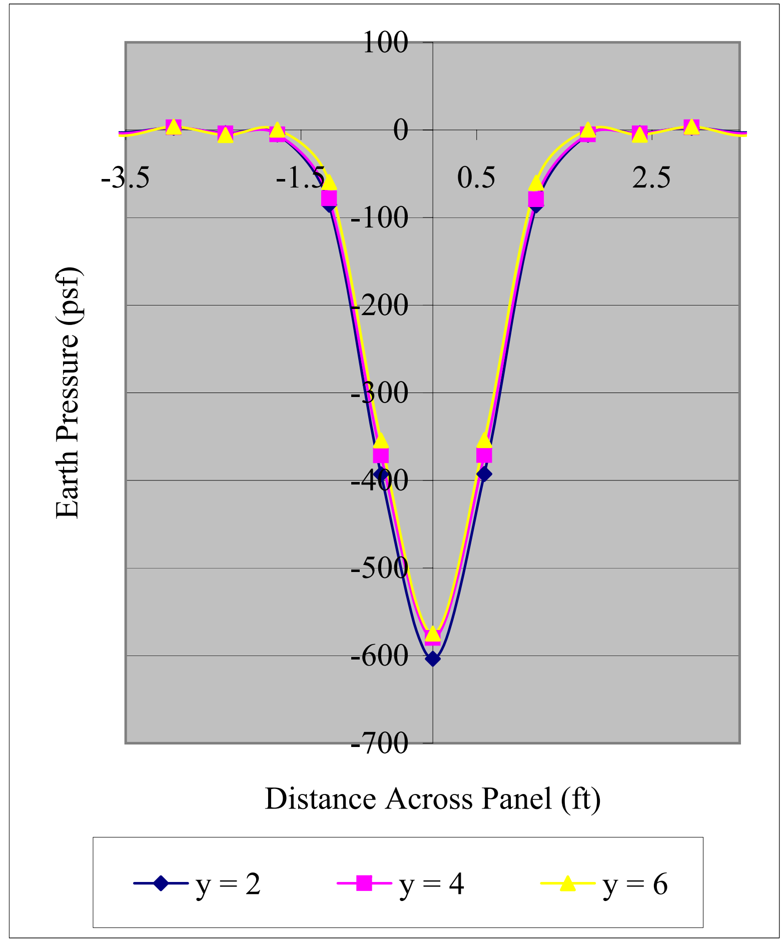


Table H-4 (Soil 1) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 3 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-5.50	-2.81	-5.50	-6.18	-5.50	-6.81
-5.32	5.97	-5.32	10.34	-5.32	12.83
-4.73	-4.12	-4.73	-7.49	-4.73	-11.14
-4.14	2.29	-4.14	3.60	-4.14	5.08
-3.55	-2.05	-3.55	-3.49	-3.55	-5.95
-2.95	0.87	-2.95	2.06	-2.95	3.10
-2.36	-4.00	-2.36	-5.78	-2.36	-4.79
-1.77	8.23	-1.77	10.28	-1.77	23.69
-1.18	-97.63	-1.18	-82.94	-1.18	-64.92
-0.59	-404.98	-0.59	-381.61	-0.59	-370.00
0.00	-589.63	0.00	-567.28	0.00	-562.86
0.59	-404.95	0.59	-381.47	0.59	-370.13
1.18	-97.39	1.18	-82.66	1.18	-64.92
1.77	7.97	1.77	10.01	1.77	23.55
2.36	-4.04	2.36	-5.83	2.36	-4.81
2.95	0.90	2.95	2.09	2.95	3.11
3.55	-2.07	3.55	-3.51	3.55	-5.96
4.14	2.29	4.14	3.61	4.14	5.08
4.73	-4.12	4.73	-7.50	4.73	-11.14
5.32	5.95	5.32	10.35	5.32	12.84
5.50	-2.81	5.50	-6.18	5.50	-6.81

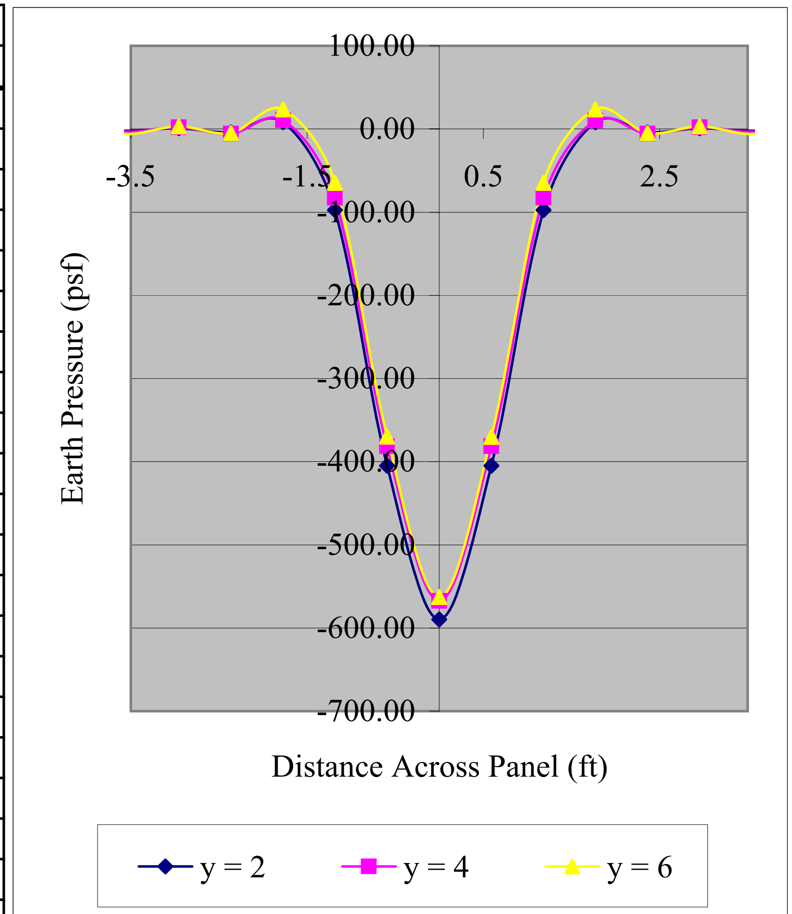


Table H-5 (Soil 1) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 4 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-5.50	-3.01	-5.50	-6.47	-5.50	-7.15
-5.32	5.39	-5.32	9.60	-5.32	12.03
-4.73	-3.43	-4.73	-6.42	-4.73	-9.99
-4.14	1.29	-4.14	2.20	-4.14	3.66
-3.55	-0.89	-3.55	-1.20	-3.55	-3.52
-2.95	-1.99	-2.95	-2.39	-2.95	0.70
-2.36	-0.84	-2.36	-2.05	-2.36	5.97
-1.77	22.48	-1.77	35.16	-1.77	53.88
-1.18	-115.88	-1.18	-101.48	-1.18	-87.50
-0.59	-416.20	-0.59	-397.25	-0.59	-389.39
0.00	-569.41	0.00	-543.78	0.00	-540.08
0.59	-416.17	0.59	-397.11	0.59	-389.44
1.18	-116.18	1.18	-101.71	1.18	-87.79
1.77	22.35	1.77	35.03	1.77	53.75
2.36	-0.79	2.36	-2.01	2.36	6.00
2.95	-2.00	2.95	-2.39	2.95	0.71
3.55	-0.89	3.55	-1.20	3.55	-3.52
4.14	1.28	4.14	2.19	4.14	3.66
4.73	-3.41	4.73	-6.42	4.73	-9.99
5.32	5.37	5.32	9.60	5.32	12.04
5.50	-3.01	5.50	-6.47	5.50	-7.15

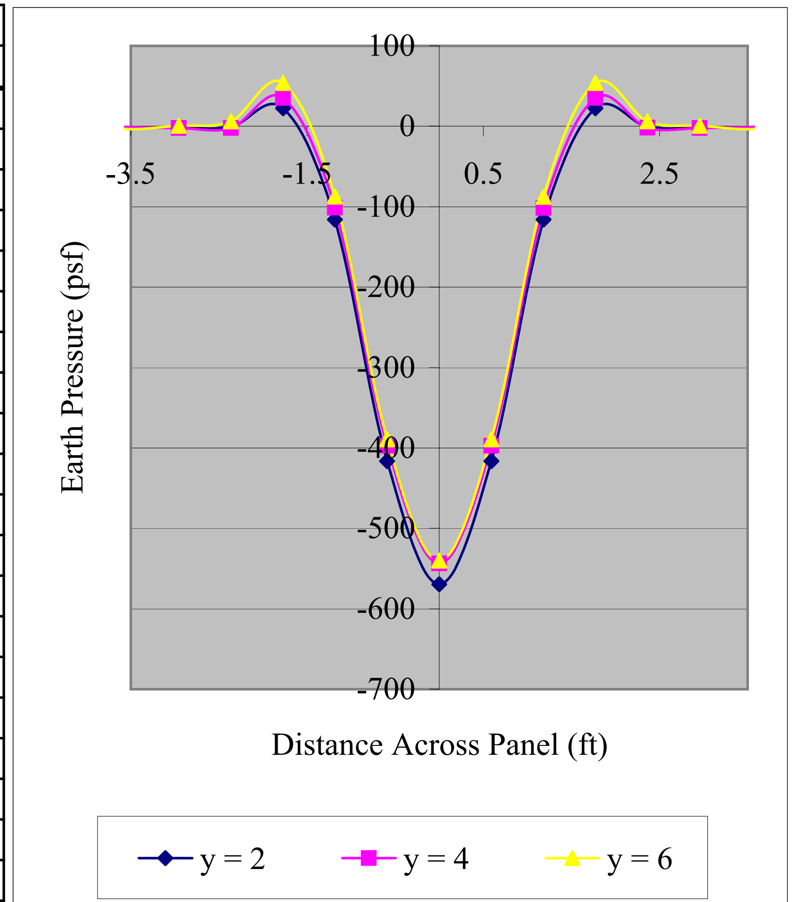


Table H-6 (Soil 1) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 6 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-5.50	-2.92	-5.50	-6.32	-5.50	-6.97
-5.32	5.39	-5.32	9.93	-5.32	12.50
-4.73	-3.63	-4.73	-6.77	-4.73	-10.39
-4.14	1.12	-4.14	2.54	-4.14	4.53
-3.55	-3.46	-3.55	-5.53	-3.55	-5.61
-2.95	5.21	-2.95	11.98	-2.95	24.33
-2.36	4.80	-2.36	12.79	-2.36	24.24
-1.77	9.08	-1.77	16.72	-1.77	32.92
-1.18	-116.16	-1.18	-105.91	-1.18	-89.04
-0.59	-409.43	-0.59	-389.95	-0.59	-380.41
0.00	-572.14	0.00	-546.76	0.00	-543.48
0.59	-409.44	0.59	-389.87	0.59	-380.51
1.18	-116.31	1.18	-106.01	1.18	-89.25
1.77	8.99	1.77	16.63	1.77	32.84
2.36	4.80	2.36	12.79	2.36	24.25
2.95	5.20	2.95	11.97	2.95	24.33
3.55	-3.43	3.55	-5.52	3.55	-5.60
4.14	1.12	4.14	2.55	4.14	4.53
4.73	-3.62	4.73	-6.77	4.73	-10.39
5.32	5.36	5.32	9.93	5.32	12.50
5.50	-2.92	5.50	-6.32	5.50	-6.97

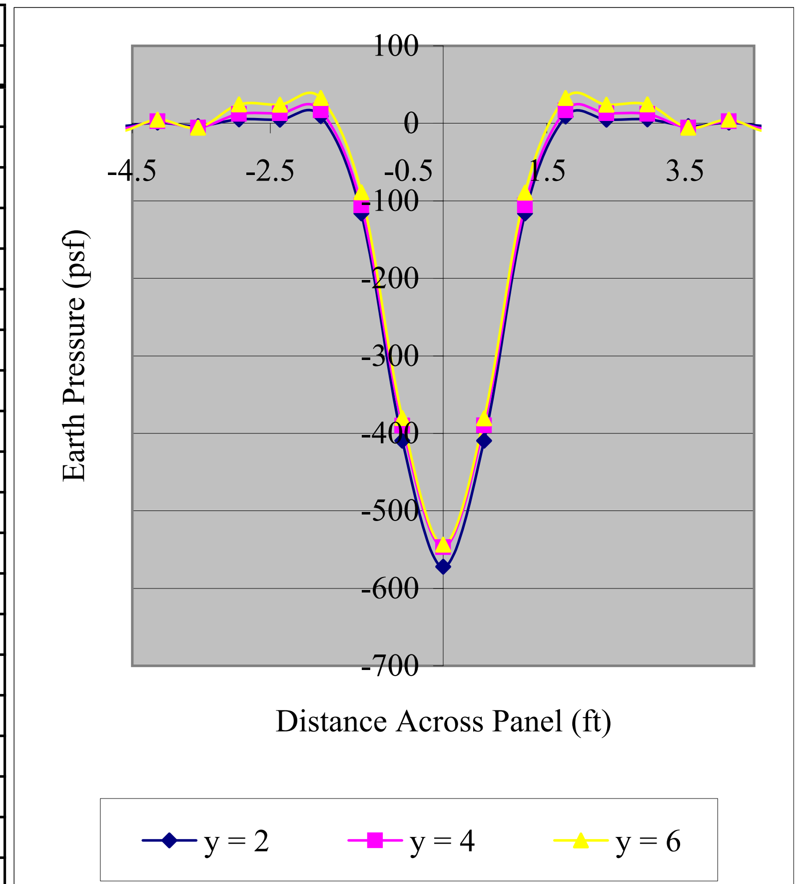


Table H-7 (Soil 1) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 8 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-5.50	-3.02	-5.50	-6.18	-5.50	-6.48
-5.32	4.86	-5.32	10.20	-5.32	13.45
-4.73	-4.86	-4.73	-8.71	-4.73	-10.79
-4.14	4.13	-4.14	10.89	-4.14	23.36
-3.55	-1.05	-3.55	6.27	-3.55	15.42
-2.95	2.85	-2.95	12.41	-2.95	26.53
-2.36	4.31	-2.36	11.42	-2.36	24.00
-1.77	10.11	-1.77	16.55	-1.77	32.67
-1.18	-114.81	-1.18	-107.31	-1.18	-90.37
-0.59	-409.68	-0.59	-392.12	-0.59	-382.23
0.00	-574.08	0.00	-549.03	0.00	-545.33
0.59	-409.71	0.59	-392.06	0.59	-382.34
1.18	-114.92	1.18	-107.37	1.18	-90.54
1.77	10.01	1.77	16.45	1.77	32.58
2.36	4.32	2.36	11.42	2.36	24.02
2.95	2.86	2.95	12.42	2.95	26.53
3.55	-1.06	3.55	6.27	3.55	15.43
4.14	4.13	4.14	10.89	4.14	23.37
4.73	-4.84	4.73	-8.71	4.73	-10.80
5.32	4.83	5.32	10.20	5.32	13.45
5.50	-3.02	5.50	-6.18	5.50	-6.48

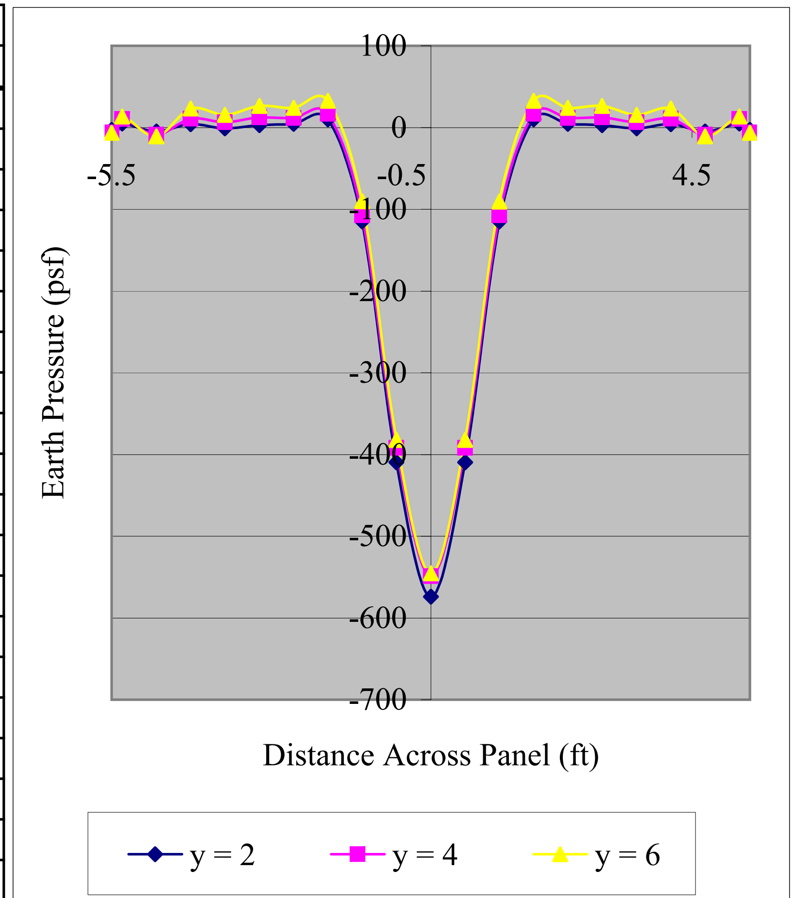


Table H-8 (Soil 2) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with struts only (no panel or strongback)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	1.86	-3.50	1.15	-3.50	-0.22
-2.90	-32.29	-2.90	-28.72	-2.90	-29.21
-2.01	50.53	-2.01	46.28	-2.01	47.55
-1.11	-84.38	-1.11	-76.33	-1.11	-78.56
-0.21	-2665.58	-0.21	-2751.04	-0.21	-2824.78
-0.21	-2690.29	-0.21	-2757.14	-0.21	-2829.55
0.21	-2692.51	0.21	-2759.54	0.21	-2832.01
0.21	-2667.82	0.21	-2753.45	0.21	-2827.25
1.11	-83.76	1.11	-75.55	1.11	-77.51
2.01	51.52	2.01	47.32	2.01	48.65
2.90	-32.91	2.90	-29.37	2.90	-29.89
3.50	1.88	3.50	1.17	3.50	-0.21

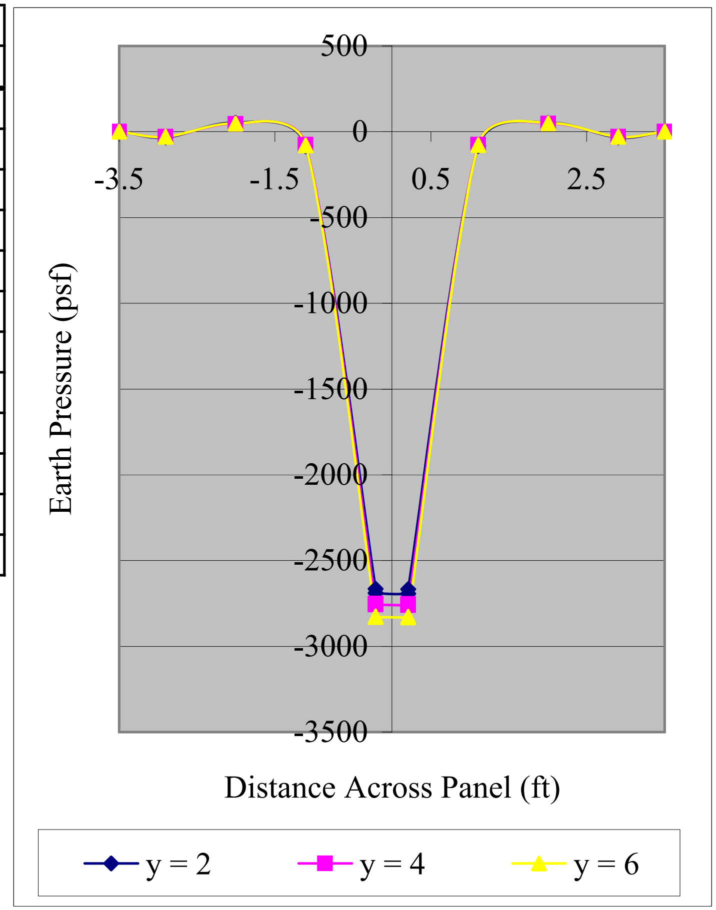


Table H-9 (Soil 2) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with a strongback only (no panel) between the struts and trench wall

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-5.53	-3.50	-6.42	-3.50	-9.19
-2.95	9.13	-2.95	9.25	-2.95	10.40
-2.36	-14.86	-2.36	-14.95	-2.36	-16.83
-1.77	25.24	-1.77	24.97	-1.77	23.93
-1.18	-68.29	-1.18	-67.71	-1.18	-53.75
-0.59	-797.78	-0.59	-770.65	-0.59	-807.88
0.00	-1338.63	0.00	-1293.48	0.00	-1395.52
0.59	-818.76	0.59	-790.75	0.59	-831.26
1.18	-71.67	1.18	-71.04	1.18	-56.66
1.77	26.87	1.77	26.55	1.77	25.50
2.36	-15.78	2.36	-15.82	2.36	-17.78
2.95	9.68	2.95	9.77	2.95	11.01
3.50	-5.82	3.50	-6.68	3.50	-9.52

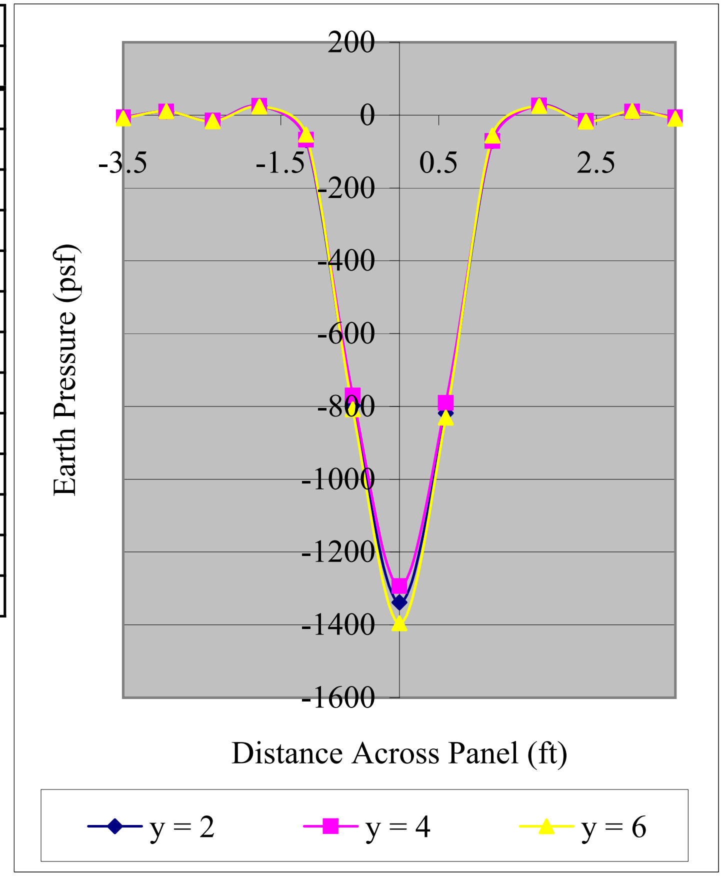


Table H-10 (Soil 2) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 2 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.76	-3.50	-1.94	-3.50	-4.09
-2.95	-0.78	-2.95	0.03	-2.95	0.93
-2.36	1.82	-2.36	1.06	-2.36	-1.15
-1.77	-16.21	-1.77	-14.84	-1.77	-6.91
-1.18	-144.50	-1.18	-132.61	-1.18	-109.19
-0.59	-320.22	-0.59	-303.24	-0.59	-279.34
0.00	-395.48	0.00	-384.42	0.00	-370.36
0.59	-319.39	0.59	-302.38	0.59	-278.74
1.18	-145.80	1.18	-133.67	1.18	-110.54
1.77	-16.44	1.77	-15.05	1.77	-7.08
2.36	1.97	2.36	1.19	2.36	-1.03
2.95	-0.87	2.95	-0.05	2.95	0.86
3.50	-0.71	3.50	-1.90	3.50	-4.05

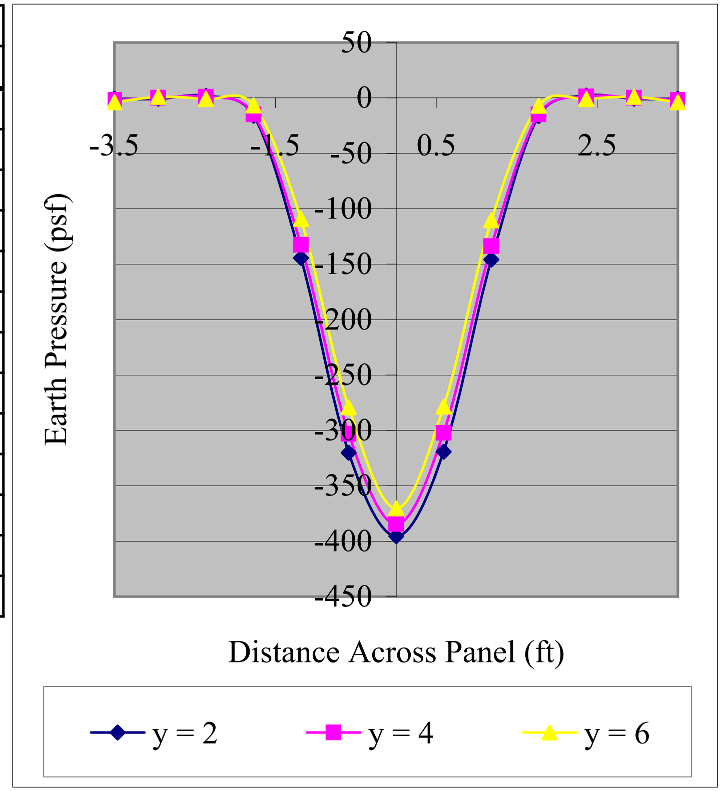


Table H-11 (Soil 2) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 3 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.02	-3.50	-2.05	-3.50	-4.06
-2.95	-0.48	-2.95	0.43	-2.95	1.45
-2.36	-3.08	-2.36	-3.54	-2.36	-2.09
-1.77	-1.76	-1.77	4.45	-1.77	19.52
-1.18	-125.23	-1.18	-111.14	-1.18	-91.50
-0.59	-329.47	-0.59	-312.82	-0.59	-293.07
0.00	-416.39	0.00	-401.96	0.00	-383.78
0.59	-329.26	0.59	-312.48	0.59	-292.95
1.18	-124.85	1.18	-110.66	1.18	-91.27
1.77	-2.17	1.77	4.00	1.77	19.19
2.36	-3.16	2.36	-3.63	2.36	-2.14
2.95	-0.43	2.95	0.49	2.95	1.49
3.50	-1.04	3.50	-2.08	3.50	-4.08

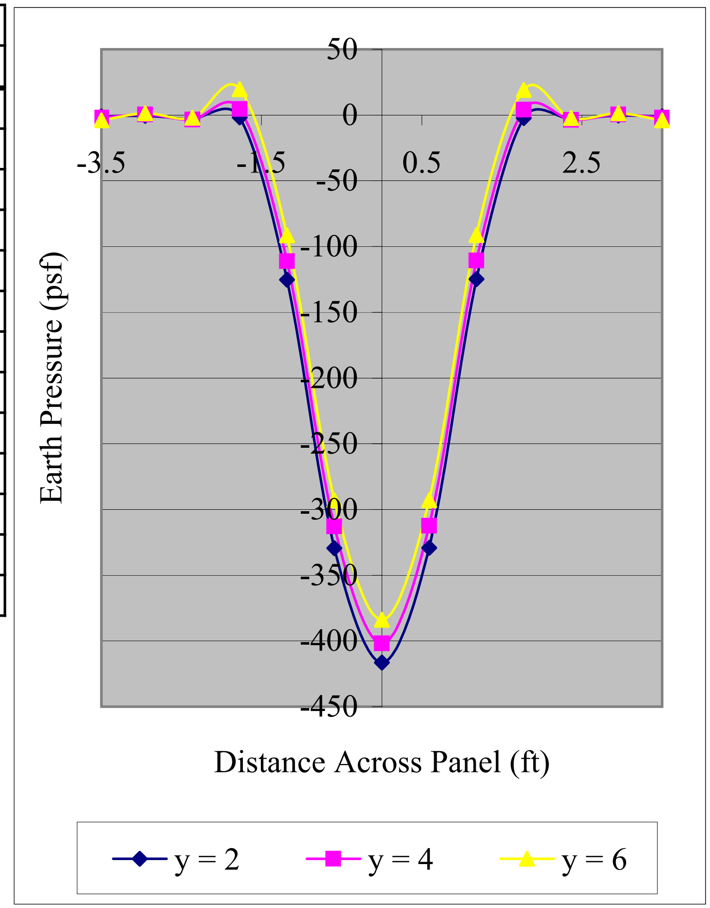


Table H-12 (Soil 2) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 4 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.92	-3.50	-1.18	-3.50	-2.98
-2.95	-1.49	-2.95	-0.88	-2.95	2.40
-2.36	0.88	-2.36	2.25	-2.36	11.71
-1.77	4.16	-1.77	18.29	-1.77	37.45
-1.18	-134.70	-1.18	-120.37	-1.18	-104.16
-0.59	-334.98	-0.59	-321.28	-0.59	-304.48
0.00	-409.05	0.00	-393.03	0.00	-376.93
0.59	-334.99	0.59	-321.18	0.59	-304.50
1.18	-135.06	1.18	-120.67	1.18	-104.48
1.77	3.98	1.77	18.11	1.77	37.29
2.36	0.93	2.36	2.30	2.36	11.76
2.95	-1.49	2.95	-0.88	2.95	2.40
3.50	-0.91	3.50	-1.18	3.50	-2.98

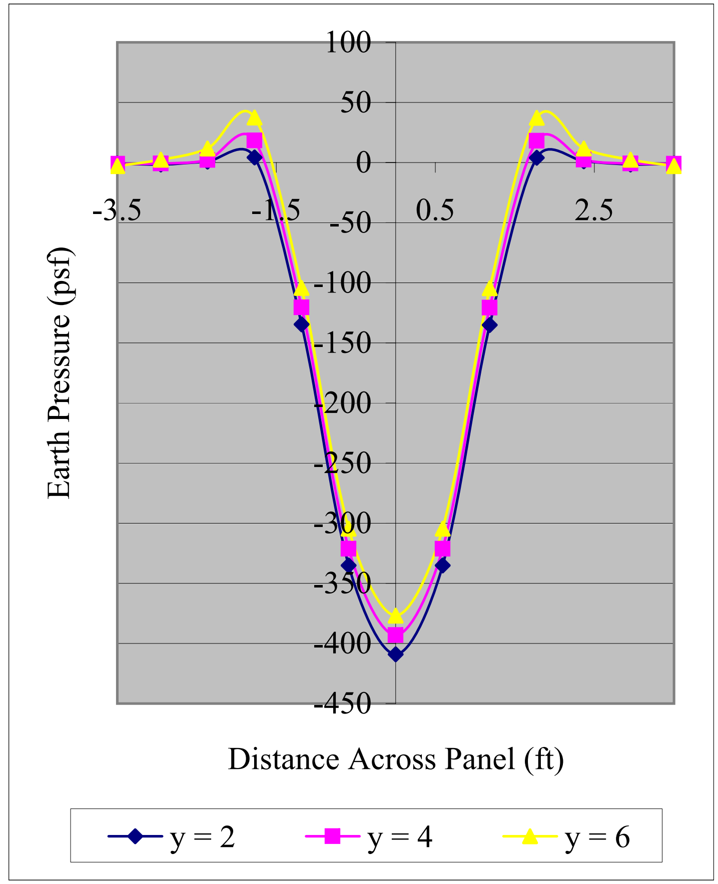


Table H-13 (Soil 2) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 6 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-4.50	-1.69	-4.50	-2.95	-4.50	-4.27
-4.14	0.58	-4.14	1.82	-4.14	3.86
-3.55	-1.31	-3.55	-2.30	-3.55	-1.75
-2.95	14.37	-2.95	24.36	-2.95	37.61
-2.36	5.56	-2.36	13.47	-2.36	24.96
-1.77	-22.84	-1.77	-16.72	-1.77	-1.24
-1.18	-140.14	-1.18	-130.06	-1.18	-111.16
-0.59	-322.82	-0.59	-307.99	-0.59	-289.26
0.00	-405.11	0.00	-388.28	0.00	-371.43
0.59	-322.89	0.59	-307.98	0.59	-289.34
1.18	-140.27	1.18	-130.14	1.18	-111.31
1.77	-22.92	1.77	-16.80	1.77	-1.32
2.36	5.51	2.36	13.42	2.36	24.93
2.95	14.35	2.95	24.33	2.95	37.59
3.55	-1.28	3.55	-2.27	3.55	-1.73
4.14	0.58	4.14	1.83	4.14	3.87
4.50	-1.68	4.50	-2.94	4.50	-4.27

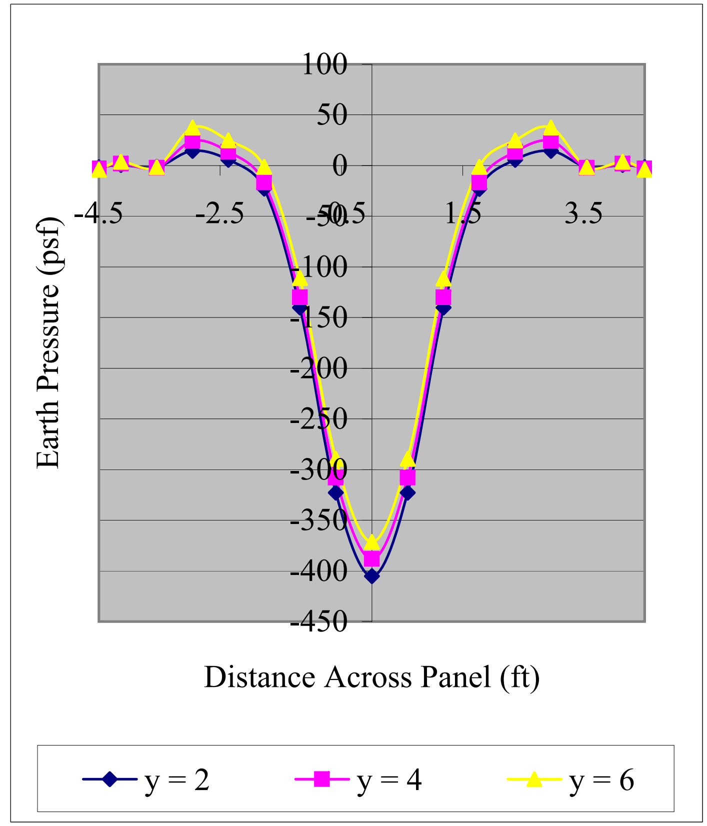


Table H-14 (Soil 2) Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with panel width = 8 ft

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-5.50	-2.95	-5.50	-6.11	-5.50	-6.45
-5.32	4.29	-5.32	9.24	-5.32	12.27
-4.73	-3.67	-4.73	-6.18	-4.73	-7.84
-4.14	7.31	-4.14	17.72	-4.14	31.77
-3.55	1.04	-3.55	7.72	-3.55	16.19
-2.95	4.58	-2.95	11.94	-2.95	24.49
-2.36	4.02	-2.36	11.38	-2.36	24.22
-1.77	-17.60	-1.77	-10.38	-1.77	5.82
-1.18	-137.91	-1.18	-129.90	-1.18	-110.54
-0.59	-323.28	-0.59	-310.85	-0.59	-291.58
0.00	-407.12	0.00	-391.98	0.00	-374.56
0.59	-323.38	0.59	-310.87	0.59	-291.68
1.18	-137.97	1.18	-129.91	1.18	-110.63
1.77	-17.67	1.77	-10.46	1.77	5.75
2.36	3.99	2.36	11.34	2.36	24.20
2.95	4.58	2.95	11.94	2.95	24.49
3.55	1.03	3.55	7.72	3.55	16.19
4.14	7.31	4.14	17.72	4.14	31.78
4.73	-3.65	4.73	-6.18	4.73	-7.84
5.32	4.26	5.32	9.24	5.32	12.28
5.50	-2.95	5.50	-6.11	5.50	-6.45

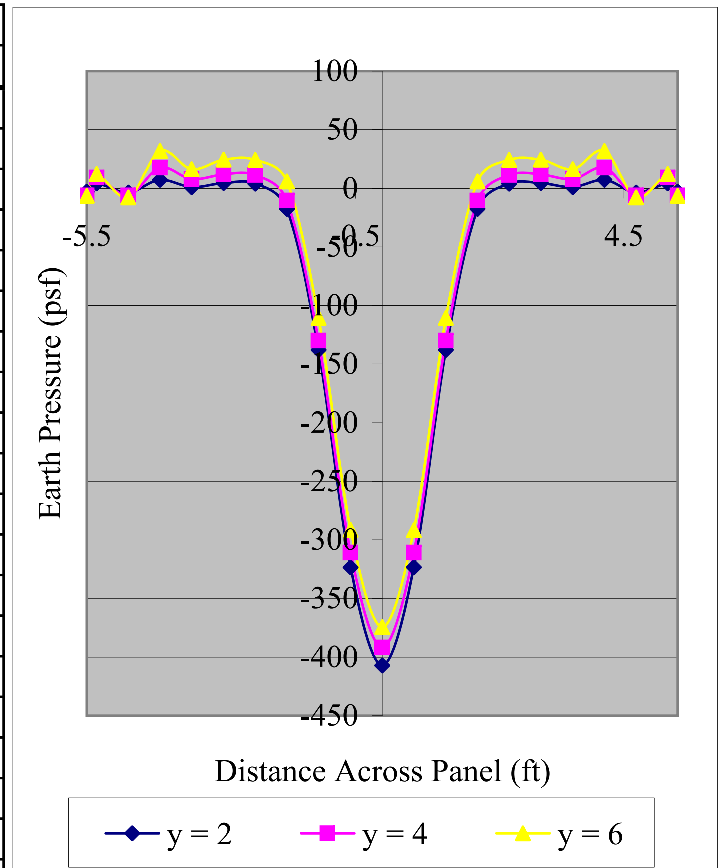


Figure H-1 (Soil 1) Combined earth pressure behind the horizontal width of the panel at depth y = 4 ft for variable panel width

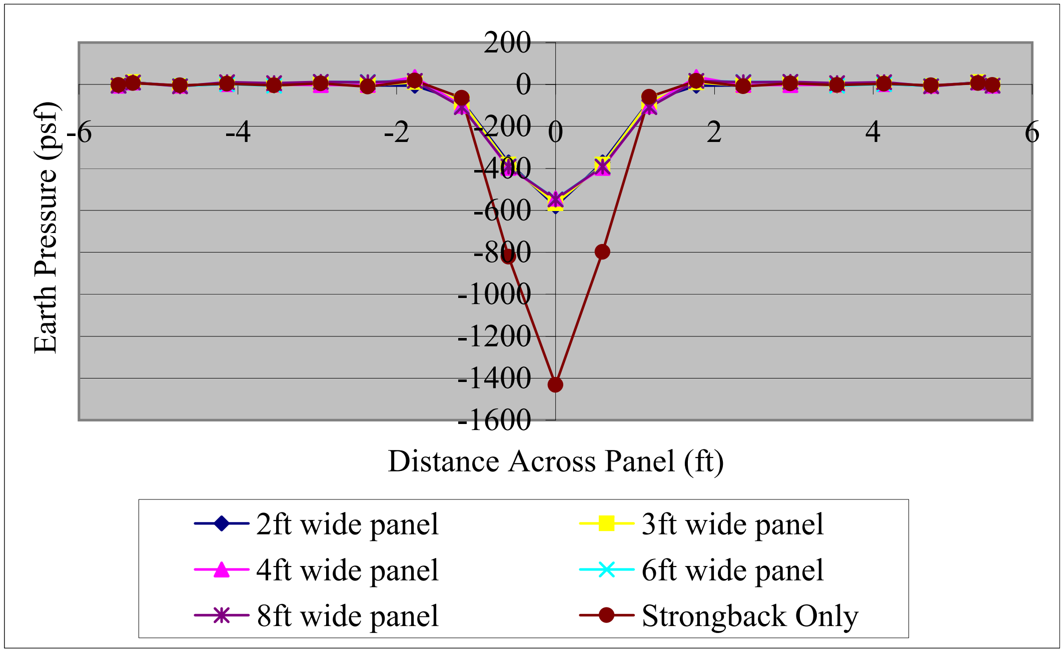


Figure H-2 (Soil 2) Combined earth pressure behind the horizontal width of the panel at depth y = 4 ft for variable panel width

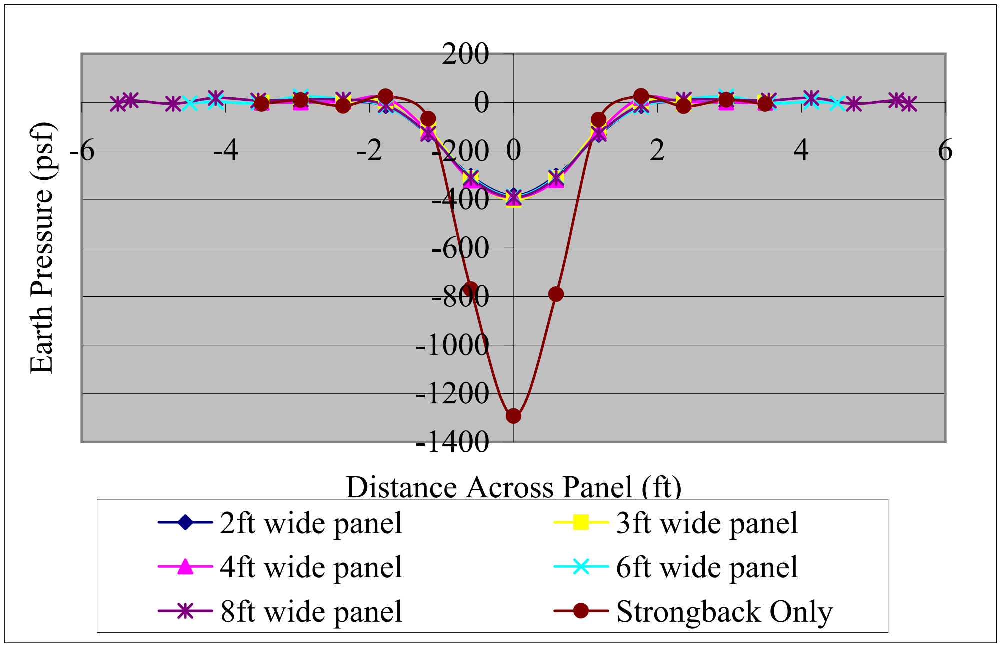


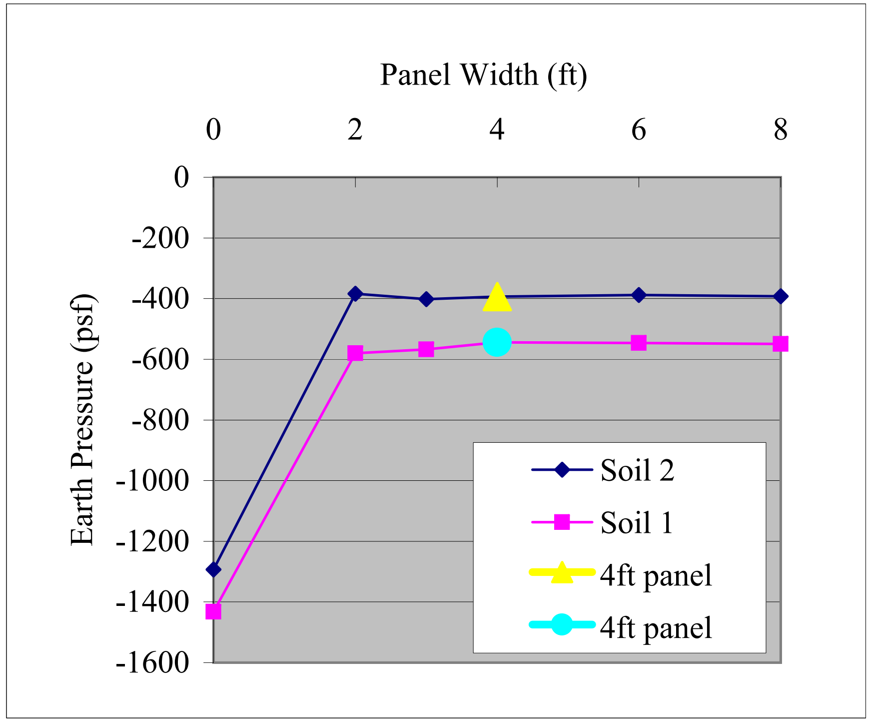
Table H-15 Panel width vs. earth pressure at a depth of y = 4 ft

Soil 1

Panel Width (ft)	EP (psf)
0	-1431.96
2	-580.03
3	-567.28
4	-543.78
6	-546.76
8	-549.03

Soil 2

Panel Width (ft)	EP (psf)
0	-1293.48
2	-384.42
3	-401.96
4	-393.03
6	-388.28
8	-391.98

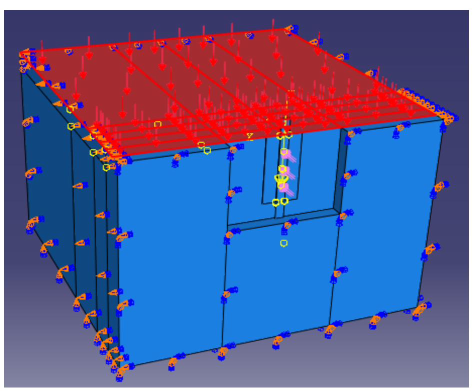


Appendix I: Parametric Study Tabulated Data – Varying Surcharge Distance from the Edge of the Trench

The following pages provide the tabulated data from the models run to evaluate the effects of varying surcharge size and distance from the edge of the trench. Strongback material properties were held constant and strut loading was set to 1298 lbs/strut for all iterations. The earth pressure due to surcharge distance from the edge of the trench was evaluated at 0 ft, 2 ft, 4 ft, 6 ft, 8 ft, and 16 ft. The final model had no surcharge at all, simulating surcharge placement 36 ft from the edge of the trench in this finite element model. Models were run for surcharge sizes of 300 psf, 600 psf, and 900 psf.

Table I-1 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 300 psf surcharge up to the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.47	-3.50	0.71	-3.50	3.25
-3.03	0.32	-3.05	3.80	-3.07	7.90
-2.45	8.96	-2.47	11.36	-2.49	23.43
-1.87	18.39	-1.90	25.23	-1.92	53.68
-1.29	-75.70	-1.32	-66.22	-1.35	-25.04
-0.72	-225.47	-0.75	-212.09	-0.78	-169.31
-0.06	-325.27	-0.11	-305.99	-0.16	-261.76
0.41	-302.65	0.38	-284.58	0.35	-239.31
0.97	-160.33	0.94	-157.56	0.91	-125.72
1.52	-21.49	1.50	-28.71	1.47	-0.48
2.08	24.63	2.05	21.80	2.03	48.72
2.63	0.49	2.61	1.35	2.59	11.50
3.18	-1.66	3.17	0.90	3.15	3.74
3.50	-1.52	3.50	-0.18	3.50	1.87



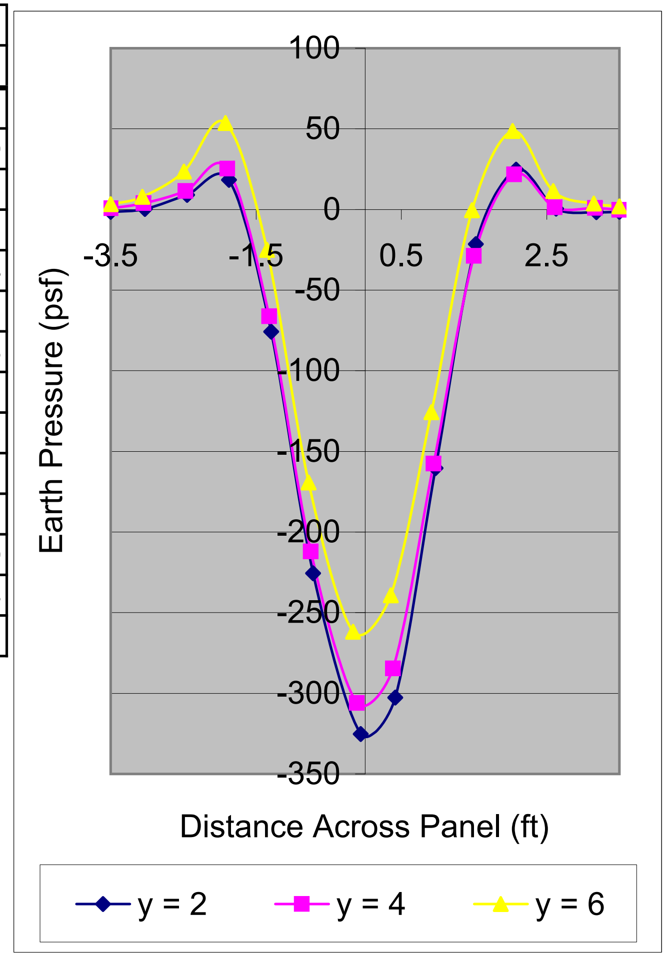
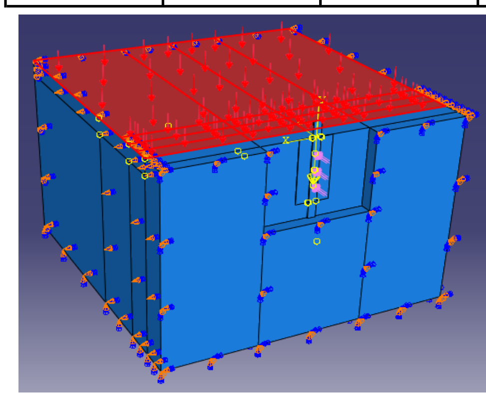


Table I-2 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 300 psf surcharge up to 2 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-4.80	-3.50	0.02	-3.50	3.23
-3.03	-3.36	-3.05	2.99	-3.07	7.83
-2.45	-1.48	-2.47	8.26	-2.49	21.26
-1.87	-10.72	-1.90	13.89	-1.92	44.67
-1.29	-105.46	-1.32	-77.68	-1.35	-36.36
-0.72	-249.66	-0.75	-221.57	-0.78	-179.77
-0.06	-350.57	-0.11	-316.87	-0.16	-272.02
0.41	-328.77	0.38	-296.06	0.35	-249.52
0.97	-187.35	0.94	-168.47	0.91	-136.13
1.52	-49.43	1.50	-38.75	1.47	-11.14
2.08	5.53	2.05	15.85	2.03	41.75
2.63	-4.11	2.61	0.67	2.59	10.40
3.18	-5.01	3.17	0.07	3.15	3.55
3.50	-4.54	3.50	-0.62	3.50	1.87



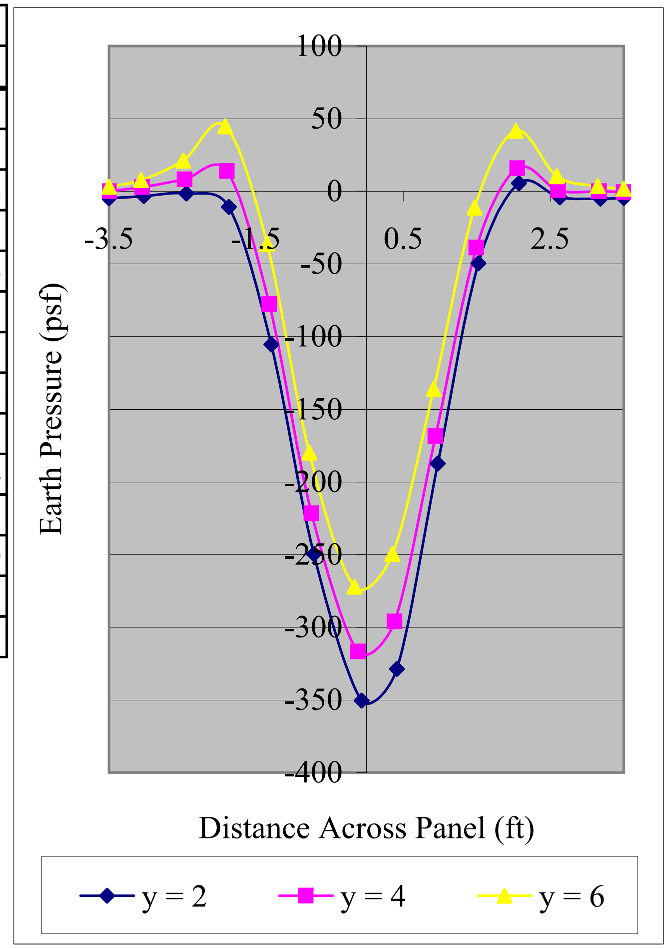
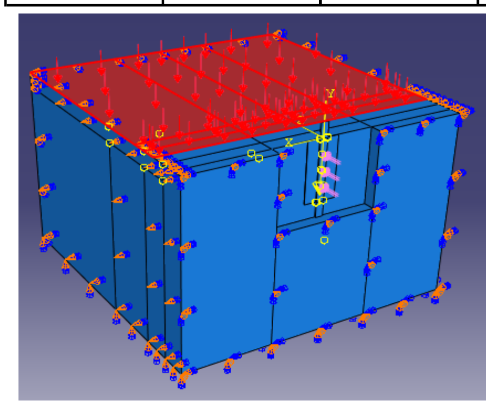


Table I-3 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 300 psf surcharge up to 4 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-2.22	-3.50	-0.79	-3.50	2.53
-3.03	-1.45	-3.05	1.23	-3.07	6.48
-2.45	0.44	-2.47	4.46	-2.49	17.85
-1.87	-6.83	-1.90	8.15	-1.92	38.05
-1.29	-99.16	-1.32	-83.45	-1.35	-43.83
-0.72	-242.04	-0.75	-227.73	-0.78	-187.27
-0.06	-340.29	-0.11	-322.70	-0.16	-279.36
0.41	-318.97	0.38	-302.02	0.35	-256.83
0.97	-180.57	0.94	-174.54	0.91	-143.37
1.52	-44.89	1.50	-44.44	1.47	-18.51
2.08	7.71	2.05	11.17	2.03	35.90
2.63	-1.87	2.61	-1.05	2.59	8.44
3.18	-2.80	3.17	-0.86	3.15	2.78
3.50	-2.14	3.50	-1.30	3.50	1.34



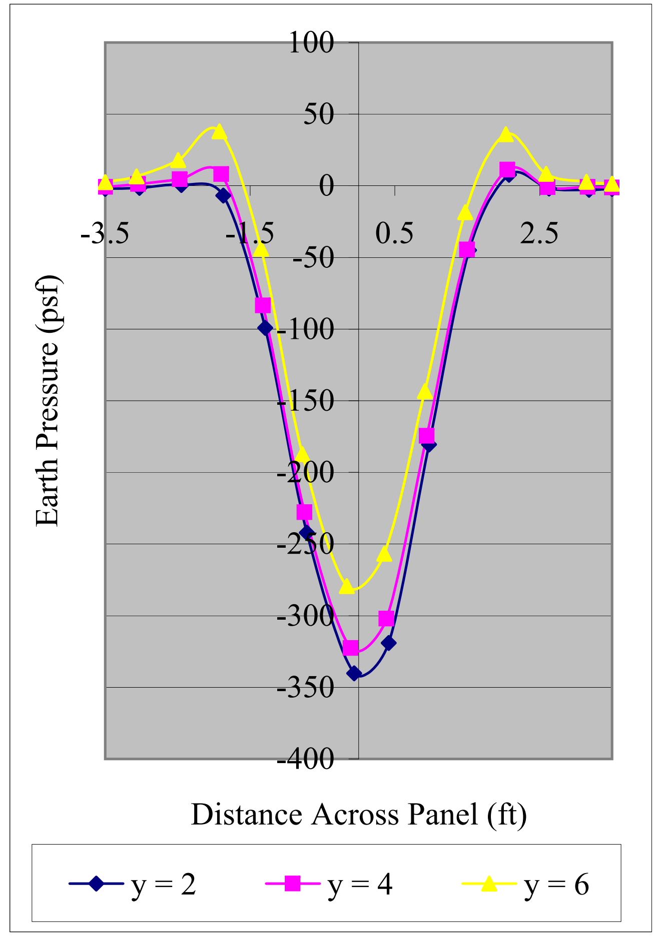
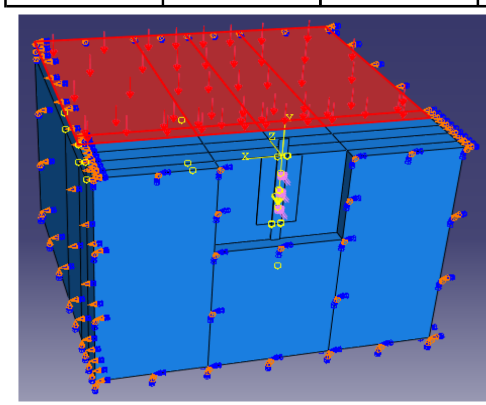


Table I-4 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 300 psf surcharge up to 6 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.00	-3.50	-0.61	-3.50	1.97
-3.03	-0.84	-3.05	0.68	-3.07	5.28
-2.45	1.22	-2.47	3.34	-2.49	15.51
-1.87	-3.57	-1.90	8.14	-1.92	34.97
-1.29	-94.88	-1.32	-83.00	-1.35	-46.98
-0.72	-238.12	-0.75	-227.80	-0.78	-190.67
-0.06	-335.18	-0.11	-321.79	-0.16	-282.30
0.41	-314.01	0.38	-301.15	0.35	-259.76
0.97	-176.62	0.94	-174.34	0.91	-146.47
1.52	-41.63	1.50	-44.51	1.47	-21.83
2.08	9.14	2.05	10.20	2.03	32.61
2.63	-1.05	2.61	-1.51	2.59	6.97
3.18	-1.77	3.17	-0.82	3.15	2.15
3.50	-1.00	3.50	-1.10	3.50	0.89



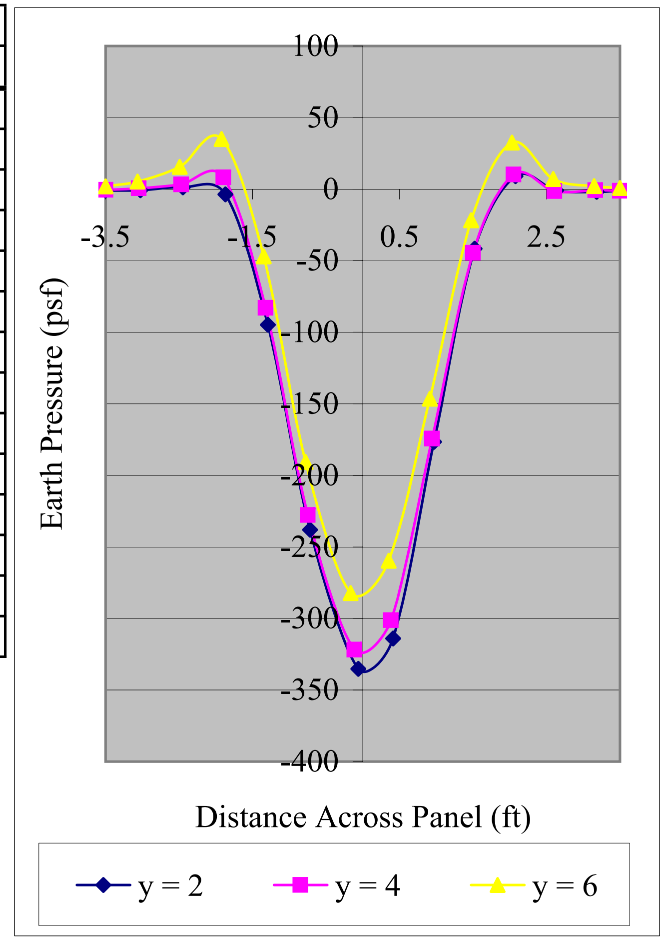
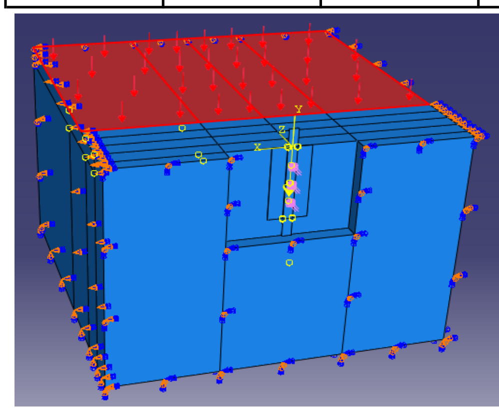


Table I-5 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 300 psf surcharge up to 8 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.69	-3.50	-0.18	-3.50	1.76
-3.03	-0.96	-3.05	0.60	-3.07	4.61
-2.45	1.08	-2.47	3.08	-2.49	14.29
-1.87	-2.10	-1.90	9.23	-1.92	34.05
-1.29	-92.97	-1.32	-81.55	-1.35	-47.63
-0.72	-236.74	-0.75	-226.83	-0.78	-191.53
-0.06	-333.29	-0.11	-320.03	-0.16	-282.61
0.41	-312.17	0.38	-299.40	0.35	-260.09
0.97	-175.09	0.94	-173.11	0.91	-147.11
1.52	-40.28	1.50	-43.56	1.47	-22.75
2.08	9.46	2.05	10.20	2.03	31.15
2.63	-1.07	2.61	-1.52	2.59	6.21
3.18	-1.55	3.17	-0.49	3.15	1.88
3.50	-0.72	3.50	-0.68	3.50	0.75



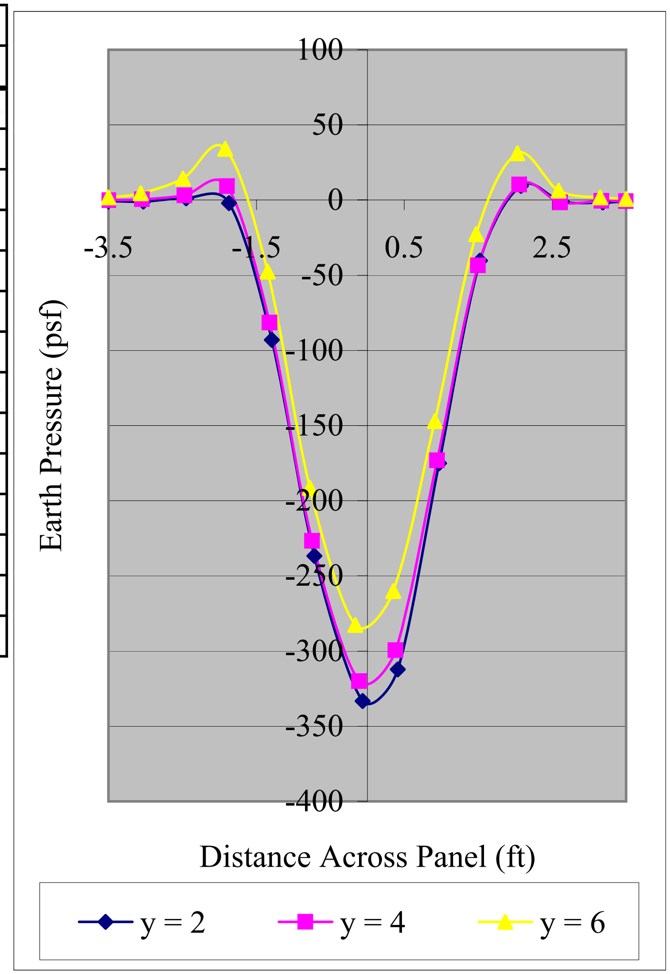
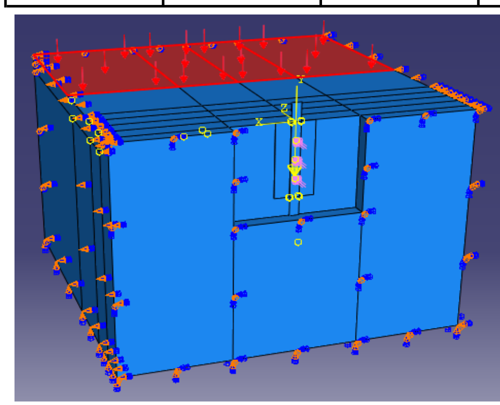


Table I-6 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 300 psf surcharge up to 16 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.82	-3.50	0.22	-3.50	1.76
-3.03	-1.56	-3.05	0.34	-3.07	3.87
-2.45	0.29	-2.47	2.91	-2.49	13.35
-1.87	-1.15	-1.90	11.71	-1.92	35.01
-1.29	-91.74	-1.32	-78.62	-1.35	-45.66
-0.72	-236.09	-0.75	-224.86	-0.78	-189.86
-0.06	-332.18	-0.11	-316.94	-0.16	-279.62
0.41	-311.10	0.38	-296.26	0.35	-257.17
0.97	-174.28	0.94	-170.63	0.91	-145.08
1.52	-39.63	1.50	-41.42	1.47	-21.41
2.08	9.19	2.05	10.62	2.03	30.52
2.63	-1.61	2.61	-1.69	2.59	5.53
3.18	-1.70	3.17	-0.22	3.15	1.71
3.50	-0.84	3.50	-0.29	3.50	0.78



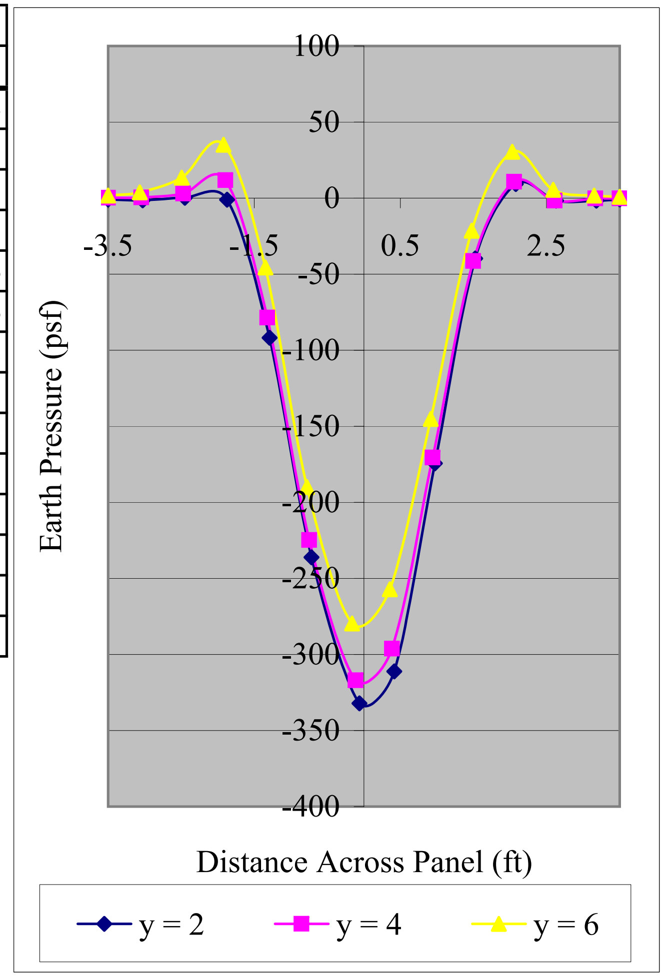
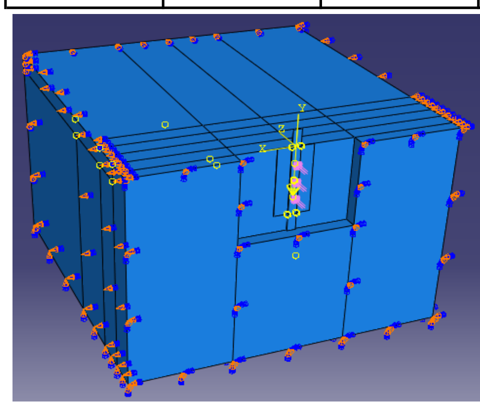


Table I-7 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 300 psf surcharge up to 36 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.35	-3.50	-0.02	-3.50	2.01
-3.03	-2.05	-3.05	-0.15	-3.07	3.73
-2.45	-0.35	-2.47	2.65	-2.49	13.69
-1.87	-1.37	-1.90	12.77	-1.92	37.61
-1.29	-92.45	-1.32	-77.35	-1.35	-42.05
-0.72	-237.33	-0.75	-223.99	-0.78	-186.49
-0.06	-333.82	-0.11	-315.66	-0.16	-275.28
0.41	-312.67	0.38	-294.95	0.35	-252.91
0.97	-175.20	0.94	-169.51	0.91	-141.50
1.52	-40.16	1.50	-40.52	1.47	-18.34
2.08	8.85	2.05	10.55	2.03	31.61
2.63	-2.23	2.61	-2.18	2.59	5.61
3.18	-2.09	3.17	-0.52	3.15	1.82
3.50	-1.30	3.50	-0.55	3.50	0.99



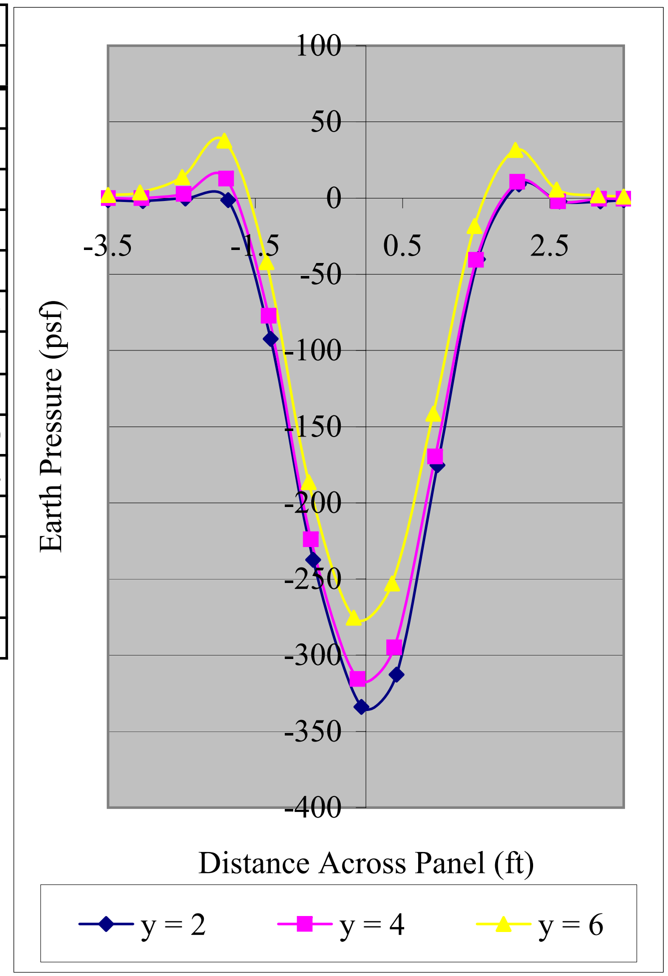
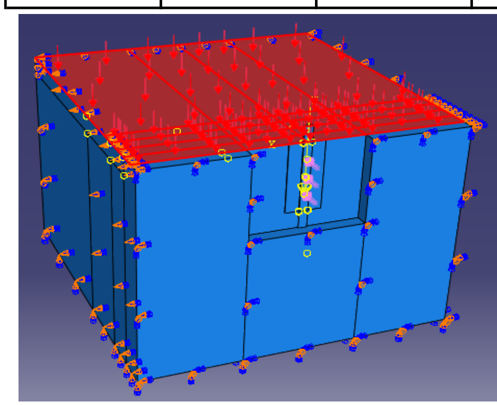


Table I-8 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 600 psf surcharge up to the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.59	-3.50	1.43	-3.50	4.49
-3.03	2.70	-3.05	7.75	-3.07	12.07
-2.45	18.26	-2.47	20.07	-2.49	33.17
-1.87	38.15	-1.90	37.69	-1.92	69.74
-1.29	-58.95	-1.32	-55.09	-1.35	-8.04
-0.72	-213.62	-0.75	-200.20	-0.78	-152.12
-0.06	-316.72	-0.11	-296.32	-0.16	-248.25
0.41	-292.63	0.38	-274.20	0.35	-225.71
0.97	-145.45	0.94	-145.60	0.91	-109.93
1.52	-2.81	1.50	-16.91	1.47	17.38
2.08	40.40	2.05	33.04	2.03	65.83
2.63	3.20	2.61	4.89	2.59	17.40
3.18	-1.22	3.17	2.33	3.15	5.67
3.50	-1.75	3.50	0.19	3.50	2.75



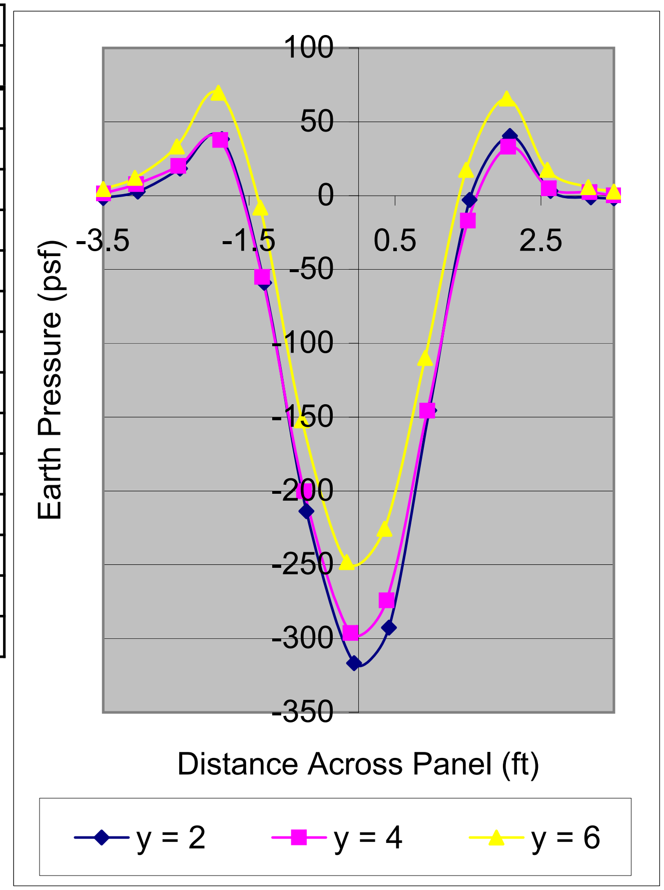
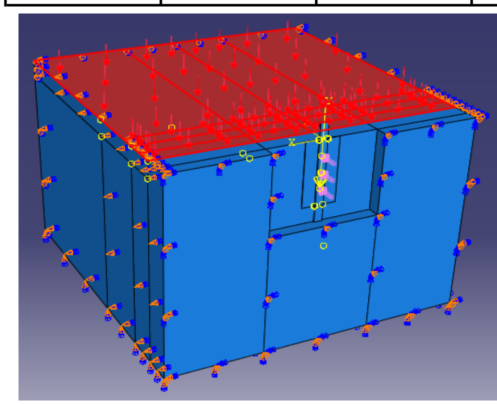


Table I-9 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 600 psf surcharge up to 2 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-8.24	-3.50	0.05	-3.50	4.44
-3.03	-4.66	-3.05	6.13	-3.07	11.92
-2.45	-2.61	-2.47	13.87	-2.49	28.83
-1.87	-20.06	-1.90	15.02	-1.92	51.74
-1.29	-118.47	-1.32	-78.01	-1.35	-30.67
-0.72	-262.00	-0.75	-219.16	-0.78	-173.06
-0.06	-367.32	-0.11	-318.07	-0.16	-268.77
0.41	-344.87	0.38	-297.18	0.35	-246.13
0.97	-199.49	0.94	-167.43	0.91	-130.76
1.52	-58.69	1.50	-36.98	1.47	-3.94
2.08	2.20	2.05	21.14	2.03	51.89
2.63	-6.00	2.61	3.53	2.59	15.19
3.18	-7.93	3.17	0.66	3.15	5.28
3.50	-7.77	3.50	-0.70	3.50	2.75



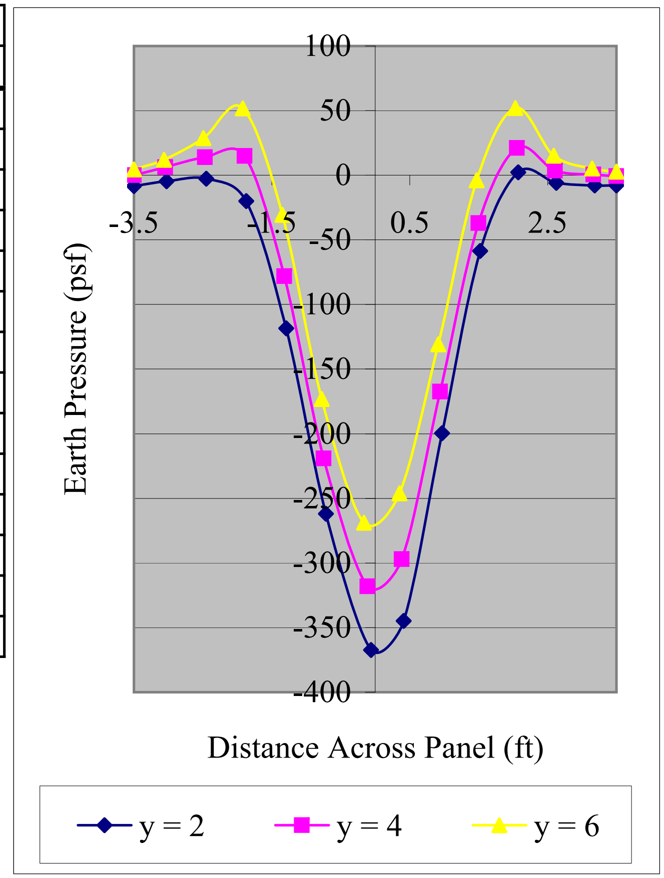
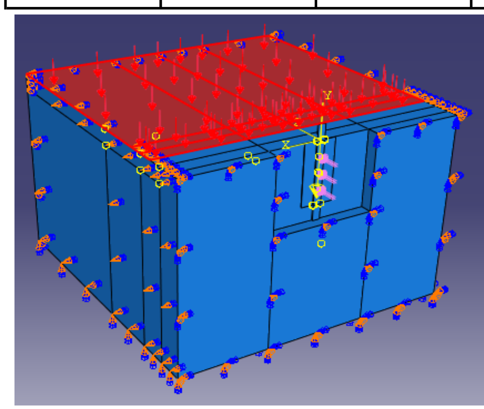


Table I-10 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 600 psf surcharge up to 4 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-3.09	-3.50	-1.56	-3.50	3.05
-3.03	-0.85	-3.05	2.62	-3.07	9.22
-2.45	1.24	-2.47	6.27	-2.49	22.02
-1.87	-12.28	-1.90	3.53	-1.92	38.50
-1.29	-105.88	-1.32	-89.55	-1.35	-45.61
-0.72	-246.76	-0.75	-231.47	-0.78	-188.05
-0.06	-346.75	-0.11	-329.74	-0.16	-283.45
0.41	-325.27	0.38	-309.10	0.35	-260.74
0.97	-185.93	0.94	-179.56	0.91	-145.23
1.52	-49.62	1.50	-48.36	1.47	-18.69
2.08	6.57	2.05	11.78	2.03	40.19
2.63	-1.51	2.61	0.07	2.59	11.26
3.18	-3.50	3.17	-1.19	3.15	3.74
3.50	-2.97	3.50	-2.05	3.50	1.69



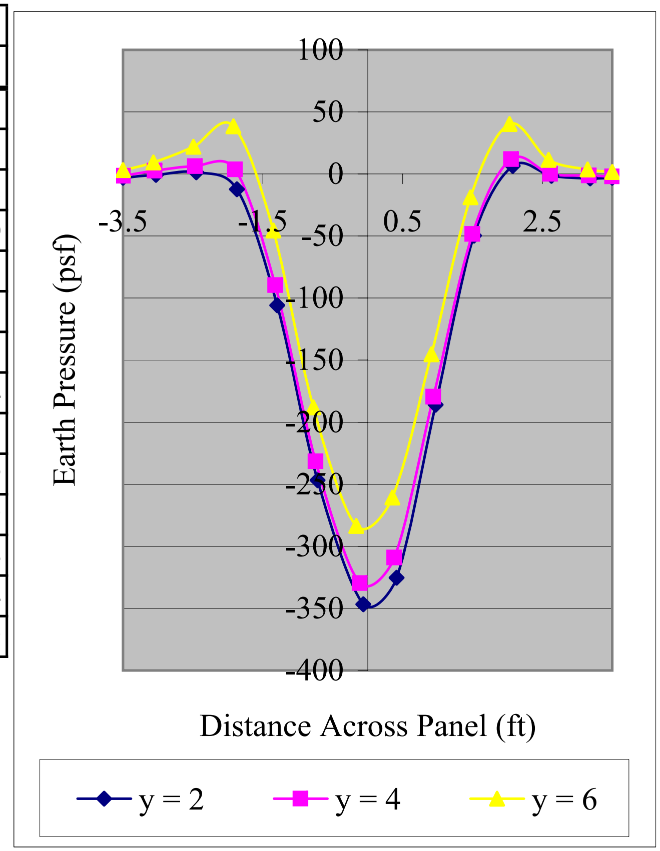
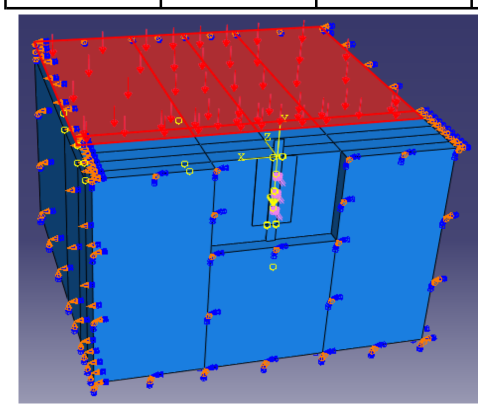


Table I-11 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 600 psf surcharge up to 6 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.65	-3.50	-1.21	-3.50	-0.65
-3.03	0.37	-3.05	1.51	-3.07	9.22
-2.45	2.78	-2.47	4.03	-2.49	22.02
-1.87	-5.78	-1.90	3.52	-1.92	38.50
-1.29	-97.30	-1.32	-88.65	-1.35	-45.61
-0.72	-238.91	-0.75	-231.61	-0.78	-188.05
-0.06	-336.55	-0.11	-327.91	-0.16	-283.45
0.41	-315.35	0.38	-307.34	0.35	-260.74
0.97	-178.05	0.94	-179.16	0.91	-145.23
1.52	-43.09	1.50	-48.50	1.47	-18.69
2.08	9.43	2.05	9.85	2.03	40.19
2.63	0.13	2.61	-0.83	2.59	11.26
3.18	-1.46	3.17	-1.11	3.15	3.74
3.50	-0.70	3.50	-1.65	3.50	1.69



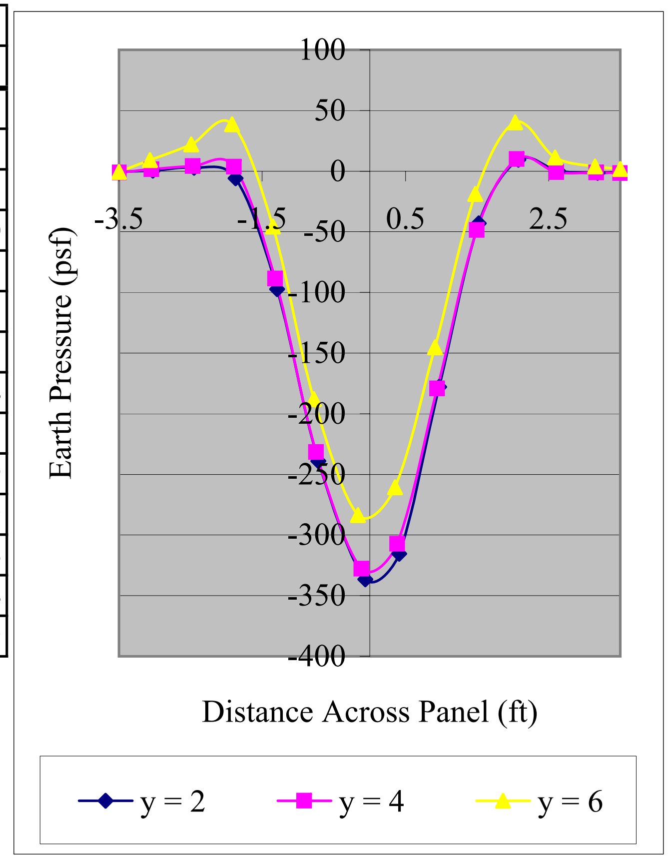
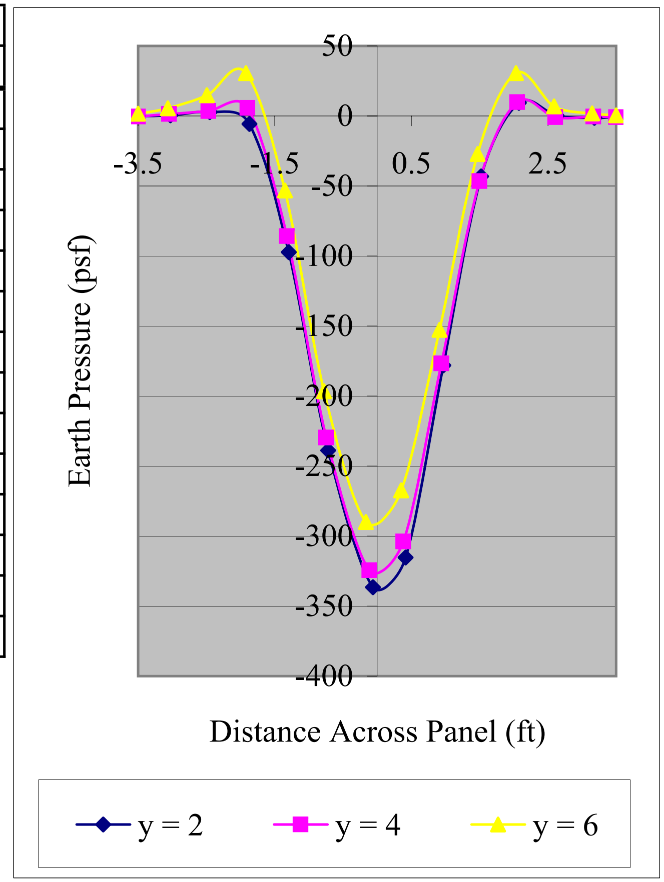


Table I-12 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 600 psf surcharge up to 8 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.03	-3.50	-0.35	-3.50	1.52
-3.03	0.37	-3.05	1.35	-3.07	5.49
-2.45	2.78	-2.47	3.51	-2.49	14.89
-1.87	-5.78	-1.90	5.69	-1.92	30.48
-1.29	-97.30	-1.32	-85.76	-1.35	-53.22
-0.72	-238.91	-0.75	-229.67	-0.78	-196.58
-0.06	-336.55	-0.11	-324.40	-0.16	-289.95
0.41	-315.35	0.38	-303.84	0.35	-267.27
0.97	-178.05	0.94	-176.70	0.91	-152.72
1.52	-43.09	1.50	-46.60	1.47	-27.15
2.08	9.43	2.05	9.84	2.03	30.68
2.63	0.13	2.61	-0.85	2.59	6.81
3.18	-1.46	3.17	-0.46	3.15	1.94
3.50	-0.70	3.50	-0.81	3.50	0.50



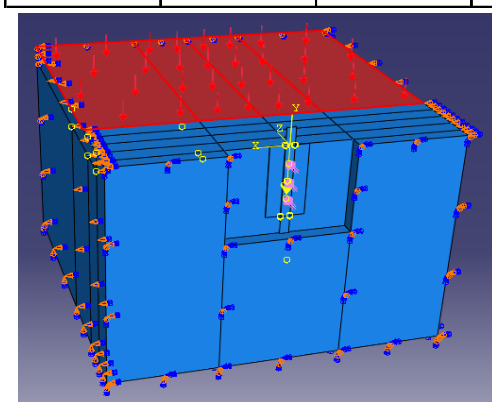
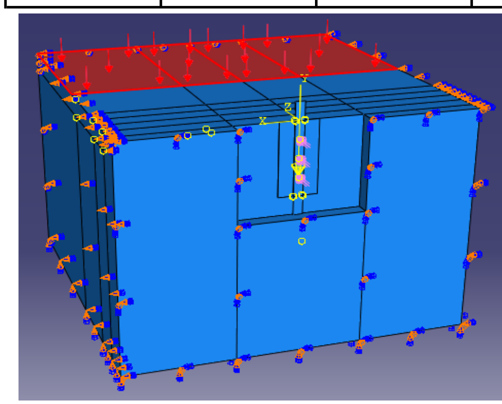


Table I-13 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 600 psf surcharge up to 16 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.29	-3.50	0.44	-3.50	1.52
-3.03	-1.05	-3.05	0.80	-3.07	4.01
-2.45	0.98	-2.47	3.17	-2.49	13.01
-1.87	-0.78	-1.90	10.57	-1.92	32.40
-1.29	-91.02	-1.32	-79.91	-1.35	-49.28
-0.72	-235.08	-0.75	-225.64	-0.78	-193.24
-0.06	-330.93	-0.11	-318.08	-0.16	-283.95
0.41	-309.90	0.38	-297.45	0.35	-261.43
0.97	-173.44	0.94	-171.69	0.91	-148.68
1.52	-39.00	1.50	-42.33	1.47	-24.49
2.08	9.65	2.05	10.60	2.03	29.43
2.63	-1.00	2.61	-1.23	2.59	5.46
3.18	-1.30	3.17	0.04	3.15	1.61
3.50	-0.37	3.50	-0.06	3.50	0.58



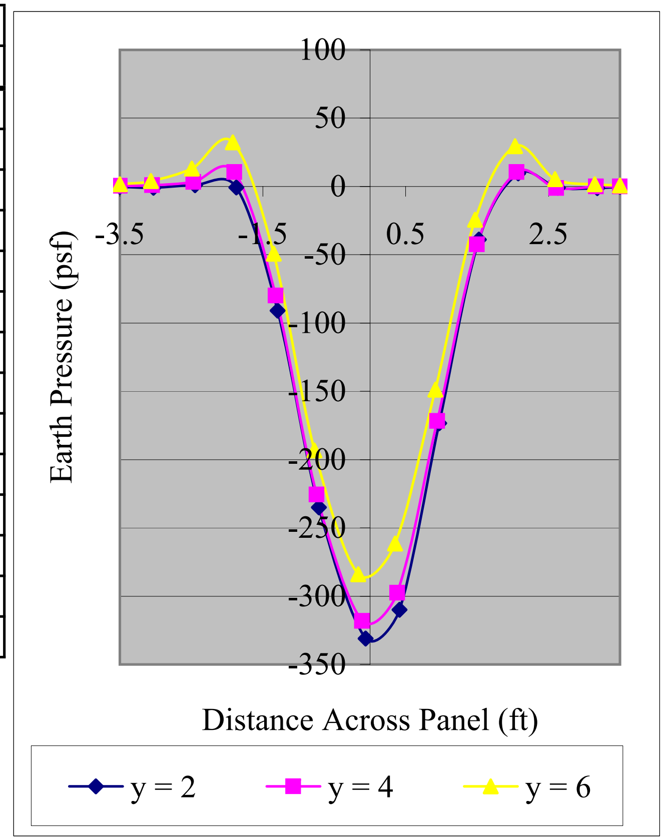
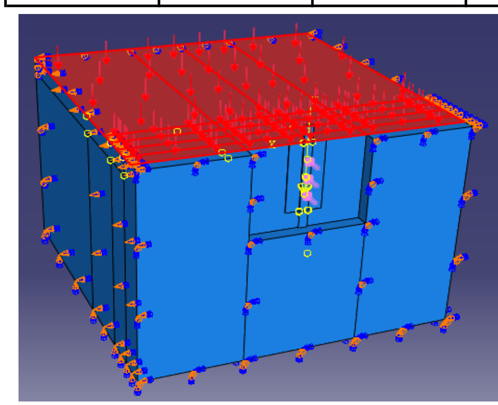


Table I-14 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6ft with 900 psf surcharge up to the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.71	-3.50	2.15	-3.50	5.73
-3.03	5.08	-3.05	11.70	-3.07	16.24
-2.45	27.56	-2.47	28.78	-2.49	42.92
-1.87	57.90	-1.90	50.15	-1.92	85.81
-1.29	-42.19	-1.32	-43.96	-1.35	8.97
-0.72	-201.76	-0.75	-188.30	-0.78	-134.94
-0.06	-308.17	-0.11	-286.65	-0.16	-234.73
0.41	-282.62	0.38	-263.83	0.35	-212.11
0.97	-130.58	0.94	-133.65	0.91	-94.14
1.52	15.86	1.50	-5.10	1.47	35.25
2.08	56.17	2.05	44.29	2.03	82.94
2.63	5.92	2.61	8.42	2.59	23.29
3.18	-0.79	3.17	3.76	3.15	7.59
3.50	-1.97	3.50	0.55	3.50	3.63



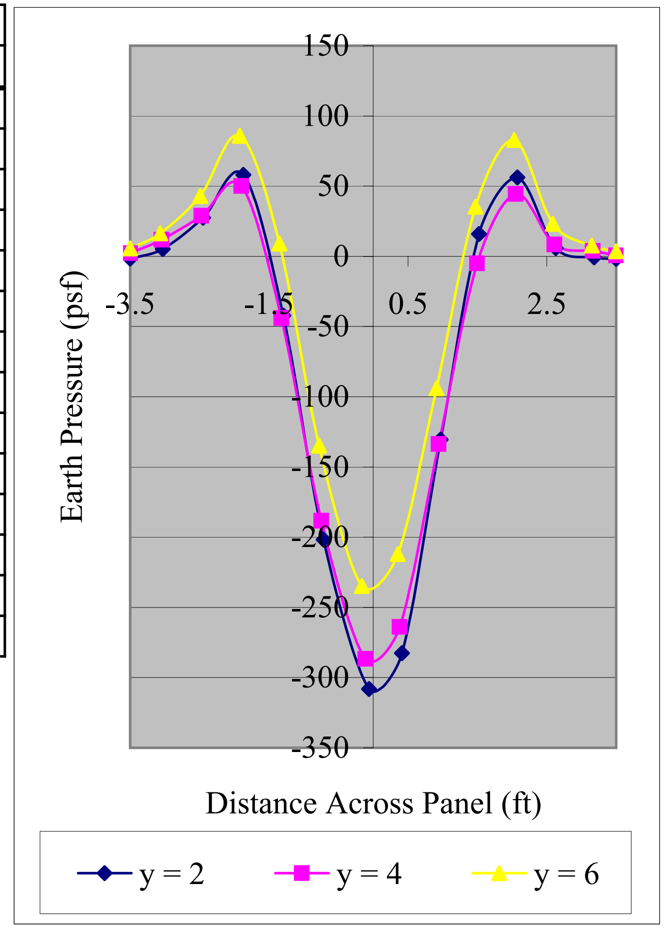
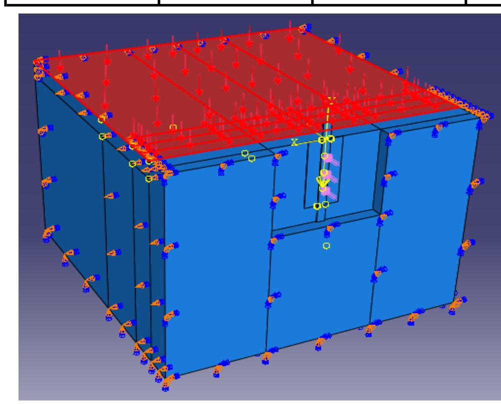


Table I-15 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 900 psf surcharge up to 2 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-11.69	-3.50	0.08	-3.50	5.66
-3.03	-5.97	-3.05	9.28	-3.07	16.02
-2.45	-3.74	-2.47	19.48	-2.49	36.40
-1.87	-29.40	-1.90	16.14	-1.92	58.80
-1.29	-131.48	-1.32	-78.33	-1.35	-24.98
-0.72	-274.34	-0.75	-216.74	-0.78	-166.35
-0.06	-384.07	-0.11	-319.27	-0.16	-265.51
0.41	-360.98	0.38	-298.29	0.35	-242.75
0.97	-211.64	0.94	-166.39	0.91	-125.39
1.52	-67.96	1.50	-35.21	1.47	3.27
2.08	-1.13	2.05	26.44	2.03	62.02
2.63	-7.88	2.61	6.38	2.59	19.97
3.18	-10.85	3.17	1.26	3.15	7.01
3.50	-11.01	3.50	-0.78	3.50	3.63



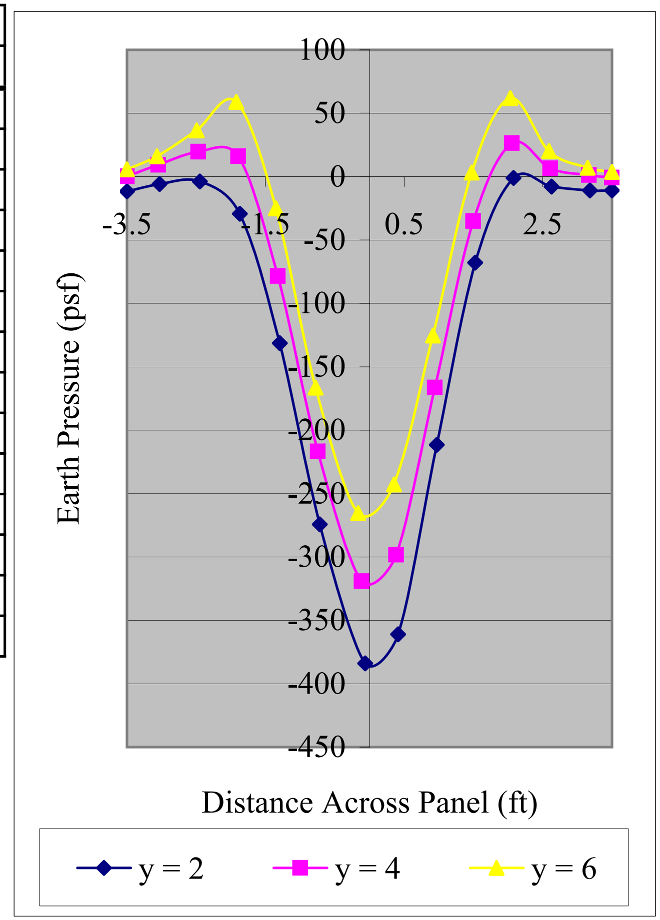
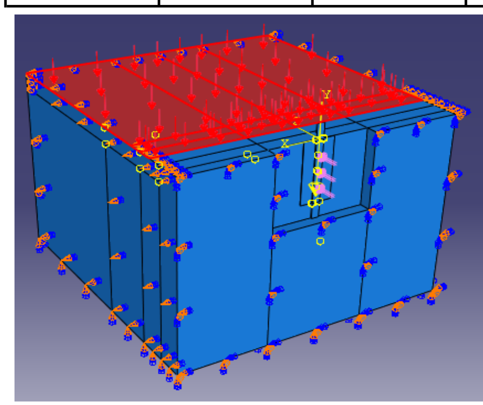


Table I-16 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 900 psf surcharge up to 4 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.30	-3.50	-1.81	-3.50	1.88
-3.03	1.58	-3.05	2.34	-3.07	8.39
-2.45	4.34	-2.47	4.73	-2.49	19.15
-1.87	-7.98	-1.90	-1.10	-1.92	29.69
-1.29	-99.73	-1.32	-94.30	-1.35	-56.83
-0.72	-239.70	-0.75	-235.42	-0.78	-199.03
-0.06	-337.91	-0.11	-334.03	-0.16	-296.35
0.41	-316.70	0.38	-313.54	0.35	-273.46
0.97	-179.47	0.94	-183.98	0.91	-156.41
1.52	-44.56	1.50	-52.50	1.47	-28.82
2.08	9.72	2.05	9.50	2.03	34.61
2.63	1.31	2.61	-0.16	2.59	9.70
3.18	-1.14	3.17	-1.40	3.15	2.80
3.50	-0.39	3.50	-2.21	3.50	0.69



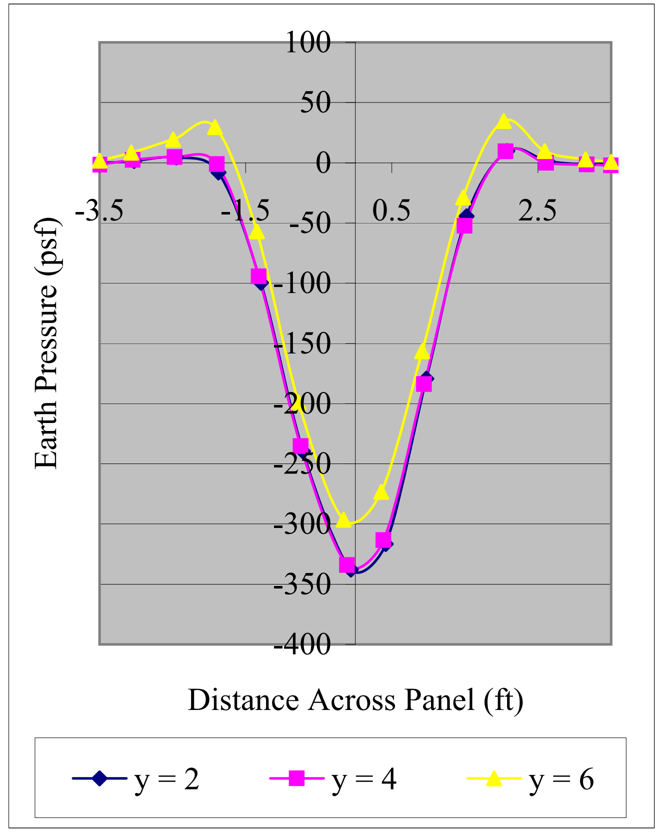
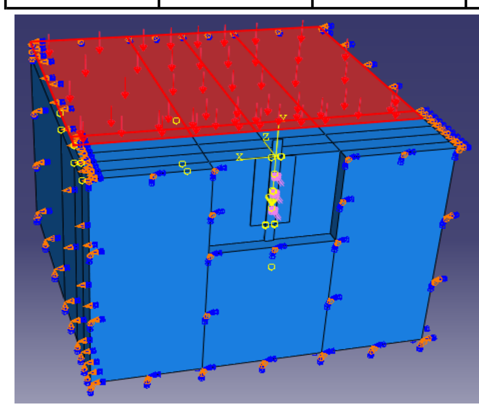


Table I-17 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 900 psf surcharge up to 6 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-0.30	-3.50	-1.81	-3.50	1.88
-3.03	1.58	-3.05	2.34	-3.07	8.39
-2.45	4.34	-2.47	4.73	-2.49	19.15
-1.87	-7.98	-1.90	-1.10	-1.92	29.69
-1.29	-99.73	-1.32	-94.30	-1.35	-56.83
-0.72	-239.70	-0.75	-235.42	-0.78	-199.03
-0.06	-337.91	-0.11	-334.03	-0.16	-296.35
0.41	-316.70	0.38	-313.54	0.35	-273.46
0.97	-179.47	0.94	-183.98	0.91	-156.41
1.52	-44.56	1.50	-52.50	1.47	-28.82
2.08	9.72	2.05	9.50	2.03	34.61
2.63	1.31	2.61	-0.16	2.59	9.70
3.18	-1.14	3.17	-1.40	3.15	2.80
3.50	-0.39	3.50	-2.21	3.50	0.69



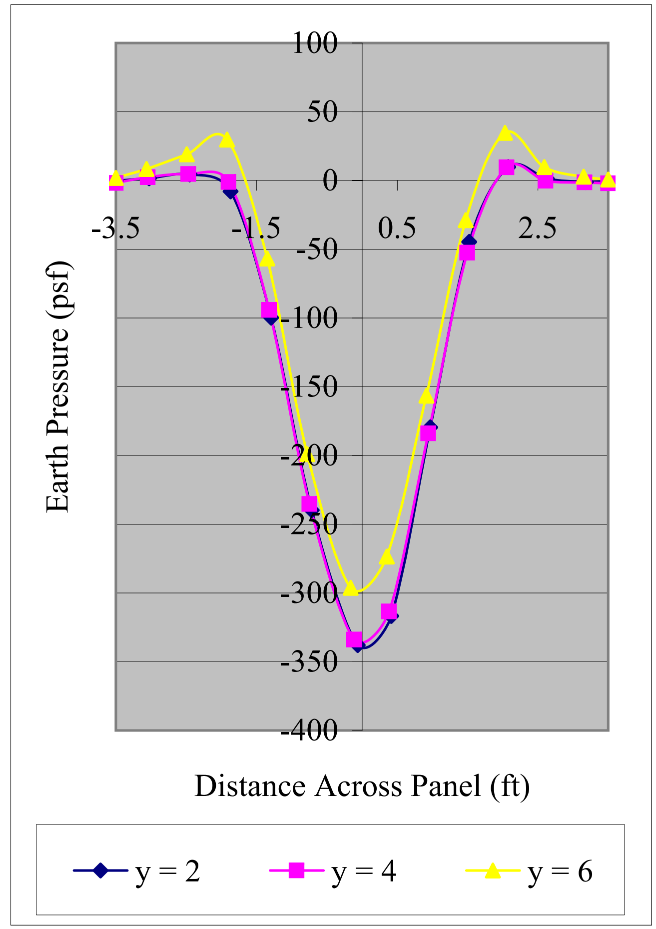
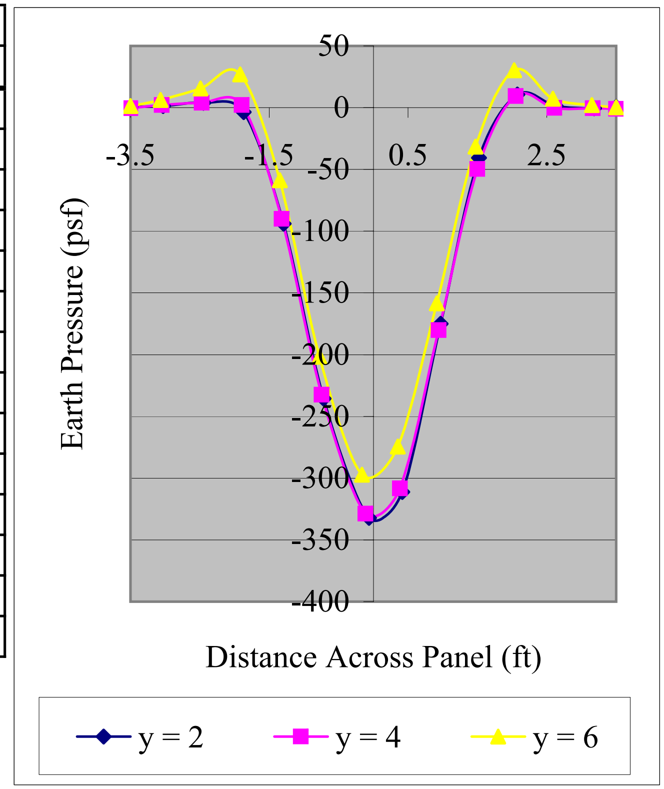


Table I-18 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 900 psf surcharge up to 8 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	0.63	-3.50	-0.52	-3.50	1.27
-3.03	1.21	-3.05	2.10	-3.07	6.37
-2.45	3.95	-2.47	3.94	-2.49	15.49
-1.87	-3.54	-1.90	2.14	-1.92	26.92
-1.29	-94.01	-1.32	-89.96	-1.35	-58.80
-0.72	-235.56	-0.75	-232.51	-0.78	-201.63
-0.06	-332.24	-0.11	-328.77	-0.16	-297.28
0.41	-311.18	0.38	-308.29	0.35	-274.45
0.97	-174.87	0.94	-180.29	0.91	-158.34
1.52	-40.53	1.50	-49.64	1.47	-31.56
2.08	10.66	2.05	9.48	2.03	30.22
2.63	1.26	2.61	-0.19	2.59	7.40
3.18	-0.48	3.17	-0.42	3.15	2.00
3.50	0.45	3.50	-0.94	3.50	0.26



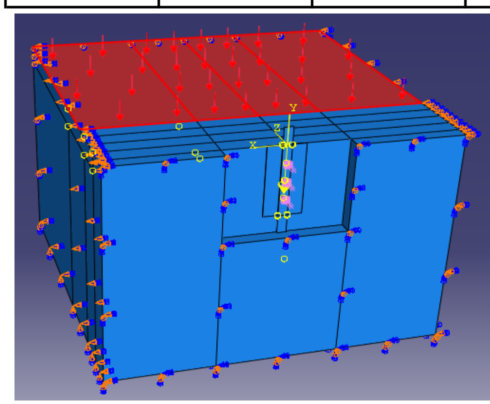
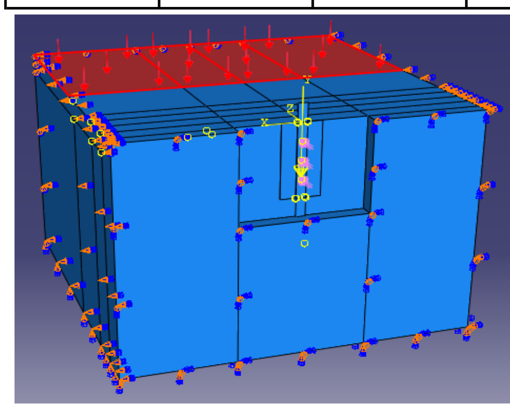


Table I-19 Earth pressure behind the width of the panel at depths y = 2 ft, 4 ft and 6 ft with 900 psf surcharge up to 16 ft from the edge of the trench

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	0.24	-3.50	0.65	-3.50	1.28
-3.03	-0.54	-3.05	1.26	-3.07	4.15
-2.45	1.66	-2.47	3.42	-2.49	12.67
-1.87	-0.41	-1.90	9.43	-1.92	29.79
-1.29	-90.30	-1.32	-81.19	-1.35	-52.91
-0.72	-234.07	-0.75	-226.41	-0.78	-196.62
-0.06	-329.68	-0.11	-319.22	-0.16	-288.28
0.41	-308.69	0.38	-298.64	0.35	-265.69
0.97	-172.60	0.94	-172.74	0.91	-152.27
1.52	-38.37	1.50	-43.25	1.47	-27.56
2.08	10.11	2.05	10.59	2.03	28.33
2.63	-0.38	2.61	-0.77	2.59	5.38
3.18	-0.89	3.17	0.30	3.15	1.50
3.50	0.10	3.50	0.16	3.50	0.37



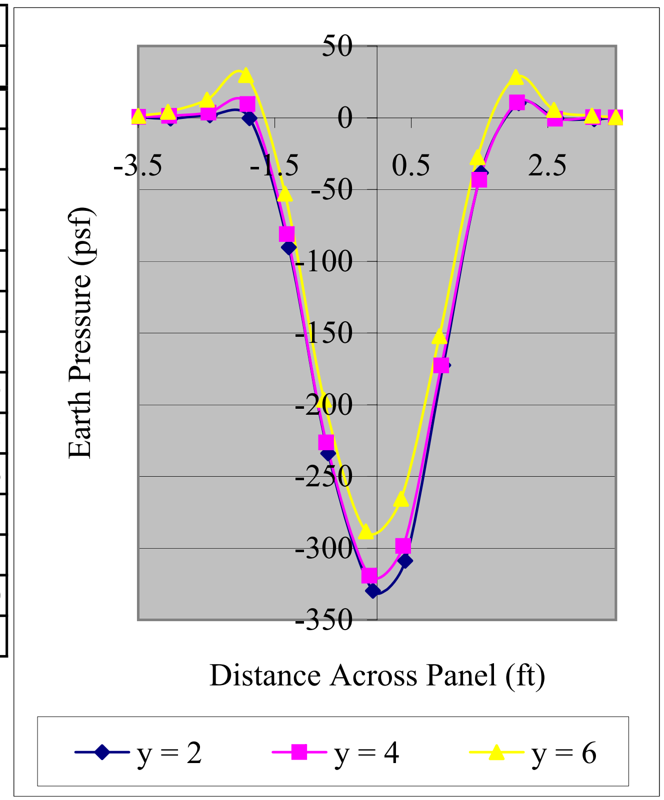
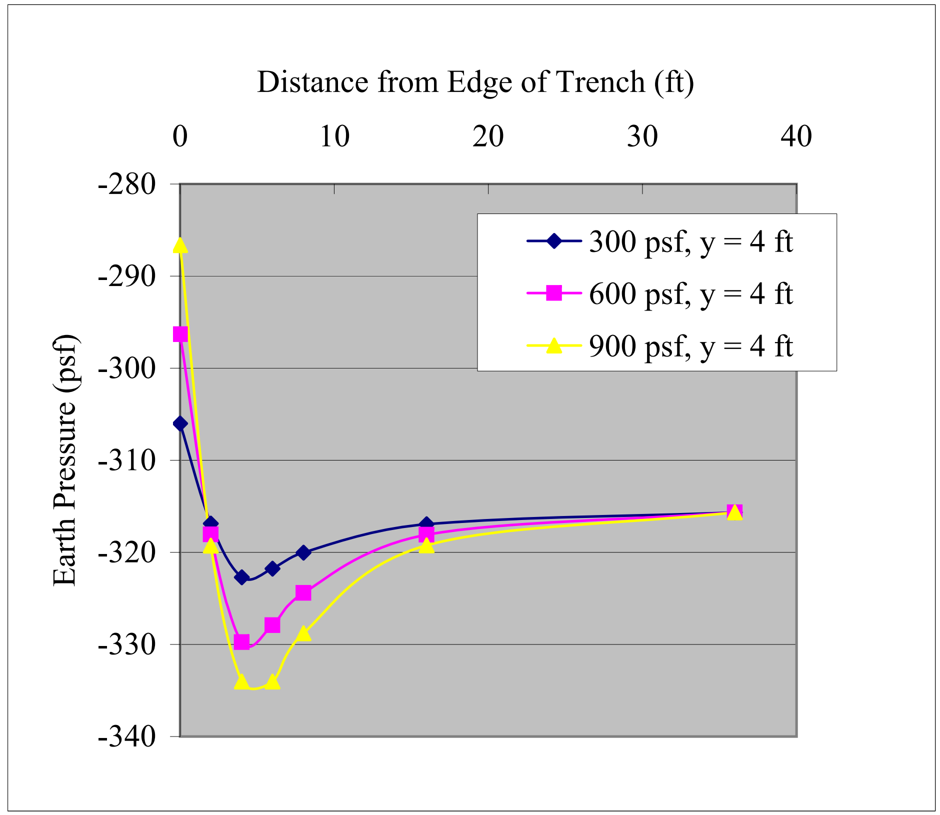


Table I-20 Surcharge distance from the edge of the trench vs. earth pressure at a depth of y = 4 ft for 300 psf, 600 psf, and 900 psf surcharges

	Earth Pressure (psf)		
300psf	y = 2 ft	y = 4 ft	y = 6 ft
0	-325.27	-305.99	-261.76
2	-350.57	-316.87	-272.02
4	-340.29	-322.70	-279.36
6	-335.18	-321.79	-282.30
8	-333.29	-320.03	-282.61
16	-332.18	-316.94	-279.62
36	-333.82	-315.66	-275.28
600psf	y = 2 ft	y = 4 ft	y = 6 ft
0	-316.72	-296.32	-248.25
2	-367.32	-318.07	-268.77
4	-346.75	-329.74	-283.45
6	-336.55	-327.91	-283.45
8	-336.55	-324.40	-289.95
16	-330.93	-318.08	-283.95
36	-333.82	-315.66	-275.28
900psf	y = 2 ft	y = 4 ft	y = 6 ft
0	-308.17	-286.65	-234.73
2	-384.07	-319.27	-265.51
4	-337.91	-334.03	-296.35
6	-337.91	-334.03	-296.35
8	-332.24	-328.77	-297.28
16	-329.68	-319.22	-288.28
36	-333.82	-315.66	-275.28



Appendix J: Parametric Study Tabulated Data – Varying Surcharge Location Laterally Along the Face of the Trench

The following pages provide the tabulated data from the models run to evaluate the effects of varying surcharge location laterally located along the trench in the X direction. Strongback material properties were held constant and strut loading was set to 1298 lbs/strut for all iterations. The soil properties were set to those of Soil 2. The earth pressure due to surcharge location laterally along the trench was located for X = 0 (the full width of the model), X = 18 (half the width of the model), and X = 25.5 (the portion of the model at the one end of the trench). Models were run for surcharge sizes of 300 psf. See figure J-1 for model orientation.

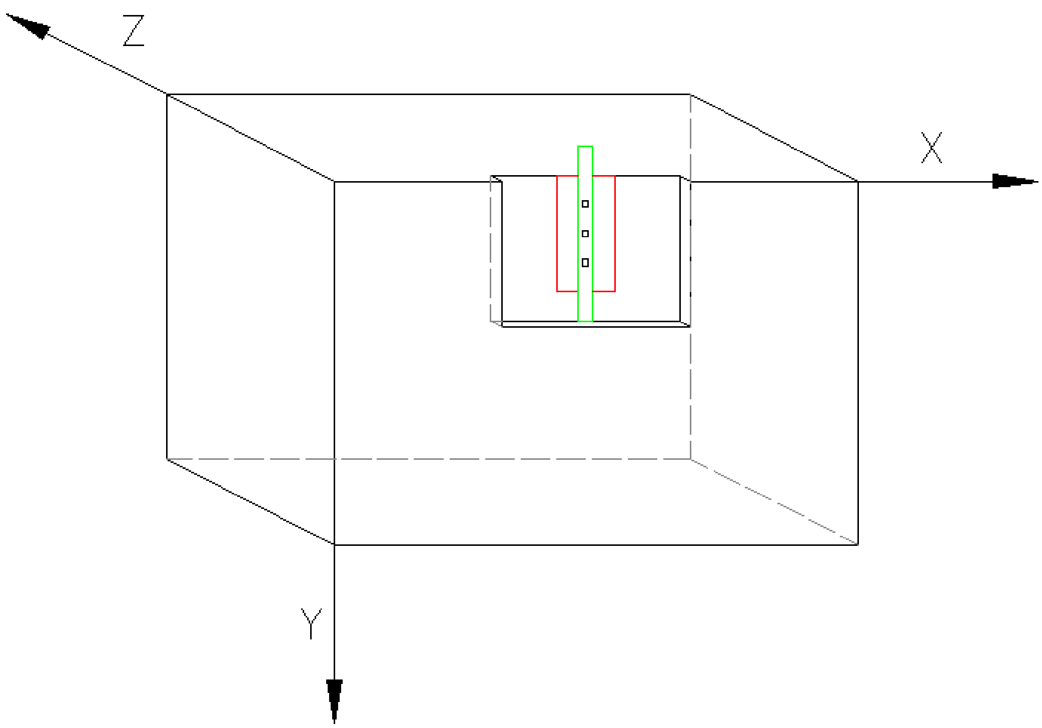
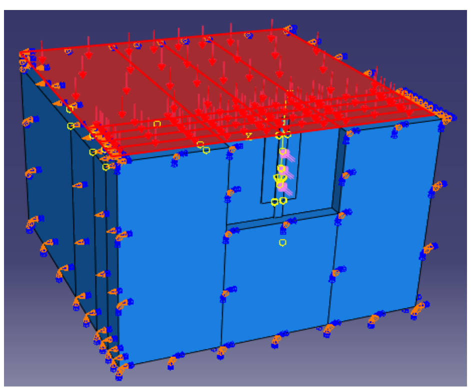


Figure J-1 Model orientation

Table J-1 Earth pressure behind the width of the panel with 300 psf surcharge up to the edge of the trench across the whole model (X = 0)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.47	-3.50	0.71	-3.50	3.25
-3.03	0.32	-3.05	3.80	-3.07	7.90
-2.45	8.96	-2.47	11.36	-2.49	23.43
-1.87	18.39	-1.90	25.23	-1.92	53.68
-1.29	-75.70	-1.32	-66.22	-1.35	-25.04
-0.72	-225.47	-0.75	-212.09	-0.78	-169.31
-0.06	-325.27	-0.11	-305.99	-0.16	-261.76
0.41	-302.65	0.38	-284.58	0.35	-239.31
0.97	-160.33	0.94	-157.56	0.91	-125.72
1.52	-21.49	1.50	-28.71	1.47	-0.48
2.08	24.63	2.05	21.80	2.03	48.72
2.63	0.49	2.61	1.35	2.59	11.50
3.18	-1.66	3.17	0.90	3.15	3.74
3.50	-1.52	3.50	-0.18	3.50	1.87



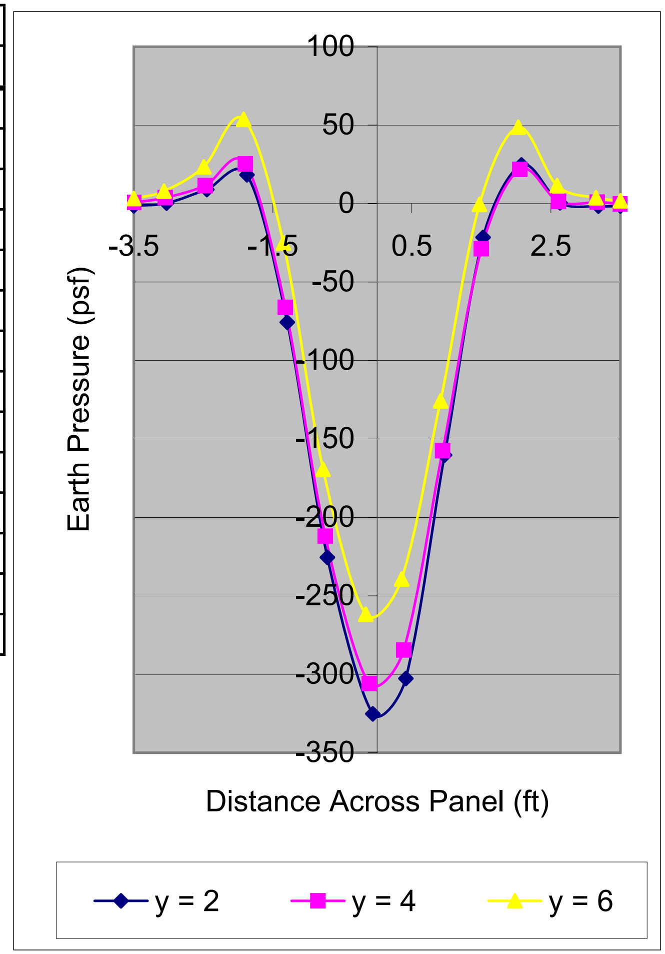
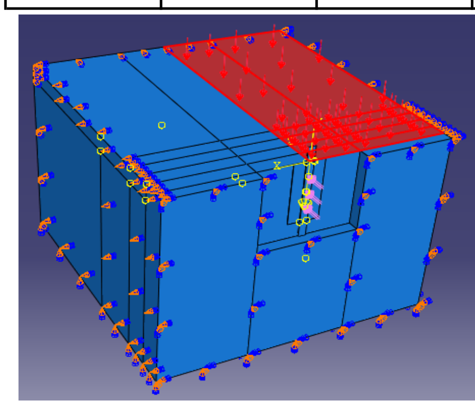


Table J-2 Earth pressure behind the width of the panel with 300 psf surcharge up to the edge of the trench across half of the model (X = 18)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-2.99	-3.50	-0.40	-3.50	4.04
-3.03	-6.20	-3.05	-0.56	-3.07	10.74
-2.45	-4.01	-2.47	2.74	-2.49	27.93
-1.87	17.19	-1.90	21.54	-1.92	46.83
-1.29	-79.23	-1.32	-71.47	-1.35	-35.30
-0.72	-237.91	-0.75	-221.79	-0.78	-176.47
-0.06	-331.08	-0.11	-311.73	-0.16	-268.06
0.41	-305.02	0.38	-288.43	0.35	-245.63
0.97	-166.34	0.94	-161.84	0.91	-133.15
1.52	-33.55	1.50	-33.35	1.47	-10.36
2.08	18.60	2.05	19.53	2.03	33.50
2.63	5.24	2.61	3.33	2.59	3.11
3.18	-0.60	3.17	0.65	3.15	1.78
3.50	-0.09	3.50	0.20	3.50	0.78



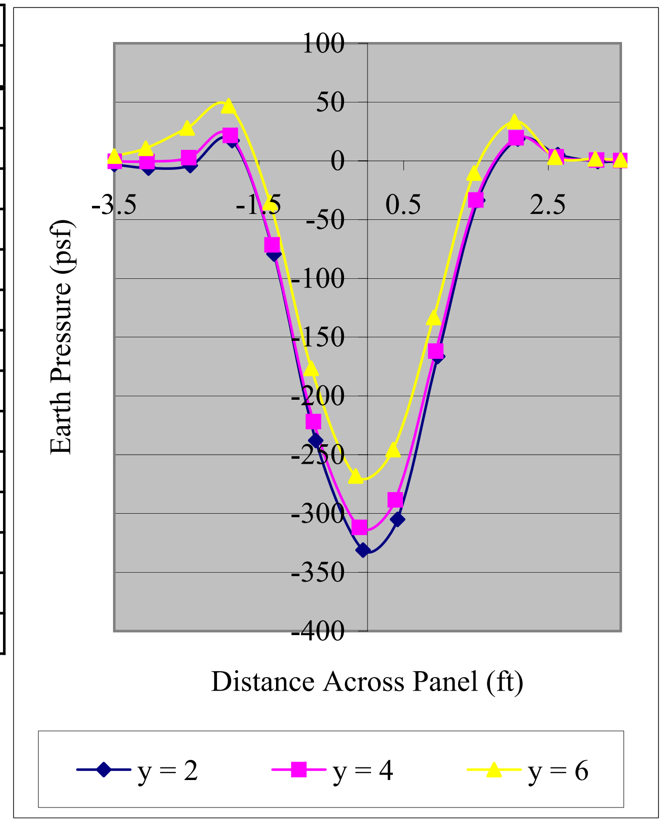
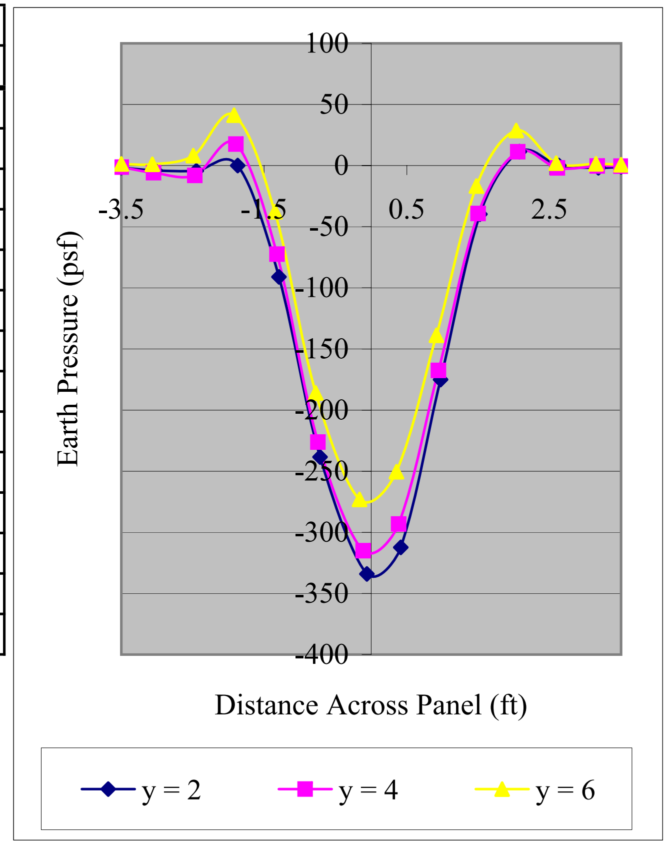
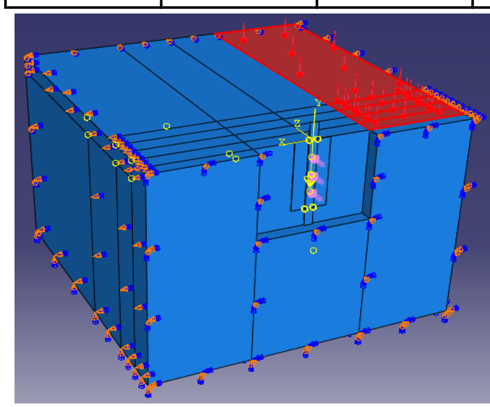


Table J-3 Earth pressure behind the width of the panel with 300 psf surcharge up to the edge of the trench for portion of model at one end of the trench (X = 25.5)

y = 2 ft		y = 4 ft		y = 6 ft	
Width (ft)	EP (psf)	Width (ft)	EP (psf)	Width (ft)	EP (psf)
-3.50	-1.72	-3.50	-1.24	-3.50	1.36
-3.03	-3.88	-3.05	-5.82	-3.07	1.08
-2.45	-4.26	-2.47	-8.11	-2.49	8.08
-1.87	-0.03	-1.90	17.53	-1.92	41.28
-1.29	-91.07	-1.32	-72.41	-1.35	-37.86
-0.72	-238.36	-0.75	-226.29	-0.78	-186.01
-0.06	-334.00	-0.11	-315.10	-0.16	-273.12
0.41	-312.35	0.38	-293.25	0.35	-250.45
0.97	-175.02	0.94	-167.62	0.91	-138.78
1.52	-39.75	1.50	-39.22	1.47	-16.72
2.08	11.39	2.05	11.27	2.03	28.69
2.63	0.08	2.61	-1.96	2.59	2.04
3.18	-1.74	3.17	-0.35	3.15	1.43
3.50	-0.96	3.50	-0.44	3.50	0.62





Appendix K: Panel (FinnForm) Laboratory Test Results and Young's Modulus Calculations

The following pages provide the laboratory test results of the panel (FinnForm) samples and the calculated modulus of elasticity to compare with the manufacturer supplied modulus of elasticity.

WOOD TESTING - To Failure

	Manufacturer Data	Average Test Result	Ratio
E (FF)	292320000 psf	**182784900** psf	**0.63**

FinnForm

b	2 in		b = sample width
h	0.75 in		h = smaple height
L	28 in		L = sample length
P	318.89439 lb		P = load
δ	1.67016 in		δ = defelction
I = 1/12*(b*h^3)	0.0703125 in^4		I = moment of inertia
E = (P*L^3)/(48*I*δ)	1241907.123 psi		E = Young's Modulus
	178834625.6 psf		

b	2 in
h	0.75 in
L	28 in
P	308.12643 lb
δ	1.58933 in
I = 1/12*(b*h^3)	0.0703125 in^4
E = (P*L^3)/(48*I*δ)	1261000.294 psi
	181584042.4 psf

b	2 in
h	0.75 in
L	28 in
P	266.98547 lb
δ	1.32872 in
I = 1/12*(b*h^3)	0.0703125 in^4
E = (P*L^3)/(48*I*δ)	1306936.453 psi
	188198849.2 psf

b	2 in
h	0.75 in
L	28 in
P	242.29392 lb
δ	1.24334 in
I = 1/12*(b*h^3)	0.0703125 in^4
E = (P*L^3)/(48*I*δ)	1267514.474 psi
	182522084.3 psf

WOOD TESTING - Elastic Range

	Manufacturer Data	Average Test Result	Ratio
E (FF)	292320000 psf	**211227022** psf	**0.72**

FinnForm

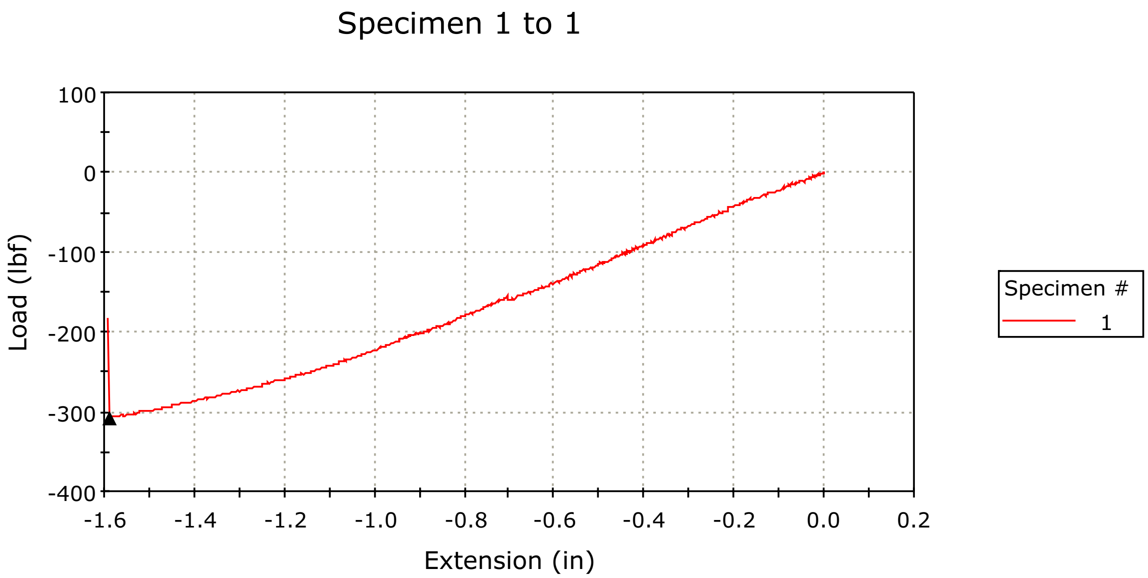
b	2	in	b = sample width
h	0.75	in	h = smaple height
L	28	in	L = sample length
P	145	lb	P = load
δ	0.6	in	δ = defelction
I = 1/12*(b*h^3)	0.0703125	in^4	I = moment of inertia
E = (P*L^3)/(48*I*δ)	1571871.605	psi	E = Young's Modulus
	226349511.1	psf	

b	2	in
h	0.75	in
L	28	in
P	180	lb
δ	0.8	in
I = 1/12*(b*h^3)	0.0703125	in^4
E = (P*L^3)/(48*I*δ)	1463466.667	psi
	210739200	psf

b	2	in
h	0.75	in
L	28	in
P	175	lb
δ	0.8	in
I = 1/12*(b*h^3)	0.0703125	in^4
E = (P*L^3)/(48*I*δ)	1422814.815	psi
	204885333.3	psf

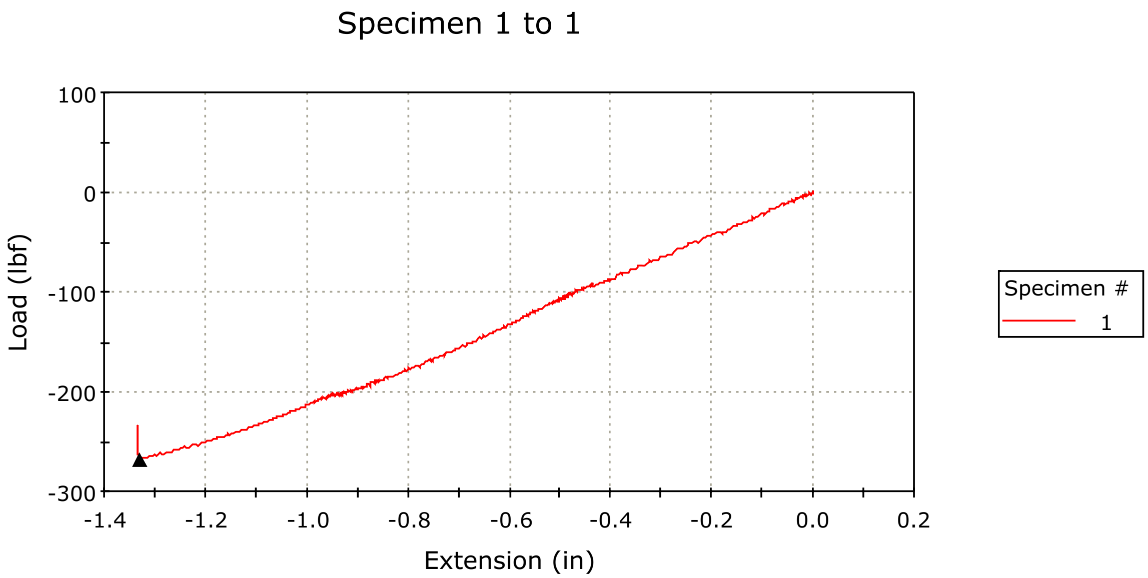
b	2	in
h	0.75	in
L	28	in
P	130	lb
δ	0.6	in
I = 1/12*(b*h^3)	0.0703125	in^4
E = (P*L^3)/(48*I*δ)	1409264.198	psi
	202934044.4	psf

3/4 x 2 x 30"



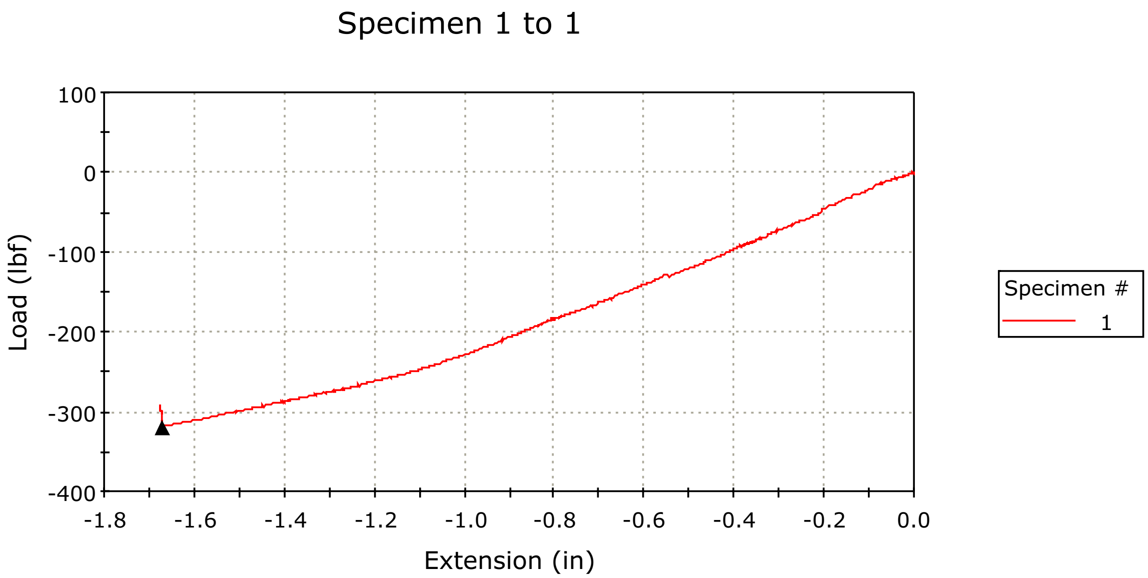
	Compressive extension at Maximum Compressive load (in)	Maximum Compressive load (lbf)
1	1.58933	308.12643

3/4 x 2 x 30"



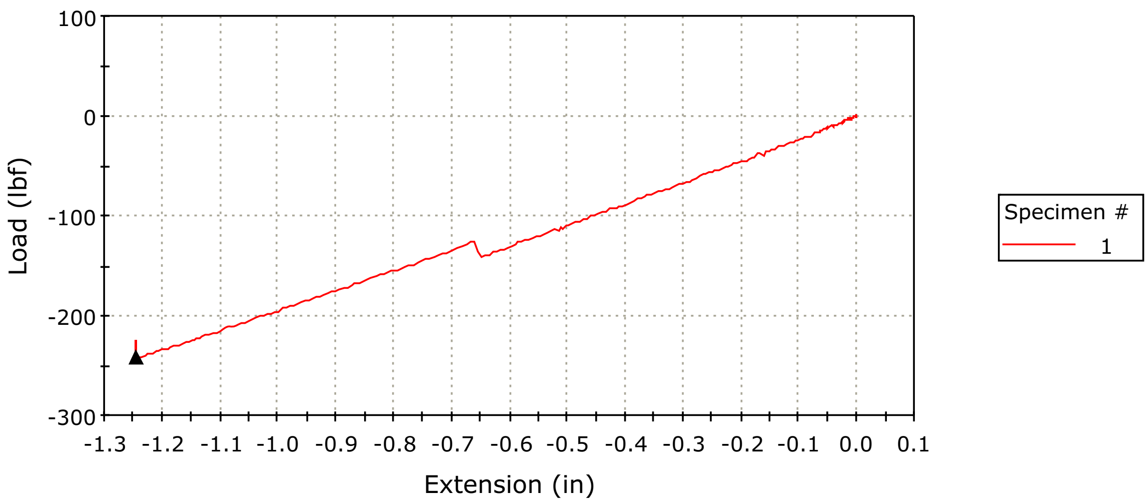
	Compressive extension at Maximum Compressive load (in)	Maximum Compressive load (lbf)
1	1.32872	266.98547

3/4 x 2 x 30"



	Compressive extension at Maximum Compressive load (in)	Maximum Compressive load (lbf)
1	1.67016	318.89439

Specimen 1 to 1



	Compressive extension at Maximum Compressive load (in)	Maximum Compressive load (lbf)
1	1.24334	242.29392

Appendix L: Strongback (#2 Kiln Dried Southern Yellow Pine) Laboratory Test Results and Young's Modulus Calculations

The following pages provide the laboratory test results of the strongback (#2 kiln dried Southern Yellow Pine) samples and the calculations to compare with the manufacturer supplied modulus of elasticity.

WOOD TESTING - To Failure

	Manufacturer Data	Average Test Result	Ratio
E (pine)	180000000 psf	**141524578** psf	**0.79**

2x12

b	2	in	b = sample width
h	1.5	in	h = smaple height
L	28	in	L = sample length
P	1444.74019	lb	P = load
δ	1.22639	in	δ = defelction
I = 1/12*(b*h^3)	0.5625	in^4	I = moment of inertia
E = (P*L^3)/(48*I*δ)	957792.6136	psi	E = Young's Modulus
	137922136.4	psf	
b	2	in	
h	1.5	in	
L	28	in	
P	1213.12714	lb	
δ	1.00439	in	
I = 1/12*(b*h^3)	0.5625	in^4	
E = (P*L^3)/(48*I*δ)	982006.2879	psi	
	141408905.5	psf	
b	2	in	
h	1.5	in	
L	28	in	
P	1249.71356	lb	
δ	1.00737	in	
I = 1/12*(b*h^3)	0.5625	in^4	
E = (P*L^3)/(48*I*δ)	1008629.808	psi	
	145242692.4	psf	

WOOD TESTING - Elastic Range

	Manufacturer Data	Average Test Result	Ratio
E (pine)	180000000 psf	**177875199** psf	**0.99**

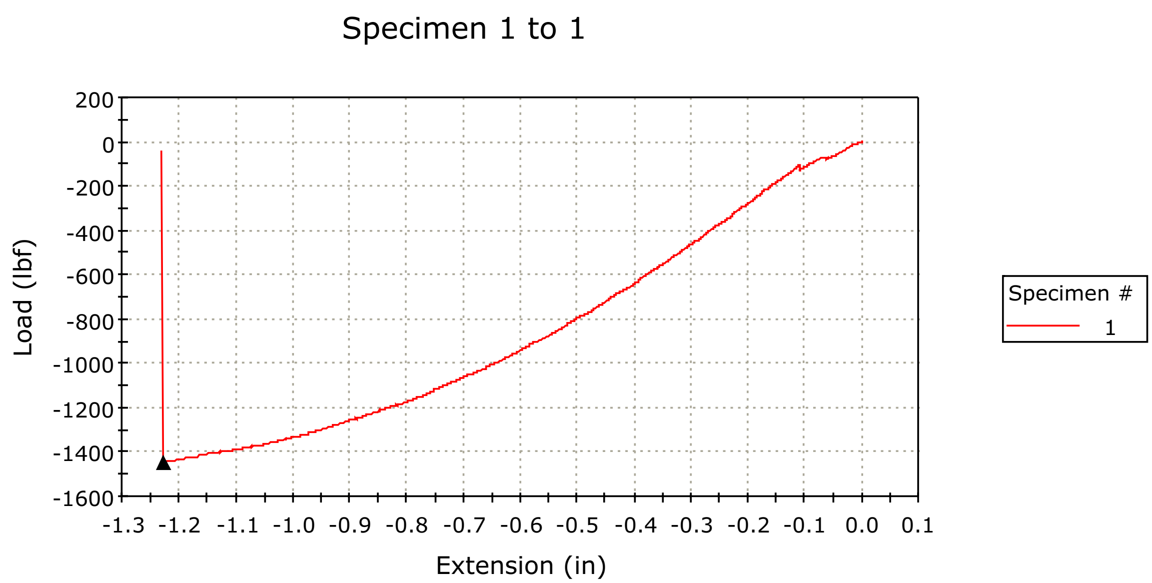
2x12

b	2	in	b = sample width
h	1.5	in	h = smaple height
L	28	in	L = sample length
P	800	lb	P = load
δ	0.5	in	δ = defelction
I = 1/12*(b*h^3)	0.5625	in^4	I = moment of inertia
E = (P*L^3)/(48*I*δ)	1300859.259	psi	E = Young's Modulus
	187323733.3	psf	

b	2	in
h	1.5	in
L	28	in
P	800	lb
δ	0.545	in
I = 1/12*(b*h^3)	0.5625	in^4
E = (P*L^3)/(48*I*δ)	1193448.862	psi
	171856636.1	psf

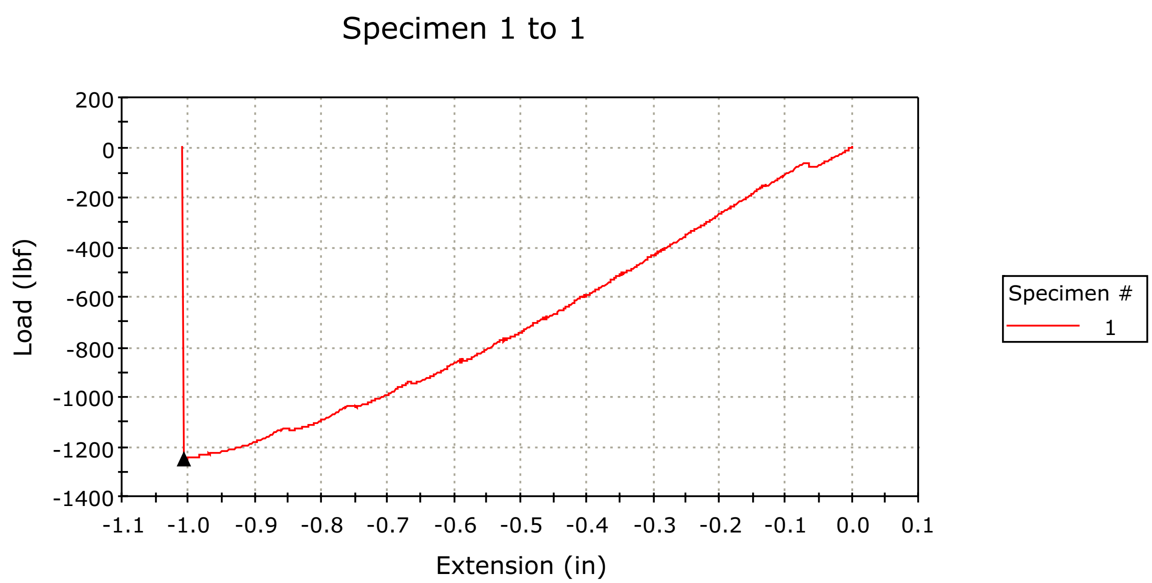
b	2	in
h	1.5	in
L	28	in
P	745	lb
δ	0.5	in
I = 1/12*(b*h^3)	0.5625	in^4
E = (P*L^3)/(48*I*δ)	1211425.185	psi
	174445226.7	psf

2 x 1-1/2 x 30"



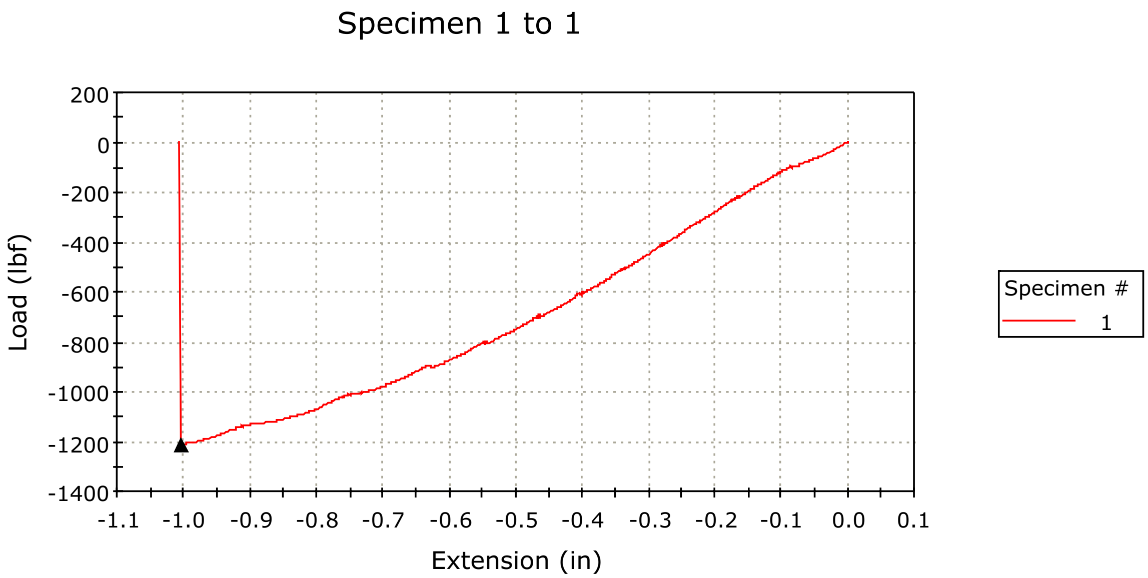
	Compressive extension at Maximum Compressive load (in)	Maximum Compressive load (lbf)
1	1.22639	1444.74019

2 x 1-1/2 x 30"



	Compressive extension at Maximum Compressive load (in)	Maximum Compressive load (lbf)
1	1.00737	1249.71356

2 x 1-1/2 x 30"



	Compressive extension at Maximum Compressive load (in)	Maximum Compressive load (lbf)
1	1.00439	1213.12714

Appendix M: Frederick Soil Lab Test Results

Approximately thirty pounds of soil was obtained from the Frederick MD trench site for testing. The sample was collected from the spoil piles on both sides of the trench and mixed together. The Standard Method for Particle-Size Analysis of Soils (ASTM D 422-63) was performed at the University of Maryland Civil Engineering Material Testing Lab on a soil samples from the trench to determine its classification. Fifty four percent of the visible grains were larger than the No. 4 sieve (4.74mm) as shown in table M-3, therefore the sample was a gravel. However, visual inspection of the soil indicates as a whole it was granular in nature. It appeared to consist of gravel and larger rock held together with small amounts of clay. The larger rocks (14 in ±) were abundant, but could not be collected as part of the test sample. Tables M-1, M-2 and M-3 and Graph M-1 summarize results of testing the sample.

Table M-1 Unit weight calculation of Frederick soil

	Test 1	Test 2
Sample Weight (g):	964.8	899
Volume of Sample (cm^3):	447.8	497
Unit Weight of Samples (g/cm^3):	2.2	1.8
Average Unit Weight (g/cm^3):	2.0	
1 g/cm^3 = 62.43 lb/ft^3		
Unit Weight of Samples (lb/ft^3):	**123.7**	

Unified Soil Classification (ASTM D 2487) of Frederick soil:

Coarse Grained soils are divided into gravelly (G) or sandy (S) soils in accordance with whether or less than 50% of the visible grains are larger than sieve No. 4 sieve (4.74mm). They are each divided further into four groups:

W: clean (less than 5% finer than 0.0074mm); well graded (uniformity coefficient Cu greater than 4 for gravels or 6 for sands, and coefficient of curvature Cc between 1 and 3)

P: clean (less than 5% finer than 0.0074mm); poorly graded (Cu less than 4 for gravels or 6 for sands, and or gap-graded because Cc is not between 1 and 3)

C: dirty (more than 12% finer than 0.0074mm); plastic clayey fines (Ip greater than 7%, also plots above A-line in plasticity chart)

M: dirty (more than 12% finer than 0.0074mm); non-plastic silty fines (Ip less than 4%, or plots below A-line in plasticity chart)

Table M-2 Water content of Frederick soil

Weight of Can (g)	Weight of Can + wet Soil (g)	Weight of Can + dry soil	Water Content (%)
203.2	620.2	549.1	20.55507
199.8	678.9	590.2	22.72029
		average:	21.63768

Table M-3 Results of Standard Test Method for Particle-Size Analysis of Soils (ASTM D422-63)

Sieve No	Sieve Opening (mm)	Weight Retained on Each Sieve (g)	Percent of Weight Retained on Each Sieve	Cumulative Percent Retained	Percent Finer
3.4 in	19.05	472	23.45	23.45	76.55
3/8 in	9.525	349	17.34	40.78	59.22
4	4.75	267	13.26	54.05	45.95
8	2.36	333	16.54	70.59	29.41
30	0.6	319	15.85	86.44	13.56
50	0.3	134	6.66	93.09	6.91
100	0.15	61	3.03	96.13	3.87
200	0.075	53	2.63	98.76	1.24
Pan		25			
	Total(g):	2013			

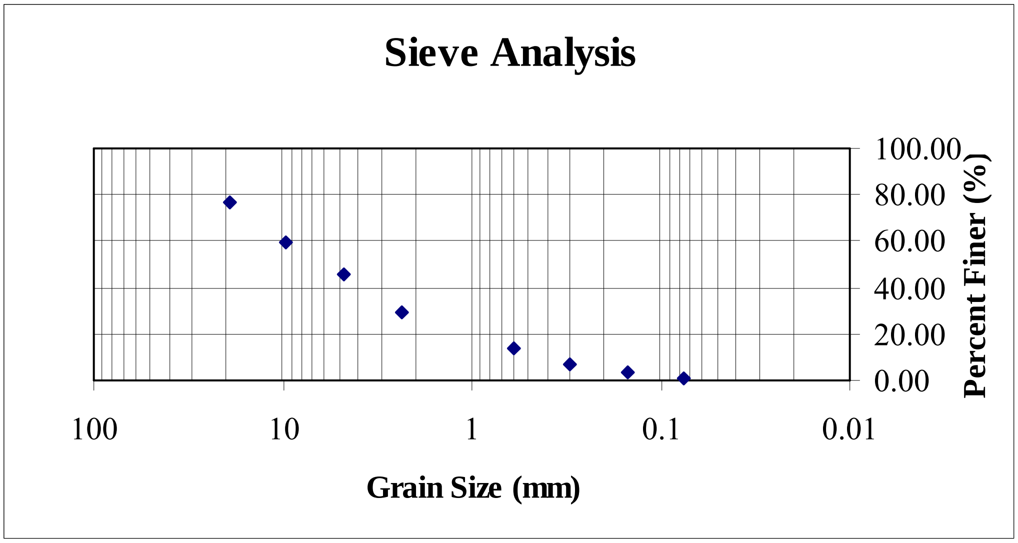


Figure M-1 Results of Standard Test Method for Particle-Size Analysis of Soils (ASTM D422-63)

Fifty four percent of the visible grains were larger than the No. 4 sieve (4.74mm), therefore the sample is granular.

Appendix N: Vishay Instructional Bulletin B-127-14 Strain Gauge Installations with M-Bond 200 Adhesive

The following pages are the Vishay Instructional Bulletin B-127-14 Strain Gauge Installations with M-Bond 200 Adhesive. They illustrate the installation process for the strain gauges.

Instruction Bulletin B-127-14

Vishay Micro-Measurements

Strain Gage Installations with M-Bond 200 Adhesive

INTRODUCTION

Vishay Micro-Measurements Certified M-Bond 200 is an excellent general-purpose laboratory adhesive because of its fast room-temperature cure and ease of application. When properly handled and used with the appropriate strain gage, M-Bond 200 can be used for high-elongation tests in excess of 60 000 microstrain, for fatigue studies, and for one-cycle proof tests to over +200 °F [+95 °C] or below -300 °F [-185°C]. The normal operating temperature range is -25° to +150°F [-30° to +65°C]. M-Bond 200 is compatible with all Vishay Micro-Measurements strain gages and most common structural materials. When bonding to plastics, it should be noted that for best performance the adhesive flowout should be kept to a minimum. For best reliability, it should be applied to surfaces between the temperatures of +70° and +85°F [+20° to +30°C], and in a relative humidity environment of 30% to 65%.

M-Bond 200 catalyst has been specially formulated to control the reactivity rate of this adhesive. The catalyst should be used sparingly for best results. Excessive catalyst can contribute many problems; e.g., poor bond strength, age-embrittlement of the adhesive, poor glueline thickness control, extended solvent evaporation time requirements, etc.

Since M-Bond 200 bonds are weakened by exposure to high humidity, adequate protective coatings are essential. This adhesive will gradually become harder and more brittle with time, particularly if exposed to elevated temperatures. For these reasons, M-Bond 200 is not generally recommended for installations exceeding one or two years.

For proper results, the procedures and techniques presented here should be used with qualified Vishay Micro-Measurements installation accessory products (refer to Catalog A-110). Those used in this procedure are:

- CSM Degreaser or GC-6 Isopropyl Alcohol
- Silicon Carbide Paper
- M-Prep Conditioner A
- M-Prep Neutralizer 5A
- GSP-1 Gauze Sponges
- CSP-1 Cotton Applicators
- PCT- 2M Gage Installation Tape

SHELF AND STORAGE LIFE

M-Bond 200 adhesive has a shelf life of three months at +75°F [+24°C] after opening and with the cap placed back onto the bottle immediately after each use.

Note: To ensure the cap provides a proper seal, the bottle spout should be wiped clean and dry before replacing the cap.

Unopened M-Bond 200 adhesive may be stored up to three months at +75°F [+24°C] or six months at +40°F [+5°C].

HANDLING PRECAUTIONS

M-Bond 200 is a modified alkyl cyanoacrylate compound. Immediate bonding of eye, skin or mouth may result upon contact. Causes irritation. The user is cautioned to: (1) avoid contact with skin; (2) avoid prolonged or repeated breathing of vapors; and (3) use with adequate ventilation. For additional health and safety information, consult the Material Safety Data Sheet, which is available upon request.

Note: Condensation will rapidly degrade adhesive performance and shelf life; after refrigeration the adhesive must be allowed to reach room temperature before opening, and refrigeration after opening is not recommended.

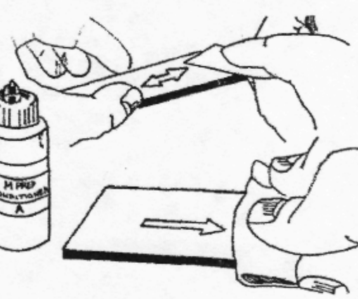
GAGE APPLICATION TECHNIQUES

The installation procedure presented on the following pages is somewhat abbreviated and is intended only as a guide in achieving proper gage installation with M-Bond 200. Vishay Micro-Measurements Application Note B-129 presents recommended procedures for surface preparation, and lists specific considerations which are helpful when working with most common structural materials.

Step 1

Thoroughly degrease the gaging area with solvent, such as CSM Degreaser or GC-6 Isopropyl Alcohol. The former is preferred, but there are some materials (e.g., titanium and many plastics) that react with strong solvents. In these cases, GC-6 Isopropyl Alcohol should be considered. All degreasing should be done with uncontaminated solvents-thus the use of "one-way" containers, such as aerosol cans, is highly advisable.

Step 2



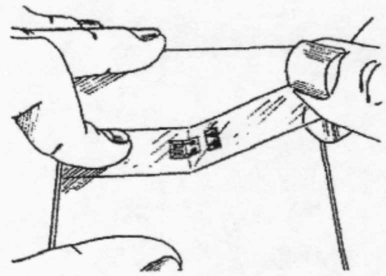
Preliminary dry abrading with 220- or 320-grit silicon- carbide paper is generally required if there is any surface scale or oxide. Final abrading is done by using 320-grit silicon-carbide paper on surfaces thoroughly wetted with M-Prep Conditioner A; this is followed by wiping dry with a gauze sponge. Repeat this wet abrading process with 400-grit silicon-carbide paper, then dry by slowly wiping through with a gauze sponge.

Using a 4H pencil (on aluminum) or a ballpoint pen (on steel), burnish (do not scribe) whatever alignment marks are needed on the specimen. Repeatedly apply M-Prep Conditioner A and scrub with cotton-tipped applicators until a clean tip is no longer discolored. Remove all residue and Conditioner by again slowly wiping through with a gauze sponge. Never allow any solution to dry on the surface because this invariably leaves a contaminating film and reduces chances of a good bond.

Step 3

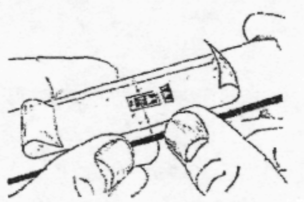
Now apply a liberal amount of M-Prep Neutralizer 5A and scrub with a cotton-tipped applicator. With a single, slow wiping motion of a gauze sponge, carefully dry this surface. Do not wipe back and forth because this may allow contaminants to be redeposited.

Step 4



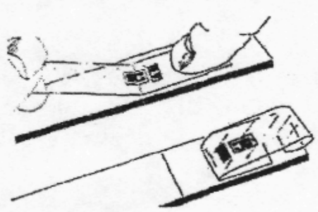
Using tweezers to remove the gage from the transparent envelope, place the gage (bonding side down) on a chemically clean glass plate or gage box surface. If a solder terminal will be used, position it on the plate adjacent to the gage as shown. A space of approximately 1/16 in [1.6 mm] or more where space allows or application requires should be left between the gage backing and terminal. Place a 4- to 6-in [100- to 150-mm] piece of Vishay Micro-Measurements PCT-2M gage installation tape over the gage and terminal. Take care to center the gage on the tape. Carefully lift the tape at a shallow angle (about 45 degrees to specimen surface), bringing the gage up with the tape as illustrated above.

Step 5



Position the gage/tape assembly so that the triangle alignment marks on the gage are over the layout lines on the specimen. If the assembly appears to be misaligned, lift one end of the tape at a shallow angle until the assembly is free of the specimen. Realign properly, and firmly anchor at least one end of the tape to the specimen. Realignment can be done without fear of contamination by the tape mastic if Vishay Micro-Measurements PCT-2M gage installation tape is used, because this tape will retain its mastic when removed.

Step 6



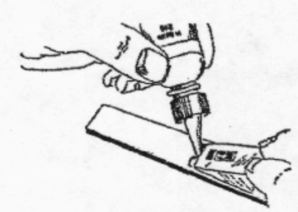
Lift the gage end of the tape assembly at a shallow angle to the specimen surface (about 45 degrees) until the gage and terminal are free of the specimen surface. Continue lifting the tape until it is free from the specimen approximately 1/2 in [10 mm] beyond the terminal. Tuck the loose end of the tape under and press to the specimen surface so that the gage and terminal lie flat, with the bonding surface exposed.
Note: Vishay Micro-Measurements gages have been treated for optimum bonding conditions and require no pre-cleaning before use unless contaminated during handling. If contaminated, the back of any gage can be cleaned with a cotton-tipped applicator slightly moistened with M-Prep Neutralizer 5A.

Step 7

M-Bond 200 catalyst can now be applied to the bonding surface of the gage and terminal. M-Bond 200 adhesive will harden without the catalyst, but less quickly and reliably. Very little catalyst is needed, and it should be applied in a thin, uniform coat. Lift the brush-cap out of the catalyst bottle and wipe the brush approximately 10 strokes against the inside of the neck of the bottle to wring out most of the catalyst. Set the brush down on the gage and swab the gage backing. Do not stroke the brush in a painting style, but slide the brush over the entire gage surface and then the terminal. Move the brush to the adjacent tape area prior to lifting from the surface. Allow the catalyst to dry at least one minute under normal ambient conditions of +75°F [+24°C] and 30% to 65% relative humidity before proceeding.

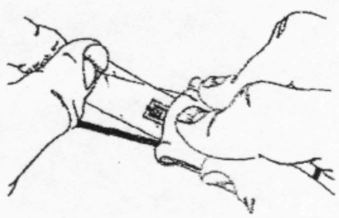
Note: The next three steps must be completed in the sequence shown, within 3 to 5 seconds. Read Steps 8, 9, and 10 before proceeding.

Step 8



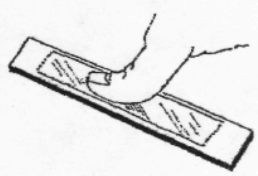
Lift the tucked-under tape end of the assembly, and, holding in the same position, apply one or two drops of M-Bond 200 adhesive at the fold formed by the junction of the tape and specimen surface. This adhesive application should be approximately 1/2 in [13 mm] outside the actual gage installation area. This will insure that local polymerization that takes place when the adhesive comes in contact with the specimen surface will not cause unevenness in the gage glueline.

Step 9



Immediately rotate the tape to approximately a 30-degree angle so that the gage is bridged over the installation area. While holding the tape slightly taut, slowly and firmly make a single wiping stroke over the gage/tape assembly with a piece of gauze bringing the gage back down over the alignment marks on the specimen. Use a firm pressure with your fingers when wiping over the gage. A very thin, uniform layer of adhesive is desired for optimum bond performance.

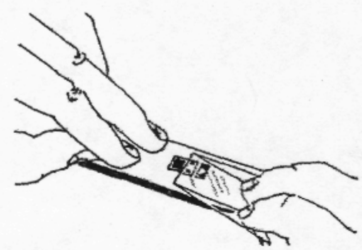
Step 10



Immediately upon completion of wipe-out of the adhesive, firm thumb pressure must be applied to the gage and terminal area. This pressure should be held for at least one minute. In low-humidity conditions (below 30%), or if the ambient temperature is below +70°F [+20°C], this pressure application time may have to be extended to several minutes.

Where large gages are involved, or where curved surfaces such as fillets are encountered, it may be advantageous to use preformed pressure padding during the operation. Pressure-application time should again be extended due to the lack of "thumb heat" which helps to speed adhesive polymerization. Wait two minutes before removing tape.

Step 11



The gage and terminal strip are now solidly bonded in place. It is not necessary to remove the tape immediately after gage installation. The tape will offer mechanical protection for the grid surface and may be left in place until it is removed for gage wiring. To remove the tape, pull it back directly over itself, peeling it slowly and steadily off the surface. This technique will prevent possible lifting of the foil on open-faced gages or other damage to the installation.

FINAL INSTALLATION PROCEDURE

1. Referring to Vishay Micro-Measurements Catalog A-110, select appropriate solder and attach leadwires. Prior to any soldering operations, open-faced gage grids should be masked with PDT-1 drafting tape to prevent possible damage.

2. Remove the solder flux with Rosin Solvent, RSK-1.

3. Select and apply protective coating according to the protective coating selection chart found in Catalog A-110.

Appendix O: York Soil Lab Test Results

Approximately thirty pounds of soil was obtained from the York PA trench site for testing. The sample was collected from the spoil piles on both sides of the trench and mixed together. Atterberg limit (ASTM D 4318-05 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soil) and water content (ASTM D 2216 Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass) lab tests were performed on the sample to determine classification in order to estimate a Young's Modulus and Poisson's Ratio. The liquid limit and plasticity index determined by the testing indicated the soil is a borderline inorganic silt/inorganic with lower liquid limit (CL-ML).

Table O-1 York soil unit weight

	Test 1	Test 2
Sample Weight (g):	755.4	730.6
Volume of Sample (cm^3):	428.4	356.6
Unit Weight of Samples (g/cm^3):	1.8	2.1
Average Unit Weight (g/cm^3):	1.9	
$1\ g/cm^3 = 62.42796\ lb/ft^3$		
Unit Weight of Samples (lb/ft^3):	**119.0**	

Unified Soil Classification of Frederick trench (Terzaghi et al. 1996):

> The fine grained soils are divided into three groups: inorganic silts (M), inorganic (C), and organic silts and clays (O). The soils are further divided into those having liquid limits lower than 50% (L), or higher (H).
>
> The distinction between the inorganic clays C and the inorganic silts M and organic soils O is made on the basis of a modified plasticity chart (Figure O-2). Soils CH and CL are represented by points above the A-line, whereas soils OH, OL, and MH correspond to positions below. Soils ML, except for a few clayey fine sands, are also represented by points below the A-line. The organic soils O are distinguished from the inorganic soils M and C by their characteristic odor and dark color or, in doubtful instances, by the influence of oven-drying on

the liquid limit. Borderline materials are represented by a double symbol, as CL-ML.

Table O-2 Water content of York soil

Weight of Can (g)	Weight of Can + wet Soil (g)	Weight of Can + dry soil	Water Content (%)
17.1	110	95	19.25546
17.8	104.9	88.2	23.72159
14.3	99.4	86.7	17.54144
		average:	20.17283

Table O-3 Liquid limit of York soil

Weight of Can (g)	Weight of Can + wet Soil (g)	Weight of Can + dry soil	Water Content (%)	Number of Blows , N	Liquid Limit
17.8	33.7	30.3	27.20	35	28.28
17.7	33.5	29.9	29.51	18	
14.2	34	29.3	31.13	8	

Liquid limit is 28.28% which is less than 50%, so the soil has a low liquid limit per the Unified Soil Classification.

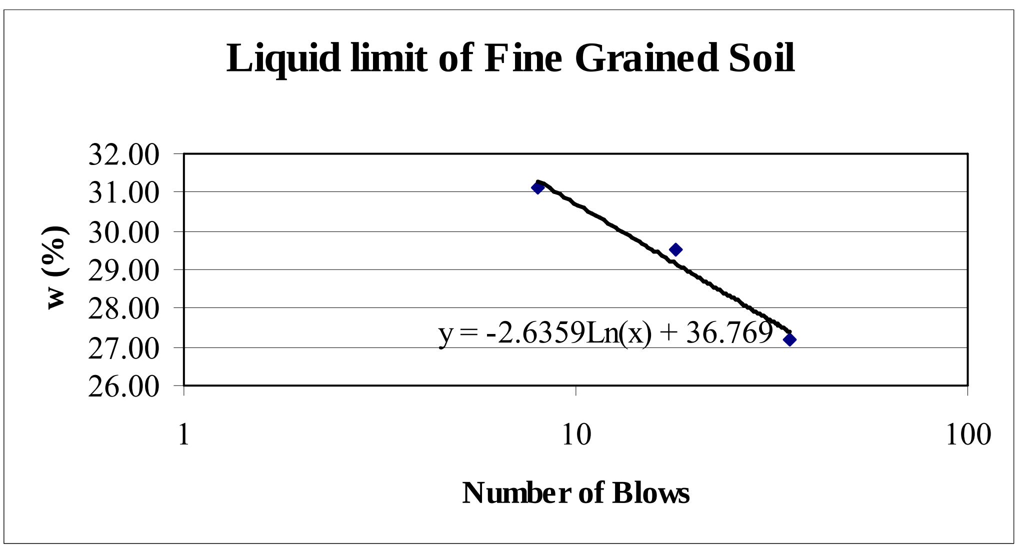


Figure O-1 Liquid limit of York soil

Table O-4 Plastic limit of York soil

Weight of Can (g)	Weight of Can + wet Soil (g)	Weight of Can + dry soil	Water Content (%)	Plastic Limit
14.6	38	33.9	21.24	21.24

Plasticity Index = LL – PL = 28.28 – 21.24 = 7.04

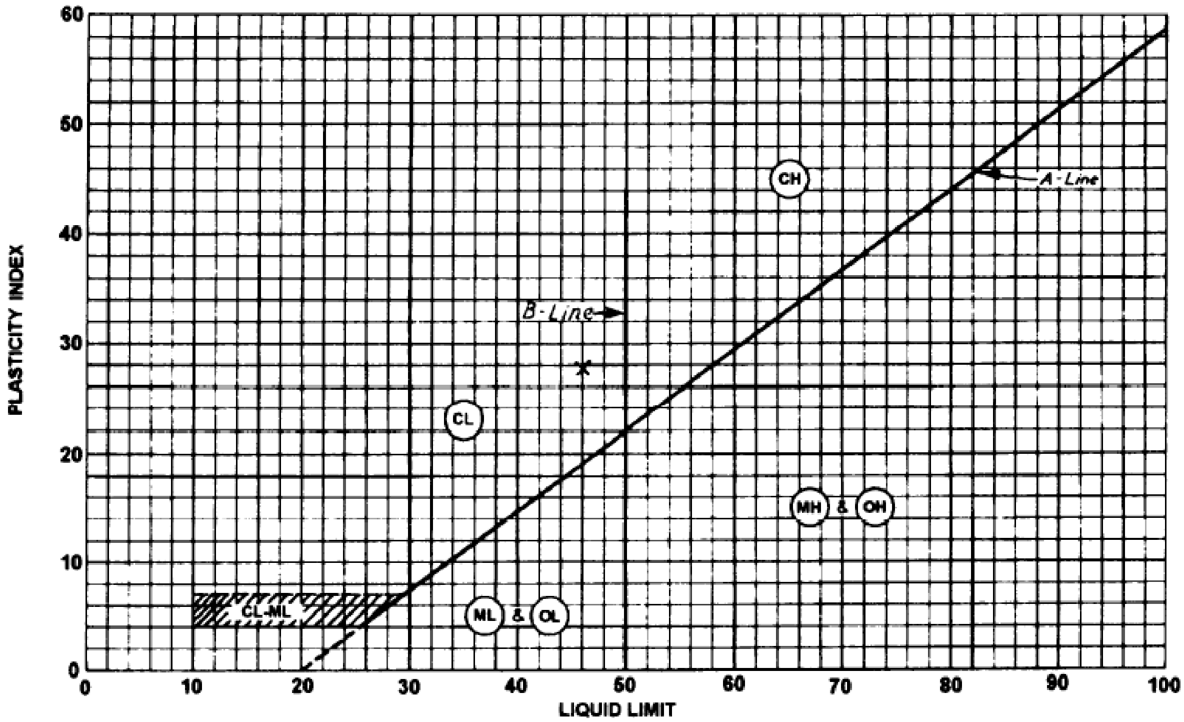


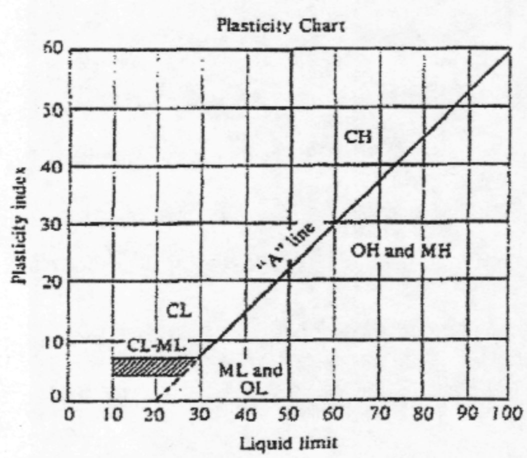
Figure O-2 Plasticity chart: LL = 28.28 & PI = 7.04, therefore subject soil is CL-ML

Appendix P: York County Fire School Soil Report – Soil Properties (ESC 2007)

The following pages are an excerpt from the March 2007 ECS LLC, Mid-Atlantic *Report of Subsurface Exploration and Geotechnical Engineering Analysis for Proposed York County Fire School Burn Tower, Manchester Township, York County, Pennsylvania* that present the results of independent laboratory test results on the York site soil.

Unified Soil Classification System (ASTM D-2487)

Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sive size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP; More than 12 percent: GM, GC, SM, SC; 5 to 12 percent: *Borderline* cases requiring dual symbols[b]	$C_u = D_{60}/D_{10}$ greater than 4; $C_c = (D_{30})^2/D_{10} \times D_{60}$ between 1 and 3	
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable Amount of fines)	GM[a] d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are *borderline* cases requiring the use of dual symbols
			GC	Clayey Gravels, gravel-sand-clay mixtures		Atterberg Limits below "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = D_{60}/D_{10}$ greater than 6; $C_c = (D_{30})^2/D_{10} \times D_{60}$ between 1 and 3	
			SP	Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a] d / u	Silty sands, sand-silt mixtures		Atterberg limits above "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are *borderline* cases requiring the use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine Grained Soils (More than half of material is smaller than No. 200 sieve size)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt	Peat and other highly organic soils			



[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing the characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

From Winterkorn and Fang, 1975

REFERENCE NOTES FOR BORING LOGS

I. Drilling Sampling Symbols:

SS	Split Spoon Sampler	ST	Shelby Tube Sampler
RC	Rock Core, NX, BX, AX	PM	Pressuremeter
DC	Dutch Cone Penetrometer	RD	Rock Bit Drilling
BS	Bulk Sample of Cuttings	PA	Power Auger (no sample)
HAS	Hollow Stem Auger	WS	Wash Sample

II. Correlation of Penetration Resistances to Soil Properties:

Standard Penetration (Blows/Ft) refers to the blows per foot of a 140 lb. Hammer falling 30 inches on a 2-inch OD split spoon sampler, as specified in ASTM D-1586. The blow count is commonly referred to as the N value.

A. Non-Cohesive Soils (Silt, Sand, Gravel and Combinations)

Density		*Relative Properties*	
Under 3 blows/ft.	Very Loose	Adjective Form	36% to 49%
4 to 6 blows/ft.	Loose	With	21% to 35%
7 to 10 blows/ft.	Firm	Some	11% to 20%
11 to 30 blows/ft.	Medium Dense	Trace	1% to 10%
31 to 50 blows/ft.	Dense		
51 to 80 blows/ft.	Very Dense		
Over 80 blows/ft.	Extremely Dense		

Particle Size Identification

Boulders		8 inches or larger
Cobbles		3 to 8 inches
Gravel	Coarse	1 to 3 inches
	Medium	½ to 1 inch
	Fine	¼ to ½ inch
Sand	Coarse	2.00mm to ¼ inch (dia. of lead pencil)
	Medium	0.42 to 2.00mm (dia. of broom straw)
	Fine	0.074 to 0.42mm (dia. of human hair)
Silt and Clay		0 .0 to 0.074mm (particles cannot be seen)

B. Cohesive Soils (Clay, Silt, and Combinations)

Blows/Ft	*Consistency*	*Unconfined Comp. Strength Q_P(tsf)*	*Degree of Plasticity*	*Plasticity Index*
Under 4	Very Soft	Under 0.25	None to Slight	0 - 4
4 to 5	Soft	0.25-0.49	Slight	5 – 7
6 to 10	Med. Stiff	0.50-0.99	Medium	8- 22
11 to 15	Stiff	1.00-1.99	High to Very High	Over 22
16 to 30	Very Stiff	2.00-3.00		
31 to 50	Hard	4.00-8.00		
Over 51	Very Hard	Over 8.00		

III. Water Level Measurement Symbols

WL	Water Level	BCR	Before Casing Removal
WS	While Sampling	ACR	After Casing Removal
WD	While Drilling	WCI	Wet Cave-In
		DCI	Dry Cave-In

The water levels are those water levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in a granular soil. In clay and plastic silts, the accurate determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally applied.

CLIENT	JOB #	BORING #	SHEET
Elliot Leboeuf & Associates	18.1438	B-1	1 OF 1

ECS LLC MID-ATLANTIC

PROJECT NAME	ARCHITECT-ENGINEER
York County Fire School	

SITE LOCATION
330 Emig Road, Emigsville, PA

-O- CALIBRATED PENETROMETER TONS/FT.² 1 2 3 4 5+

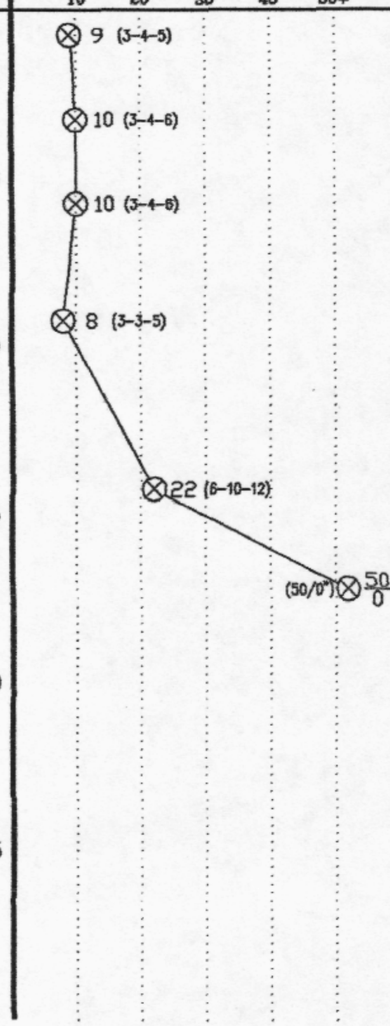
PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY
RQD% — — — REC.% ——
20% — 40% — 60% — 80% — 100%

⊗ STANDARD PENETRATION BLOWS/FT. 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS
BOTTOM OF CASING — LOSS OF CIRCULATION 100%
SURFACE ELEVATION 100.0

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
0	1	SS	18	18	ASPHALT DEPTH 5" / GRAVEL DEPTH 9"	100	9 (3-4-5)
	2	SS	18	18	Silty CLAY, With Fine to Medium Gravel, Trace Fine Sand, Dark Brown, Moist, Medium Stiff, (CL/ML)		10 (3-4-6)
5	3	SS	18	18		95	10 (3-4-6)
	4	SS	18	18			8 (3-3-5)
10					Sandy SILT, With Fine to Medium Gravel, Purplish Brown, Moist, Medium Dense, (ML/SM)	90	
	5	SS	18	18			22 (6-10-12)
15						85	
	6	SS	1	0	AUGER REFUSAL @ 17.0'		(50/0") 50/0
20						80	
25						75	
30							



THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL Dry — WS OR WD	BORING STARTED 04/19/07		
WL(BCR) Dry — WL(ACR) Dry	BORING COMPLETED 04/19/07	CAVE IN DEPTH @ 9.5	
WL	RIG CME 450 — FOREMAN S. Lind	DRILLING METHOD 2 1/4" HSA	

DKK (04-25-07) DKK (05-16-07)

CLIENT	JOB #	BORING #	SHEET
Elliot Leboeuf & Associates	18.1438	B-2	1 OF 1

ECS LLC MID-ATLANTIC

PROJECT NAME: York County Fire School

ARCHITECT-ENGINEER:

SITE LOCATION: 330 Emig Road, Emigsville, PA

-O- CALIBRATED PENETROMETER TONS/FT.² 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % Δ

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — — — REC.% ——— 20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT. 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION 100.0

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
0	1	SS	18	18	ASPHALT DEPTH 5" / GRAVEL DEPTH 7"	100	5 (2-2-3)
	2	SS	18	12	Silty CLAY, With Fine to Medium Gravel, Trace Fine Sand, Brown to Purplish Brown, Moist, Soft to Medium Stiff, (CL/ML)		8 (2-3-5)
5	3	SS	18	18		95	8 (4-5-3)
	4	SS	18	18			13 (4-6-7)
10						90	
	5	SS	18	18			6 (2-3-3)
15	6	SS	1	0	AUGER REFUSAL @ 16.0'	85	(50/0") 50/0
20						80	
25						75	
30							

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL Dry — WS OR WD	BORING STARTED 04/19/07		
WL(BCR) Dry — WL(ACR) Dry	BORING COMPLETED 04/19/07	CAVE IN DEPTH @ 8.0	
WL	RIG CME 450 FOREMAN S. Lind	DRILLING METHOD 2 1/4" HSA	

DKK (04-25-07) DKK (06-18-07)

ECS MID-ATLANTIC, LLC.

York, Pennsylvania

Laboratory Testing Summary

Date: 3-May-07

Project Number: 18.1438 **Project Name:** York Co. Fire School

Project Engineer: DKK **Principal Engineer:** WDF **Summary By:** DKK

Boring Number	Sample Number	Depth (feet)	Moisture Content (%)	Soil Description	LL	PL	PI	Percent Passing No. 200 Sieve	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	CBR	Other
B-1	S-1	1.0-2.5'	18.9	Silty CLAY (CL/ML)								
B-1	S-2	2.5-4.0'	17.1	Silty CLAY (CL/ML)								
B-1	S-3	5.0-6.5'	17.7	Silty CLAY (CL-ML)	25	20	5					
B-1	S-4	8.5-10.0'	15.3	Silty CLAY (CL/ML)								
B-1	S-5	13.5-15.0'	10.8	Sandy SILT (ML/SM)				25.7				
B-2	S-1	1.0-2.5'	24.2	Silty CLAY (CL/ML)								
B-2	S-2	2.5-4.0'	18.9	Silty CLAY (CL/ML)								
B-2	S-3	5.0-6.5'	20.3	Silty CLAY (CL/ML)								
B-2	S-4	8.5-10.0'	18.7	Silty CLAY (CL/ML)								
B-2	S-5	13.5-15.0'	18.4	Silty CLAY (CL/ML)								

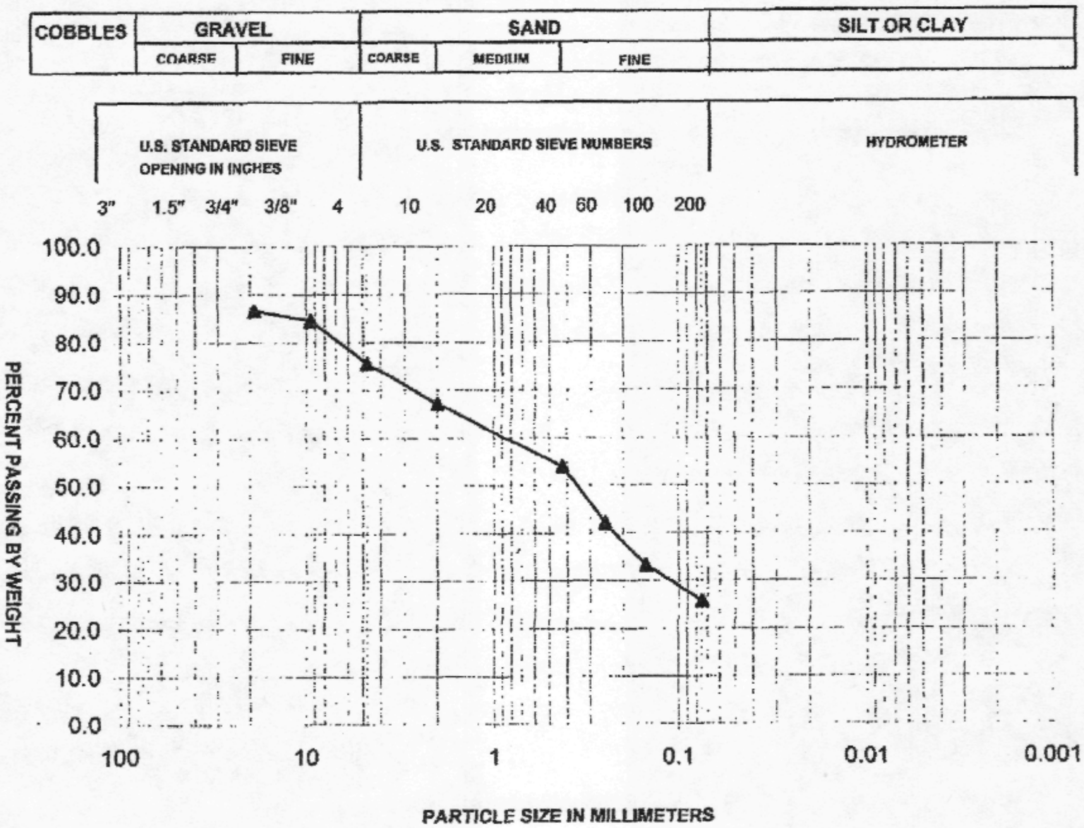
Summary Key:

SA = See Attached
S = Standard Proctor
M= Modified Proctor
V = Virginia Test Method
OC = Organic Content

LL= Liquid Limit
PL= Plastic Limit
PI= Plasticity Index

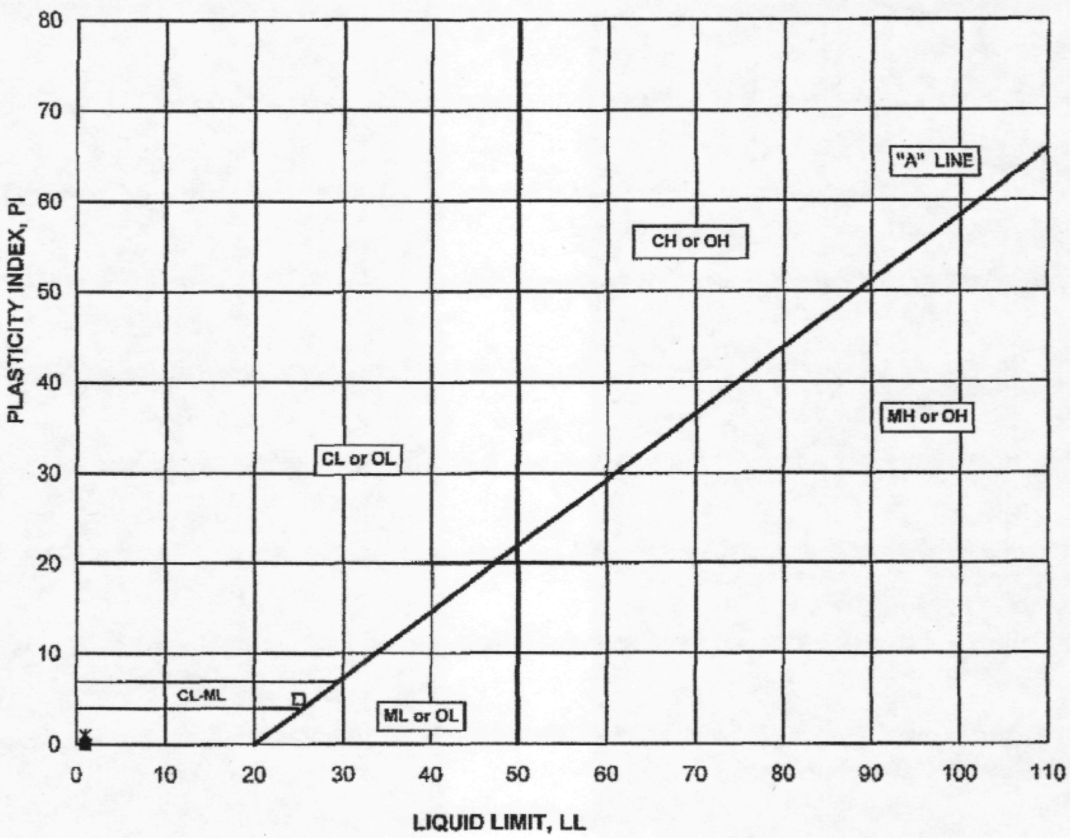
Hyd = Hydrometer
Con = Consolidation
DS = Direct Shear
GS = Specific Gravity

UCS = Unconfined Compression Soil
UCR = Unconfined Compression Rock
LS = Lime Stabilization
CS = Cement Stabilization



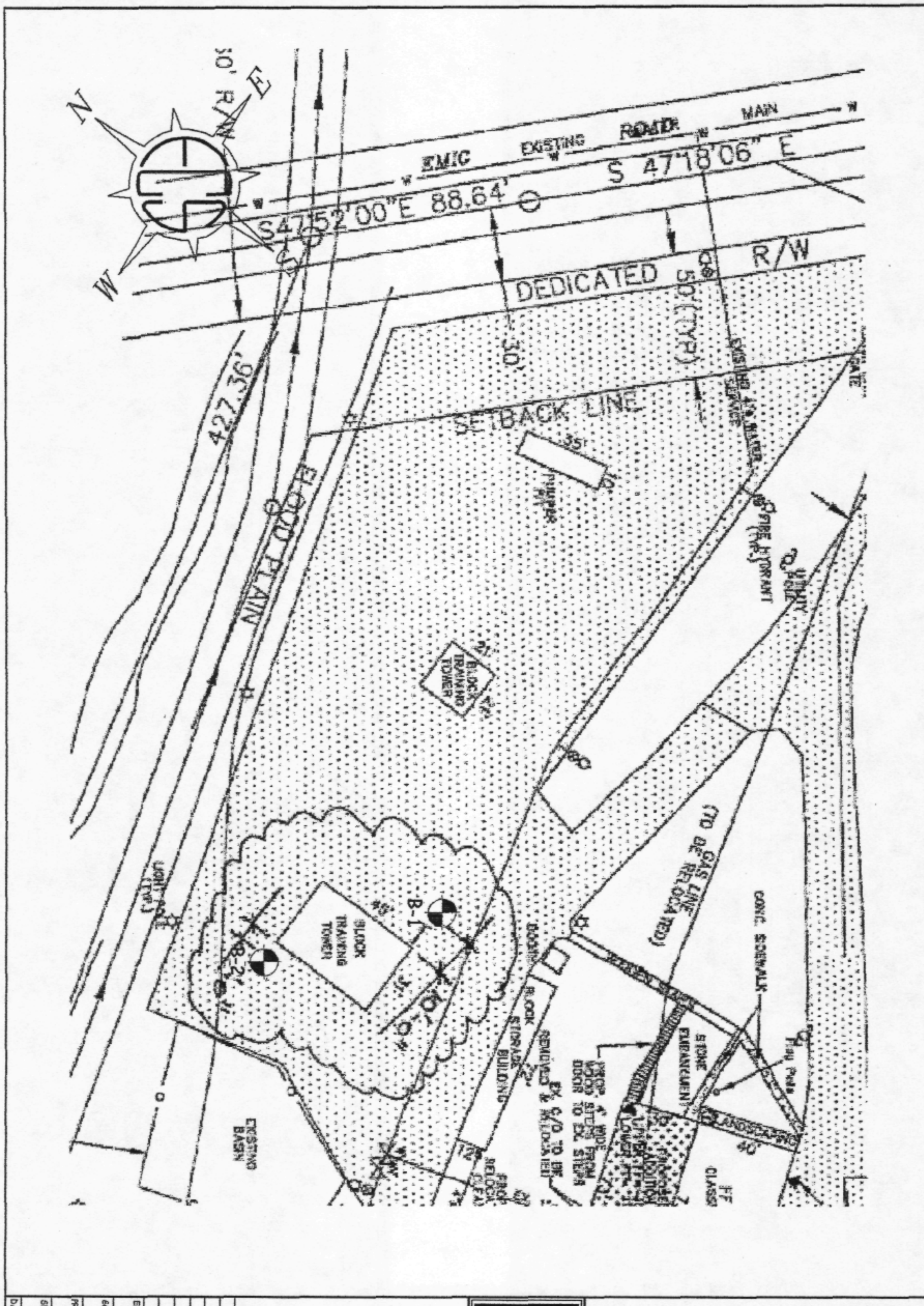
BORING/ SAMPLE	DEPTH (FEET)	SYMBOL	LL	PI	DESCRIPTION
B-1 / S-5	13.5-15.0'	▲			Sandy SILT (ML/SM)

Project: York Co. Fire School Project No.: 18.1438 Date: 2-May-07	ECS Mid-Atlantic, LLC. York, Pennsylvania Grain Size Analysis



BORING/ SAMPLE No.	DEPTH (feet)	TEST SYMBOL	DESCRIPTION	WATER CONTENT (%)	LL	PL	PI
B-1 / S-3	5.0-6.5'	□	Silty CLAY (CL/ML)	17.7	25	20	5
		■			-	-	-
		△			-	-	-
		▲			-	-	-
		x			-	-	-
		○			-	-	-
		●			-	-	-
		◇			-	-	-
		◆			-	-	-

Project: York Co. Fire School	**ECS Mid-Atlantic, LLC.**
Project No.: 18.1438	**York, Pennsylvania**
Date: May 2,2007	**Plasticity Chart**



ECS REVISIONS	ENGINEER	DRAFTING	SCALE	PROJECT NO.	SHEET	DATE
	WDF	DKK	NTS	18.1438	1	04/26/07

BORING LOCATION DIAGRAM

Elliot, LeBoeuf & Assoc.

ECS LLC MID-ATLANTIC

York County Fire School
Emigsville
York County, PA

Appendix Q: York Trench Field Test Results

The following pages provide summary charts and the field data collected during the York trench tests. Figure Q-1 shows the gauge locations on the panel.

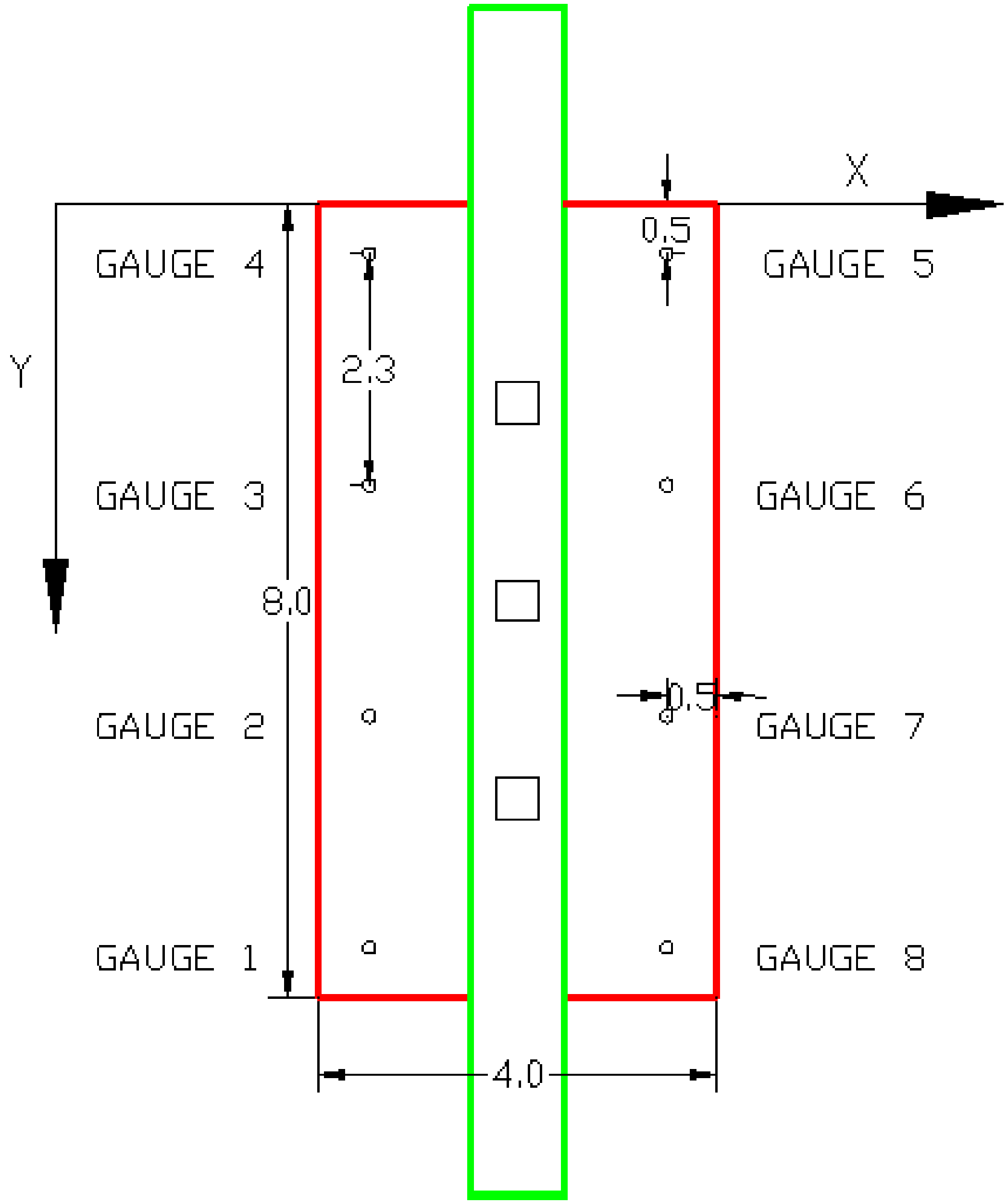


Figure Q-1 Gauge locations (gauges 4-1 are the "left" side of the panel; gauges 5-8 are the "right" side of the panel)

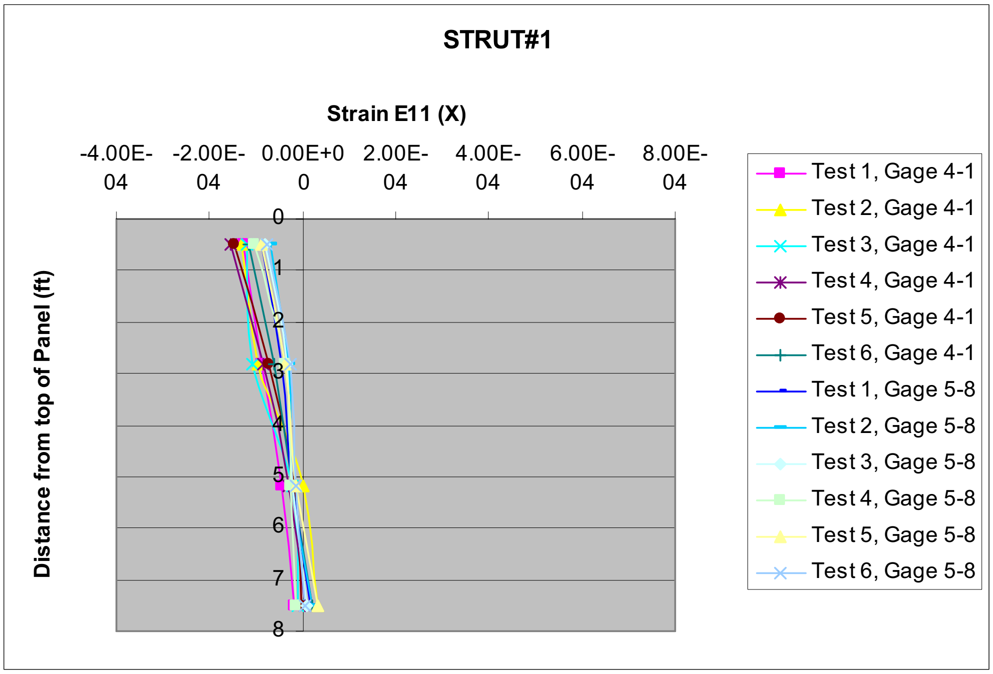


Figure Q-2 (a) Strains in the X direction after loading Strut #1 (the top strut) at approximately 1298 lbs

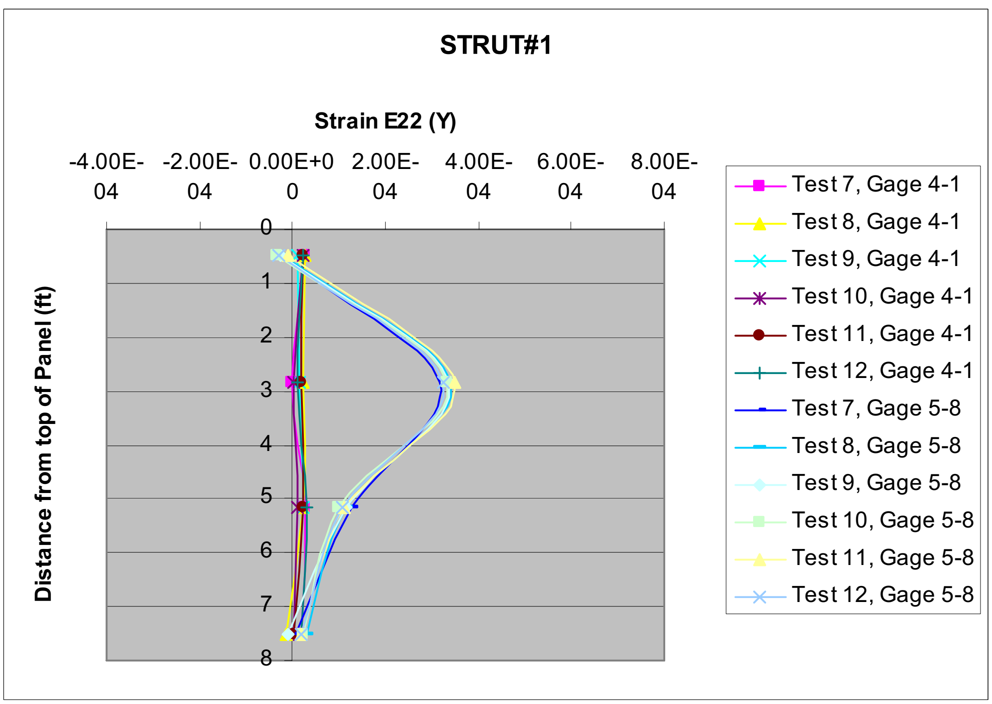


Figure Q-2 (b) Strains in the Y direction after loading Strut #1 (the top strut) at approximately 1298 lbs

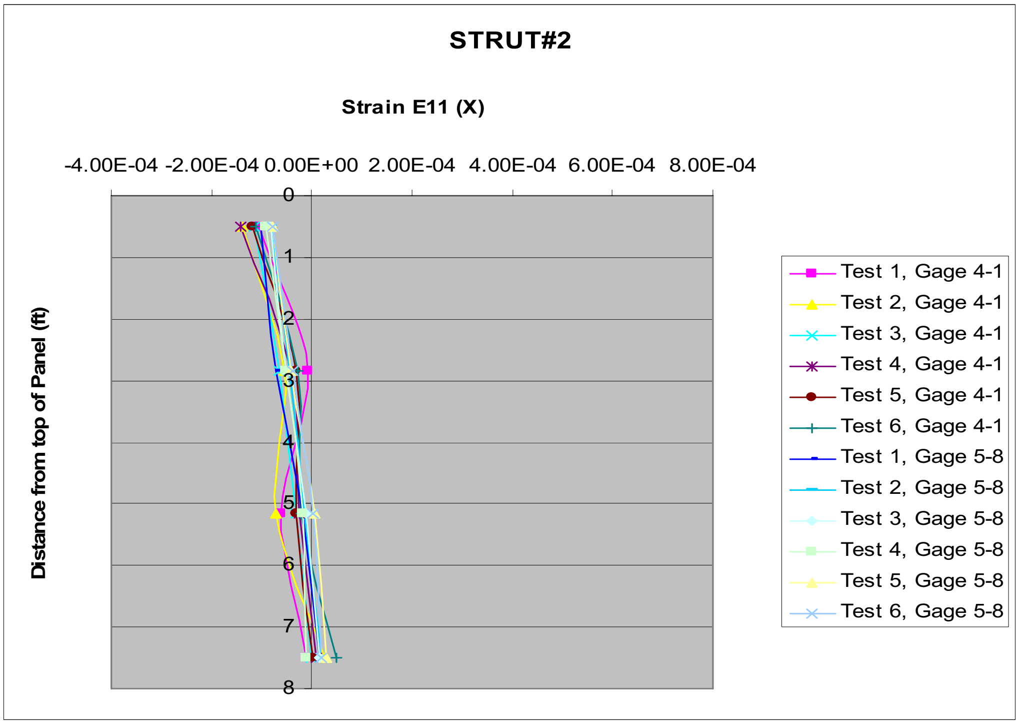


Figure Q-3 (a) Strains in the X direction after loading Struts #1 and #2 (the top two struts) at approximately 1298 lbs/strut

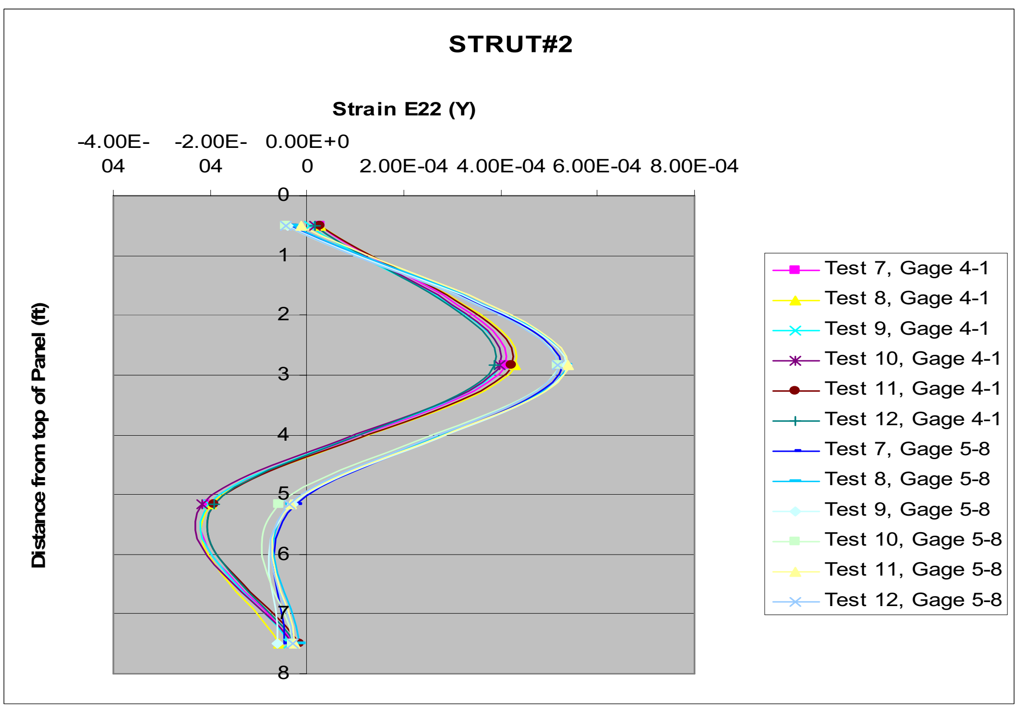


Figure Q-3 (b) Strains in the Y direction after loading Struts #1 and #2 (the top two struts) at approximately 1298 lbs/strut

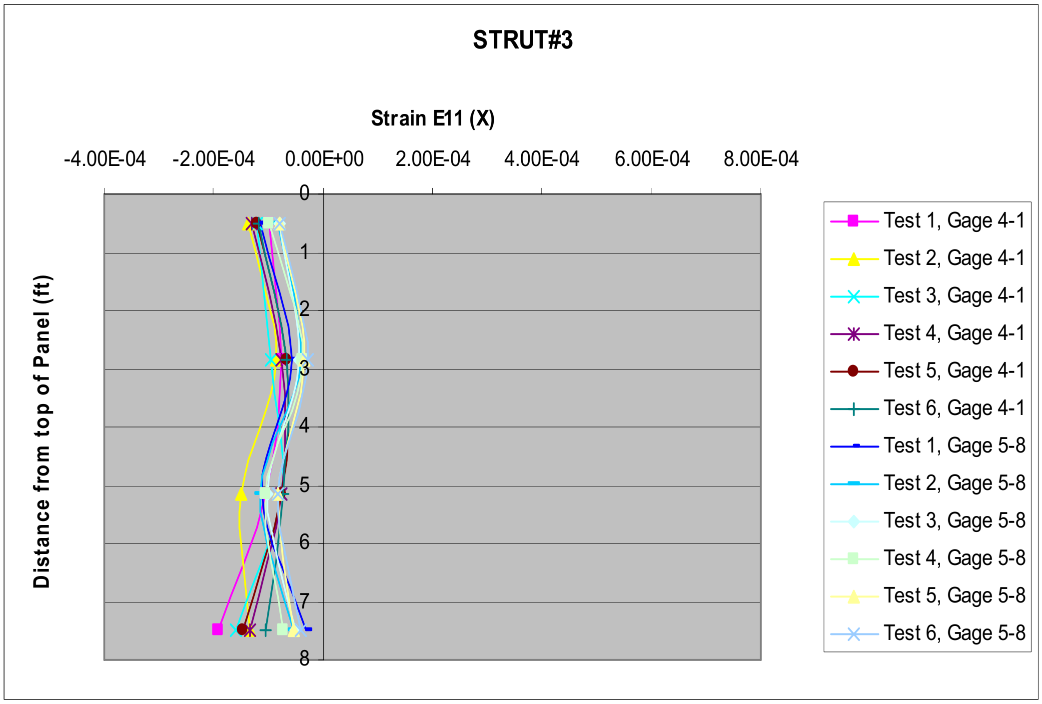


Figure Q-4 (a) Strains in the X direction after loading Struts #1, #2, and #3 (all three struts) at approximately 1298 lbs/strut

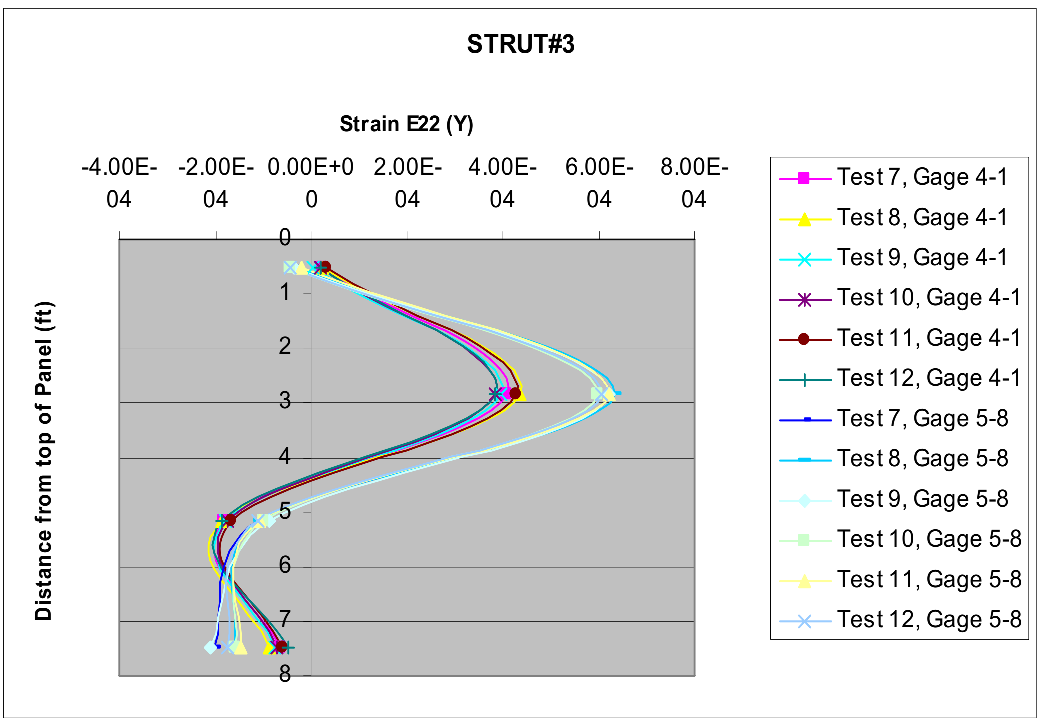


Figure Q-4 (b) Strains in the Y direction after loading Struts #1, #2, and #3 (all three struts) at approximately 1298 lbs/strut

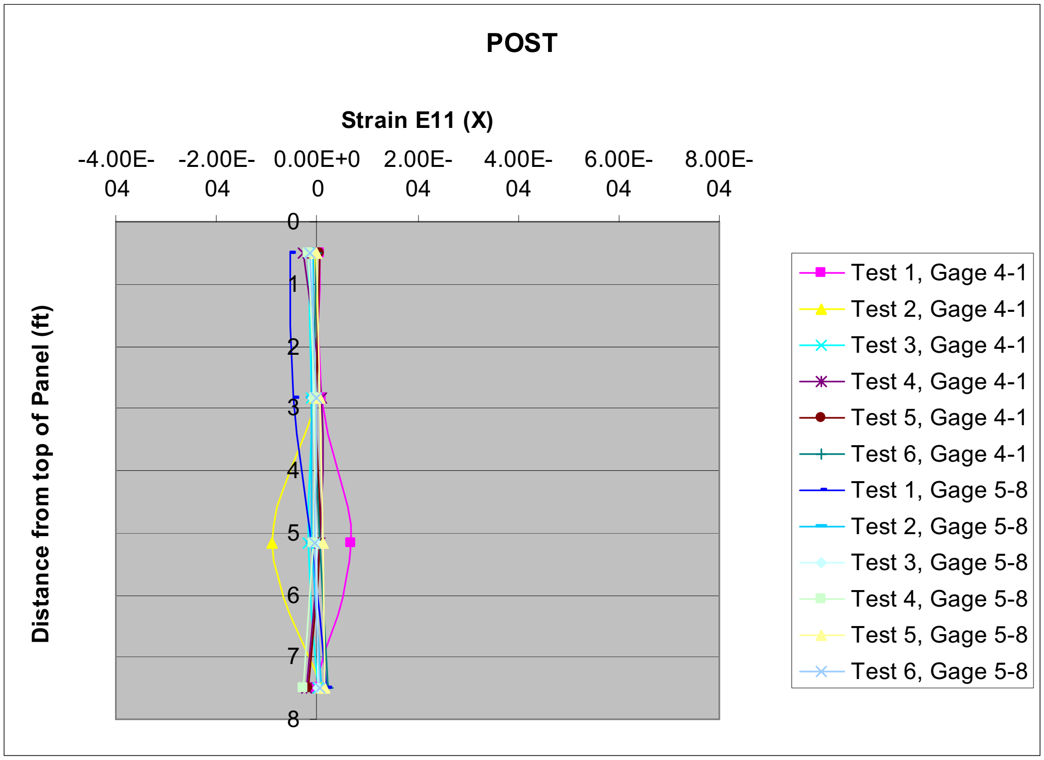


Figure Q-5 (a) Residual strains in the X direction after unloading Struts #1, #2, and #3 (all three struts)

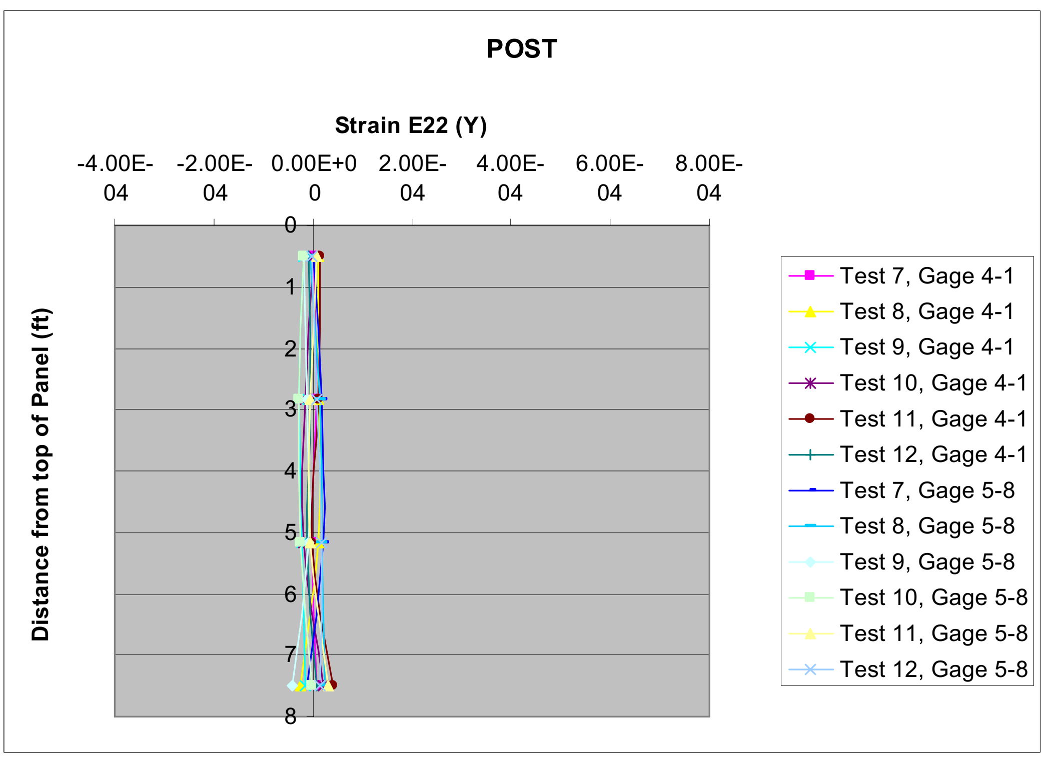


Figure Q-5 (b) Residual strains in the Y direction after unloading Struts #1, #2, and #3 (all three struts)

Test # 1 @ 200psi

Trench Shoring Test Form 200psi

Date	7/15/08	Order of Shoring	top, middle, bottom
Location	York Fire School		

με

Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	1	-1	-18	-10	-192	-4
1 ε = 0						
2 ε DNE 0	2	-1	-44	-59	-104	+66
2 ε = 0						
3 ε DNE 0	3	-1	-89	-7	-75	+8
3 ε = 0						
4 ε DNE 0	4	-1	-128	-105	-98	+6
4 ε = 0						
5 ε DNE 0	5	-1	-89	-100	-115	-56
5 ε = 0						
6 ε DNE 0	6	0	-47	-70	-58	-46
6 ε = 0						
7 ε DNE 0	7	-1	-20	-16	-111	-13
7 ε = 0	10	0	-8	-33	-125	-52
8 ε DNE 0	8	0	+15	+15	-30	+19
8 ε = 0						
6 2 DNE 0	9	-1	-31	-65	+11	+43

Test #2 @ 200psi

$\not{\!A}$ = Small wires

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	top, middle, bottom
Location	York Fire School		

με

Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	1	+5	+28	+21	-140	+15
1 ε = 0						
2 ε DNE 0	2	+1	0	-73	-152	-89
2 ε = 0						
$\not{\!A}$ 3 ε DNE 0	3	-4	-91	-50	-81	-8
3 ε = 0						
4 ε DNE 0	4	-5	-126	-132	-132	-8
4 ε = 0						
$\not{\!A}$ 5 ε DNE 0	5	-5	-64	-78	-78	-11
5 ε = 0						
$\not{\!A}$ 6 ε DNE 0	6	-3	-26	-42	-38	-14
6 ε = 0						
7 ε DNE 0	7	-5	-16	-9	-109	-15
7 ε = 0	10	-2	+66	-135	-111	+22
8 ε DNE 0	8	+4	+18	+15	-58	+12
8 ε = 0						
$\not{\!A}$ 6 2 DNE 0	9	-5	-18	-39	-42	-7

Test #3 @ 200psi

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	T, M, B
Location	York Fire School		

με

Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	1	-2	-10	0	-157	+0
1 ε = 0						
2 ε DNE 0	2	1	-22	-32	-78	-15
2 ε = 0						
3 ε DNE 0	3	-4	-102	-60	-92	-16
3 ε = 0						
4 ε DNE 0	4	5	-126	-120	-125	-11
4 ε = 0						
5 ε DNE 0	5	2	-84	-85	-82	-8
5 ε = 0						
6 ε DNE 0	6	-3	-30	-47	-41	-11
6 ε = 0						
7 ε DNE 0	7	-3	-20	-8	-100	-3
7 ε = 0	10	-1	+118	-52	-103	+23
8 ε DNE 0	8	+2	+9	+12	-53	4
8 ε = 0						
6Z DNE 0	9	+3	-26	-40	-35	-4

Test 4 @ 200psi

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	T, M, B
Location	York fire school		

με

Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	1	-5	+10	+15	-128	-26
1 ε = 0						
2 ε DNE 0	2	-2	-27	-24	-75	+2
2 ε = 0						
3 ε DNE 0	3	-3	-82	-37	-73	+5
3 ε = 0						
4 ε DNE 0	4	-5	-149	-137	-126	-31
4 ε = 0						
5 ε DNE 0	5	-4	-98	-87	-94	-21
5 ε = 0						
6 ε DNE 0	6	-5	-32	.48	-32	-9
6 ε = 0						
7 ε DNE 0	7	-2	-24	-16	-104	-8
7 ε = 0	10	+2	+112	-54	-128	+6
8 ε DNE 0	8	-5	-8	-5	-68	-32
8 ε = 0						
62 DNE 0	9	-5	-25	-39	-42	+7

Test #5 @ 200psi

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	
Location	York fire school		

Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
		με				
1 ε DNE 0	1	+3	-6	0	-149	-14
1 ε = 0						
2 ε DNE 0	2	-1	-18	-30	-74	+6
2 ε = 0						
3 ε DNE 0	3	+2	-75	-33	-69	+2
3 ε = 0						
4 ε DNE 0	4	-2	-145	-118	-120	+2
4 ε = 0						
5 ε DNE 0	5	-2	-86	-75	-76	-4
5 ε = 0						
6 ε DNE 0	6	-3	-33	-39	-30	+1
6 ε = 0						
7 ε DNE 0	7	+3	-19	+3	-85	+14
7 ε = 0	10	-5	+114	-49	-116	+8
8 ε DNE 0	8	+4	+28	+25	-56	+18
8 ε = 0						
6Z DNEO	9	-6	-29	-40	-38	-5

test #6 @ 200psi

more like 190psi

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	T M B
Location	York Fireschool		

Gage	Circuit #	At Rest	Shore #1 (190)	Shore #2 (με 190)	Shore #3 (180)	Post
1 ε DNE 0	1	+1	+19	+48	-105	+24
1 ε = 0						
2 ε DNE 0	2	-5	-16	-16	-69	0
2 ε = 0						
3 ε DNE 0	3	+3	-64	-30	-68	-3
3 ε = 0						
4 ε DNE 0	4	-2	-114	-109	-115	-3
4 ε = 0						
5 ε DNE 0	5	+3	-80	-82	-83	-10
5 ε = 0						
6 ε DNE 0	6	-1	-25	-38	-26	-2
6 ε = 0						
7 ε DNE 0	7	+2	-16	+1	-85	-2
7 ε = 0	10	+2	+104	-50	-119	-5
8 ε DNE 0	8	+2	+5	+18	-44	+6
8 ε = 0						
6Z DNE0	9	+2	-29	-44	-47	-6

test #7 @ 200psi

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	T M B
Location	York Fire School		

Gage	Circuit #	At Rest psi	Shore #1 210	Shore #2 με 200	Shore #3 200	Post
1 ε DNE 0						
1 ε = 0	1	+2	+17	-33	-66	+10
2 ε DNE 0						
2 ε = 0	2	-3	+30	-199	-180	-6
3 ε DNE 0	10	+0	-86	-62	-88	-15
3 ε = 0	3	-3	+2	+412	+412	+5
4 ε DNE 0						
4 ε = 0	4	+4	+23	+22	+20	+5
5 ε DNE 0						
5 ε = 0	5	-2	-28	-31	-31	-2
6 ε DNE 0						
6 ε = 0	6	-3	+318	+529	+625	+13
7 ε DNE 0						
7 ε = 0	7	0	+128	-23	-112	+22
8 ε DNE 0						
8 ε = 0	8	-2	+8	-44	-199	-13
62 ε=0	9	-1	+398	+498	+605	+22

test #8 @ 200 psi

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	T-M-B
Location	York fire school		

Gage	Circuit #	At Rest	200 Shore #1	με 200 Shore #2	200 Shore #3	Post
1 ε DNE 0						
1 ε = 0	1	-2	-13	-57	-85	-38
2 ε DNE 0						
2 ε = 0	2	-3	+25	-197	-184	+8
3 ε DNE 0	10	-3	-76	-28	-71	+25
3 ε = 0	3	+2	+22	+426	+433	+12
4 ε DNE 0						
4 ε = 0	4	+3	+22	+23	+21	+15
5 ε DNE 0						
5 ε = 0	5	+3	-24	-32	-33	-1
6 ε DNE 0						
6 ε = 0	6	0	+340	+537	+635	+14
7 ε DNE 0						
7 ε = 0	7	0	+114	-34	-107	+18
8 ε DNE 0						
8 ε = 0	8	0	+30	-15	-158	+24
6-2 ε = 0	9	-2	+324	+509	+610	+15

Test #9 @ 200psi

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	T, M, B
Location	York Fire School		

200 με 200 200

Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0						
1 ε = 0	1	-4	+6	-33	-73	-19
2 ε DNE 0						
2 ε = 0	2	+2	+22	-205	-180	-20
3 ε DNE 0	10	+3	-55	-25	-52	+13
3 ε = 0	3	+3	+11	+394	+394	-27
4 ε DNE 0						
4 ε = 0	4	-5	+16	+11	+10	-22
5 ε DNE 0						
5 ε = 0	5	-2	-25	-37	-33	-22
6 ε DNE 0						
6 ε = 0	6	+2	+335	+532	+620	-10
7 ε DNE 0						
7 ε = 0	7	0	+115	-30	-88	-9
8 ε DNE 0						
8 ε = 0	8	-3	-6	-58	-205	-45
G2 ε=0	9	-2	+335	+524	+620	+21

Test # 10 @ 200psi

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	T, M, B
Location	York Fire School		

Gage	Circuit #	At Rest	Shore #1 (200 με)	Shore #2 (200)	Shore #3 (200)	Post
1 ε DNE 0						
1 ε = 0	1	-6	+10	-24	-67	+14
2 ε DNE 0						
2 ε = 0	2	-3	+14	-212	-170	-23
3 ε DNE 0	10	-1	-80	-15	-58	+10
3 ε = 0	3	-2	+5	+398	+388	-16
4 ε DNE 0						
4 ε = 0	4	+1	+20	+17	+18	+1
5 ε DNE 0						
5 ε = 0	5	-3	-30	-40	-41	-21
6 ε DNE 0					#	
6 ε = 0	6	0	+332	+519	+598	-29
7 ε DNE 0						
7 ε = 0	7	-4	+104	-54	-96	-30
8 ε DNE 0						
8 ε = 0	8	0	+21	-30	-160	-3
	9	+3	+320	+497	+583	-24

Test #11
@ 200psi

Trench Shoring Test Form

Date	7/15/08	Order of Shoring	T, 7, 15
Location	York Fire School		

			205	με 200	200	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0						
1 ε = 0	1	-3	+6	-11	-56	+38
2 ε DNE 0						
2 ε = 0	2	-2	+24	-190	-164	-3
3 ε DNE 0	10	-5	-68	-48	-70	+1
3 ε = 0	3	-1	+21	422	+429	+9
4 ε DNE 0						
4 ε = 0	4	+2	+22	+26	30	+15
5 ε DNE 0						
5 ε = 0	5	+2	-13	-15	-20	+10
6 ε DNE 0						
6 ε = 0	6	-0	+349	+540	+621	-4
7 ε DNE 0						
7 ε = 0	7	-4	+115	-30	-102	-8
8 ε DNE 0						
8 ε = 0	8	-2	+21	-25	-144	+29
	9	-3	+333	+509	+596	0

Test #12
@200psi

Trench Shoring Test Form

Date	7/5/08	Order of Shoring	1, 7, 5
Location	York Fire School		

Gage	Circuit #	At Rest	Shore #1 (200 με)	Shore #2 (200)	Shore #3 (195)	Post
1 ε DNE 0						
1 ε = 0	1	-4	+23	-21	-45	+4
2 ε DNE 0						
2 ε = 0	2	-2	+31	-191	-186	-13
3 ε DNE 0	10	+3	-70	-31	-75	+16
3 ε = 0	3	+1	+8	+386	+384	-5
4 ε DNE 0						
4 ε = 0	4	+2	+20	+15	+16	-6
5 ε DNE 0						
5 ε = 0	5	-6	-23	-39	-39	-7
6 ε DNE 0						
6 ε = 0	6	-4	+329	+523	+608	+3
7 ε DNE 0						
7 ε = 0	7	-4	+110	-33	-105	+14
8 ε DNE 0						
8 ε = 0	8	-3	+23	-25	-171	+11
	9	+5	+318	+502	+595	+6

Appendix R: Frederick Trench Field Test Results

The following pages provide summary charts and the field data collected during the Frederick trench tests. Figure R-1 shows the gauge locations on the panel.

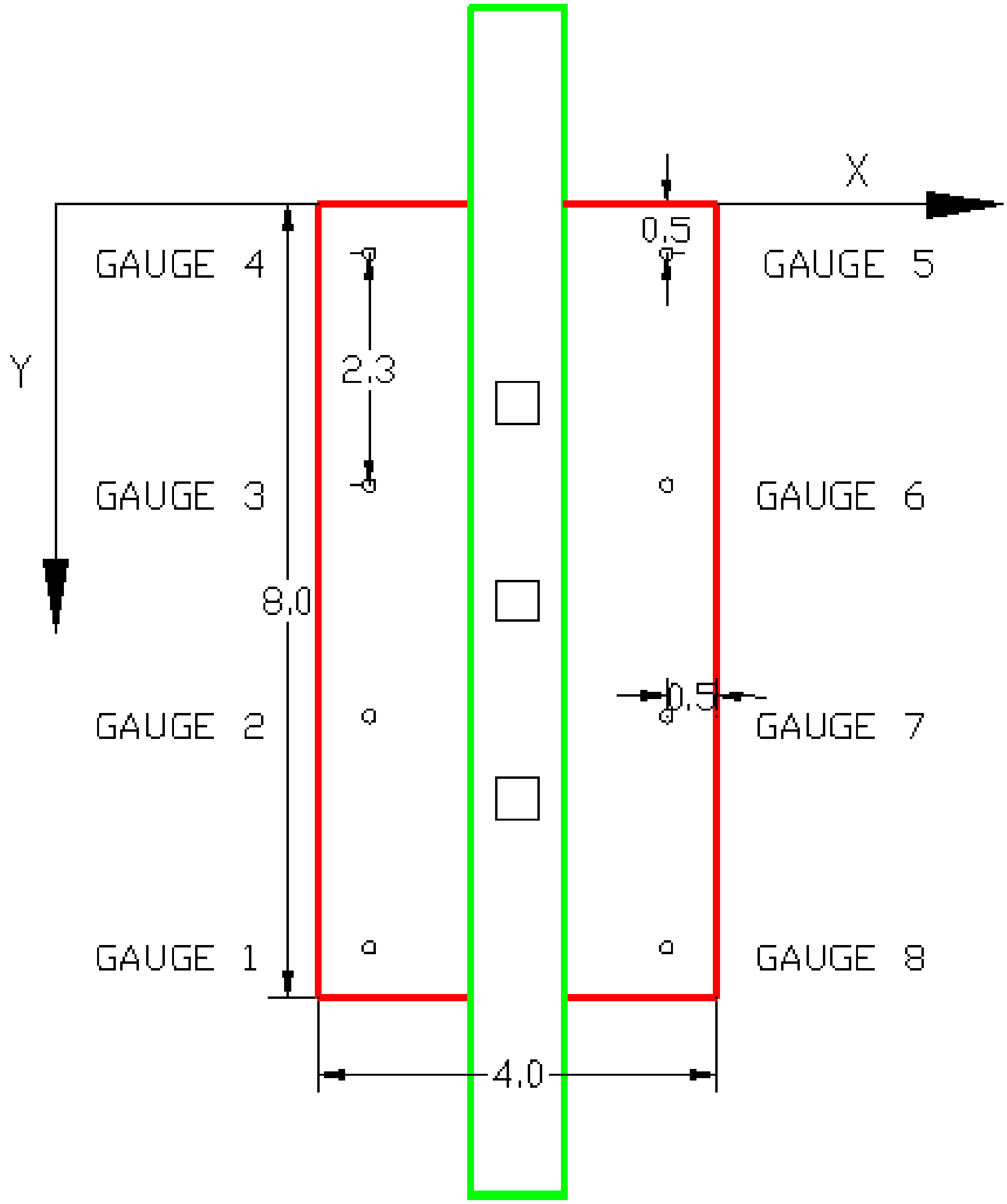


Figure R-1 Gauge locations (gauges 4-1 are the "left" side of the panel; gauges 5-8 are the "right" side of the panel)

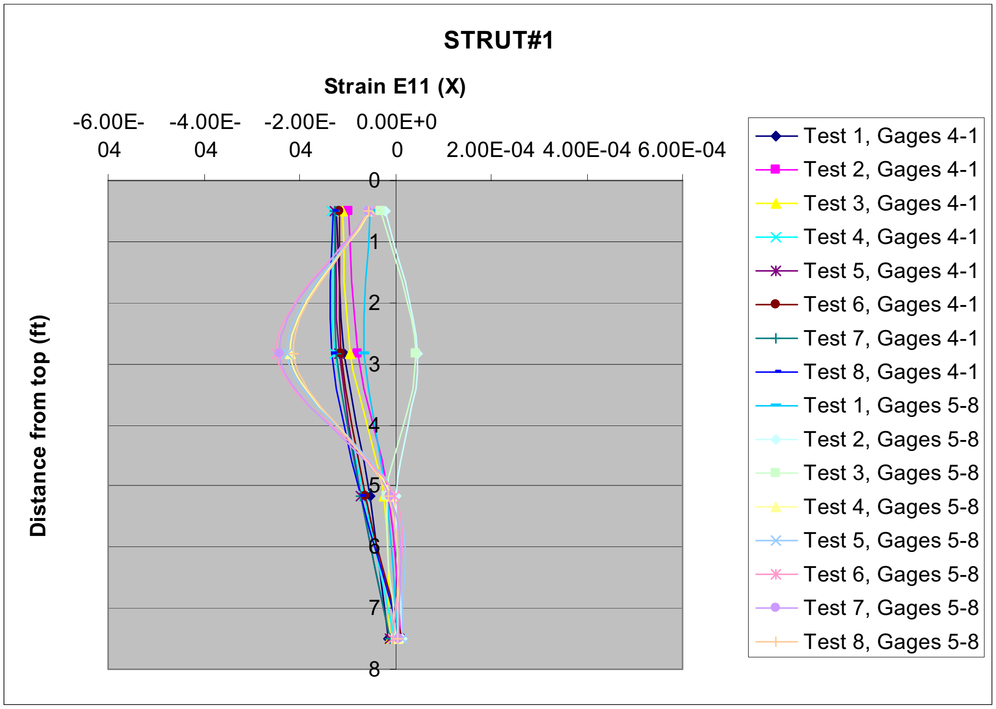


Figure R-2 (a) Strains in the X direction after loading Strut #1 (the top strut) at approximately 1298 lbs

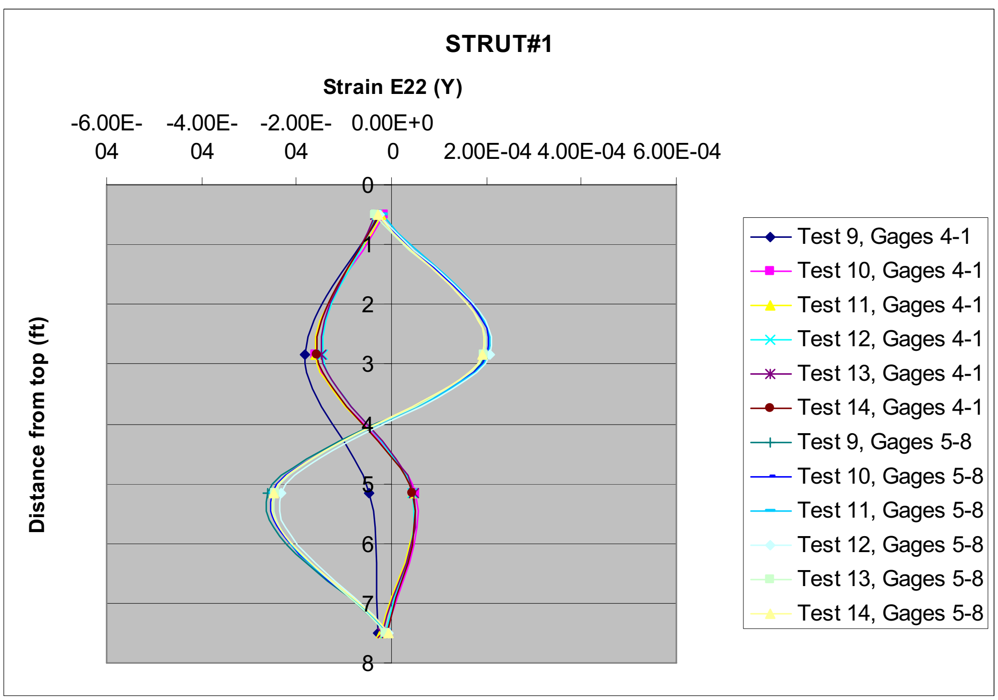


Figure R-2 (b) Strains in the Y direction after loading Strut #1 (the top strut) at approximately 1298 lbs

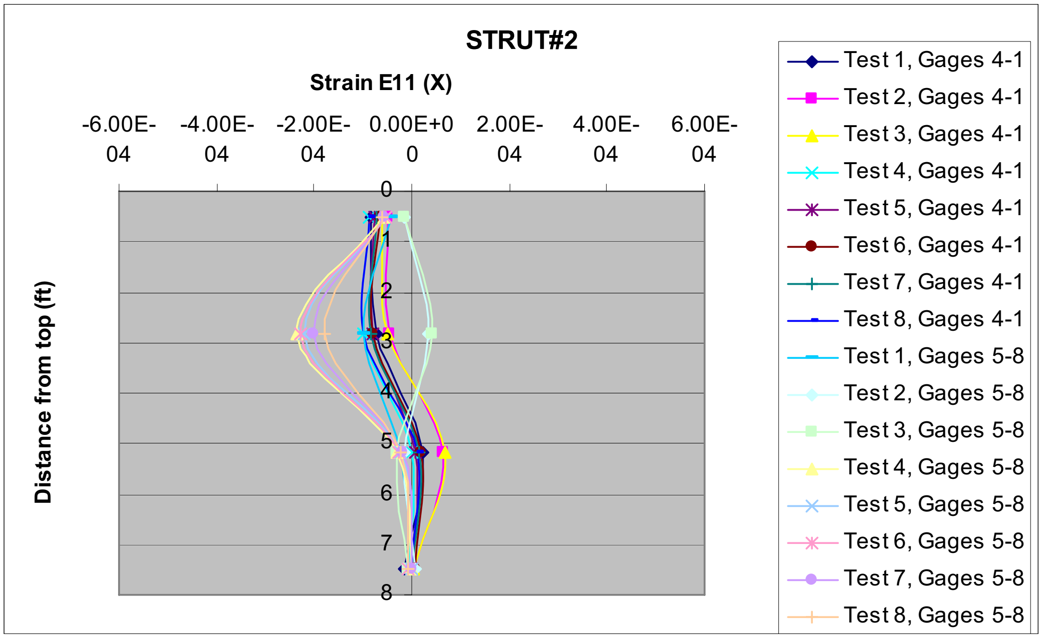


Figure R-3 (a) Strains in the X direction after loading Struts #1 and #2 (the top two struts) at approximately 1298 lbs/strut

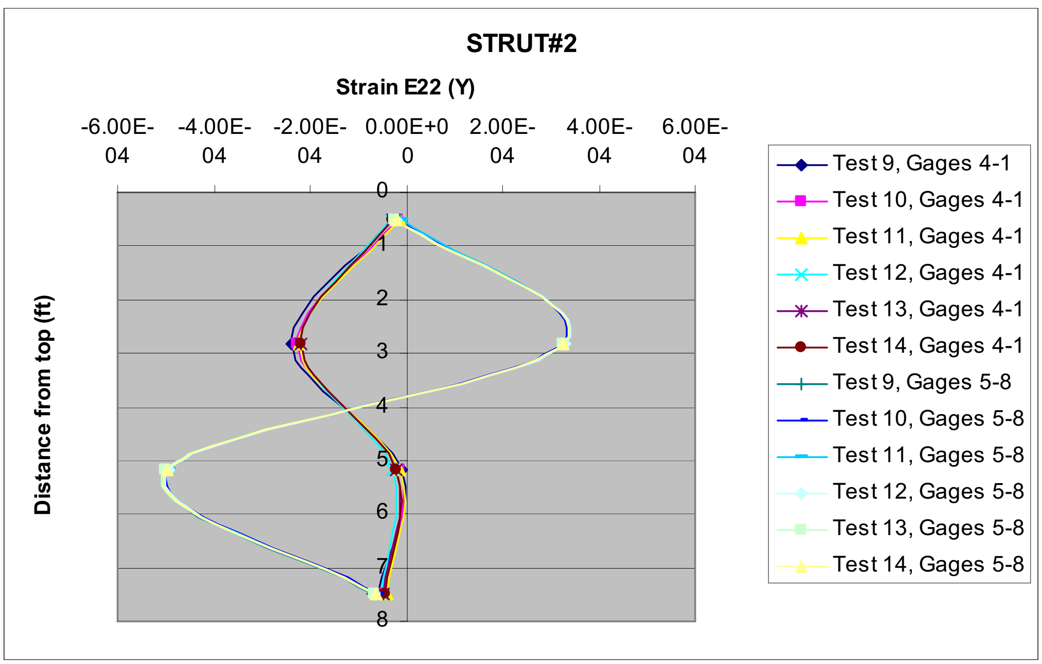


Figure R-3 (b) Strains in the Y direction after loading Struts #1 and #2 (the top two struts) at approximately 1298 lbs/strut

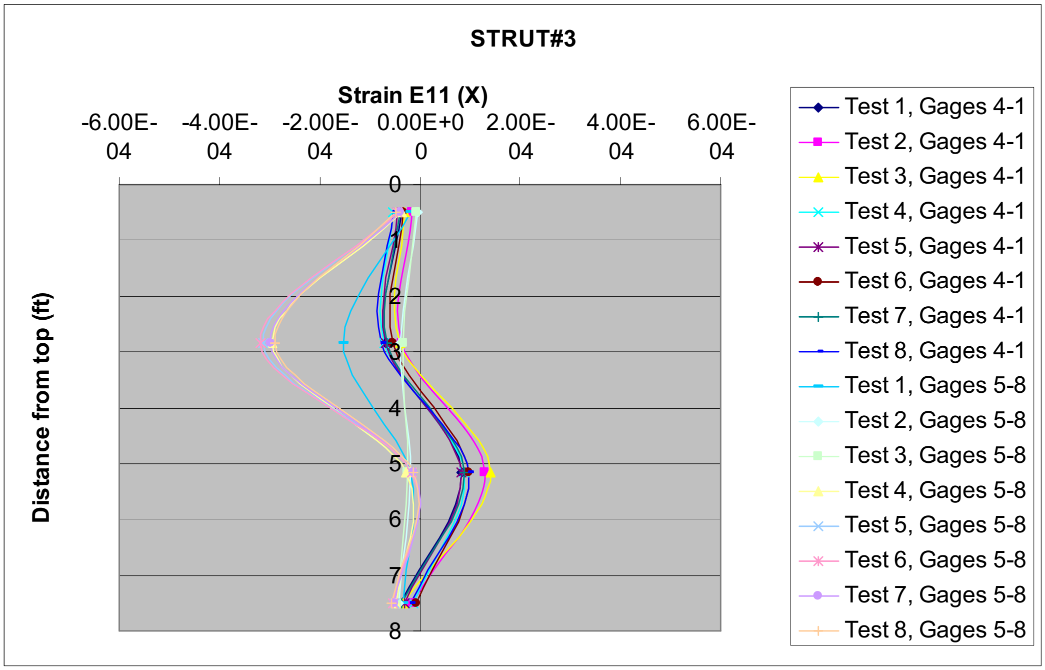


Figure R-4 (a) Strains in the X direction after loading Struts #1, #2, and #3 (all three struts) at approximately 1298 lbs/strut

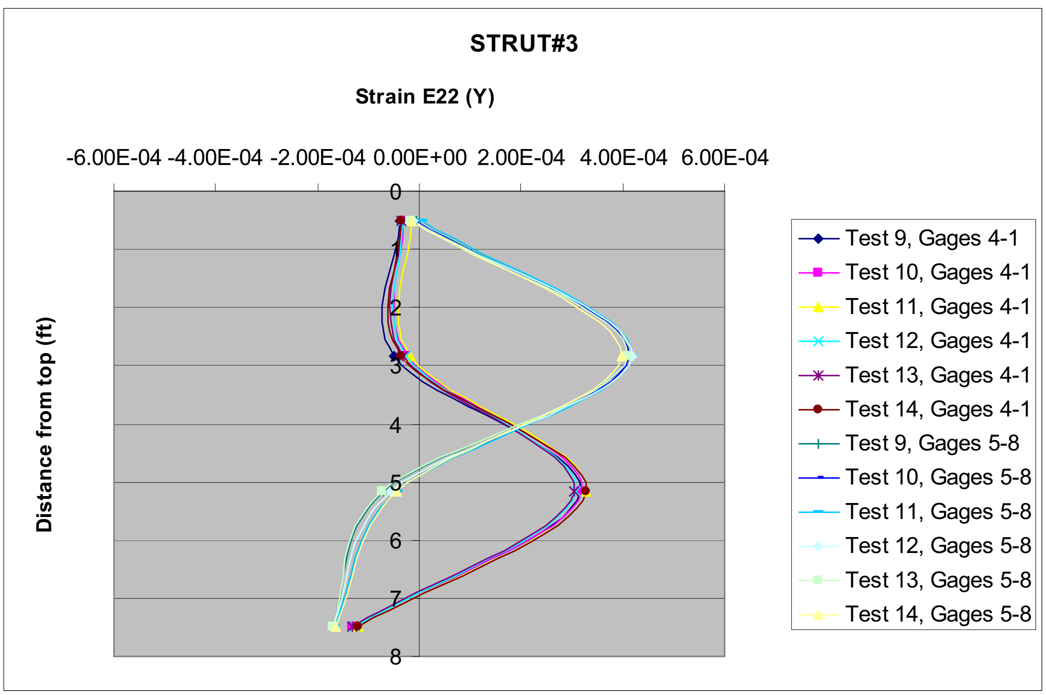


Figure R-4 (b) Strains in the Y direction after loading Struts #1, #2, and #3 (all three struts) at approximately 1298 lbs/strut

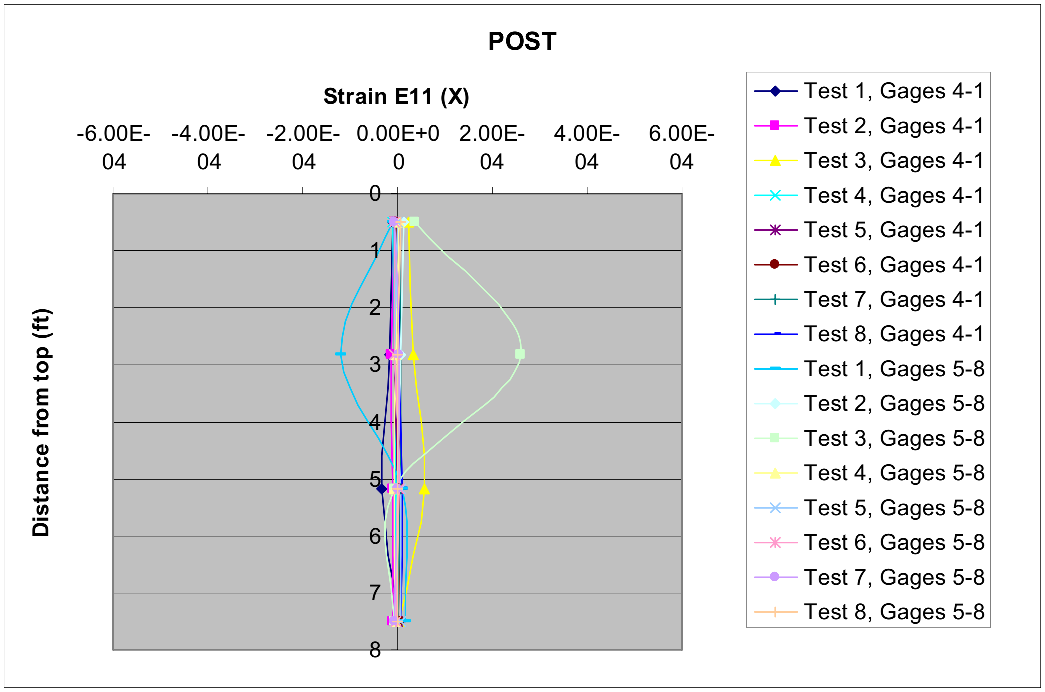


Figure R-5 (a) Residual strains in the X direction after unloading Struts #1, #2, and #3 (all three struts)

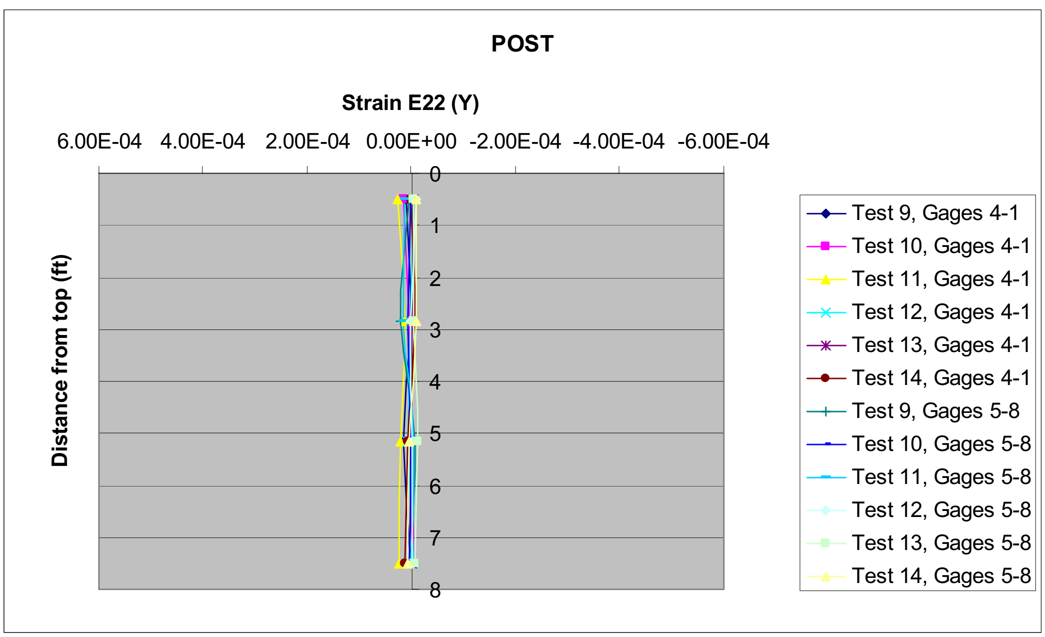


Figure R-5 (b) Residual strains in the Y direction after unloading Struts #1, #2, and #3 (all three struts)

Trench Shoring Test Form

Date	7/21		Test #	1		
Location	frederick					
psi			185	185	185	
Gage	**Circuit #**	**At Rest**	**Shore #1**	**Shore #2**	**Shore #3**	**Post**
1 ε DNE 0	1	0	-13	-14	-40	+2
1 ε = 0	10	-4	+8	-21	+77	+8
2 ε DNE 0	2	0	-54	+24	+82	-33
2 ε = 0						
3 ε DNE 0	3	+3	-105	-69	-64	-13
3 ε = 0						
4 ε DNE 0	4	+4	-120	-81	-32	-7
4 ε = 0						
5 ε DNE 0	5	+2	-52	-41	-16	-8
5 ε = 0						
6 ε DNE 0	6	-4	-69	-102	-156	-123
6 ε = 0						
62 ε DNE 0						
62 ε = 0	9	-3	+65	+146	+282	+165
7 ε DNE 0	7	-2	-21	-24	-20	+8
7 ε = 0						
8 ε DNE 0	8	0	+6	+1	-34	+17
8 ε = 0						

Trench Shoring Test Form

Date	7/21		Test #	2		
Location	Frederick					
psi			185	185	185	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0 1	-5	-5	+7	0	-22	-15
1 ε = 0 10	-5	+2	+14	-8	-88	-4
2 ε DNE 0 2	-6	-6	-22	+58	+124	-15
2 ε = 0						
3 ε DNE 0 3	+1	+1	-76	-44	-36	-11
3 ε = 0						
4 ε DNE 0 4	+4	+4	-93	-45	-12	-1
4 ε = 0						
5 ε DNE 0 5	-3	-3	-23	-17	-7	+11
5 ε = 0						
6 ε DNE 0 6	0	0	+47	+36	-38	+6
6 ε = 0						
62 ε DNE 0 9						
62 ε = 0	9	-1	-58	+14	+125	-7
7 ε DNE 0 7		-3	-2	-18	-22	-3
7 ε = 0						
8 ε DNE 0 8		-4	+10	+5	-45	-6
8 ε = 0						

Trench Shoring Test Form

Date	7/21		Test #	3		
Location	Frederick					
psi			195	195	195	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	1	~~8~~ 0	-11	+2	-33	2
1 ε = 0	10	1	+18	-15	-83	+30
2 ε DNE 0	2	+1	-22	+71	+141	+57
2 ε = 0						
3 ε DNE 0	3	+1	-93	-47	-38	+35
3 ε = 0						
4 ε DNE 0	4	-1	-112	-61	-31	+24
4 ε = 0						
5 ε DNE 0	5	-2	-31	-19	-8	+35
5 ε = 0						
6 ε DNE 0	6	-8	+36	+33	-42	+253
6 ε = 0						
62 ε DNE 0						
62 ε = 0	9	-2	-68	-1	+ 119	-445
7 ε DNE 0	7	-2	-20	-34	-27	-7
7 ε = 0						
8 ε DNE 0	8	-5	-2	-11	-50	-11
8 ε = 0						

Trench Shoring Test Form

Date	7/21		Test #	#4		
Location	frederick					
psi			190	190	190	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	1	-5	-6	-14	-17	-9
1 ε = 0	10	-2	-21	-40	-116	+3
2 ε DNE 0	2	-6	-72	-6	+84	-10
2 ε = 0						
3 ε DNE 0	3	0	-125	-98	-~~8~~72	-5
3 ε = 0						
4 ε DNE 0	4	+1	-133	-88	-52	-8
4 ε = 0						
5 ε DNE 0	5	+2	-52	-54	-35	+4
5 ε = 0						
6 ε DNE 0	6	-5	-224	-241	-301	-9
6 ε = 0						
62 ε DNE 0						
62 ε = 0	9	0	+426	+494	+601	+14
7 ε DNE 0	7	-5	-12	-37	-32	-11
7 ε = 0						
8 ε DNE 0	8	-5	0	-6	-52	-7
8 ε = 0						

Trench Shoring Test Form

Date	7/21		Test #	#5		
Location	Frederick					
psi			190	190	190	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	1	-3	-15	-3	-34	-3
1 ε = 0	10	+1	-10	-45	-113	+4
2 ε DNE 0	2	-3	-76	+6	+80	-3
2 ε = 0						
3 ε DNE 0	3	-4	-121	-83	-72	-9
3 ε = 0						
4 ε DNE 0	4	+3	-125	-72	-41	-4
4 ε = 0						
5 ε DNE 0	5	+2	-53	-53	-44	+2
5 ε = 0						
6 ε DNE 0	6	+1	-224	-221	-312	-6
6 ε = 0						
62 ε DNE 0						
62 ε = 0	9	-3	+446	+509	+609	+11
7 ε DNE 0	7	+4	+3	-17	-11	+9
7 ε = 0						
8 ε DNE 0	8	-3	+10	-2	-53	~~-4~~ +5
8 ε = 0						

Trench Shoring Test Form

Date	7/21		Test #	#6		
Location	frederick					
psi			190	185	185	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	1	+4	+20	+13	-3	+9
1 ε = 0	10	-2	-29	-58	-139	-6
2 ε DNE 0	2	-4	-66	+16	+93	-2
2 ε = 0						
3 ε DNE 0	3	-4	-118	-82	-59	-2
3 ε = 0						
4 ε DNE 0	4	-2	-120	-68	-36	-5
4 ε = 0						
5 ε DNE 0	5	+2	-52	-52	-43	-5
5 ε = 0						
6 ε DNE 0	6	-3	-252	-232	-320	-11
6 ε = 0						
62 ε DNE 0						
62 ε = 0	9	+3	+445	+512	+600	+10
7 ε DNE 0	7	+1	+1	-24	-16	+4
7 ε = 0						
8 ε DNE 0	8	-1	-6	-9	-56	-3
8 ε = 0						

Trench Shoring Test Form

Date	7/21		Test #	#7		
Location	Frederick					
psi			185	180	200	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	1	-4	-16	-4	-40	-4
1 ε = 0	10	+3	-15	-50	-125	-7
2 ε DNE 0	2	-2	-74	+12	+87	+5
2 ε = 0						
3 ε DNE 0	3	-1	-123	-86	-67	+5
3 ε = 0						
4 ε DNE 0	4	+0	-127	-73	-38	+2
4 ε = 0						
5 ε DNE 0	5	+2	-55	-54	-38	-3
5 ε = 0						
6 ε DNE 0	6	+3	-239	-200	-296	+6
6 ε = 0						
62 ε DNE 0						
62 ε = 0	9	-1	~~9~~ +429	+494	+595	+2
7 ε DNE 0	7	-4	-15	-24	-18	-3
7 ε = 0						
8 ε DNE 0	8	-1	+6	-1	-53	-8
8 ε = 0						

Trench Shoring Test Form

Date	7/21		Test #	#8		
Location	Frederick					
psi			200	185	200	
Gage	**Circuit #**	**At Rest**	**Shore #1**	**Shore #2**	**Shore #3**	**Post**
1 ε DNE 0	1	+4	+16	+3	-14	+12
1 ε = 0	10	+2	-23	-49	-136	-1
2 ε DNE 0	2	-1	-72	+11	+97	+10
2 ε = 0						
3 ε DNE 0	3	+2	-131	-98	-76	+6
3 ε = 0						
4 ε DNE 0	4	-2	-133	-90	-56	0
4 ε = 0						
5 ε DNE 0	5	-3	-59	-62	-52	+3
5 ε = 0						
6 ε DNE 0	6	+5	-208	-173	-285	+4
6 ε = 0						
62 ε DNE 0	~~4~~					
62 ε = 0	9	+4	+407	+491	+588	+14
7 ε DNE 0	7	-4	-17	-25	-18	-4
7 ε = 0						
8 ε DNE 0	8	-1	+5	-7	-58	-2
8 ε = 0						

Trench Shoring Test Form

Date	7/21		Test #	#9		
Location	frederick					
psi			170	190	185	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0						
1 ε = 0	1	-2	-29	-61	-135	-2
2 ε DNE 0						
2 ε = 0	2	-4	-52	-14	+311	+11
3 ε DNE 0						
3 ε = 0	3	+3	-179	-235	-47	+4
4 ε DNE 0						
4 ε = 0	4	+3	-23	-27	-35	+12
5 ε DNE 0						
5 ε = 0	5	-2	-26	-16	-7	+1
6 ε DNE 0	10	+2	-237	-191	-302	-22
6 ε = 0	6	-2	+199	+322	+409	+18
62 ε DNE 0	9	+4	-131	-74	-145	0
62 ε = 0						
7 ε DNE 0						
7 ε = 0	7	+2	-260	-499	-69	-5
8 ε DNE 0						
8 ε = 0	8	+6	-4	-65	-163	+6

Trench Shoring Test Form

Date	9/21		Test #	#10		
Location	Frederick					
psi			205	200	200	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0						
1 ε = 0	1	+5	-5	-44	-126	+2
2 ε DNE 0						
2 ε = 0	2	-1	+50	-14	+318	-2
3 ε DNE 0						
3 ε = 0	3	-1	-162	-229	-24	+2
4 ε DNE 0						
4 ε = 0	4	0	-17	-18	-32	+15
5 ε DNE 0						
5 ε = 0	5	-3	-27	-20	-11	-2
6 ε DNE 0	10	-2	-256	-188	-320	-10
6 ε = 0	6	-1	+200	+319	+411	+6
62 ε DNE 0	9	+3	-132	-72	-147	-1
62 ε = 0						
7 ε DNE 0						
7 ε = 0	7	-2	-254	-494	-46	-2
8 ε DNE 0						
8 ε = 0	8	-2	-11	-60	-169	+1

Trench Shoring Test Form

Date	7/21		Test #	#11		
Location	Frederick					
psi			200	200	205	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0						
1 ε = 0	1	-5	-27	-46	-126	+18
2 ε DNE 0						
2 ε = 0	2	-3	+40	-15	+326	+17
3 ε DNE 0						
3 ε = 0	3	-4	-163	-228	-21	+8
4 ε DNE 0						
4 ε = 0	4	+2	-20	-11	-12	+28
5 ε DNE 0						
5 ε = 0	5	-4	-24	-12	+3	+8
6 ε DNE 0	10	-3	-236	-173	-295	-11
6 ε = 0	6	-4	+201	+323	+415	+10
62 ε DNE 0	9	+1	-138	-66	-148	-2
62 ε = 0						
7 ε DNE 0						
7 ε = 0	7	-4	-251	-499	-47	-7
8 ε DNE 0						
8 ε = 0	8	-3	-13	-68	-171	-3

Trench Shoring Test Form

Date	7/21		Test #	#12		
Location	Frederick					
psi			205	200	200	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0						
1 ε = 0	1	+4	-10	-49	-123	0
2 ε DNE 0						
2 ε = 0	2	-2	+41	-31	+306	+5
3 ε DNE 0						
3 ε = 0	3	-4	-148	-224	-31	-3
4 ε DNE 0						
4 ε = 0	4	0	-30	-32	-31	-4
5 ε DNE 0						
5 ε = 0	5	+3	-23	-17	-8	-6
6 ε DNE 0	10	-4	-257	-191	-316	-5
6 ε = 0	6	-2	+204	+326	+417	-2
62 ε DNE 0	9	-2	-148	-82	-158	-9
62 ε = 0						
7 ε DNE 0						
7 ε = 0	7	-1	-235	-495	-56	-3
8 ε DNE 0						
8 ε = 0	8	-1	-7	-66	-171	-2

Trench Shoring Test Form

Date	7/21		Test #	#13		
Location	Frederick					
psi			200	200	200	
Gage	**Circuit #**	**At Rest**	**Shore #1**	**Shore #2**	**Shore #3**	**Post**
1 ε DNE 0						
1 ε = 0	1	+3	-17	-46	-129	+2
2 ε DNE 0						
2 ε = 0	2	+4	+51	-16	+308	+6
3 ε DNE 0						
3 ε = 0	3	-1	-149	-219	-34	-3
4 ε DNE 0						
4 ε = 0	4	+1	-34	-27	-33	-3
5 ε DNE 0						
5 ε = 0	5	-4	-37	-27	-22	-8
6 ε DNE 0	10	+1	-243	-177	-301	-5
6 ε = 0	6	0	+196	+327	+411	-4
62 ε DNE 0	9	-3	-152	-76	-158	-7
62 ε = 0						
7 ε DNE 0						
7 ε = 0	7	+2	-242	-500	-71	-11
8 ε DNE 0						
8 ε = 0	8	+2	-11	-63	-168	-4

Trench Shoring Test Form

Date	7/21		Test #	#14		
Location	frederick					
psi			200	200	200	
Gage	Circuit #	At Rest	Shore #1	Shore #2	Shore #3	Post
1 ε DNE 0	4					
1 ε = 0	1	+5	-4	-40	-114	+17
2 ε DNE 0						
2 ε = 0	2	+3	+48	-18	+330	+11
3 ε DNE 0						
3 ε = 0	3	+1	-156	-218	-33	-5
4 ε DNE 0						
4 ε = 0	4	-3	-31	-30	-37	-3
5 ε DNE 0						
5 ε = 0	5	-3	-32	-25	-18	-14
6 ε DNE 0	10	+2	-238	-156	-294	-5
6 ε = 0	6	+2	+195	+326	+400	-7
62 ε DNE 0	9	-2	-136	-73	-145	-2
62 ε = 0						
7 ε DNE 0						
7 ε = 0	7	-4	-252	-503	-49	+1
8 ε DNE 0						
8 ε = 0	8	0	-7	-62	-165	+7

Appendix S: Determining the Effective Height and Width of the Panel

The following pages present the tabulated data for finding the effective panel width and height. The procedure found the areas under the curves determined by the finite element models for 1298 lbs/strut, 2356 lbs/strut, and 4712 lbs/strut loading and calculated an average width and height based on the maximum earth pressure ordinate. The effective width of the panel was found to be 1.92 ft for all three strut loads and the effective height of the panel was found to 4.67 ft.

Table S-1 Finding the area under the curve of the earth pressure across the width of the panel for 1298 lbs/strut and finding the effective width while holding the maximum eath pressure ordinate

y = 4 ft		
Width (ft)	EP (psf)	Area (lbs/ft)
-3.50	-1.18	
-2.95	-0.88	
-2.36	2.25	
-1.77	18.29	
-1.69	0.00	211.02
-1.18	-120.37	211.05
-1.10	-150.00	130.55
-0.59	-321.28	130.49
0.00	-393.03	30.87
0.59	-321.18	31.00
1.10	-150.00	
1.18	-120.67	
1.70	0.00	
1.77	18.11	
2.36	2.30	
2.95	-0.88	
3.50	-1.18	

744.97 lb/ft Total Area

1.90 ft Effective Width

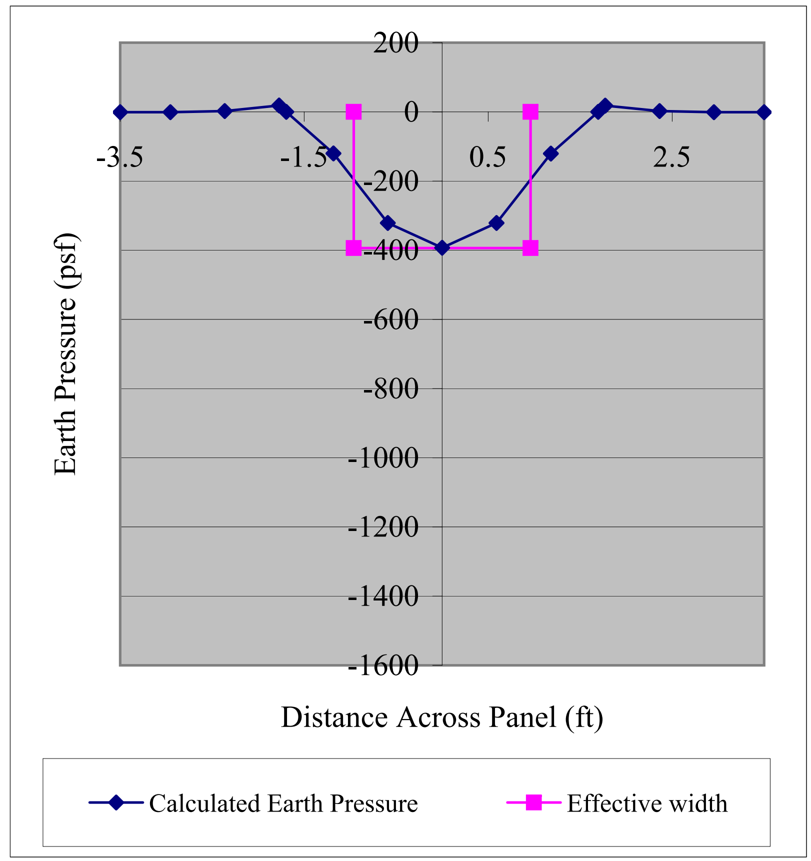


Table S-2 Finding the area under the curve of the earth pressure across the width of the panel for 2356 lbs/strut and finding the effective width while holding the maximum eath pressure ordinate

y = 4 ft		
Width (ft)	EP (psf)	Area (lbs/ft)
-3.50	-0.71	
-2.95	-3.92	
-2.36	-0.81	
-1.77	17.84	
-1.73	0.00	388.92
-1.18	-230.60	388.88
-1.10	-275.00	243.31
-0.59	-593.39	243.45
0.00	-722.83	63.24
0.59	-593.52	62.98
1.10	-275.00	
1.18	-230.01	
1.73	0.00	
1.77	18.18	
2.36	-0.90	
2.95	-3.92	
3.50	-0.71	

1390.78 lb/ft Total Area
1.92 ft Effective Width

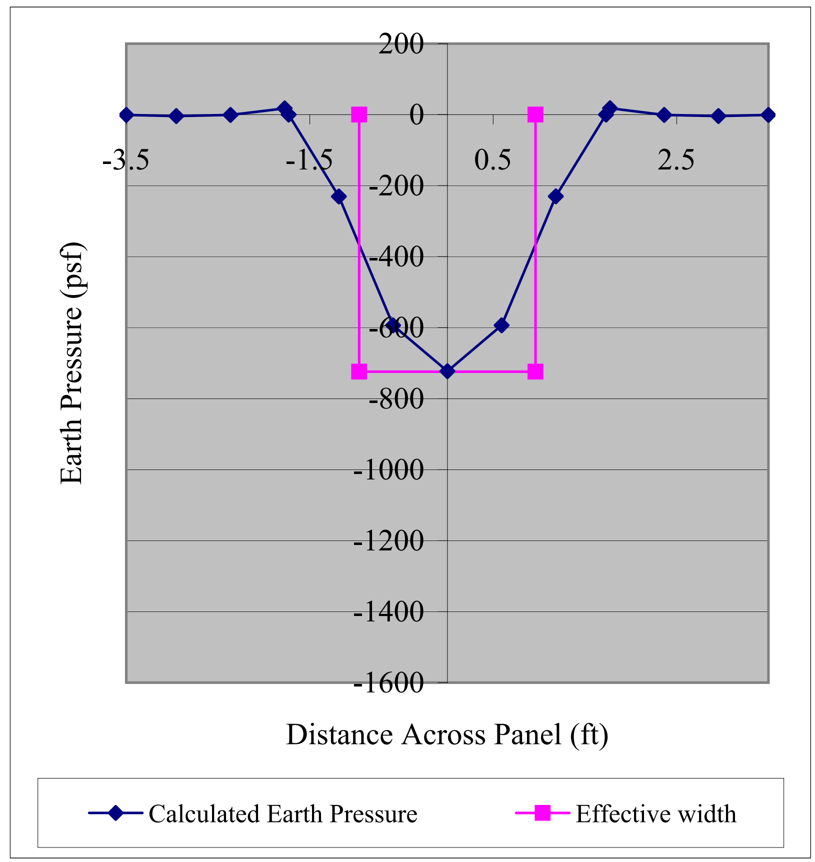


Table S-3 Finding the area under the curve of the earth pressure across the width of the panel for 4712 lbs/strut and finding the effective width while holding the maximum eath pressure ordinate

y = 4 ft		
Width (ft)	EP (psf)	
-3.50	0.37	
-2.95	-10.83	
-2.36	-7.89	
-1.77	17.21	Area (lbs/ft)
-1.75	0.00	793.58
-1.18	-480.68	793.52
-1.10	-575.00	499.99
-0.59	-1212.64	500.30
0.00	-1473.11	137.11
0.59	-1212.84	136.54
1.10	-575.00	
1.18	-479.42	
1.75	0.00	
1.77	17.92	
2.36	-8.07	
2.95	-10.84	
3.50	0.37	

2861.04 lb/ft Total Area

1.94 ft Effective Width

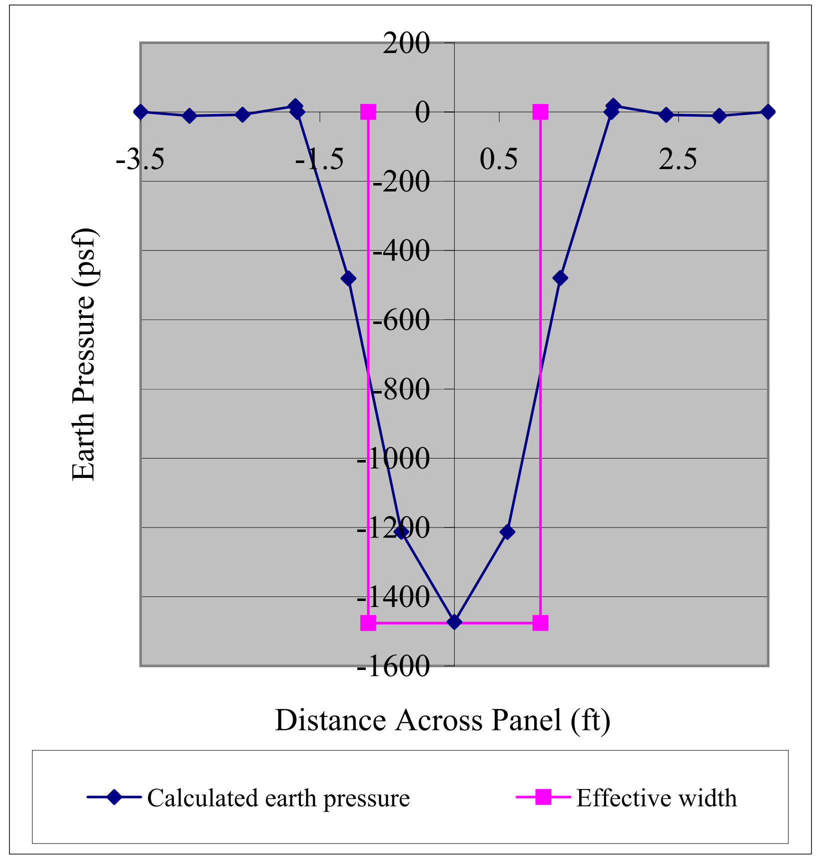


Table S-4 Finding the area under the curve of the earth pressure vertically behind the panel for a 1298 lbs/strut load

Height (ft)	EP (psf)	Area (lb/ft)	Height (ft)	EP (psf)	Area (lb/ft)
0.00	99.41		4.21	-367.66	-68.86
0.20	40.55		4.41	-316.32	-58.93
0.30	0.00	-1.97	4.62	-263.78	-50.16
0.40	-39.94	-5.19	4.82	-229.11	-45.59
0.50	-60.08	-6.64	5.02	-222.35	-30.93
0.60	-78.72	-19.09	5.16	-234.72	-12.89
0.80	-112.81	-26.31	5.21	-238.64	-49.29
0.99	-151.36	-35.24	5.40	-271.17	-62.48
1.19	-200.79	-46.49	5.62	-320.78	-70.01
1.40	-262.06	-59.36	5.82	-364.52	-69.96
1.60	-329.10	-71.69	6.01	-377.09	-68.05
1.80	-385.70	-79.00	6.20	-340.54	-63.32
2.00	-409.48	-80.24	6.40	-273.26	-49.04
2.20	-387.06	-73.17	6.61	-201.83	-34.36
2.40	-337.41	-62.59	6.82	-135.25	-22.81
2.60	-284.26	-53.85	7.02	-89.20	-15.06
2.80	-245.28	-6.75	7.22	-61.82	-10.56
2.83	-243.54	-41.73	7.42	-44.28	-3.00
3.01	-232.60	-48.03	7.50	-26.26	-1.61
3.21	-246.05	-52.75	7.62	-1.92	-0.01
3.41	-282.24	-61.82	7.62	0.00	
3.61	-333.63	-72.21	7.82	48.77	
3.81	-379.37	-77.90	8.00	-78.90	
4.01	-393.79	-75.89		Total Area	-1844.84 lb/ft
				Effective Height	**4.68** ft

Table S-5 Finding the area under the curve of the earth pressure vertically behind the panel for a 2356 lbs/strut load

Height (ft)	EP (psf)	Area (lb/ft)
0.00	170.61	
0.20	88.02	
0.34	0.00	-0.89
0.40	-32.90	-5.35
0.50	-70.13	-8.35
0.60	-104.55	-27.60
0.80	-172.35	-42.01
0.99	-249.46	-59.59
1.19	-346.01	-81.26
1.40	-462.93	-105.59
1.60	-588.53	-128.69
1.80	-694.54	-142.55
2.00	-740.35	-145.26
2.20	-701.69	-132.87
2.40	-613.97	-114.13
2.60	-519.61	-98.66
2.80	-450.64	-12.40
2.83	-447.56	-76.75
3.01	-428.09	-88.21
3.21	-451.02	-96.59
3.41	-516.36	-113.22
3.61	-611.64	-132.49
3.81	-696.58	-143.15
4.01	-724.31	-139.80

Height (ft)	EP (psf)	Area (lb/ft)	
4.21	-678.34	-127.38	
4.41	-586.92	-109.69	
4.62	-492.80	-93.90	
4.82	-429.89	-85.27	
5.02	-414.47	-57.43	
5.16	-434.22	-23.83	
5.21	-440.88	-91.58	
5.40	-506.32	-117.01	
5.62	-602.27	-131.05	
5.82	-680.42	-130.78	
6.01	-705.88	-128.46	
6.20	-648.79	-121.92	
6.40	-533.13	-96.48	
6.61	-401.49	-69.03	
6.82	-275.70	-46.65	
7.02	-183.35	-30.46	
7.22	-122.11	-19.50	
7.42	-73.75	-5.01	
7.50	-43.93	-2.72	
7.62	-3.79	-0.02	
7.62	0.00		
7.82	83.19		
8.00	-39.92		
	Total Area	-3383.61	lb/ft
	Effective Height	**4.67**	ft

Table S-6 Finding the area under the curve of the earth pressure vertically behind the panel for a 4712 lbs/strut load

Height (ft)	EP (psf)	Area (lb/ft)
0.00	332.59	
0.20	196.02	
0.38	0.00	-0.13
0.40	-16.88	-5.70
0.50	-93.01	-12.26
0.60	-163.31	-46.96
0.80	-307.79	-77.74
0.99	-472.66	-114.98
1.19	-676.38	-160.36
1.40	-919.92	-210.75
1.60	-1178.74	-258.35
1.80	-1397.16	-287.13
2.00	-1493.09	-293.20
2.20	-1417.47	-268.71
2.40	-1243.13	-231.39
2.60	-1055.01	-200.62
2.80	-917.83	-25.27
2.83	-911.71	-156.41
3.01	-872.83	-179.63
3.21	-917.32	-196.33
3.41	-1048.99	-230.17
3.61	-1244.10	-269.63
3.81	-1418.24	-291.62
4.01	-1476.23	-285.19

Height (ft)	EP (psf)	Area (lb/ft)
4.21	-1385.14	-260.51
4.41	-1202.51	-225.15
4.62	-1013.80	-193.40
4.82	-886.67	-175.54
5.02	-851.55	-117.73
5.16	-888.08	-48.72
5.21	-900.98	-187.79
5.40	-1041.29	-241.07
5.62	-1242.65	-269.89
5.82	-1399.09	-269.13
6.01	-1453.85	-265.88
6.20	-1350.08	-255.25
6.40	-1124.34	-204.39
6.61	-855.73	-147.91
6.82	-595.22	-100.90
7.02	-397.55	-65.50
7.22	-259.29	-39.83
7.42	-140.80	-9.58
7.50	-84.13	-5.26
7.62	-8.06	-0.04
7.63	0.00	
7.82	161.50	
8.00	48.76	

Total Area -6885.99 lb/ft

Effective Height **4.66** ft

Figure S-1 Plot of effective panel height vs. finite element earth calculated earth pressure for 1298 lbs/strut

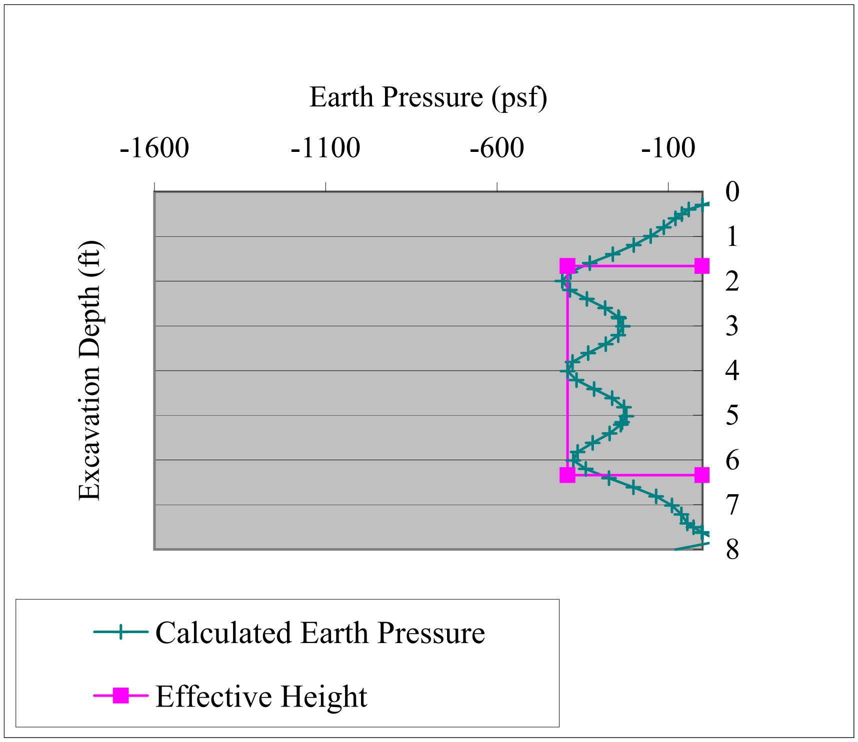


Figure S-2 Plot of effective panel height vs. finite element earth calculated earth pressure for 2356 lbs/strut

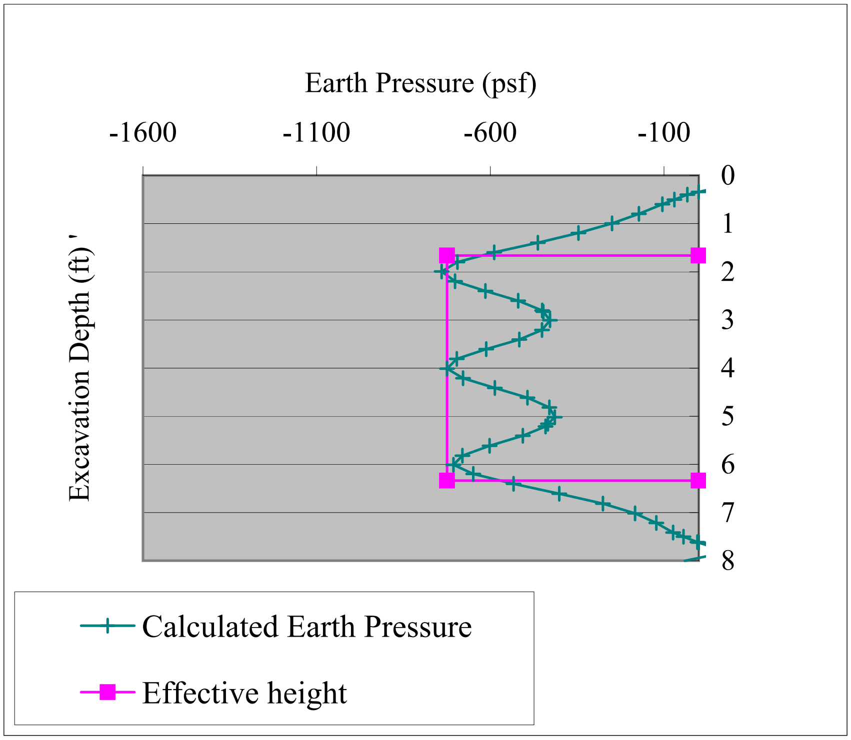
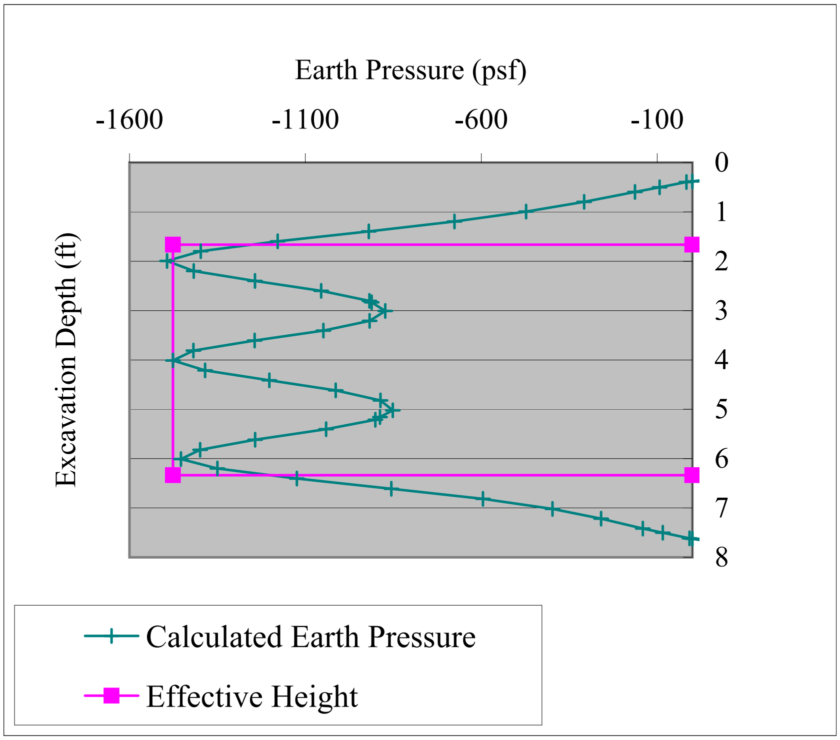


Figure S-3 Plot of effective panel height vs. finite element earth calculated earth pressure for 4712 lbs/strut



Appendix T: Determining the Limit Factor from the Finite Element Results

This appendix presents the tabulated data for calculating the limit factor from the finite element results. The maximum ordinate as determined in various parametric studies and presented in the referenced appendices was divided by the uniform earth pressure determined by distributing the known strut loads over the panel area (8 ft x 4 ft). Table T-1 presents the factors calculated from the results in Appendix E for variable strut loads set at 1298 lbs/strut, 2356 lbs/strut, and 4712 lbs/strut.

Table T-1 Variable strut load limit factor calculation

Variable		**EP (psf)**	**Uniform**	**Limit**
Strut Load		**@ y = 4 ft**	**EP (psf)**	**Factor**
1298	lbs/strut	-393.79	-121.69	3.2
2356	lbs/strut	-724.31	-220.88	3.3
4712	lbs/strut	-1476.23	-441.75	3.3

Tables T-2, T-3, and T-4 provide the limit factors calculated by dividing the earth pressure obtained from the parametric studies by the uniform earth pressure found by distributing three 1298 lbs loads over the panel area (8 ft x 4 ft), or 121.69 psf.

Table T-2 Variable panel stiffness limit factor calculation (divide by 121.69 psf) – earth pressure is from Appendix F

Variable		**EP (psf)**	**Limit**
Panel Stiffness		**@ y = 4 ft**	**Factor**
7.31E+07	psf	-521.72	4.3
1.46E+08	psf	-446.53	3.7
2.92E+08	psf	-393.79	3.2
1.38E+08	psf	-368.09	3.0
5.85E+08	psf	-352.00	2.9

Table T-3 Variable panel thickness limit factor calculation (divide by 121.69 psf) – earth pressure is from Appendix G

Variable		**EP (psf)**	**Limit**
Panel Thickness		**@ y = 4 ft**	**Factor**
0.25	in	-575.34	4.3
0.50	in	-457.27	3.7
0.75	in	-393.03	3.2
1.00	in	-351.24	3.0
1.25	in	-321.22	2.9

Table T-4 Variable soil properties limit factor calculation (divide by 121.69 psf) – earth pressure is from Appendix H

Variable					
Soil				**EP (psf)**	**Limit**
Unit Weight		Young's Modulus		**@ y = 4 ft**	**Factor**
119	pcf	6.27E+05	psf	-393.79	3.2
124	pcf	1.67E+06	psf	-543.78	4.5

Appendix U: Finite Element Model Results from Loading Top Strut Only

The following pages present the tabulated data for earth pressure and panel displacement due to the loading of only one strut at a depth of y = 2 ft to 1298 lbs.

Table U-1 Earth pressure behind the vertical height of the panel for one strut loaded to 1298 lbs at a depth of y = 2 ft

Depth (ft)	EP (psf)
0.00	89.60
0.20	31.24
0.40	-48.17
0.50	-67.79
0.60	-85.96
0.80	-119.41
0.99	-157.62
1.19	-206.99
1.40	-268.34
1.60	-334.59
1.80	-388.62
2.00	-407.72
2.20	-376.09
2.40	-311.37
2.60	-235.64
2.80	-165.47
2.83	-158.37
3.01	-113.44
3.21	-74.85
3.41	-41.04
3.61	-13.72
3.81	2.82
4.01	11.60
4.21	17.24
4.41	21.36
4.62	23.04
4.82	22.41
5.02	17.24
5.16	13.51
5.21	12.75
5.40	20.99
5.62	28.30
5.82	26.09
6.01	28.83
6.20	39.78
6.40	46.97
6.61	43.73
6.82	37.00
7.02	26.02
7.22	12.00
7.42	-7.56
7.50	-3.68
7.62	1.72
7.82	10.06
8.00	-120.54

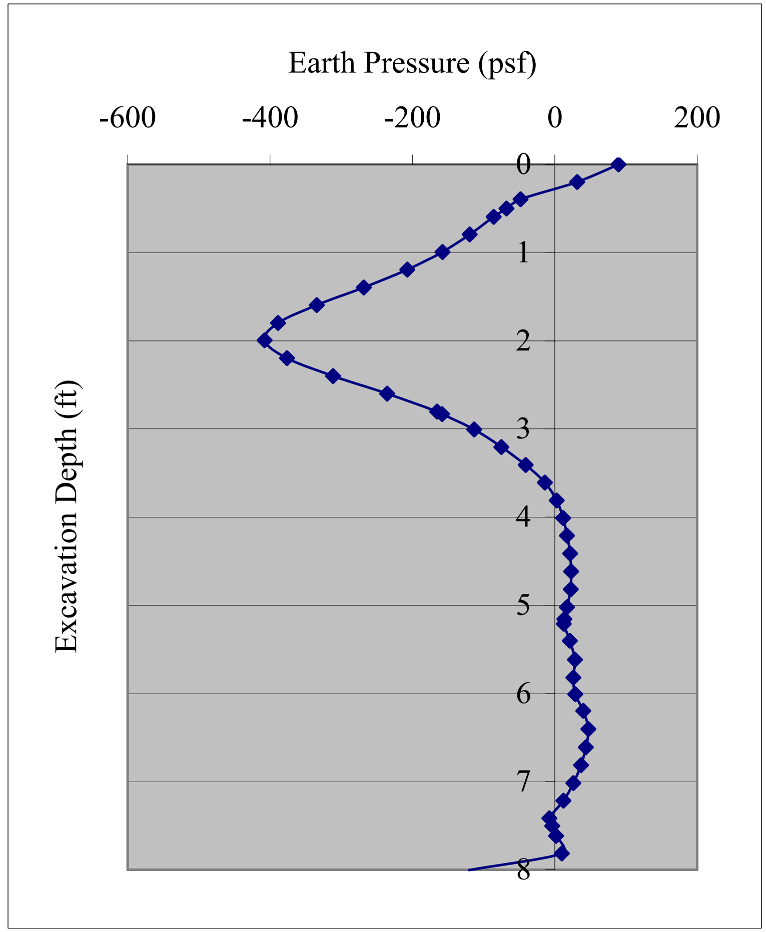
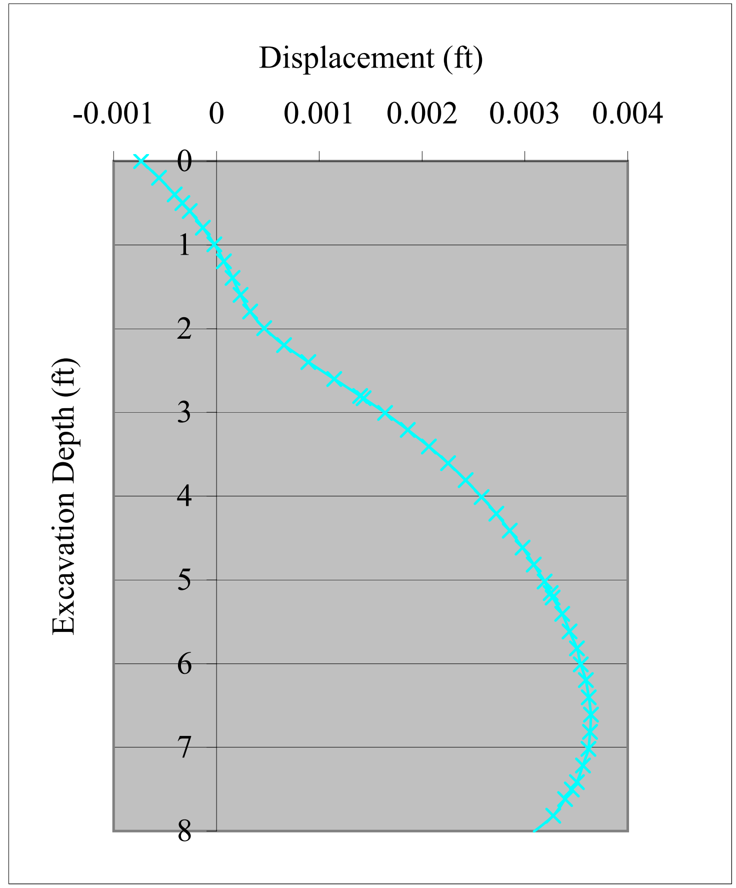


Table U-2 Earth pressure behind the vertical height of the panel for one strut loaded to 1298 lbs at a depth of y = 2 ft

Depth (ft)	Displacement (ft)
0.00	-7.32E-04
0.20	-5.59E-04
0.40	-4.06E-04
0.50	-3.31E-04
0.60	-2.61E-04
0.80	-1.33E-04
0.99	-2.12E-05
1.19	7.35E-05
1.40	1.57E-04
1.60	2.35E-04
1.80	3.29E-04
2.00	4.64E-04
2.20	6.56E-04
2.40	8.93E-04
2.60	1.15E-03
2.80	1.40E-03
2.83	1.43E-03
3.01	1.64E-03
3.21	1.86E-03
3.41	2.06E-03
3.61	2.25E-03
3.81	2.42E-03
4.01	2.58E-03
4.21	2.72E-03
4.41	2.85E-03
4.62	2.98E-03
4.82	3.09E-03
5.02	3.19E-03
5.16	3.25E-03
5.21	3.27E-03
5.40	3.36E-03
5.62	3.43E-03
5.82	3.51E-03
6.01	3.54E-03
6.20	3.59E-03
6.40	3.62E-03
6.61	3.64E-03
6.82	3.63E-03
7.02	3.62E-03
7.22	3.57E-03
7.42	3.50E-03
7.50	3.46E-03
7.62	3.39E-03
7.82	3.28E-03
8.00	3.09E-03

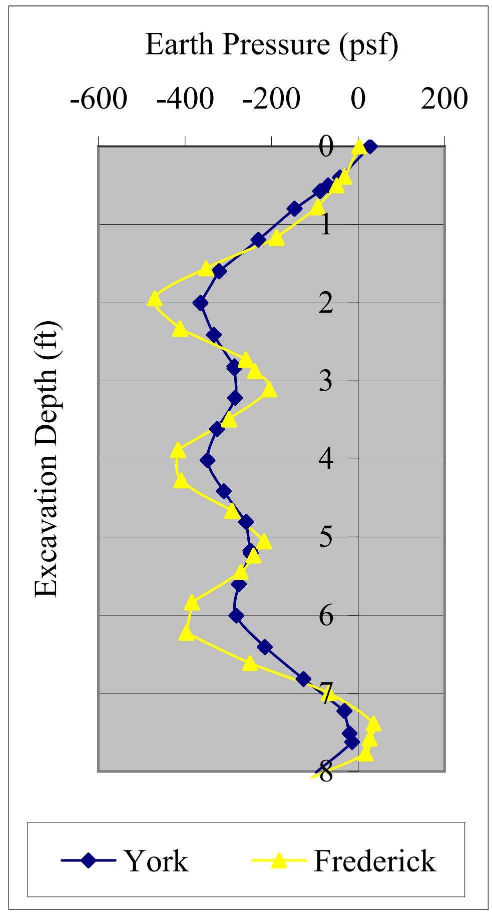


Appendix V: Frederick and York Finite Element Model Results

The following pages present the tabulated data for earth pressure determined by the finite element models that simulate the Frederick and York trench. The models simulate the surcharge size and configuration, panel placement, and strut loading as described in Chapter 4.

Table V-1 Plot of finite element earth pressure in Frederick trench vs. the York trench for 1298 lbs/strut

York		Frederick	
Height (ft)	EP (psf)	Height (ft)	EP (psf)
0.00	28.02	0.00	2.36
0.40	-42.03	0.39	-31.77
0.50	-68.84	0.50	-49.54
0.57	-87.84	0.77	-93.16
0.80	-146.95	0.78	-93.06
0.80	-146.95	1.16	-188.21
1.20	-230.51	1.18	-188.02
1.60	-321.38	1.56	-351.10
1.60	-321.49	1.94	-470.10
2.00	-364.10	2.33	-411.49
2.01	-363.93	2.34	-410.99
2.41	-333.09	2.72	-258.89
2.82	-285.91	2.73	-259.15
2.82	-286.00	2.87	-238.19
2.84	-285.86	3.11	-205.44
3.22	-284.25	3.49	-298.06
3.62	-325.43	3.50	-297.67
3.62	-325.37	3.88	-415.55
4.02	-347.55	3.89	-416.06
4.41	-310.30	4.27	-408.55
4.81	-258.05	4.66	-290.80
4.81	-258.05	4.67	-290.62
5.17	-248.88	5.05	-216.89
5.21	-248.24	5.06	-216.96
5.21	-248.24	5.24	-241.59
5.61	-275.48	5.45	-271.24
6.01	-281.63	5.83	-383.76
6.41	-215.66	5.84	-383.44
6.41	-215.65	6.22	-396.40
6.82	-126.06	6.23	-396.78
7.23	-31.75	6.61	-249.42
7.23	-31.76	7.00	-67.35
7.51	-19.27	7.39	35.75
7.62	-14.04	7.58	26.84
8.01	-97.39	7.77	17.45
		8.08	-106.60



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