ANALYSIS OF LATERAL EARTH PRESSURE DISTRIBUTIONS WITH NUMERICAL AND ANALYTICAL METHODS: THE CASE OF BRACED DEEP EXCAVATION IN ADDIS ABABA CITY

MSc THESIS

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A THESIS SUBMITTED TO
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Therefore I/we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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Declaration

I hereby declare that this MSc thesis entitled "Analysis of Lateral Earth Pressure Distributions with Numerical and Analytical Methods: The Case of Braced Deep Excavation in Addis Ababa City" is my original work performed under the supervision of my research advisor Dr. Ing. Siraj Mulugeta (PhD) and has not been presented for a degree in any other university, and all sources of material used for this thesis / dissertation have been duly acknowledged.

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Table of Contents

List of Symbols/Abbreviation

BEF: Beam on Elastic Foundation

BEM: Boundary Element Method

EA: Axial Stiffness

FEM: Finite Element Method

HSM: Hardening Soil Model

SPT: Standard Penetration Test

SPW: Sheet Pile Wall

E: Young's Modulus of Elasticity

G: Shear Modulus of Elasticity

γ: Unit Weight of Soil

$\boldsymbol{\gamma_{sat}}$: Saturated Unit Weight

Φ: Angle of Internal Friction

C: Cohesion of Soils

V: Poisson's Ratio

$\mathbf{D_{emb}}$: Depth of Embedment

$\mathbf{D_{exc}}$: Depth of Excavation

$\mathbf{K_a}$: Coefficient of Active Earth Pressure

$\mathbf{K_p}$: Coefficient of Passive Earth Pressure

$\mathbf{K_o}$: Coefficient Earth Pressure at Rest

$\boldsymbol{\lambda}^*$: Modified compression index

$\boldsymbol{\kappa}^*$: Modified Swelling Index

$\mathbf{U_{x,max}}$: Maximum Horizontal Displacement of the Ground or Sheet Pile Wall

$\mathbf{U_{y,max}}$: Maximum Vertical Ground Settlement

$\mathbf{U_{x,SPW}}$: Maximum Total Displacement in the Sheet Pile Wall

$\mathbf{M_{1\text{-}1}}$**:** Maximum Bending Moment in the Sheet Pile That Form at the Center

$\mathbf{M_{2\text{-}2}}$**:** Maximum Bending Moment in the Sheet Pile That Form at the Corner

Ψ: Delatancy Angle

List of Tables

List of Figures

ABSTRACT

The lateral earth pressure distributions caused by braced deep excavation are paramount important to determine in order to assure both safety against displacement, settlement and bending moment of the ground and support systems. The study of the effects of braced deep excavations in different soil types which is located in Kirkos Sub City was presented in this paper. The main objective of this thesis is to investigate the effects of change in soil type and structural parameters on the performance of deep excavation support system and adjacent ground.

The parameters considered as an input in software are soil type, excavation depth (12 m, 16 m, and 20 m), embedment depth, stiffness, strut and walling spacing, anchors and grouting. Following this the numerical analysis was proven by experiment as a useful tool for analysis. The results showed that as parameters were varied the recorded average value of the maximum and minimum horizontal displacements of the ground and SPW, $U_{y,max}$, M_{1-1} and M_{2-2} were 0.3643 m - 0.03169 m, 0.4891m - 0.03772m, 0.317m - 0.029m, 4237KN m - 85.55KN m and 2061KN m - 14.40KN m respectively.

The comparison of empirical and numerical method of analysis result shows that the maximum U_y is recorded by Peck whereas the minimum is recorded by PLAXIS-3D analysis. But the maximum $U_{x,SPW}$ is recorded by PLAXIS-3D whereas the minimum is recorded by Pecks method. Finally, the overall precious summary and sufficient conclusion were made for exact design of braced deep excavation and its retaining system.

Keywords: Bending moment, braced deep excavation, FE method, ground displacement, PLAXIS-3D, tieback anchored sheet pile wall.

1. INTRODUCTION

1.1. Background of the study

Internationally a considerable number of multi–story buildings and transport facilities constructions are take place near the existing structures due to the congested life style of urbanized area. Thus to design deep excavation for such types of construction, it would be very useful for geotechnical engineers to estimate the lateral earth pressure distribution and probable ground settlement of soil at specified area. Worldwide experiences shows that, tie-back anchored SPW should have been designed by having high stiffness and sufficient safety to comply with strict specification on the limitation of ground movement.

In our country especially in Addis Ababa major constructions in many directions has common standard. There are a large volume of deep excavations for the requirement of building constructions and transport facilities in the City. Then, the case of ground settlement is of a major concern due to the probability of additional settlement that might be over the design limit. Additionally as depth of excavation increases, there may be excessive displacement of the ground and deformation of soil supporting system (SPW in this case) because; lateral earth pressure distribution may also be increase in depth.

Theoretical methods like Coulomb’s and Rankin’s theories of earth pressure cannot be directly used for the computation of lateral earth pressure on flexible walls, because those theories are mostly applicable to rigid retaining wall that rotates about their bases. The sheeting with anchor and bracing system is somewhat flexible, and rotation takes place at the top of walls (Arora, 2004). The general wedge theory also does not provide the relationships required for estimating the lateral earth pressure variation with depth. Even, Peck (1969), Clough and O’Rourke (1990) cannot always provide reasonable prediction on the deformation pattern in complex modern construction. Concerning deep excavation the lateral earth pressure distribution is dependent on the type of soil, construction method and type of construction equipment used. Apparent pressure envelopes(diagram) are simple to use for the design of deep excavation, but actual pressure distribution is a function of construction sequence, relative flexibility of the wall, type of soil, location of water table, etc. (Das, 2007).

By using 3D-finite element method, geotechnical aspects such as soil properties, construction sequences, surcharge load, water pressure and the detail of structures of braced deep excavation can be simulated in a more realistic manner.

Mostly used constitutive models coded in geotechnical software such as ABAQUS, PLAXIS-3D and FLAC-3D requires various input parameters obtained from different soil investigation techniques. PLAXIS incorporates constitutive models whose parameters may be obtained directly or correlated from SPT or Triaxial tests. In this thesis PLAXIS 3D, version 2013 was selected to model soil and support structures by Hardening Soil Model which is the most reliable and accurate soil model as compared to others.

1.2. Statement of the problem

Currently a lot of multi-story buildings with basements for parking, storage and road facilities specifically bridges are being constructed by using braced deep excavation in Addis Ababa soil. However any deep excavation in congested areas leads to some movement of the surrounding ground that will affect the confinement of soil that needs for its stability.

The cause of excessive ground movement and deformation that affects the functionality of structures in terms of efficiency, durability, aesthetics and its long life span are due to the lateral earth pressure, soil-structure interaction, water pressure, earth quakes. In general the effects of construction activities near adjacent ground and structures should be given special attention before excavation.

1.3. Objectives

1.3.1. General Objectives

- The main objective of this thesis is to analyze the effect of lateral earth pressure distributions that results due to braced deep excavation on support structure (SPW) and adjacent ground by Numerical and Analytical methods.

1.3.2. Specific Objectives

- To study the surface settlement and total displacement develop that caused by braced deep excavation on adjacent ground behind retaining structures.
- To assess the displacement and bending moment in SPW that results from lateral earth pressure distributions due to braced deep excavation in stratified soil of Addis Ababa.
- To investigate the effects of change in soil type and structural parameters on the performance of braced deep excavation support system and adjacent ground.
- To identify the most appropriate methods for the analysis of lateral earth pressure distribution effects on adjacent ground and structures.

1.4. Significance of the study

The most importance of this study is to provide the valuable solution for braced deep excavation, mainly its effects on nearby ground and structures such as ground settlement, total displacement and bending moment that might happen as a result of deep excavation problem on Addis Ababa soil. Because, still now there was no sufficient and accurate researches were studied on lateral earth pressure distribution due to braced deep excavation in Addis Ababa soil. So hopefully, this research fills the gaps that doesn't gain much attention and have been a very serious problem for geotechnical engineering structures. In fact, a significance of this study is to save various ground and structures in Addis Ababa, from lack of advanced geotechnical analysis in order to save loss of life and property, collapse of structures, to increase the awareness of geotechnical engineers, especially, for civil engineers those have been working in the design practice at Addis Ababa City.

1.5. Scope and limitation

The simulations are based on the silty clay, clayey silt and clayey silt with sand including silty sand that found in Kirkos Sub City of Addis Ababa. For these simulations, the soil types, depth of excavations, embedment depth and stiffness of tieback anchored sheet pile wall, strut and walling stiffness and vertical spacing of strut and walling are varied. The results of these analyses were recorded in terms of displacement, bending moment and the ground settlement. The supporting system used in this thesis is tieback anchored SPW that was modeled as an equivalent wall so as to effectively use PLAXIS-3D software. Only finite element modeling is done for all cases, the effect of heave and on site measurement of deformations using inclinometers and other instruments is out of the scope of this thesis.

1.6. Research Questions

- ✓ From the numerical and empirical methods of analysis which one is more important for braced deep excavation analysis in terms of accuracy?
- ✓ How much variation is there between FE method (PLAXIS-3D) and empirical methods result of braced deep excavations supported by tie-back anchored sheet pile wall?
- ✓ Does this variation have an effect on the design and economy of tie-back anchored sheet pile wall and braced deep excavation on adjacent ground or structures?

2. LITERATURE REVIEW

2.1. General Concepts

The excavations in soil deeper than 6 m for deep foundation of multi-story buildings as well as for transport facility structures are conventionally defined as deep excavation. It can be done either with side slope or with vertical side and bracings. Its analysis is usually required before going in to its design, because deep excavation analysis is a typical soil-structure interaction problem. The analyses necessary for excavation design include stress and deformation analyses.

Clough and O'Rourke (1990) found that the wider the excavation, the larger the deformation of the retaining wall. In fact, for a typical excavation the wider the excavation, the larger are the unbalanced forces; the larger the unbalanced force, the greater are the wall deformation. As far as the analysis of lateral earth pressure distribution that results from braced deep excavation is concerned, it depends on a number of factors. These are the time dependent nature and physical properties of soil, drainage problems, amount of surcharge load, the interaction between soil and retaining structure and location of ground water table.

There are several types of in-situ walls that are used to support excavations. The criteria to select the types of wall are size of excavation, ground conditions, groundwater level, vertical and horizontal displacements of adjacent ground and limitations of various structures, availability of construction, cost, speed of work and others. The following types of in-situ walls are the common types of walls.

1. Sheet pile walls
2. Soldier pile and lagging walls
3. Diaphragm walls or slurry trench walls
4. Contiguous, Secant pile walls
5. Caissons
6. Cofferdams

2.1.1. Sheet pile walls

Sheet pile walls are widely used for both large & small waterfront structures ranging from small boat launching facilities to large ocean going ship dock facilities. Piers jutting, in to the harbour consisting of two rows of sheeting is widely used. Sheet pile walls are also used in slope stability and erosion protection works.

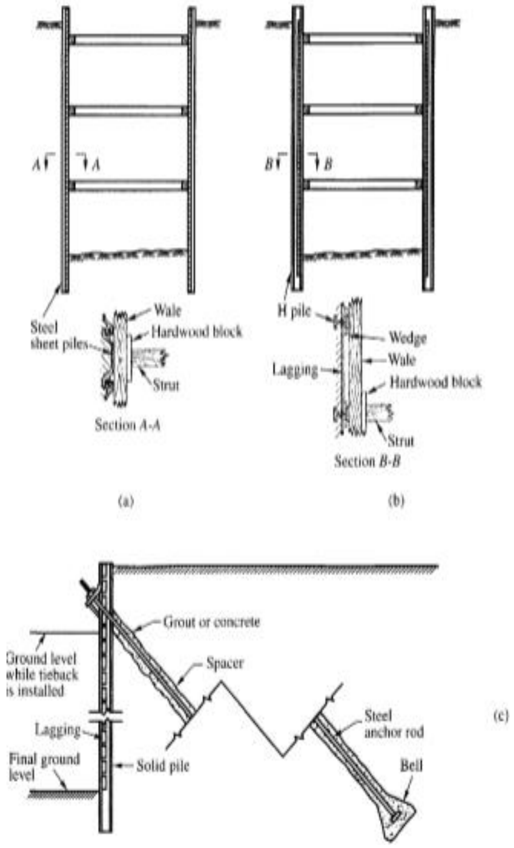


Figure 2.1 Cross sections, through typical bracing in deep excavation, (a) sides retained by steel sheet piles, (b) sides retained by H piles and lagging, (c) one of several tieback systems for supporting vertical sides of open cut. Several sets of anchors may be used, at different elevations (Peck, 1969)

The sheet pile problem is one of lateral earth pressure with the Coulomb or Rankine methods being directly applicable. In particular, the Coulomb method is recommended since a sheet pile wall is sufficiently flexible to produce large lateral displacements so that relative slip occurs between wall and soil. Friction resistance is developed from soil slip at the interface to reduce the earth pressure coefficient Ka. The Rankine method is also commonly used since the earth pressure co efficient is more conservative i.e. slightly larger and the method is simple.

- **Types of Sheet Pile**

1. Timber sheet piling: It is used for short spans, light lateral loads and commonly for temporary structures in the form of braced sheeting. If it is used in permanent structures above water level, it requires preservative treatment, and even so, the useful life is relatively short.

2. Reinforced concrete sheet piles: These sheet piles are precast concrete members, usually with a tongue and groove joint. They are designed for computed service stresses and also to withstand driving and handling stresses. They are relatively bulky and as such tend to increase the driving resistance.

3. Steel sheet piling: Steel sheet piling is the most common type used because of several advantages over other materials; viz. (a) it is resistant to high driving stresses as developed in hard material. (b) It is relatively lightweight when compared to piles of other materials for the same purpose. (c) It may be reused several times. (d) It has a long service life above or below water table with modest protection. (e) It is easy to increase the pile length by either welding or bolting. (f) Joints are less apt to deform when wedged full with soil & stones during driving.

2.1.2. Lateral Yielding of Sheet Pile Wall and Vertical Ground Settlement

In braced cuts, some lateral movement (yielding) of sheet pile walls may be expected. Lateral yielding of the walls will cause the ground surface surrounding the cut to settle. (See Figure 2.2) The amount of lateral yielding (δ_H) of sheet pile wall depends on several factors, thus are the elapsed time between excavation, placement of wales and struts and type of soil below the bottom of excavation.

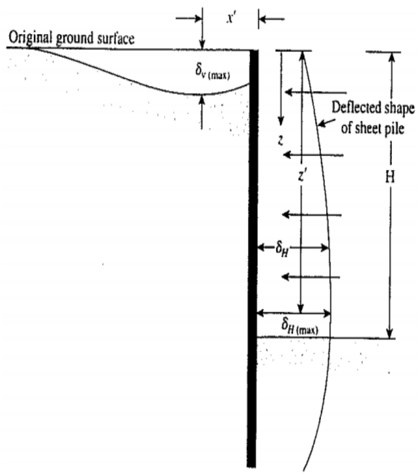


Figure 2.2 Lateral Yielding of SPW and Vertical Ground Settlement (Peck (1969))

The literature review is divided into the following four categories:

I. Approaches to design and analysis of excavation support structures
II. previous researches on case studies of similar deep excavation conditions
III. Numerical studies of deep excavation.
IV. Field performance studies of deep excavations.

2.2. Approaches to design and analysis of excavation support structures

There are three major approaches to design and analyze excavation support structures

1. The classical approach
2. The beam–column approach
3. Beam on Elastic Foundation Method
4. The finite element approach.

The major characteristics of the classical approach are quoted from A. Teferra and M. Leikun (1999) and A. Teferra (1992). Some of the major characteristics of the beam column approach are presented from the design manual for excavation support by C. J. Rutherford (2004) and its details can be found in literature and beam–column program manuals (e.g. Briaud and Kim, 1998 and Matlock et al.,1981respectively).

2.2.1. The Classical Approach

The classical method for sheet pile walls bases its calculations on either Rankine or Coulomb failure criteria. Based on loads on sheet piling and embedment depth, sections of sheet pile walls are categorized in to two. Namely, cantilever and anchored sheet pile wall.

2.2.1.1. Cantilever sheet pile wall

Cantilever sheet pile wall is normally recommended for moderate height about 6m or less measured above dredge line. In such walls the sheet pile acts as a wide cantilever beam above the dredge line.

2.2.1.2. Anchored sheet pile Walls

Anchored sheet pile walls are held in tact by tie-back anchored and dead man anchored into the soil. The principal methods of analyzing the equilibrium of anchored sheet pile wall are based on two assumptions related to the method of support of the driven end.
Thus are:

2.2.1.2.1. Free earth support method

The free earth support method involves a minimum penetration depth. Below the dredge line, no pivot point exists for the static system.

2.2.1.2.2. Fixed earth support method

When using the fixed earth support method, we assume that the toe of the pile is restrained from rotating (see figure 2.3) below. In the fixed earth support solution, a simplified method called the equivalent beam solution is generally used to calculate L3 and, thus, D.

The development of the equivalent beam method is generally attributed to Blum (1931).

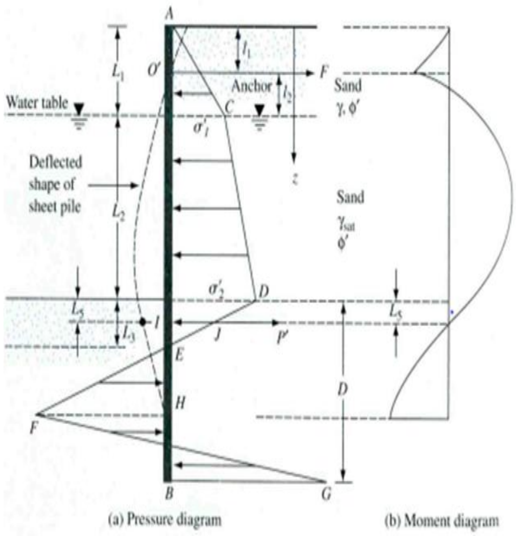


Figure 2.3 Fixed earth support method for penetration of sandy soil (Cornfield, 1975)

2.2.2. Beam-Column Method

The boundary element method, BEM (Figure 2.4), consists of modeling the wall as a set of vertical elements Δz long with a bending stiffness, EI, and an axial stiffness, AE. The soil is represented by a series of vertical and horizontal springs placed along the wall. Spring models for tieback walls have been recommended by Briaud and Kim (1998). A typical input for the BEM is the length of the wall, the length of the wall elements, and the wall stiffness.

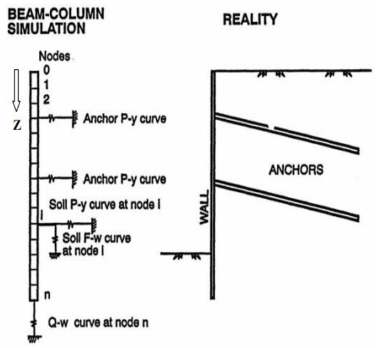


Figure 2.4 Boundary elements model of repeatable section of wall (Briaud and Kim, 1998)

The beam-column method for tieback walls deals with the analysis of the wall as a structural element interacting with the soil and the anchors (Briaud and Kim, 1998).

Three general methods have been used for staged construction analyses:

- ✓ Equivalent Beam Method
- ✓ Beam on Elastic Foundation Method
- ✓ Finite Element Method

The equivalent beam method is obsoleted and seldom used in current actual application. But our discussion focused on the beam on elastic foundation and finite element methods. Both approaches can be used to predict stresses, loads, and ground movements.

2.2.3. Beam on Elastic Foundation Method (BEF)

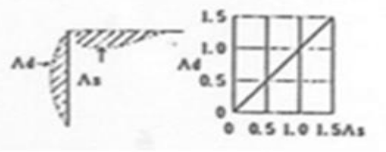
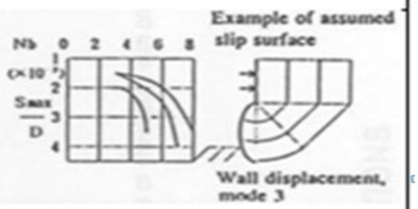
The elastic foundation model (Winkler) approximates the wall-soil interaction with a 1D - model instead of 2D - model that includes the soil mass, and hence does not include the effects of mass. Typically, the required soil arching within the soil parameters include: unit weight; at-rest, active, and passive earth pressure coefficients; and values for the modulus of subgrade reaction for the various soils that may affect the system.

The BEF method does not directly estimate ground movements behind the wall. Ground movements behind the wall are evaluated using the calculated wall displacement from the model. An empirical relationship between wall movement and ground movements must then be used. Beam on Elastic Foundation Method of analytical model can provide useful insights into the behavior of the wall and the wall-soil boundary, and the automated computer programs make it easy to perform multiple analyses.

2.2.4. Finite Element Method

FEM is mostly used for the soil and retaining structures model as small elements and assigning the elements properties which control their behavior. The typical input for FEM is the mesh description including geometry of elements for the supporting wall, anchors, soil, models of retaining wall parameters, water, boundary conditions, and the loads of boundary. Typical programs include PLAXIS, FLAC, etc.

Table 2.1 Methods for predicting settlement of Adjacent ground for braced excavations (S.S Gue & Y.C. Tan (1998))

Proposers	Outline of prediction method	Ground	Conceptual diagram of the method
Naito, *et al.* (1958) (Empirical)	Solve for volume A_d (amount of deformed soil) and A_s (amount of settlement of ground surface) due to deformations of walls, which are the same. In recent years, amount of deformations of walls is computed by the "Elasto-plastic method" and amount of settlement is predicted from the relationship shown in the conceptual diagram.	Plastic Cohesive soil	
Maruoka, Ikuta, Aoki, Sato, *et al.* (1978) (1991) (Theoretical)	Determination of the relationship between the maximum amounts of settlement, S_{max}. (Calculated as the following condition: a sliding surface is assumed according to the stress characteristic curve defined by Bransby (1975), and the wall displaces along with the sliding surface and appears on the ground surface.), and the heaving stability number N_b, proposed by Peck. After that, sliding surfaces are assumed by various methods, and the sliding surfaces are patterned for computing settlement of surface ground. The settlements are compared with measured data. D is the depth of excavation.	Soft Cohesive soil	

Note: The content in the parenthesis under the proposers' column are the category of the prediction method, either theoretical or empirical.

2.2.5. Comparison of Design Methods

Even though classical methods are quick and simple, computer aided analysis may be required for complex soil, site conditions and represents the future trend. Comparison of the design methods are presented as below.

Table 2.2 Comparisons of Design Methods

Method	Data Required	Advantages	Disadvantages
Classical	c, φ, S_u, k, EI, f_{max}	Simple. Quick. No computer necessary.	Many assumptions made. Limited application.
Beam–Column	c, φ, S_u, k, y_a, y_p, EI, k_{anchor}, f_{max}	Better bending moment and reaction predictions. Multilayer soils. Relatively simple.	No seepage condition. Limited precision on movement. Requires computer and program.
Finite Element	c, φ, S_u, k, E_{soil}, EI, f_{max}, k_{anchor}	Simulates construction sequence. Good bending moment predictions. Good movement predictions. Unlimited geometry. Multilayer. Any loading. Simulate construction sequence.	Difficult and time consuming to use properly. Requires computer and program.

As compared to BEF analysis, the FEM of analysis can provide direct information of the ground movements outside and inside the excavation. It also can be used to model the soil structure interaction response of nearby structures to the excavation-induced ground movements. Another difference between the FEM and BEF methods is that variations in the soil stiffness (modulus) can have a greater effect on predicted loads and movements due to the inclusion of soil arching in the FEM model.

2.2.6. Correlations for different parameters

2.2.6.1. Poisson's Ratio

When investigating or predicting the deformation of the ground its properties are mostly described by Young's modulus (E) and Poisson's ratio (v). These properties are obtained mostly from the results of triaxial compression tests. When modulus is the ratio of the stress to strain or is obtained from the slope of deviator stress-axial strain curves then Poisson's ratio is the ratio of the radial strain to the axial strain. The modulus and Poisson's ratio is both nonlinear and stress-dependent. However, the range of V is relatively small as compared to the range of E. For elastic materials, Young's modulus and Poisson's ratio are interrelated uniquely with the shear modulus (G):

Poisson's ratio for isotropic elastic materials, the entire range of V is from 0 to 0.5. For dilatant soils that are inelastic, V may exceed 0.5. However, it should be remembered that the behavior is no longer elastic in this case. For undrained loading of saturated cohesive soil, no volume change occurs; therefore the v_u is equal to 0.5 by definition. For drained loading, volume changes occur and the drained Poisson's ratio varies with soil type and consistency.

Table 2.3 Typical Ranges of Poisson's Ratio (Carter, M. and Bentley, S. (1991))

Soil Type	Description	V'
Clay	Soft	0.35 – 0.40
	Medium	0.30 – 0.35
	stiff	0.20 – 0.30
Sand	Loose	0.15 – 0.25
	Medium	0.25 – 0.30
	dense	0.25 – 0.35

2.2.6.2. Modulus of Cohesive Soils

The response of cohesive soils to loading is time dependent. For initial quick loading conditions, the response is undrained. The excess pore water pressures developed will dissipate with time leading to consolidation. For undrained loading, the modulus of cohesive soils can be described by either the Young's modulus (E_u) or the shear modulus (G). The summarized ranges of undrained modulus are as follows

Table 2.4 Typical Ranges of Modulus for Clay (Carter, M. and Bentley, S. (1991))

Consistency	Normalized undrained modulus , E_u/P_a
Soft	15 to 40
Medium	40 to 80
Stiff	80 to 200

2.2.6.3. Modulus for Cohesion less soils

Cohesion less soil doesn't exhibit significant time dependency to loading caused by excess pore water stress dissipation. Therefore the modulus under undrained loading conditions exists briefly.

Table 2.5 Typical Values of modulus for cohesion less soils (Carter. M and Bentley, S. (1991))

Consistency	Normalized elastic modulus, E_d/P_a	
	Typical	Driven piles
Loose	100 to 200	275 to 550
Medium	200 to 500	550 to 700
Dense	500 to 1000	700 to 1100

2.3. Empirical Methods of Analysis

Based on field experience, empirical or semi-empirical procedures for estimating apparent pressure diagrams may be justified. Several commonly used empirical methods in engineering practice are reviewed below (Prof. V. N. S. Murthy, 958).

2.3.1. Moormann's Database (2004)

Moormann (2004) had carried out extensive empirical studies by taking 530 case histories of retaining wall and ground movement due to excavation in soft soil (Cu < 75kpa) into account. It is concluded that the maximum horizontal wall displacement (δ_{Hm}) lie between 0.5%H and 1.0%H, on average at 0.87%H (Figure 2.5). The location of maximum horizontal displacement is at 0.5H to 1.0H below the ground. The maximum vertical settlement at the ground surface behind a retaining wall (δ_{vm}) lies in the range of 0.1% H to 10% H, on average at 1.1% H. The settlement δ_{vm} occurs at a distance of less than 0.5 % H behind the wall, but there are cases in soft soil with this distance to be up to 2.0 H. The ratio δ_{vm}/δ_{hm} varies mainly between 0.5 and 1.0. The ground conditions and the excavation depth H are found to be the most influential parameter for deformation due to excavation.

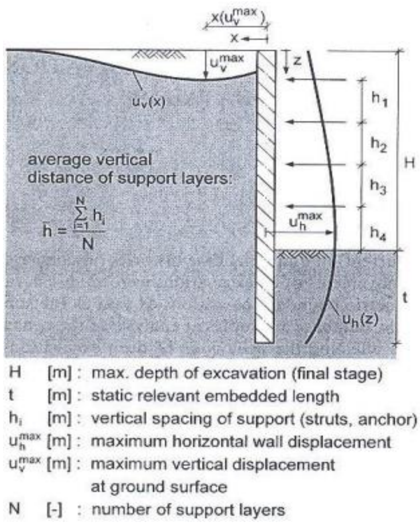


Figure 2.5 Definition of symbols by (Moormann (2004))

2.3.2. Clough and O'Rourke (1990)

Clough and O'Rourke (1990) presented the current state-of-the-art of empirical observations for estimating the lateral wall deflections and surface settlements in excavations. They also conduct a finite element parametric study on excavation in sands, stiff clays, and residual soils supported by soldier piles, sheet piles, diaphragm walls, soil nail walls, drilled pier walls, and soil cement walls. Figure 2.6 and Figure 2.7 were prepared to show the maximum lateral wall deflections (= 0.2%H), δ_{Hm} .vs. excavation depth, H; and the maximum surface settlements < 0.15%H, Δ_{max} .vs. excavation depth, H, respectively. The maximum lateral wall deflections and maximum surface settlements were usually less than 0.5%H.

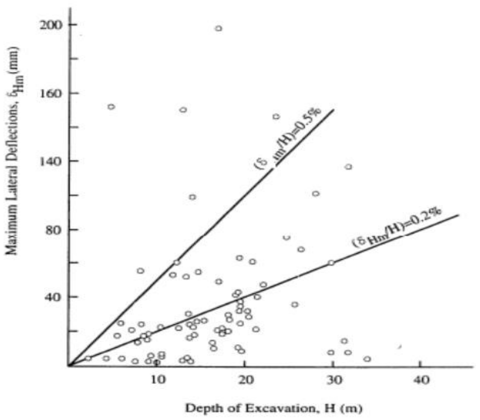


Figure 2.6 Maximum Lateral Wall Deflections in Sands, Stiff Clays, and Residual Soils (Clough and O'Rourke, 1990)

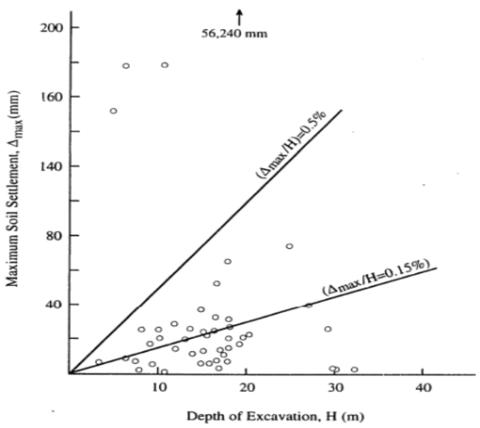


Figure 2.7 Maximum Surface Settlements in Sands, Stiff Clays, and Residual Soils (Clough and O'Rourke, 1990)

Clough and O'Rourke (1990) presented a dimensionless settlement profile as shown in Figure 2.8 to Figure 2.9 for sand, stiff to very hard clays, and soft to medium clays to further support these findings. Based on the figures below, the settlement influence zone is

3H for excavations in stiff to very hard clays and 2H for excavations in sands and soft to medium clays. They also occluded from Figure 2.8 to Figure 2.9 as follows:

- If the deep lateral wall deflections were the predominant mode, which would be the case with deep excavations in soft clay, the surface settlement would tend to have a trapezoidal distribution.
- If the cantilever mode predominated, which would be the case for excavations in sand and stiff to hard clay, the surface settlements would tend to be a triangular distribution.

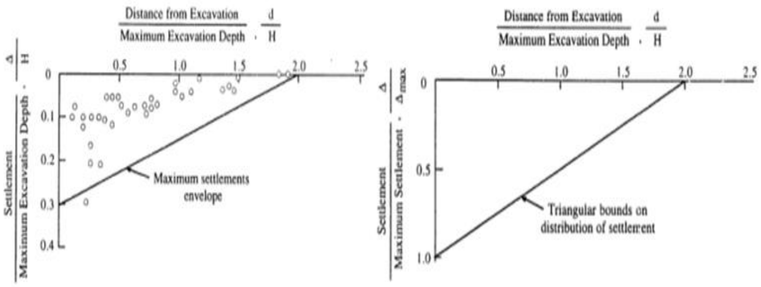


Figure 2.8 Surface settlements adjacent to excavations in sand (Clough & O’Rourke 1990)

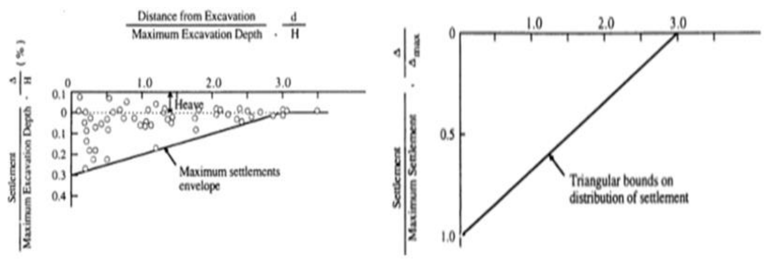


Figure 2.9 Surface settlements adjacent to excavations in stiff to hard clay
(Clough and O’Rourke, 1990)

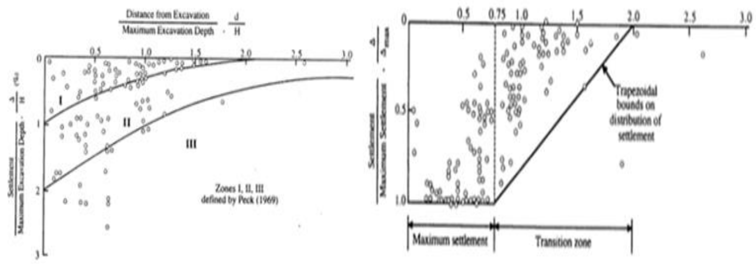


Figure 2.10 Surface settlements adjacent to excavations in soft to medium clay (Clough and O’Rourke, 1990)

2.3.3. Hsieh and Ou (2011)

Hsieh and Ou suggested that there are two types of settlement profiles caused by excavations. One is spandrel type and the other is concave type. In the first one, the spandrel one, the maximum settlement occurs very close to the wall and in the concave type the maximum settlement occurs at a distance away from the retaining wall as shown in figure 2.11. The spandrel type of settlement profile occurs if a large amount of wall deflection occurs at the first stage of excavation when cantilever conditions exist and the wall deflection is relatively small due to subsequent excavation. After the initial stages of excavation, additional cantilever wall deflection is restrained by installation of support as the excavation proceeds to deeper elevations. The concave settlement profile reflects the ground settlement profile that develops when the movements are more deep-seated.

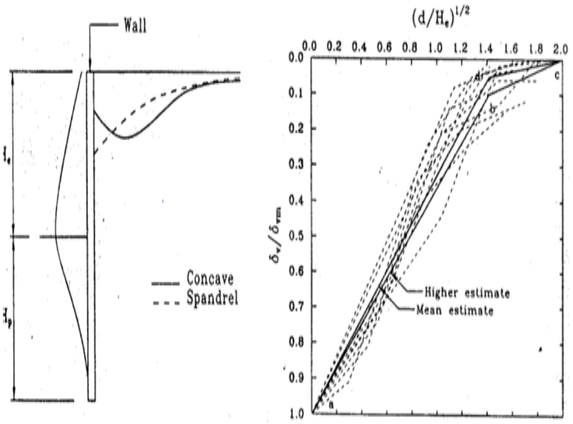


Figure 2.11 (a) Concave and spandrel type settlement profiles as suggested by Hsieh and Ou and Figure 2.11 (b) the settlement profile for a spandrel-type condition.

The data are presented as normalized settlement, $^{v}/_{v_m}$ where v_m is the maximum ground surface settlement, versus the square root of the distance from the edge of excavation divided by the excavation depth $(d/He)^{1/2}$. This relationship was based on 10 case histories from Taipei, Taiwan (Hsieh, P. and Ou, C). In figure 2.11(b) the broken lines show the 10 cases and the solid lines show the higher and the mean estimate.

2.3.4. Peck's Method (1969)

Peck (1969) presented pressure distribution diagrams on braced cuts. These diagrams are based on a wealth of information collected by actual measurements in the field. Peck (1969) developed the first empirical method to predict the wall movement based on the actual vertical ground deformation data collected from temporary braced sheet pile wall and soldier pile walls with tie back anchor. He mainly employed the results of case histories in London, Oslo and Houston and established the relation curves between the ground surface settlement and the distance from braced wall or the edge of excavation for three types of soil which are Soft clay, Stiff clay and Sand as shown in Figure 2.12.

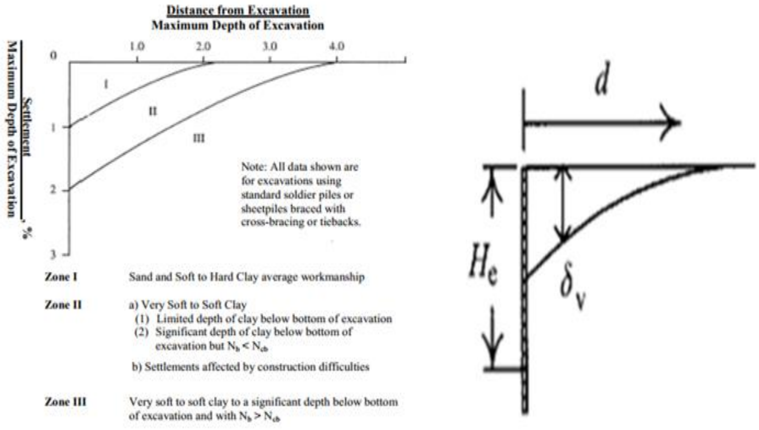


Figure 2.12 Summary of Settlements adjacent to open cuts in various soils as a function of distance from edge of excavation (Peck, 1969)

Figure 2.12 gives the settlements, divided by the maximum excavation depth H, and plotted against the distance from the walls also divided by excavation depth H. According to soil types three categories of behavior were defined:

1. Sand and soft to hard clay, (zone I)

2. Very soft to soft clay:

a. Limited depth of clay below the bottom of excavation.

b. Significant depth of clay below the bottom of excavation but $N_b < N_{cb}$, (zone II); and

3. Very soft to soft clay to a significant depth below the bottom of excavation and with $N_b > N_{cb}$ (zone III).

- Maximum horizontal deflection of sheet piles, $\delta_{H(max)}$

  For 40% of excavation in soft clay, $0.5\% \leq \delta_{H(max)} \leq 1\%$

  The average value of $\delta_{H(max)}/H$ is about 0.87%.

  In stiff clays, the average value of $\delta_{H(max)}/H$ is about 0.25%.

  In non-cohesive soils, the average value of $\delta_{H(max)}/H$ is about 0.27%.

  Location of $\delta_{H(max)}$ that is z' (figure 2.2)

For deep excavation of soft to stiff cohesive soils, z'/H is about 0.5 to 1.0.

2.3.5. Bowles's method (1986)

Bowles (1986) proposed a method for estimating the spandrel-type settlement profile induced by excavation. Bowles is one of the researchers who suggested a procedure to estimate braced excavation-induced ground surface settlements which is the geotechnical problem of braced deep excavation.

The steps are given as follows

1. Lateral wall displacement is estimated
2. Volume of lateral movement of soil mass is calculated
3. The influence zone (D) using the method suggested by Caspe (1966) is adopted

$D = (H_e + H_d) \tan (45 - \phi'/2)$

Where:-

H_e is the final excavation depth, ϕ' is the internal frictional angle of soil.

For cohesive soil, H_d = B, where B = width of excavation.

For cohesion less soil $H_d = 0.5B \tan (45 + \phi'/2)$

4. By assuming that maximum ground settlement occurs at the wall, maximum ground Settlement can be estimated by the following

$\delta_{vm} = 4Vs/D$

5. The settlement curve is assumed to be parabolic. The settlement (δ_v) at a distance (d) from the supported wall can be calculated as.

$\delta_v = \delta_{vm}(l_x/D)^2$

Where:-

D & l_x are the distance from the wall

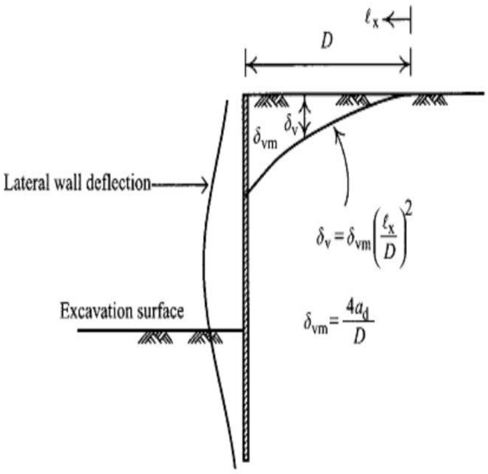


Figure 2.13 Bowles's method for estimating ground surface settlement (Bowles (1986))

2.4. Numerical studies of deep excavations

Hoe, N. H. (2007) carried out a parametric study using finite element method to investigate the effects of soil stiffness, the initial coefficient of lateral earth pressure and rock socket length. He also investigated the suitability of constitutive models (Hardening Soil model and Mohr-Coulomb model) by comparison with field monitored measured data. From the study he concluded that wall deflection is very sensitive to change in soil stiffness and coefficient of lateral earth pressure. Through comparison with measured field monitored data, it was observed that Hardening Soil model can predict ground deformation more precisely than Mohr Coulomb model.

Balasubramaniam, A. S. et al (1994) analyzed the performance of six deep excavations with different support systems and construction methods in Bangkok sub soils. Parametric finite element studies of the effects of pre-loading, barrette pile and foundation pile and foundation pile installation, embedment depth, and surcharge are also presented. Analytical results agreed in general with the observed behavior and they concluded that the stiffness of the retaining wall and bracing element control deformations. They also found that diaphragm walls performed better (smaller movements) than SPW and wall embedment depth was a more significant Performance factor with SPWs than diaphragm walls.

Goh, A. T. C. (1994) performed parametric studies using the finite element method to assess the effects of the wall properties, depth of competent soil, excavation width, and wall embedment on deep excavation stability in clay. The parametric studies showed that thickness of the clay layer beneath the excavation, embedment depth of wall, and the stiffness of the wall are important factors affecting basal stability.

2.5. Field performance studies of deep excavations

Ou, C. Y. et al (1993) studied the characteristics of ground surface settlement during excavation by examining data from ten case histories of deep excavations in Tapei. Maximum lateral wall deflection often occurs near the bottom of an excavation. Upper limit on maximum ground settlement is the maximum lateral wall deflection; in general ground settlement is about 50% to 70% of the maximum wall deflection. Maximum wall deflections were usually between 0.2% and 0.5% of the excavation depth.

Garvin, R. and Boward, J. (1992) described the cut-and-cover construction of a five level underground parking structure in Pittsburgh. A diaphragm wall with tiebacks was used to support the 7 m to 8 m excavation. Slurry wall system performed well and permitted dewatering within the excavation within a minimum influence on groundwater levels outside excavation. Maximum lateral movements for the excavation were between 10 - 20 mm.

2.6. Summary

Braced deep excavation is a complex soil-structure interaction problem, and its performance is influenced by a number of factors such as soil conditions, the type of retaining structures, construction methods, and the workmanship. Several methods in the analysis of deep excavations and some correlations are reviewed in this section, e.g. some correlations, empirical and Numerical methods and field performance measurements.

The empirical methods provide some basic understanding of the behavior of deep excavations, but they have limited applications due to their simplicity. Field measurements reflect the real performance of deep excavations, and the field data are valuable to calibrate the numerical analyses. However, field measurements are expensive and take a long time to obtain the data, and the process is not repeatable. Moreover, field measurements can only record the data, and are not appropriate for predictions. Numerical modeling is an efficient tool to investigate the behavior of deep excavations, and can be used for predictions. However, for more reliable prediction purposes, numerical analyses need to consider appropriately both geotechnical and structural aspects such as the irregular geometries, detailed retaining structures, correct construction sequences, realistic material models, and reliable input parameters. But, it should be noted that it may not be practical to take in to account all these aspects in a single analysis due to the complexity of the problem, and engineers need to focus on the most important ones and make appropriate simplification.

3. MATERIALS AND METHODS

3.1. Parametric identification and modeling

The input data's needed for both Numerical analysis and Empirical methods are discussed with brief explanation in this part of the thesis. In this paper, variables that have practical importance and have significant influence in the performance of braced deep excavations are considered. The identified variables considered in the parametric studies are soil type, excavation depth, SP diameter, stiffness and embedment depth, strut and walling stiffness and vertical spacing. When one of the above parameters is varied, the rest are kept constant to determine the overall deformation and settlement properties. The issue of heave doesn't address in this study as PLAXIS is not include this parameter during analysis.

3.1.1. Soil parameter identification

The soil types take into account for the parametric study are silty clay, clayey silt, clayey silt with sand and silty sand at Mexico and Lagahar (Kirkos Sub City of Addis Ababa) area (see table 3.1-3.3). Ground water level is about 25 meters below existing ground level. Relevant parameters for the above soil types had taken from Radice Engineering Plc, Geotechnical investigation reports done for Noah Real Estate (2019) (-6+G+35) and for National Theatre (2015) (-3B+G+13) building at Mexico and Lagahar respectively.

The detail properties of the soil model are presented in Appendix A.1. The undrained soil condition for silty clay and clayey silt soil was adopted, whereas for clayey silt with sand and silty sand soil drained soil condition was adopted during analysis. These analyses are based on the assumption that there will be no drainage of water on the front side of the wall during excavation phase and the water table is below the base of excavation depth. Furthermore, soil parameters that have been taken as an input for FEM (PLAXIS 3D, version 2013) software was presented in tables bellow (Radice Engineering Plc).

Table 3.1 Silty Clay and Clayey Silt soil parameters for base model (own construction)

	Name	Silty Clay	Clayey Silt
General			
Material model	Model	Hardening Soil Model	Hardening Soil Model
Drainage type	Type	Undrained A	Undrained A
Unit wt. of soil above phreatic line(KN/m^3)	γ_{unsat}	16.04	17.07

Unit wt. of soil below phreatic line(KN/ m^3)	γ_{sat}	18.2	19.3
parameters			
Secant modulus(KN/ m^2)	E_{50}^{ref}	18755.6	16295.9
Oedometric stiffness (KN/ m^2)	E_{50}^{ref}	18755.6	1629.9
Unloading-reloading stiffness (KN/ m^2)	E_{ur}^{ref}	56266.8	48887.7
Power for stress level dependency of stiffness	m	0.5	0.5
Cohesion (KN/ m^2)	C'_{ref}	36	43
Friction angle of soil (^{o})	ф	34.5	32.97
Delatancy angle	ψ	0.0	0.0
Poisson's ratio	V'	0.24	0.22
Interface			
Interface strength	-	Manual	Manual
Interface reduction factors	R_{inter}	0.60	0.63
Automatic			
	K_o	0.434	0.456
	OCR	1.0	1.0
	POP	0.0	0.0

Table 3.2 Clayey Silt with Sand and Silty Sand soil parameters (own construction)

	Name	Clayey Silt with Sand	Silty Sand
General			
Material model	Model	Hardening Soil Model	Hardening Soil Model
Drainage type	Type	Drained	Drained
Unit wt. of soil above phreatic line(KN/ m^3)	γ_{unsat}	15.3	17.3
Unit wt. of soil below phreatic line(KN/ m^3)	γ_{sat}	17.6	19.8
parameters			
Secant modulus(KN/ m^2)	E_{50}^{ref}	18047.62	26406.25
Oedometric stiffness(KN/ m^2)	E_{oed}^{ref}	18047.62	26406.25
Unloading-reloading stiffness(KN/ m^2)	E_{ur}^{ref}	54142.86	79218.75
Power for stress level dependency of stiffness	m	0.5	0.5

Cohesion (KN/ m ^{2})	C'_{ref}	21	6
Friction angle of soil (^{o})	ϕ	37.94	40.9
Delatancy angle	ψ	3.5	3.7
Poisson's ratio	V'	0.28	0.30
Interface			
Interface strength	-	Manual	Manual
Interface reduction factors	R_{inter}	0.67	0.70
Automatic			
	K_o	0.3852	0.3453
	OCR	1.0	1.0
	POP	0.0	0.0

Table 3.3 Silty Clay, Clayey Silt, Clayey Silt with Sand and Silty Sand soil parameters for base model (own construction)

	Name	Silty Clay	Clayey Silt	Clayey Silt with Sand	Silty Sand
General					
Material model	Model	Hardening Soil Model	Hardening Soil Model	Hardening Soil Model	Hardening Soil Model
Drainage type	Type	Undrained A	Undrained A	Drained	Drained
Unit weight of soil (KN/ m ^{3})	γ_{unsat}	16.04	17.07	15.3	17.3
Unit weight of soil (KN/ m ^{3})	γ_{sat}	18.2	19.3	17.6	19.8
Parameters					
Secant modulus(KN/ m ^{2})	E_{50}^{ref}	18755.6	16295.9	18047.62	26406.25
Oedometric stiffness(KN/ m^2)	E_{oed}^{ref}	18755.6	1629.9	18047.62	26406.25
U-R stiffness(KN/ m ^{2})	E_{ur}^{ref}	56266.8	48887.7	54142.86	79218.75
Power	m	0.5	0.5	0.5	0.5
Cohesion (KN/ m ^{2})	C'_{ref}	36	43	21	6
Friction angle of soil (^{o})	ϕ	34.5	32.97	37.94	40.9
Delatancy angle	ψ	0.0	0.0	3.5	3.7

Poisson's ratio	V'	0.24	0.22	0.28	0.30
Interface					
Interface strength	-	Manual	Manual	Manual	Manual
Interface reduction factors	R_{inter}	0.60	0.63	0.67	0.70
Automatic					
	K_o	0.434	0.456	0.3852	0.3453
	OCR	1.0	1.0	1.0	1.0
	POP	0.0	0.0	0.0	0.0

Where: $E_{50}^{ref} = E_{oed}^{ref} = 3^* E_{ur}^{ref}$

3.1.2. Structural parameter identification

The main structural parameters include the SPW, struts, walling, anchors and grout bodies. Struts are spaced 5 m interval horizontally and 3 m interval vertically in order to observe the real construction condition. Those parameters are described in the following tables.

Table 3.4 First material properties for beam (own construction)

Parameter	Name	Strut	Waling	Unit
Cross-section area	A	0.008467	0.009782	m^2
Unit weight	γ	80.5	80.5	KN/m^3
Material behavior	Type	Linear	Linear	-
Young's modulus	E	$1.85*10^8$	$1.85*10^8$	KN/m^2
Moment of Inertia	I_3 I_2	$6.083*10^{-5}$ $6.083*10^{-5}$	$1.845*10^{-4}$ $4.05*10^{-4}$	m^4 m^4

Table 3.5 Second material properties for beam (own construction)

Parameter	Name	Strut	Waling	Unit
Cross-section area	A	0.008467	0.009782	m^2
Unit weight	γ	80.5	80.5	KN/m^3
Material behavior	Type	Linear	Linear	-
Young's modulus	E	$2.5*10^8$	$2.5*10^8$	KN/m^2
Moment of Inertia	I_3 I_2	$6.083*10^{-5}$ $6.083*10^{-5}$	$1.845*10^{-4}$ $4.05*10^{-4}$	m^4 m^4

Table 3.6 Third material properties for beam (own construction)

Parameter	Name	Strut	Waling	Unit
Cross-section area	A	0.008467	0.009782	m^2
Unit weight	γ	80.5	80.5	KN/m^3
Material behavior	Type	Linear	Linear	-
Young's modulus	E	$3.6*10^8$	$3.6*10^8$	KN/m^2
Moment of Inertia	I_3 I_2	$6.083*10^{-5}$ $6.083*10^{-5}$	$1.845*10^{-4}$ $4.05*10^{-4}$	m^4 m^4

Table 3.7 Material properties for anchors (own construction)

Parameters	Name	Node - to - node anchor	unit
Material type	Type	Elastic	-
Axial stiffness	EA	$6.9*10^5$	KN

Table 3.8 Material properties for the embedded pile (grout body) (own construction)

Parameters	Name	Grout	Unit
Young's modulus	E	$3.2*10^7$	KN/m^2
Unit weight	γ	24.5	KN/m^2
Pile type	-	Pre-defined	-
Pre-defined pile type	-	Massive circular pile	-
Diameter	Diameter	0.333	m
Skin friction distribution	type	Linear	-
Skin resistance at the top of embedded pile	$T_{top\ max}$	210	KN/m
Skin resistance at the bottom of embedded pile	$T_{bot\ max}$	0.0	KN/m
Base resistance	$F_{bot\ max}$	0.0	KN

Table 3.9 First material properties for sheet pile wall (own construction)

Parameters	Name	Sheet pile wall	Unit
Thickness	d	0.90	m
Unit weight	γ	4.56	KN/m^3
Type or behavior	Type	Linear, non-isotropic	-
Young's modulus	E_1 E_2	$1.66*10^7$ $7.4*10^5$	KN/m^2 KN/m^2

Poisson's ratio	V	0.0	-
Shear modulus	G_{12} G_{13} G_{23}	$7.4*10^5$ $1.58*10^6$ $4.02*10^5$	KN/m^2 KN/m^2 KN/m^2

Table 3.10 Second material properties for sheet pile wall (own construction)

Parameters	Name	Sheet pile wall	Unit
Thickness	d	0.90	m
Unit weight	γ	4.56	KN/m^3
Type or behavior	Type	Linear, non-isotropic	-
Young's modulus	E_1 E_2	$21.55*10^7$ $27.6*10^5$	KN/m^2 KN/m^2
Poisson's ratio	V	0.0	-
Shear modulus	G_{12} G_{13} G_{23}	$27.6*10^5$ $21.88*10^6$ $24.2*10^5$	KN/m^2 KN/m^2 KN/m^2

Table 3.11 Third material properties for sheet pile wall (own construction)

Parameters	Name	Sheet pile wall	Unit
Thickness	d	0.90	m
Unit weight	γ	4.56	KN/m^3
Type or behavior	Type	Linear, non-isotropic	-
Young's modulus	E_1 E_2	$41.5*10^7$ $47.8*10^5$	KN/m^2 KN/m^2
Poisson's ratio	V	0.0	-
Shear modulus	G_{12} G_{13} G_{23}	$47.8*10^5$ $41.8*10^6$ $44.5*10^5$	KN/m^2 KN/m^2 KN/m^2

Table 3.12 Depth of excavation for the base model (own construction)

Parameters	Unit	Dex1	Dex2	Dex3
Depth of excavation	m	-12	-16	-20

Table 3.13 Depth of embedment for the base model (own construction)

Parameters	Unit	Dem1	Dem2	Dem3
Depth of embedment	m	-4	-6.5	-9

3.1.3. Soil and structural modeling using PLAXIS 3D software

The required soil parameters had been used is organized and the typical 3D base model geometry is provided as shown in figure 3.1 - 3.4. The model is simulated as an elasto-plastic strain Hardening Cap Model. When the specified soil has both cohesive and cohesion less properties the drained and undrained material model is used.

Besides of this, sheet pile is modeled as elastic wide beam element in which the unloading of the ground during installation of sheet pile doesn't considered. The change in thickness of SP represents a change in both bending stiffness and shearing stiffness of sheet pile wall (Table 3.9 - 3.11). The tieback anchors were fixed to the SP which was inclined at 45° and have pre-stressed load of 200 KN. At the end of tieback anchors the grout body was also fixed which have no axial stiffness but having bending stiffness only, allows flexibility during load transfer from the anchor to the ground along its entire length (Table 3.8).

❖ **Identification of the Maximum Soil and Structural Model Size**

The maximum soil and structural model size is clearly observed in figure 3.1. In this figure it is tried to show the whole soil and structural model size with clear dimensions (90 m*60 m*40 m), the excavation part model size having the dimensions of (30 m*16 m*20 m) and the strut and walling provided for excavation. The horizontal spacing between struts is 5 m as strut and walling vertical spacing is 3 m for convenience of work (Radice Eng...Plc)

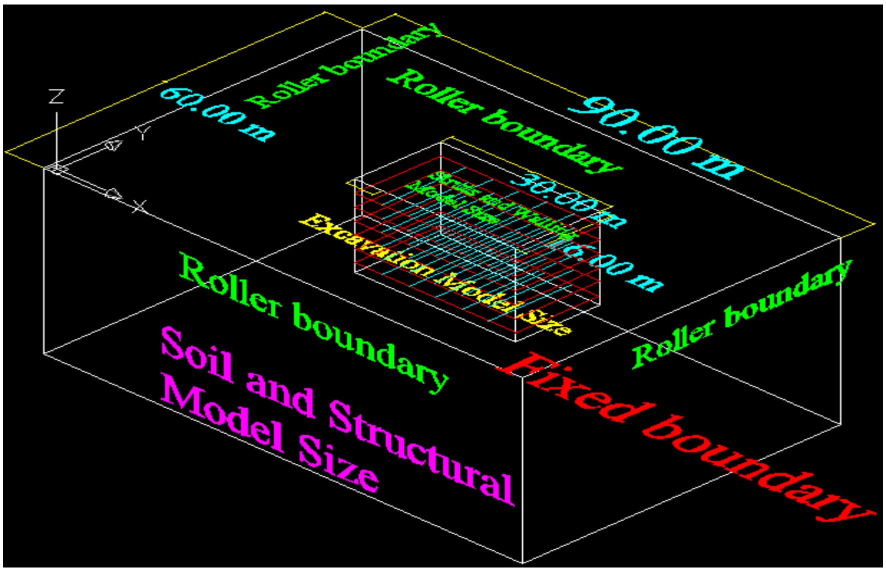


Figure 3.1 descriptions of soil and structural model size with estimated boundaries

The region to be analyzed in deep excavation is relatively small as compared with those of the surrounding medium. In FE analysis, the normal practice is to extend mesh to some extent away from the zone of interest and apply appropriate boundary conditions there. The boundary is sufficiently remote from excavation edge (around 30 m, 22 m and 20 m away in x*y*z direction). Four vertical boundaries provided are roller boundaries and the bottom is bounded by fixed boundaries. Figure 3.2 shows the general soil model has 16-noded solid elements and mesh has 48040 of triangular elements having 71226 nodes.

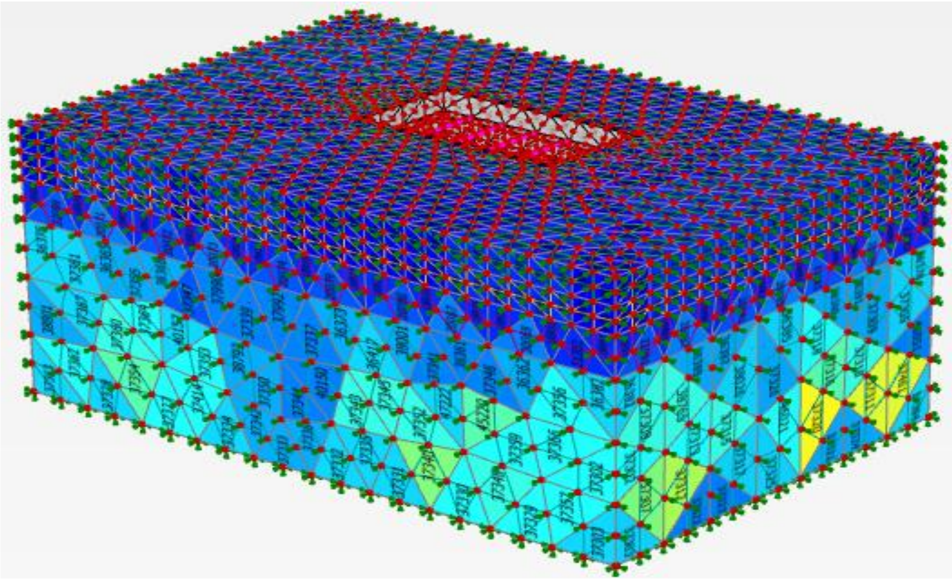


Figure 3.2 Finite element mesh and boundary conditions

In this study the coarse mesh (triangular element) was used, because it is somewhat difficult to complete calculation of results by using more fine mesh for software analysis.

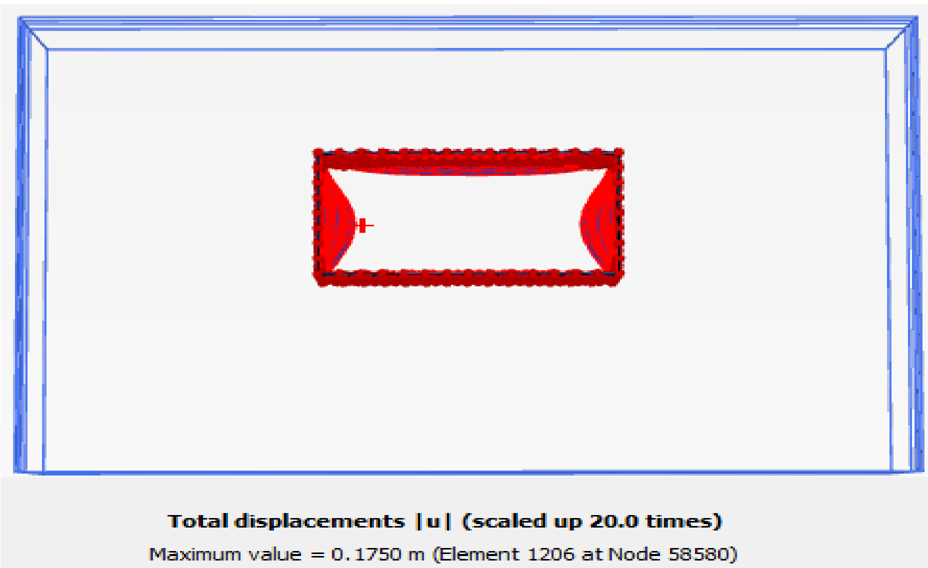


Figure 3.3 Maximum total displacement of the sheet pile wall due to braced deep excavation in soil type three

As shown in Fig.3.3 the largest wall deflection is concentrated at the wall central area close to the excavation formation level, while the deflection at the corner is smaller due to the corner effect. The shape of the SPW also affects the wall deflection.

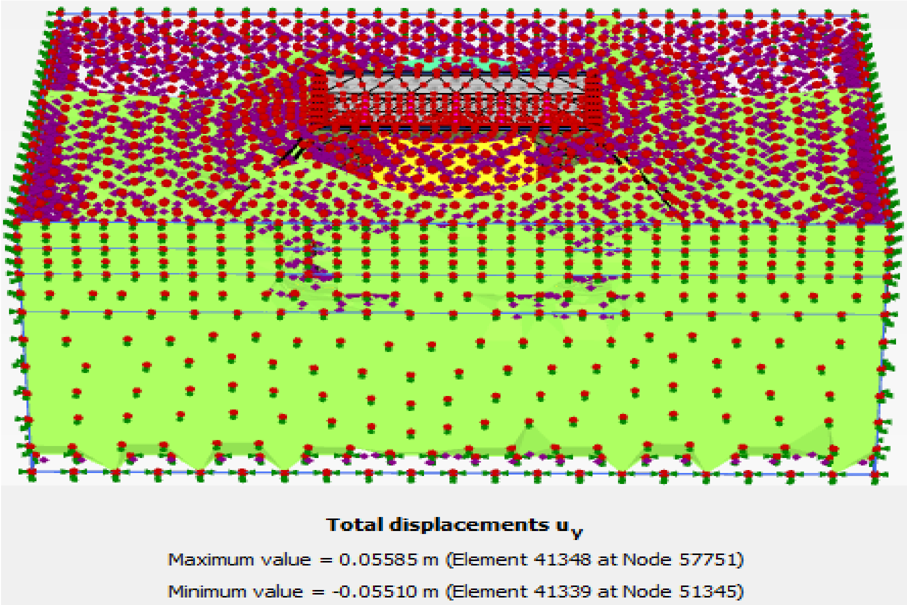


Figure 3.4 Maximum vertical ground settlements behind sheet pile wall that happen due to braced deep excavation in soil type three

The ground vertical displacement contour is displayed in Fig 3.4. It is clearly seen that the largest ground settlement is concentrated behind the wall center outside the excavation, while the settlement around the corner is smaller due to the corner effect. The longer the wall, the larger is the ground settlement area.

3.2. Methodology

3.2.1. Description of the Study Area

Addis Ababa is selected for study, because it is the Capital City of Ethiopia and also the most Commercial, Industrial, Government and non-Government institutional area that makes it very fast developing City in the Country. In order to counter balance its fast developing with Commercial, Industrial, institutional area and aesthetics there are a high increase of multi-story buildings, highway and rail way construction. For these reasons there are a lot of braced deep excavations have been happening on adjacent ground and structures at the City. Frankly speaking there is no researcher that studies the actual possibility of failure of adjacent ground and structures. By considering these all, this thesis was decided to study the analysis of the effects of lateral earth pressure distribution with Numerical and Empirical methods for braced deep excavations in Addis Ababa.

- **Location of the Site**

The site is located in Kirkos Sub city around Leghar, besides Yeha Building. Following is a table presenting the co-ordinates and elevation of the exploratory boreholes measured using Total Station and hand held GPS (UTM Adindan Datum; zone 37).

Table 3.14 Co-ordinates and elevation of Boreholes (Radice Engineering Plc)

Borehole ID	Easting	Northing	Elevation, m	Drilled Depth(m)
BH-1	472918	995924	2371	50
BH-2	472928	995882	2371	50
BH-3	472965	995928	2371	50
BH-4	472978	995888	2371	50

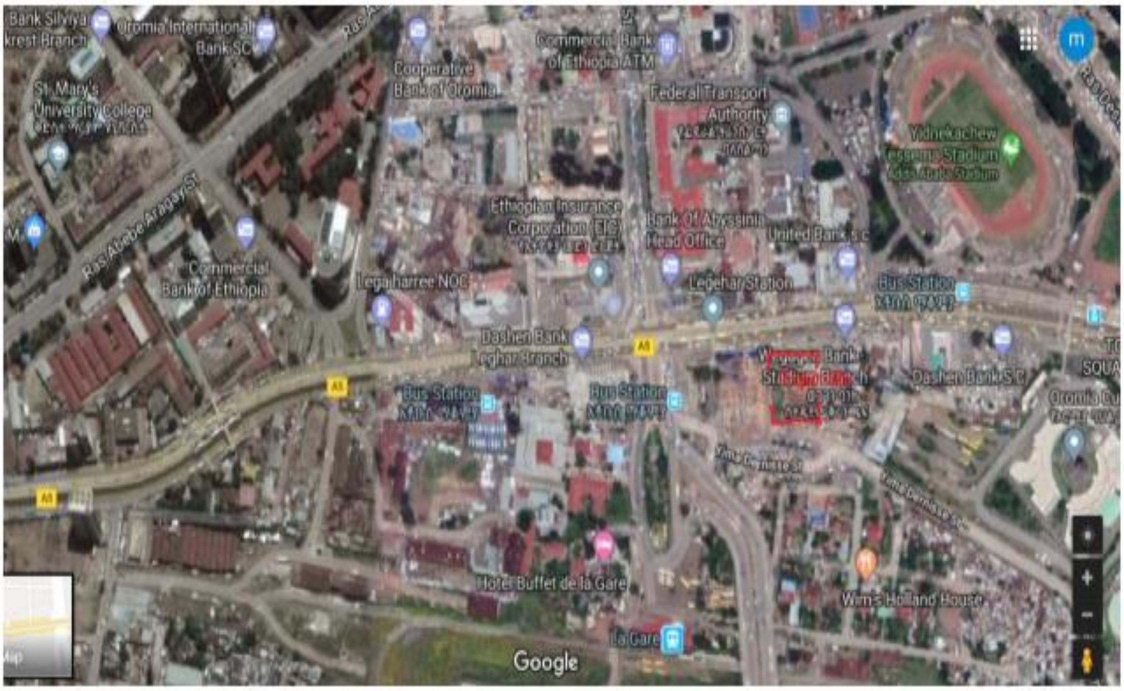


Figure 3.5 Google earth image of site location (Radice Engineering Plc)

- **Description of Geotechnical Layer**

Visual description of core sample, in-situ tests and laboratory test results of representative sample reveal the occurrence of the following geotechnical layers.

Layer 1: Reddish brown, highly plastic **silty clay** of stiff consistency

This reddish brown, highly plastic silty clay of stiff consistency is the first layer in all the boreholes. This soil formation is encountered up to a depth of 4.00m, 3.50m, 1.40m and 2.00m in BH-1, BH-2, BH-3, and BH-4 respectively. This layer has a slightly different yellowish color in BH-3 and BH-4.

Figure 3.6 Typical Layers 1 (BH-1, BH-3 & BH-4) (Radice Engineering Plc)

Layer 2: Light and Reddish brown, highly plastic **clayey silt** of firm consistency

This layer is the second formation in all boreholes. It is encountered with in a depth of 4.00m- 9.00m, 3.50m6.00m, 1.40m- 8.00m and 2.00m-11.00m in BH-1, BH-2, BH-3 and BH-4 respectively. The SPT results indicate that the soil has firm consistency. According to Unified Soil Classification System, the soil is classified as elastic SILT with sand (MH), except in BH-4 where it is classified as Sandy silt (ML) of medium plasticity.

Figure 3.7 Typical Layers 2 (BH-1 & BH-4) (Radice Engineering Plc)

Layer 3: Grayish to brown, highly plastic, stiff, **clayey silt with sand**

This layer is the third formation in all boreholes and the layer with the largest thickness. It is encountered with in a depth of 9.00m-30.00, 6.00m-21.85m, 8.00m-33.60m and 11.00m-29.00 in BH-1, BH-2, BH-3 and BH-4 respectively. In BH-1, this layer is encountered again within a depth of 35.00m-46.00m. This layer is encountered again within the depth of 27.10m-35.00m in BH-3.

Figure 3.8 Typical Layers 3 (BH-2 & BH-3) (Radice Engineering Plc)

Layer 4: Grayish, **silty sand** of high relative density This layer is found in BH-1, BH-3 and BH-4 within the depth of 30.00m-35.00m, 33.60m-43.70m and 29.00m35.00m respectively. According to Unified Soil Classification System, the soil is classified as silty SAND with gravel (SM).

Figure 3.9 Typical Layers 4 (BH-1 & BH-3) (Radice Engineering Plc)

Layer 5: Grayish, **sandy gravel with silt** of medium relative density and traces of boulders (created due to fracturing of rock layer underneath)

This layer is found in BH-2 and BH-4. It is encountered within the depth of 21.85m-27.10m and 35.00m37.00m in BH-2. The top one is more of clayey gravel while the one underneath is sandy gravel. In BH4 it is encountered within the depth of 35.00m- 42.00m. According to Unified Soil Classification System, the soil is classified as sandy gravel with silt (GM).

Figure 3.10 Typical Layers 5 (BH-2 & BH-4) (Radice Engineering Plc)

Layer 6: Dark grey, slightly fractured, **aphanitic strong basalt** of RQD value 0%-70%

This BASALT formation is encountered in BH-2, BH-3 and BH-4; it is also seen at the end of the explored depth in BH-1. In BH-2 it is encountered from a depth of 37.00m-48.80m. But the rock type from a depth of 44.80m-48.80m is ignimbrite with a yellowish brown color. In BH-3 it is encountered from a depth of 43.70m-45.40m and again from 48.00m-50.00m. In BH-4 it is encountered with in a depth of 42.00m- 50.00m.

Figure 3.11 Typical Layers 6 (BH-2, BH-3 & BH-4) (Radice Engineering Plc)

Layer 7: Reddish and brownish, **highly plastic clayey silt** of firm consistency

This formation is found in BH-2 and BH-3 with in the depth of 48.80m-50.00m and 45.40m-48.00m respectively. It is the last formation within the investigated depth in BH-2 and it is found sandwiched in between basalt layers in BH-3. According to Unified Soil Classification System, the soil is classified as highly plastic silt (MH) in BH-2 and as highly plastic clay with sand (CH) in BH-3

Figure 3.12 Typical Layers 7 (BH-2 & BH-3) (Radice Engineering Plc)

3.2.2. Methods of Data collection

- The most important soil parameters from field test (SPT) and laboratory tests of silty clay, clayey silt; clayey silt with sand including silty sand that are found in Kirkos Sub City of Addis Ababa was taken from Radice Engineering Plc.
- The appropriate literature to the study area was conducted and examined to establish the factors that influence sheet pile wall and for further information.
- Tie-back anchored sheet pile wall is provided as deep excavation support system which was implemented for National Theatre and Noah Real Estate building project.

3.2.3. Methods of Data Analysis

- ✓ First, the parameters of different soils were obtained directly from geotechnical investigation report taken from Radice Engineering Plc.
- ✓ Secondly the soil parameters that doesn't gained directly was obtained by correlation methods. Then, the representative properties of soil had been taken and used as an input in simplified soil model.
- ✓ Parametric identification such as change in soil type, diameter and stiffness of SPW, struts and walling stiffness, strut and walling vertical spacing, depth of excavation and embedment depth had been carried out and the FEM of analysis was proceed.
- ✓ The effects of lateral earth pressure distributions were examined for the dominant types of soils by some empirical methods of analysis.
- ✓ Depending on the results had been gained from both empirical and FE method, the comparison was done in terms of geotechnical engineering justifications.

4. RESULT AND DISCUSSION

4.1. PLAXIS-3D Software Analysis

Supporting structures are temporary structures that may or may not be used as permanent structures after its purpose is completed. Thus if they are used only as temporary retaining systems, then the soil retained could be modeled as undrained material since the time of construction is relatively short as compared to permanent structure construction.

PLAXIS has both undrained and drained options in its material property inputs. For undrained case, the effective soil parameters or the drained soil parameters are entered because, PLAXIS automatically adds bulk stiffness for the water and distinguishes between effective stresses and excess pore water pressure.

4.1.1. Parametric study

This section has presents a detail result of 6 parameters. The parametric study conducted includes a number of alternative arrangements of variables under consideration (Appendix-3). Accordingly, the effect of varying a parameter is demonstrated by obtaining the variation in total [$U_{x,max}$] of tieback anchored sheet pile wall and the ground, $U_{y,max}$ and maximum bending moments [BM] in the sheet pile wall.

The model used for parametric study was the same as the original model, except for certain parameters being changed in order to determine the influence of these specified parameters. Soil type, D_{exc} thickness, D_{emb} and stiffness of sheet pile, spacing of strut and walling are the variables considered in this study. When one of the above variables is varied, the rest are kept constant to determine the influence in various properties of retaining structure and supported soil.

- **Representation of Soil Types**

A. First case: when depth of excavation is 12 m, thus the soil parameters are discussed as below.

  - Soil type one: - Represents silty clay and clayey silt with -8 m and -20 m depth from ground level.
  - Soil type two: - Represents silty clay, clayey silt and clayey silt with sand with -4 m, -8 m and -20 m depth.
  - Soil type three: - Represents silty clay, clayey silt, clayey silt with sand and silty sand with -4 m, -8 m, -12 m and -20 m depth from ground.

B. Second case: When depth of excavation is 16 m.
   - ❖ Soil type one: - Represents silty clay and clayey silt with -12 m and -30 m depth from ground level respectively.
   - ❖ Soil type two: - Represents silty clay, clayey silt and clayey silt with sand with -6 m, -14 m and -30 m depth.
   - ❖ Soil type three: - Represents silty clay, clayey silt, clayey silt with sand and silty sand with -4 m, -8 m, -14 m and -30 m depth.

C. Third case: When depth of excavation is 20 m.
   - ❖ Soil type one: - Represents silty clay and clayey silt with -12 m and -40 m depth from ground level respectively.
   - ❖ Soil type two: - Represents silty clay, clayey silt and clayey silt with sand with -6 m, -10 m and -40 m depth.
   - ❖ Soil type three: - Represents silty clay, clayey silt, clayey silt with sand and silty sand with -4 m, -8 m, -14 m and -40 m depth.

This representation of stratified soil type by one general name is used to simplify a discussion and comparison of the result provided from various soil at a required depth. However this general name didn't use when the stratified soil material had been filling step by step in PLAXIS-3D for soil modeling, rather its original soil name is used in soil model. Thus the reader shall be understood from this discussion what a soil type is and why the representation is needed simply.

4.1.1.1. Effect of Change in Soil Type

In this section of study, the specific objective of this thesis focus on how change in soil type affects the performance of braced deep excavations supported by tieback anchored sheet pile wall for each of 12 m 16 m and 20 m depth of excavation. The result of those analyses is presented for each soil types supported by tieback anchored sheet pile wall with various thicknesses in three cases.

In all analyses conducted in this part of parametric study, the following parameters are kept the same (unchanged):

- The parameters of Soil types at all, i.e. Silty Clay, Clayey Silt and Clayey Silt with Sand and Silty Sand soils.
- Tieback anchor spacing (L_h = 4 m)
- Strut horizontal spacing (L_h = 5 m)

- Stiffness of structures (Sheet pile, Strut, Walling, Anchor and Grouting)
- Strut and Walling vertical spacing (L_v = 3 m)
- Embedment depth of SPW (D_{emb} = 4 m) and Construction sequences

Figure 4.1-4.55 Show the effect of change in Soil type, D_{exc} D_{emb}, stiffness of (SP, strut and walling) and spacing of strut and walling on the total ($U_{x,max}$) of SP and the ground, vertical settlement (U_y) and maximum BM in the SPW due to braced deep excavations.

❖ **First Case: When Depth of Excavation (D_{exc}) = 12 m**

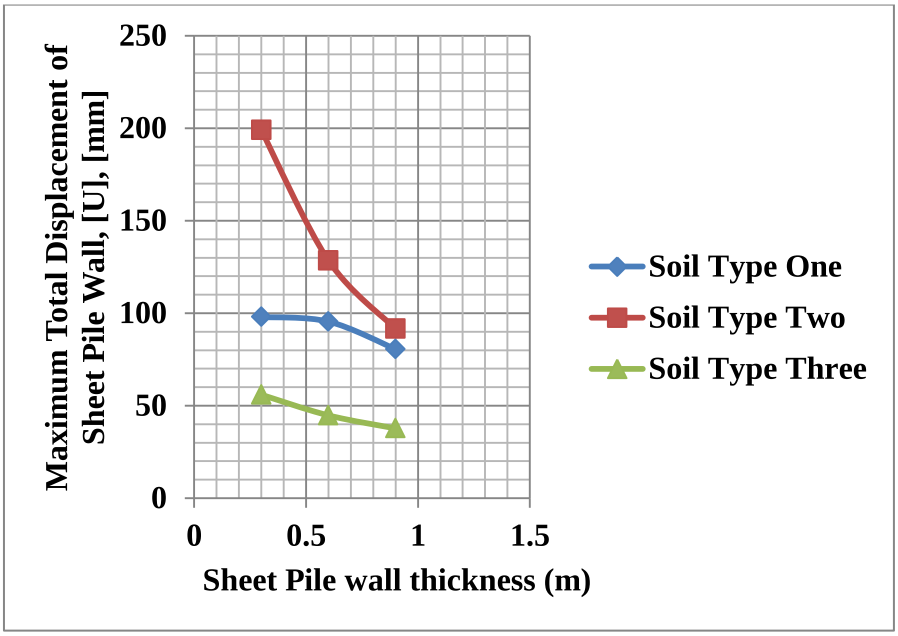


Figure 4.1 Maximum [U] of sheet pile wall with change in sheet pile wall thickness

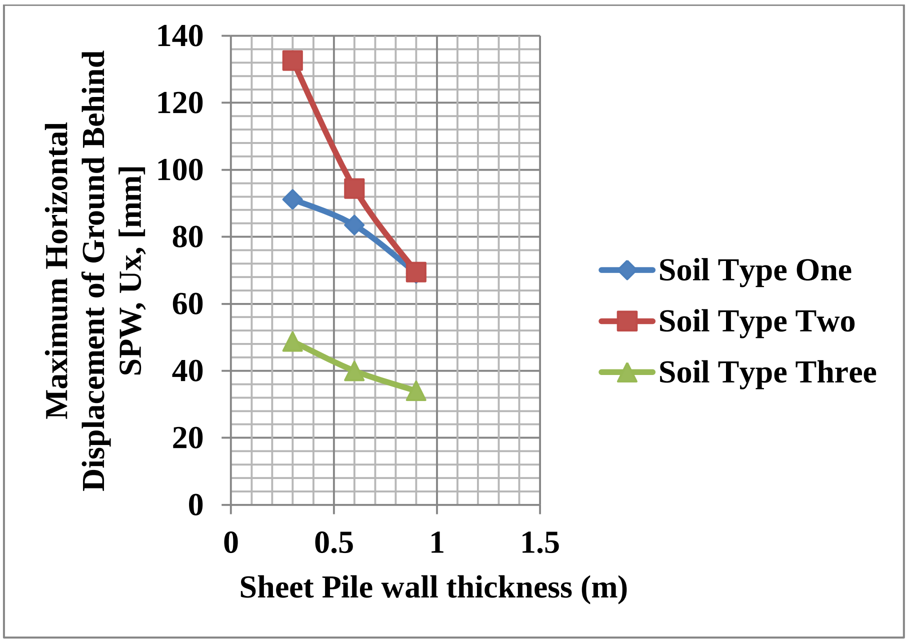


Figure 4.2 $U_{x,max}$ of the ground behind SPW with change in sheet pile wall thickness

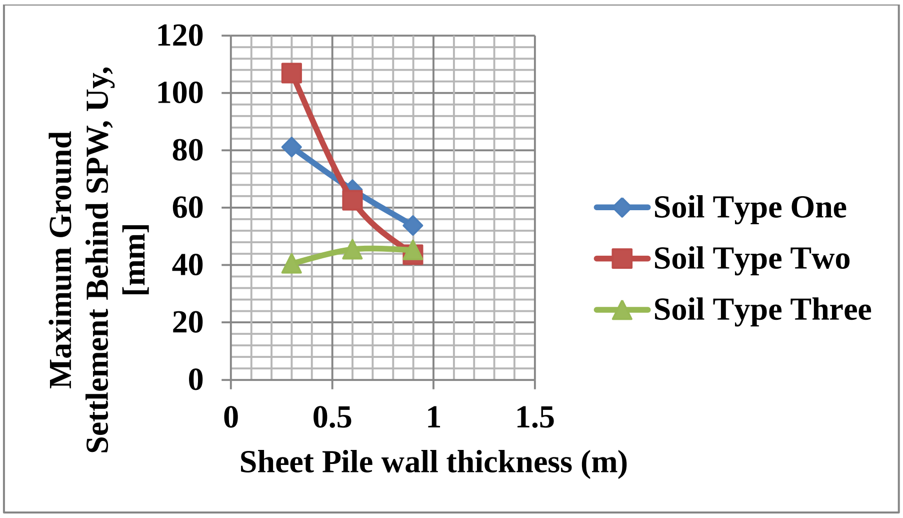


Figure 4.3 U_y behind the sheet pile wall with change in sheet pile wall thickness

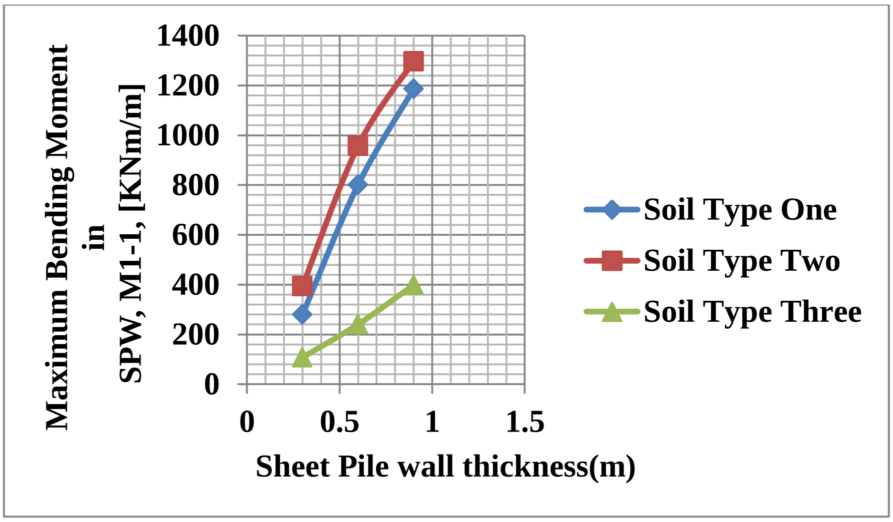


Figure 4.4 BM_{max} [M1-1] in the sheet pile wall with change in sheet pile wall thickness

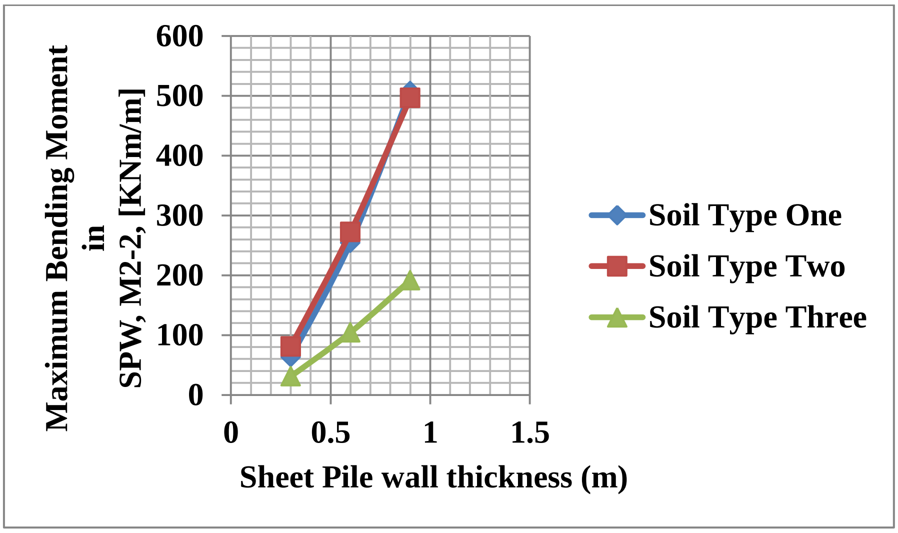


Figure 4.5 BM_{max} [M2-2] in the sheet pile wall with change in sheet pile wall thickness

The wall and ground deformation

The extensive decrease in sheet pile wall and the ground total horizontal displacement (U_x) around braced deep excavation is observed for soil type two as compared to soil type one and three that cause a medium decrement of U_x in sheet pile wall and the ground with increase in thickness of sheet pile wall.

The maximum bending moment (M1-1, M2-2) in the sheet pile wall is increase extensively for soil type one and two when compared to soil type three, because BM (M1-1, M2-2) in the SPW for soil type three is increase with medium variations as thickness of the SPW is increase. Actually the braced deep excavation in soil type one, two and three that supported by 0.3 m, 0.6 m and 0.9 m diameter SPW is produce the percentage increase of maximum BM (M1-1) in the SPW of 286.79%, 424.33% for soil type one, 242.87%, 329.02% for soil type two and 222.66%, 368.82% for soil type three as thickness of SPW is increase.

Ground settlement

Figure 4.3 Observes that soil type two has much more vertical ground settlement (U_y) than the others and its $U_{y,max}$ decreases significantly. Considering soil type one the $U_{y,max}$ is decrease by medium variation as the $U_{y,max}$ is increase with medium variation for braced deep excavation in soil type three with increase in thickness of SPW from original value.

In general, as it can be observed from the above figures soil type two provide the highest result as soil type three provide the minimum value that varies with increase in thickness of sheet pile wall. Thus, it is happen because of the stiffness of soil type three is higher than that of soil type two and one as a result of the general soil stratification with in depth.

Second Case: When Depth of Excavation (D_{exc}) = 16 m

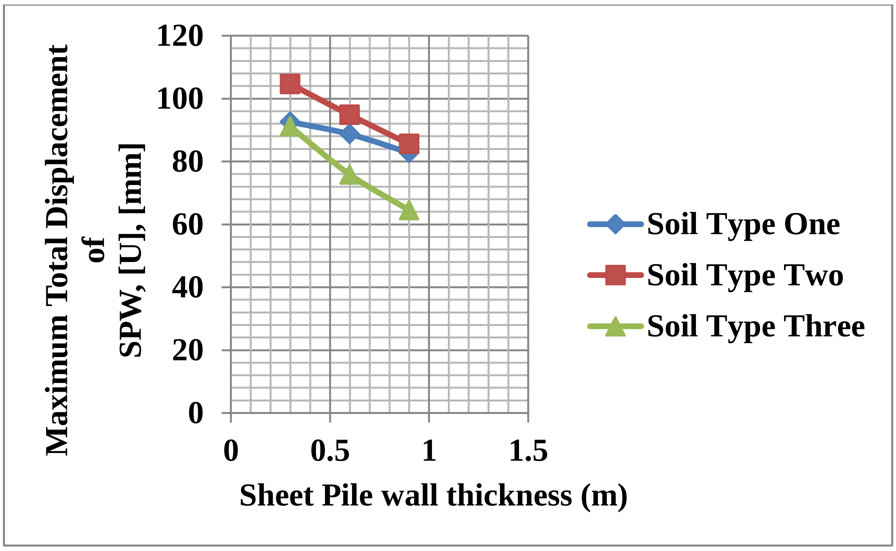


Figure 4.6 Maximum [U] of sheet pile wall with change in sheet pile wall thickness

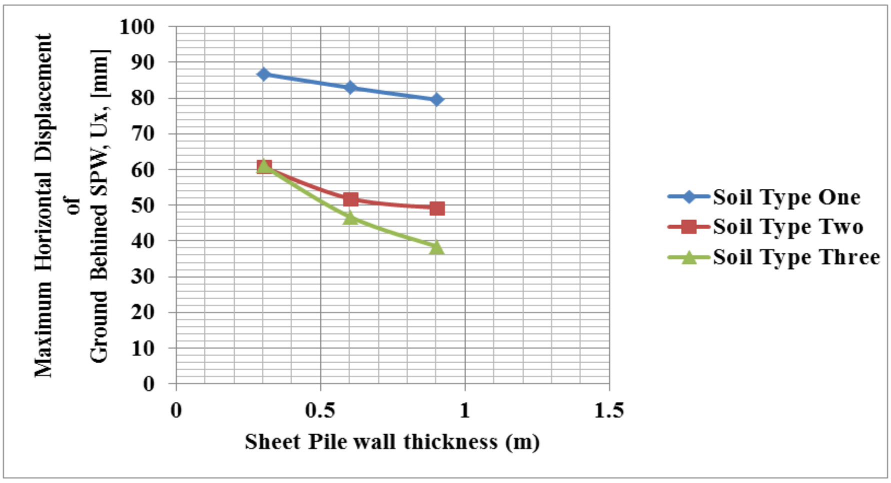


Figure 4.7 $U_{x,max}$ of the ground behind sheet pile wall with change in SPW thickness

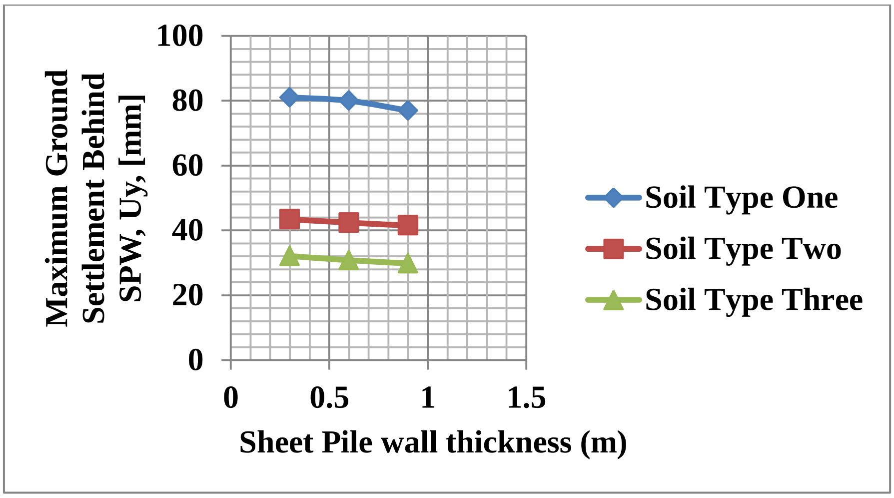


Figure 4.8 $U_{y,max}$ behind the sheet pile wall with change in sheet pile wall thickness

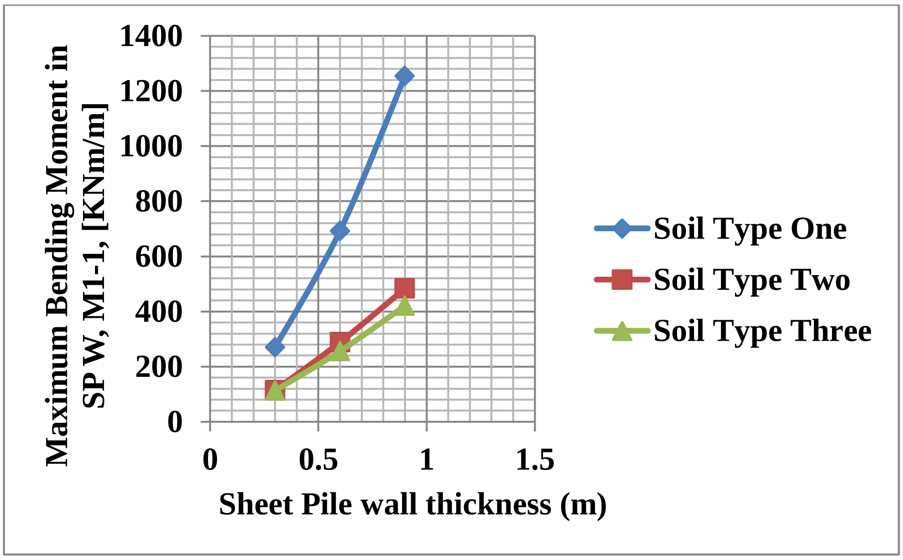


Figure 4.9 BM_{max} [M1-1] in the sheet pile wall with change in sheet pile wall thickness

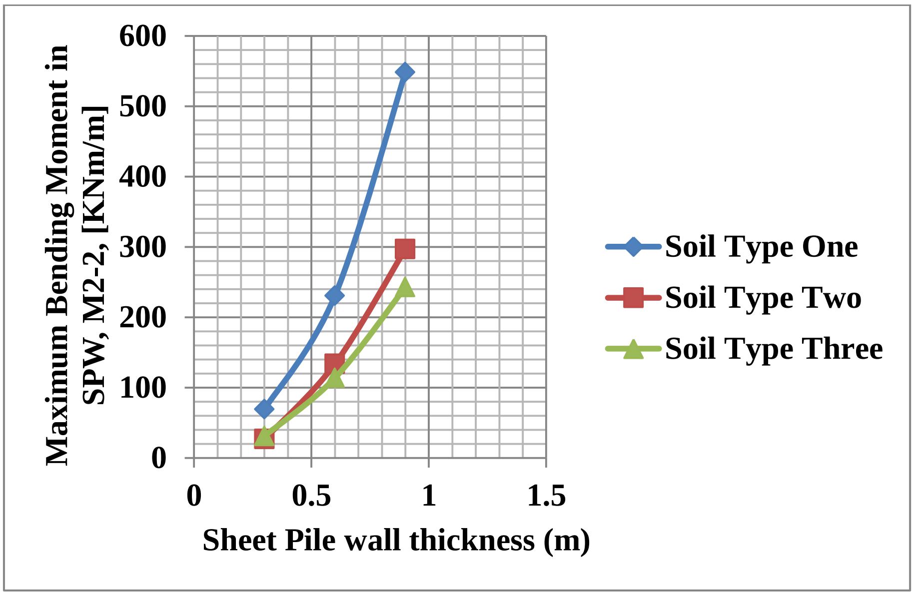


Figure 4.10 BM_{max} [M2-2] in the sheet pile wall with change in sheet pile wall thickness

The wall and ground deformation

Figure 4.6-7 Observes that the decrease of total U_x in sheet pile wall and the ground by small variation for soil type one and by medium variation for soil type three. The total U_x in sheet pile wall is decrease by medium variation as U_x of the ground is decrease very slightly for braced deep excavation in soil type two as thickness of SPW is increase.

The maximum bending moment (M1-1, M2-2) in the sheet pile wall is increase extensively for soil type one when compared to soil type two and three, since the maximum BM (M1-1, M2-2) in the SPW is increase with medium variations for soil type two and three.

Ground settlement

Figure 4.8 observes that soil type one has much more ground settlement than the others and $U_{y,max}$ is decrease by very small variation. Considering soil type two and three the maximum $U_{y,max}$ behind the sheet pile wall is decrease with very small variation as thickness of sheet pile wall is increase from the original value.

In this case, soil type one provide the highest result than soil type two that results from the effects of depth on the stiffness of stratified soil . Therefore, it is happen because of the stiffness of soil type one is smaller than that of soil type two and three.

- **Third Case: When Depth of Excavation (D_{exc}) = 20 m**

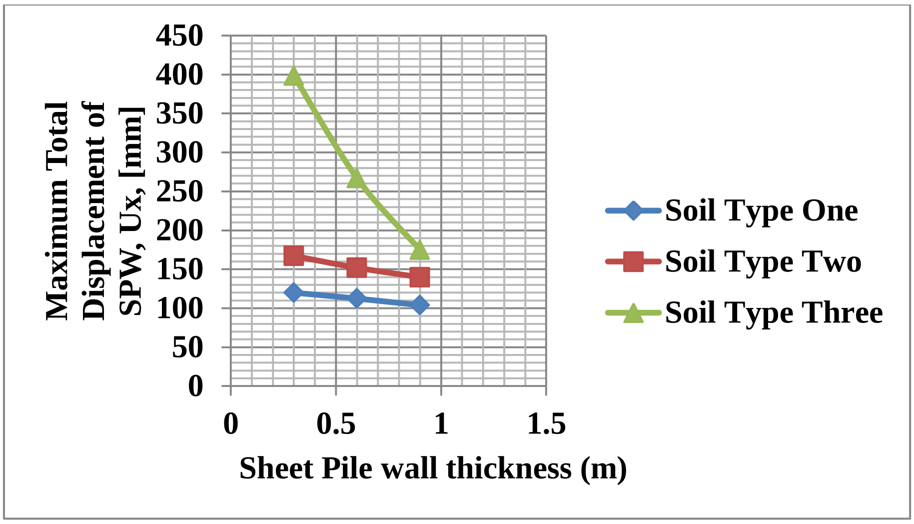


Figure 4.11 Maximum [U] of sheet pile wall with change in sheet pile wall thickness

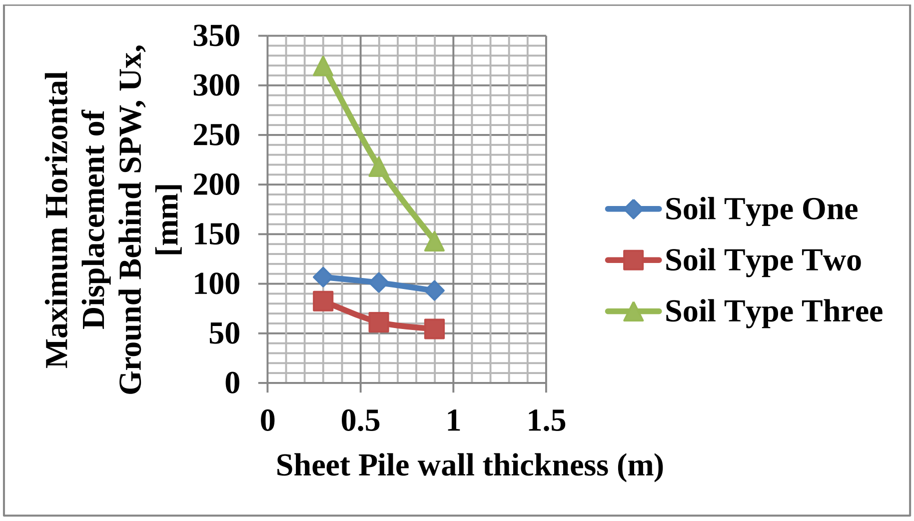


Figure 4.12 U_x of the ground behind sheet pile with change in sheet pile wall thickness

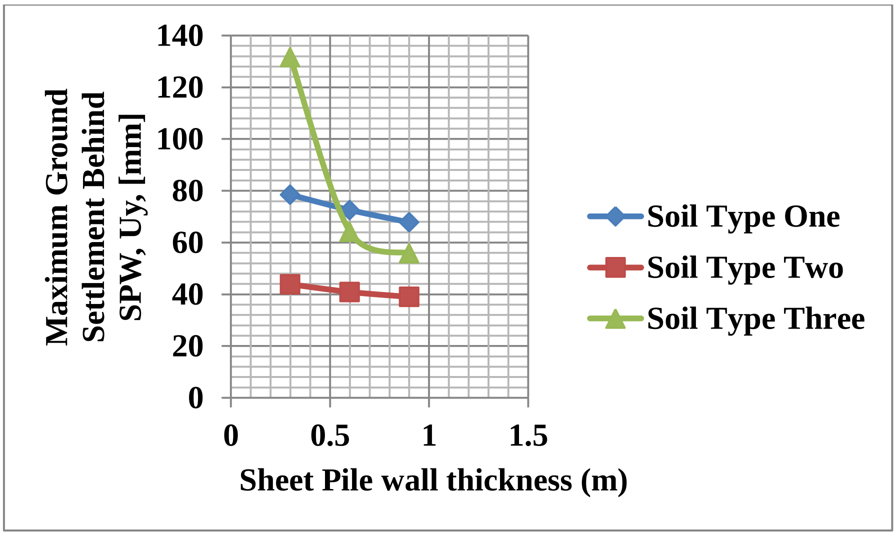


Figure 4.13 U_y behind the sheet pile wall with change in sheet pile wall thickness

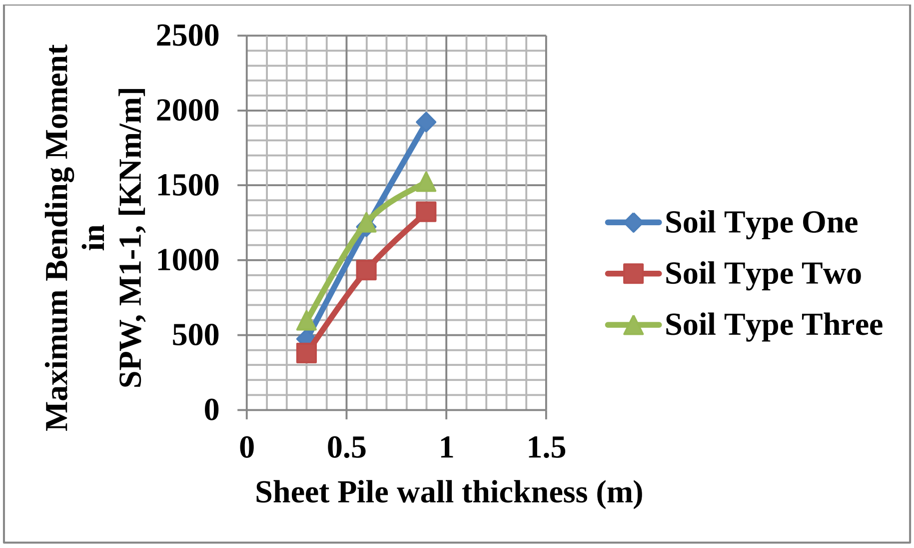


Figure 4.14 BM_{max} [M1-1] in the sheet pile wall with change in sheet pile wall thickness

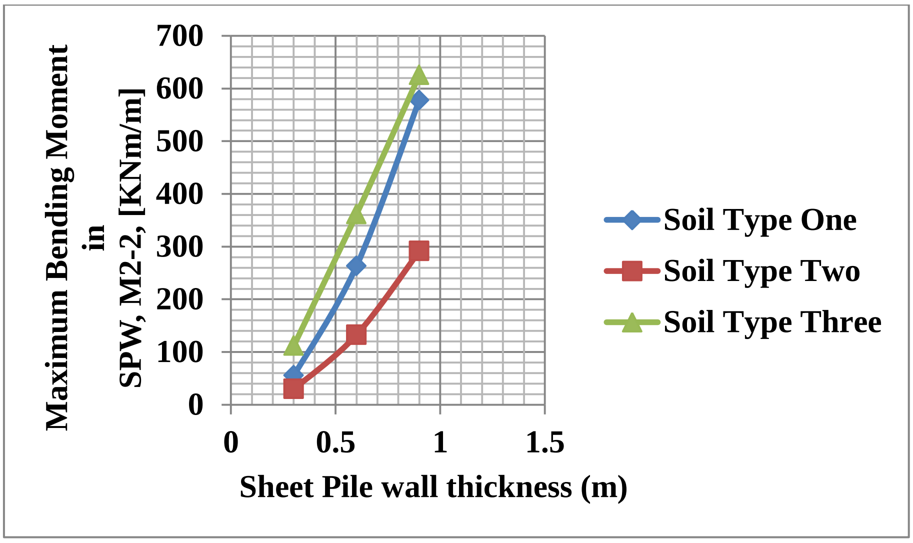


Figure 4.15 BM_{max} [M2-2] in the sheet pile wall with change in sheet pile wall thickness

The wall and ground deformation

The extensive decrease of the $U_{x,max}$ in SPW and the ground is observed around braced deep excavation in soil type three whereas soil type one and two cause the $U_{x,max}$ in SPW and the ground by very small variation. In general braced deep excavation in soil type one, two and three produce the percentage decrease in $U_{x,max}$ of (93.74%, 86.57%), (90.9%, 83.66%) and (67%, 43.91%) in the SPW as thickness of SPW is increase respectively.

The maximum bending moment (M1-1, M2-2) in the SPW is extensively increase as a result of braced deep excavation in soil type one and three when compared to soil type two, because the maximum BM (M1-1, M2-2) in the SPW around braced deep excavation in soil type two is increase with medium variations as thickness of SPW is increases.

Ground settlement

Figure 4.13 Notify that soil type three become much more vertical ground settlement (U_y) than others and extensively decreases $U_{y,max}$ with very small change, whereas in soil type one and two U_y is decrease with small variation as thickness of SPW is increase.

4.1.1.2. Effect of Change in Depth of Excavation

In this section of studies, the specific objective of this thesis is focus on how change in depth of excavation affects the performance of braced deep excavations supported by tieback anchored SPW having 12 m 16 m and 20 m depth of excavation and thickness of SPW (d = 0.9 m). The other parameters are same with that is written in section 4.1.1.1.

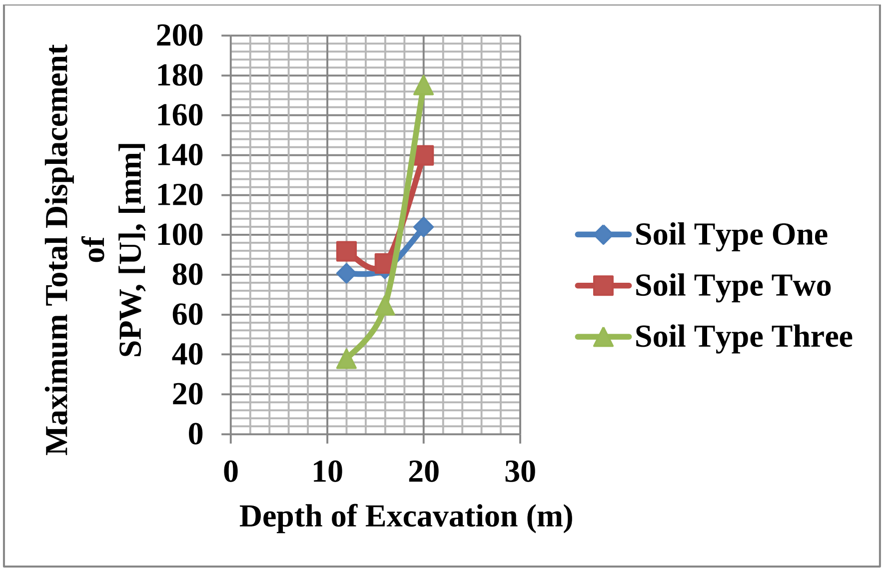


Figure 4.16 Maximum [U] of sheet pile wall with change in depth of excavations

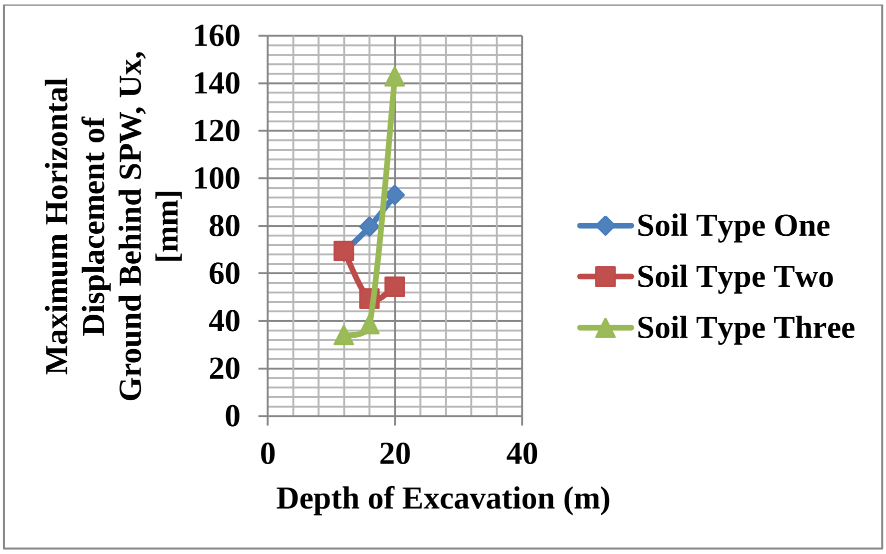


Figure 4.17 U_{xmax} of the ground behind sheet pile wall with change in depth of excavations

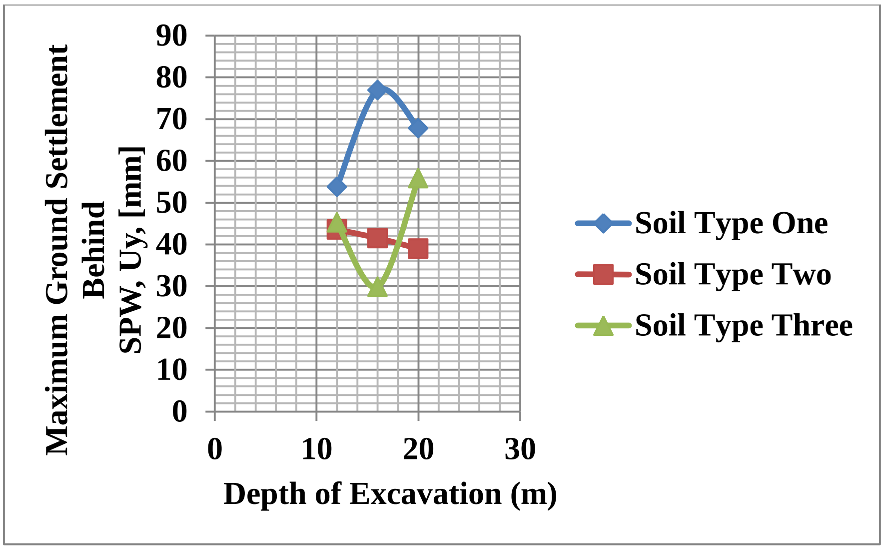


Figure 4.18 $U_{y,max}$ behind the sheet pile wall with change in depth of excavations

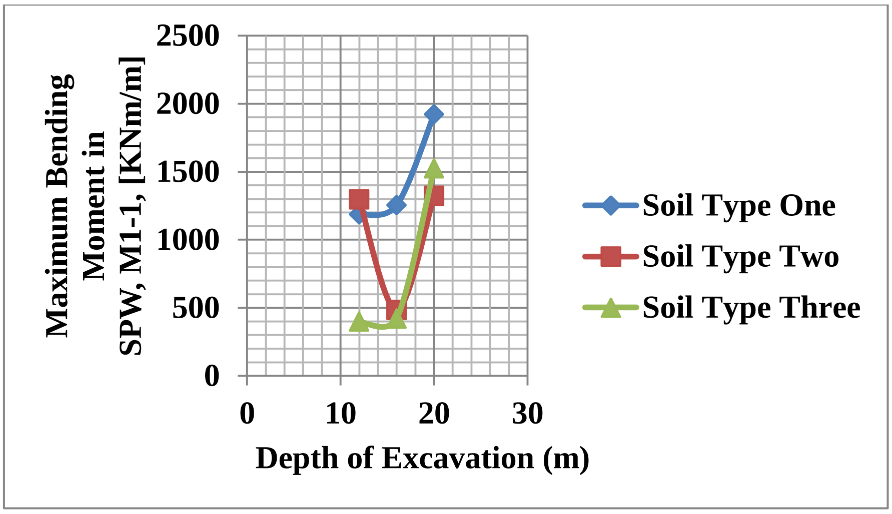


Figure 4.19 BM_{max} [M1-1] in the sheet pile wall with change in depth of excavations

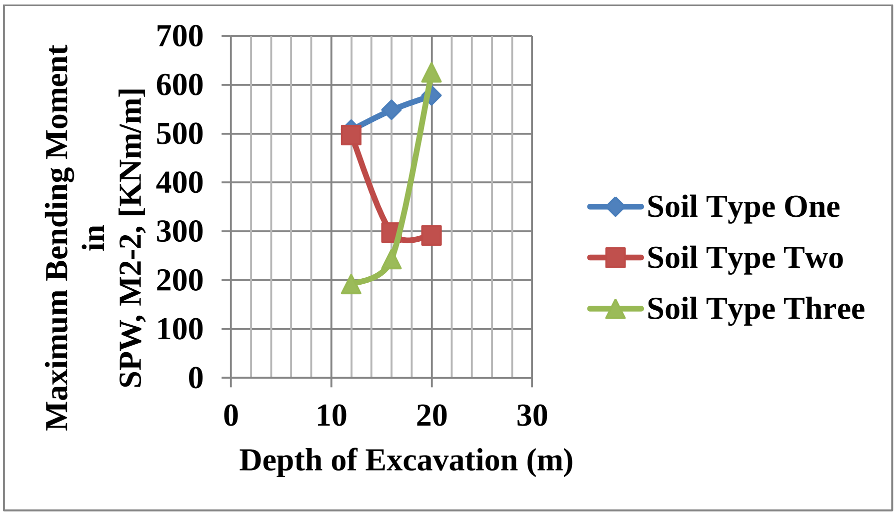


Figure 4.20 BM_{max} [M2-2] in the sheet pile wall with change in depth of excavations

The wall and ground deformation

Figure 4.16-17 Observes that soil type three has much more total $U_{x,max}$ in sheet pile and the ground at D_{exc} = 20 m that increase extensively, since soil type two provide U_x in SPW and the ground that decrease and then increase with medium variation as excavation depth is increase. When soil type one is considered the $U_{x,max}$ in sheet pile wall and the ground is increase by medium variation as depth of excavation is increase from original value.

The maximum BM (M1-1) in the SPW is increase slightly up to some degree and increase significantly, whereas maximum BM (M2-2) in the SPW is increase with medium change due to braced deep excavation in soil type one. When the maximum BM (M1-1) in the SPW decrease to some extent and increase extensively, on the other hand the maximum BM (M2-2) in the SPW highly decrease to some degree and increase slightly as a result of braced deep excavation in soil type two. However the maximum BM (M1-1) decrease by small variation and then increase significantly as maximum BM (M2-2) increase slightly and then increase extensively for soil type three with increase in depth of excavation.

Ground settlement

Figure 4.18 Observes that soil type one has much more vertical settlement than others and it is highly increases to some extent and then decrease with medium variation whereas the settlement behind sheet pile decrease slightly in soil type two. Finally as far as soil type three is concerned the maximum settlement behind sheet pile wall is decrease and increase significantly as depth of excavation increases from original value.

From the above figure, we can observes that as depth of excavation is increase the displacement of the ground and SPW, settlement and bending moments are almost increases. However the reverse of the graph with increase in depth is results from the stiffness of stratified soil found at that depth. Therefore, the linearity of the graph is provide from uniform soil rather than stratified soil.

4.1.1.3. Effect of Change in Stiffness of Sheet Pile Wall

This part concentrates on how the change in stiffness of sheet pile wall affects the performance of braced deep excavations. The result of this analysis is presented for each soil types retained by tieback anchored sheet pile wall having varying stiffness and thickness = 0.9 m and D_{exc} = 16 m of the original model. The other parameters are same with that is written in section 4.1.1.1.

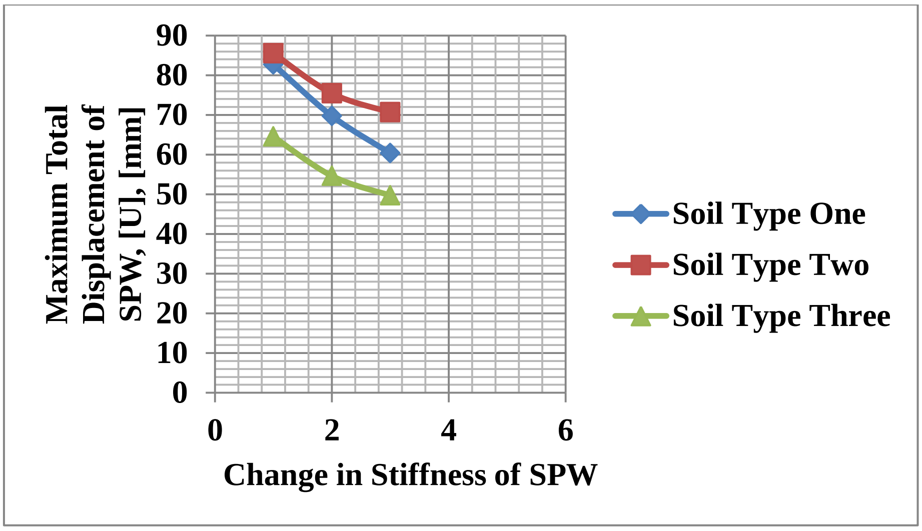


Figure 4.21 Maximum [U] of sheet pile wall with change in stiffness of sheet pile wall

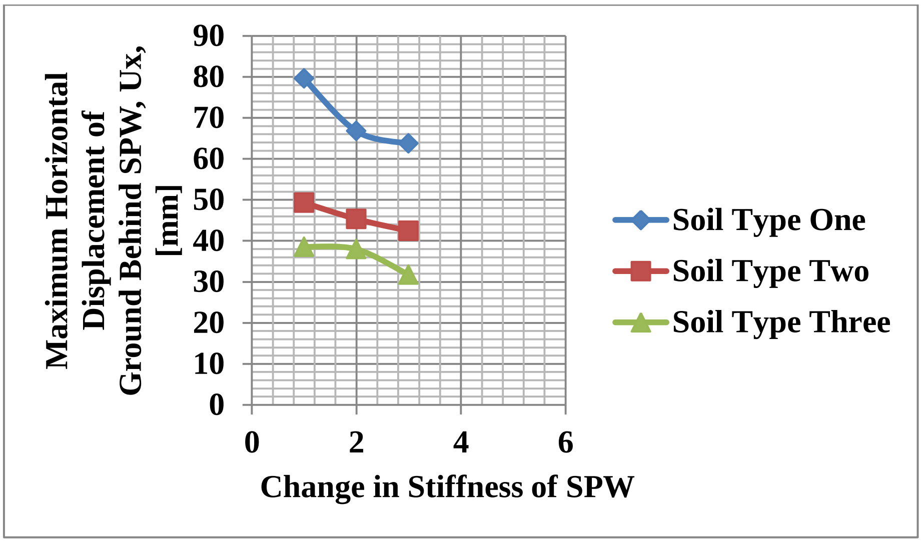


Figure 4.22 $U_{x,max}$ of the ground behind sheet pile wall with change in stiffness of SPW

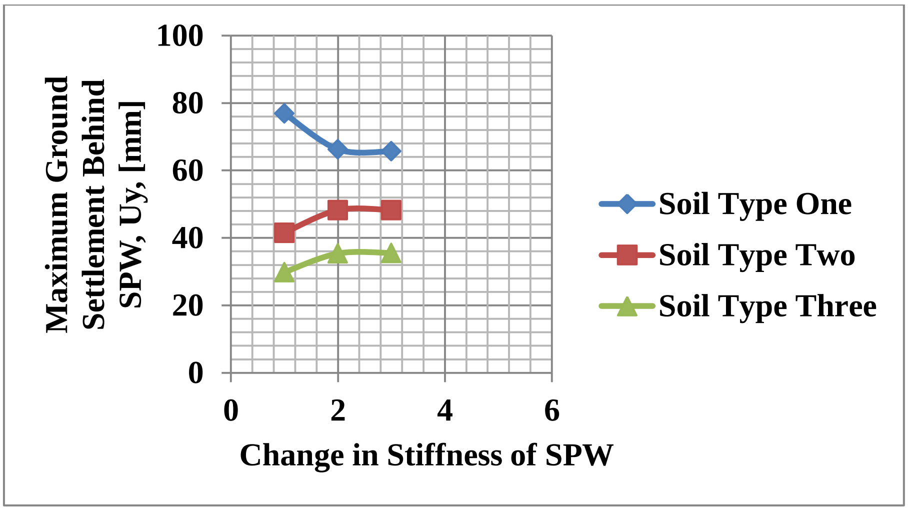


Figure 4.23 $U_{y,max}$ behind the sheet pile wall with change in stiffness of sheet pile wall

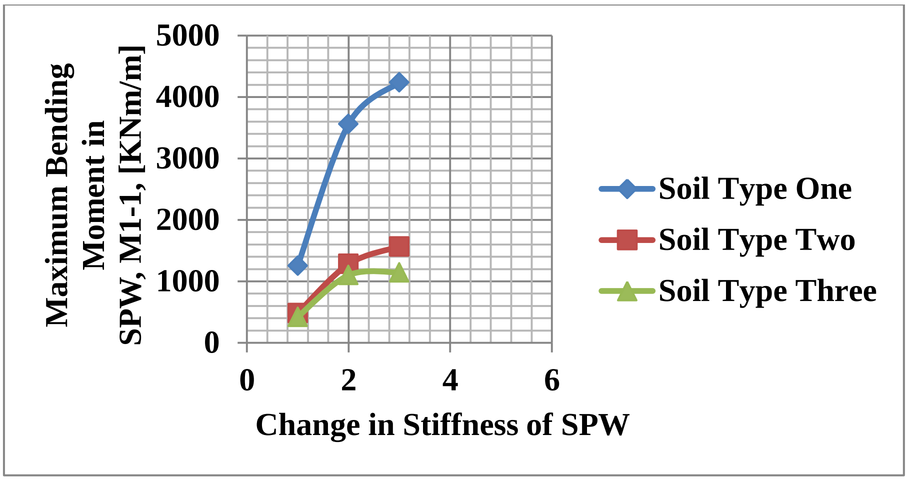


Figure 4.24 BM_{max} [M1-1] in the sheet pile wall with change in stiffness of sheet pile wall

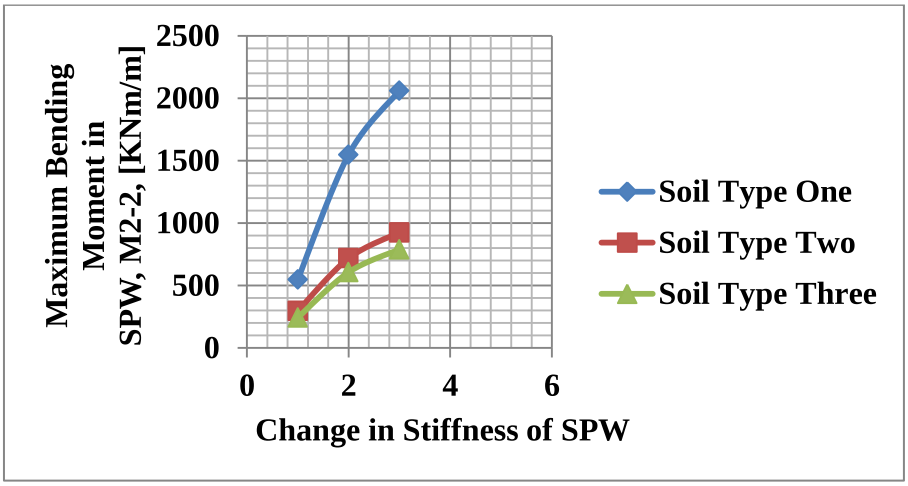


Figure 4.25 BM_{max} [M2-2] in the sheet pile wall with change in stiffness of sheet pile wall

The wall and ground deformation

Figure 4.21-22 Observes that soil type one, two and soil type three provides $U_{x,max}$ in the sheet pile wall that decrease by medium variation. Besides of this, as far as soil type one is concerned $U_{x,max}$ of the ground is decrease by medium variation as $U_{x,max}$ of the ground behind sheet pile wall is also decrease by small variation. However the $U_{x,max}$ of the ground due to braced deep excavation in soil type three is decrease by very small change and then decrease with medium variation.

Soil type one has very much more maximum bending moment (M1-1, M2-2) in the sheet pile wall as compared to others and provide significant increase of the maximum bending moment (M1-1, M2-2) in the sheet pile wall, whereas maximum bending moment (M1-1, M2-2) in the sheet pile wall is increase with medium variations due to braced deep excavation in soil type two and three as stiffness of SPW increases from the original value.

Ground settlement

Figure 4.23 Observe that soil type one has much more vertical ground settlement (U_y) than other soil types and it is decrease $U_{y,max}$ with medium variation up to some extent and then approach to constant. In addition, the maximum U_y behind the sheet pile increase with medium variation up to some degree and then approach to constant as a result of braced deep excavation in soil type two and three with increase in stiffness of SPW.

In general it is observed that as stiffness of SPW is increase the displacement of the ground and SPW and ground settlement is decrease. But, bending moment of sheet pile wall is increase. However, when bending moment is increase significantly the surface settlement is also increases by small increment as well.

4.1.1.4. Effect of Change in Embedment Depth of Sheet Pile Wall

The specific objective of this thesis in this section of study is concentrate on how the change in embedment depth of SPW affects the performance of braced deep excavations supported by tieback anchored SPW in two cases. The first case is for less stiff SPW with thickness = 0.3 m and the second case is for stiffer SPW having thickness = 0.9 m that studied for both 16 m and 20 m depth of excavation. The result of this analysis is presented for each soil types with varying depth of embedment from 4 m→ 6.5 m→ 9 m during simulations. The other parameters are same with that is written in section 4.1.1.1.

- **Case One: Less Stiff Sheet Pile Wall With Diameter (d = 0.3 m)**

- **When Depth of Excavation (D_{exc}) = 16 m**

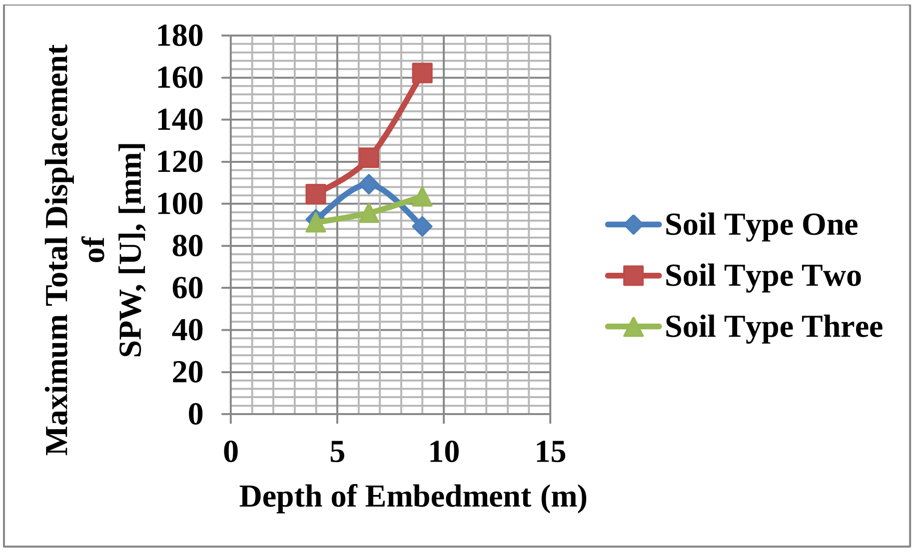


Figure 4.26 Maximum [U] of sheet pile wall with change in embedment depth

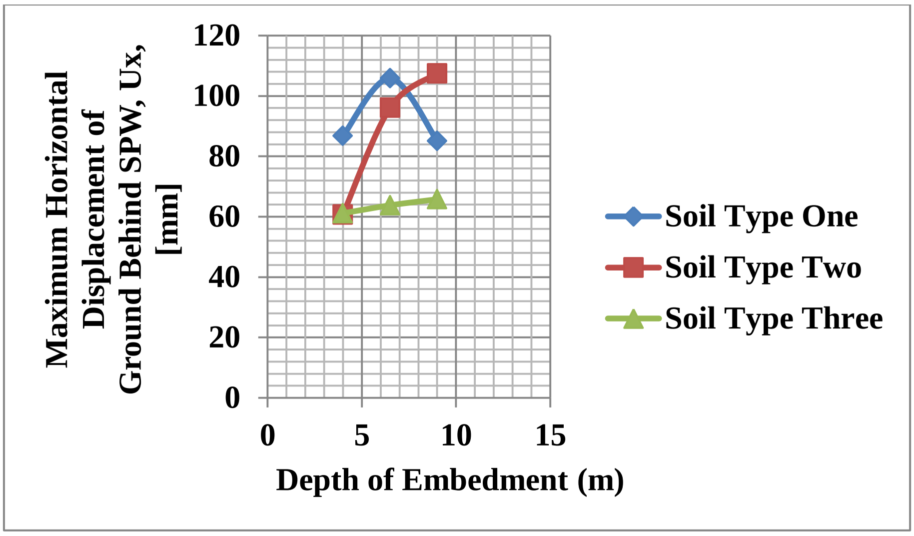


Figure 4.27 $U_{xmax,}$ of the ground behind sheet pile wall with change in embedment depth

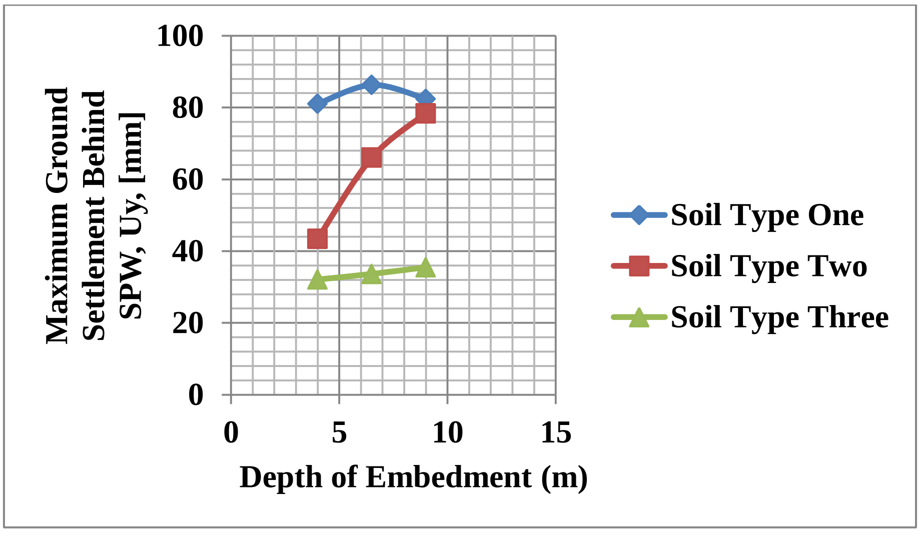


Figure 4.28 Maximum ground settlement behind the sheet pile wall with change in D_{emb}

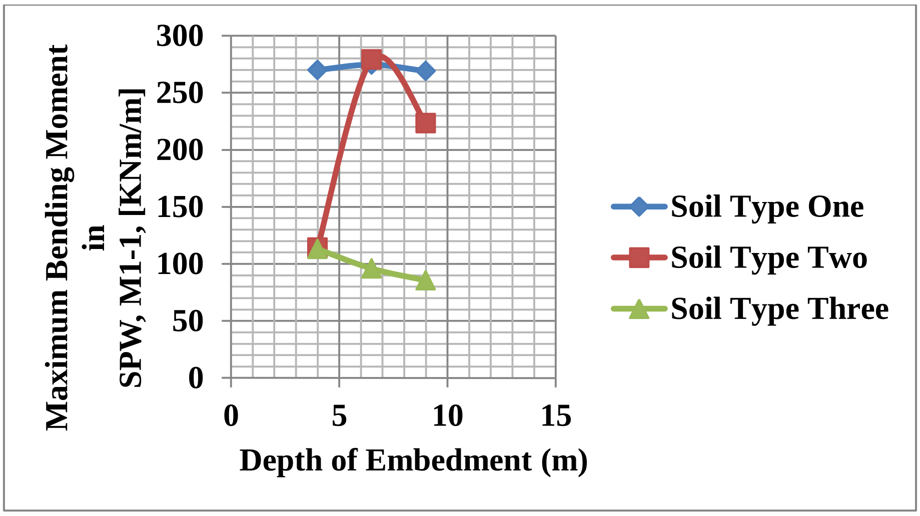


Figure 4.29 Maximum BM [M1-1] in the sheet pile wall with change in embedment depth

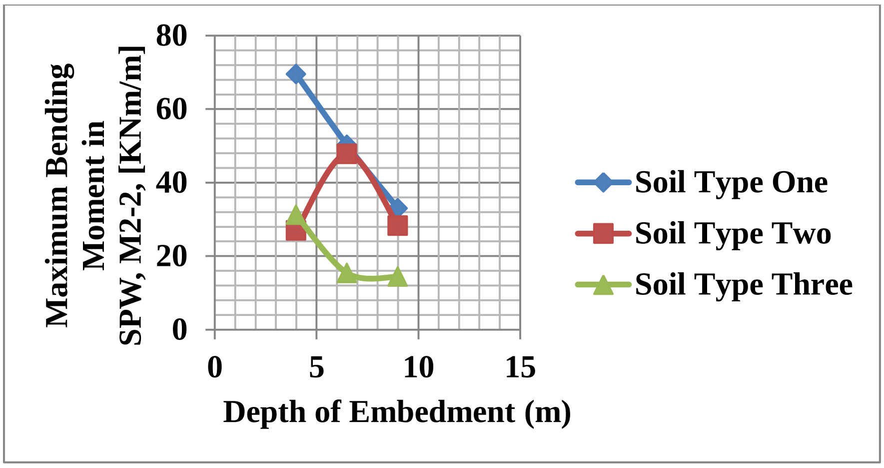


Figure 4.30 Maximum BM [M2-2] in the sheet pile wall with change in embedment depth

The wall and ground deformation

Figure 4.26-27 Observes that soil type one provides slight increase of total $U_{x,max}$ in sheet pile wall and the ground around braced deep excavation to some degree and then decrease it. Additionally, the $U_{x,max}$ in sheet pile wall and the ground due to braced deep excavation in soil type two is increase with medium variation and the $U_{x,max}$ in sheet pile wall and the ground for soil type three is increase by small change as embedment depth is increase.

Soil type one produce the increment and decrement of maximum bending moment (M1-1) in the sheet pile by very small variation, whereas maximum BM (M2-2) is significantly decrease for the same type of soil as depth of embedment is increase. As far as soil type two is concerned, the maximum BM (M1-1, M2-2) in the sheet pile wall is increase and decrease extensively. Even though it is so the maximum BM (M1-1) in the SPW is decrease by small variation as maximum BM (M2-2) is also decrease by medium variation.

Ground settlement

Figure 4.28 Observe that soil type one has more vertical $U_{y,max}$ than others and U_y is increase and decrease by very small variation as the $U_{y,max}$ of soil type two is significantly increase. However the vertical $U_{y,max}$ behind the SPW is increase by small variation in soil type three as embedment depth of sheet pile is increase from the original value.

All in all the figure above shows that increasing the embedment depth of SP only does not provide significant decrease in displacement of the ground and SP, settlement and bending moment using less stiff ground and SPW. But, the reverse of the graph is results from the stiffness of stratified soil found at that depth. Therefore, the linearity of the graph is provide from uniform soil rather than stratified soil.

➢ **When Depth of Excavation (D_{exc}) = 20 m**

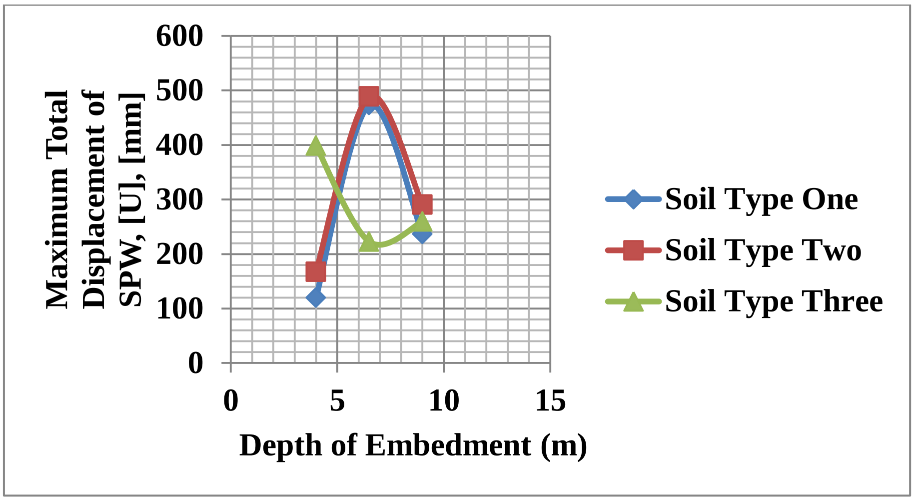


Figure 4.31 Maximum [U] of sheet pile wall with change in embedment depth

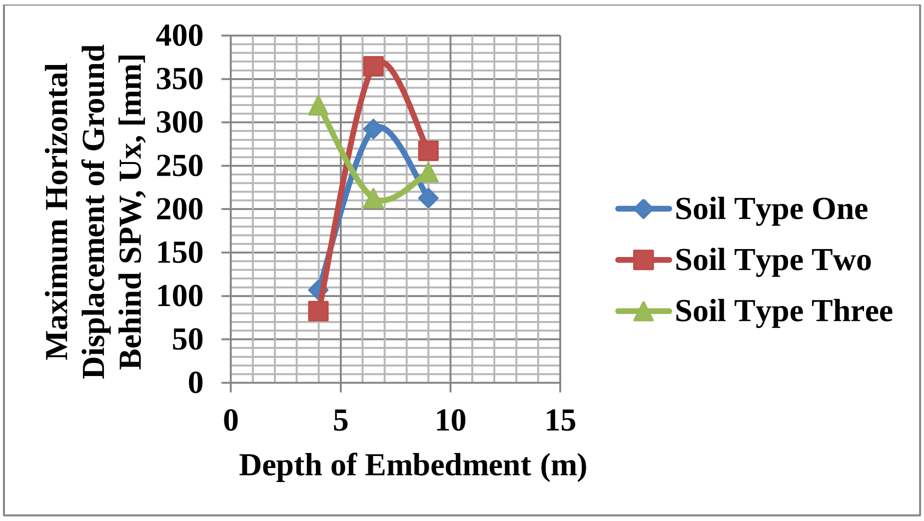


Figure 4.32 Maximum $U_{x\ max}$ of the ground behind SPW with change in embedment depth

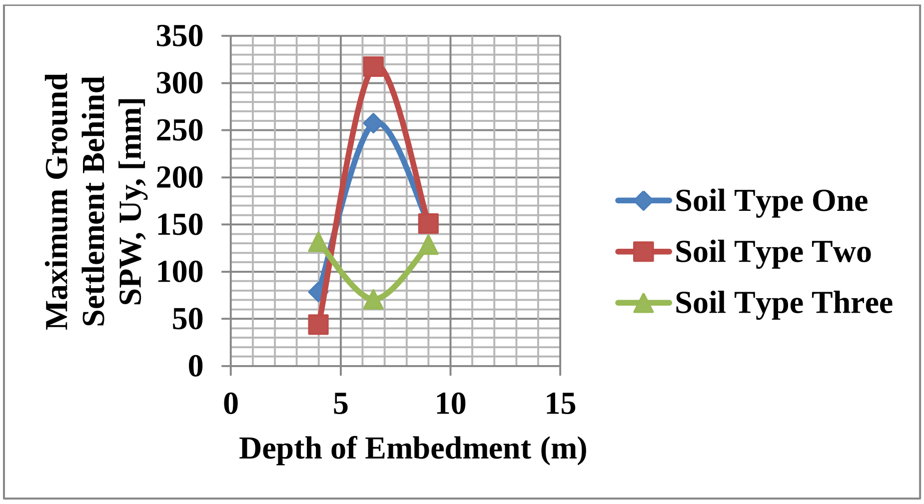


Figure 4.33 Maximum ground settlement behind the sheet pile wall with change in D_{emb}

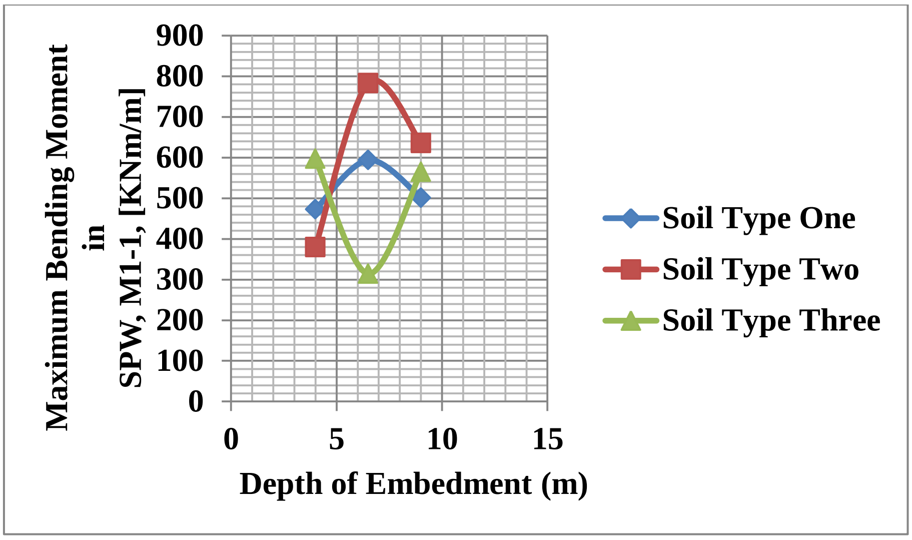


Figure 4.34 Maximum BM [M1-1] in the sheet pile wall with change in embedment depth

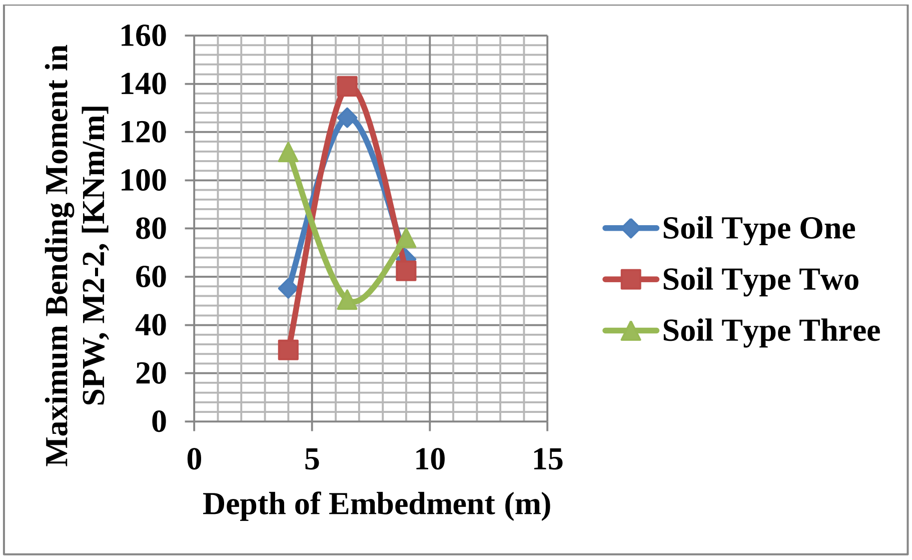


Figure 4.35 Maximum BM [M2-2] in the sheet pile wall with change in embedment depth

The wall and ground deformation

Figure 4.31 - 32 Observes that soil type one and two provides the increase and decrease of $U_{x,max}$ in SPW and the ground around braced deep excavation. The $U_{x,max}$ in the SPW and that of the ground decrease by medium variation to some degree and then increase slightly due to braced deep excavation in soil type three as embedment depth of SPW is increase.

Soil type one and two has maximum bending moment (M1-1, M2-2) in the SPW which is significantly increase and decrease as D_{emb} is increase. The maximum BM (M1-1, M2-2) due to braced deep excavation in soil type three is decrease and increase extensively. The percentage increment and decrement of the maximum BM (M1-1, M2-2) in the SPW is 125.76%, 106.09% and 228.22%, 121.88% in soil type one, 206.38%, 167.46% and

467.68%, 210.4% in soil type two, 52.61%, 94.64% and 45.14%, 68.1% in soil type three as D_{emb} of sheet pile wall is increase from the original value in order already mentioned.

Ground settlement

Figure 4.33 Observes that soil type one and two has vertical $U_{y,max}$ that significantly increase and decrease with increase in D_{emb} of sheet pile wall. Finally $U_{y,max}$ is decrease and increase with medium variation in soil type three as D_{emb} of sheet pile wall is increase.

Case Two: Stiffer Sheet Pile Wall with Diameter (d = 0.9 m)

➢ **When Depth of Excavation (D_{exc}) = 16 m**

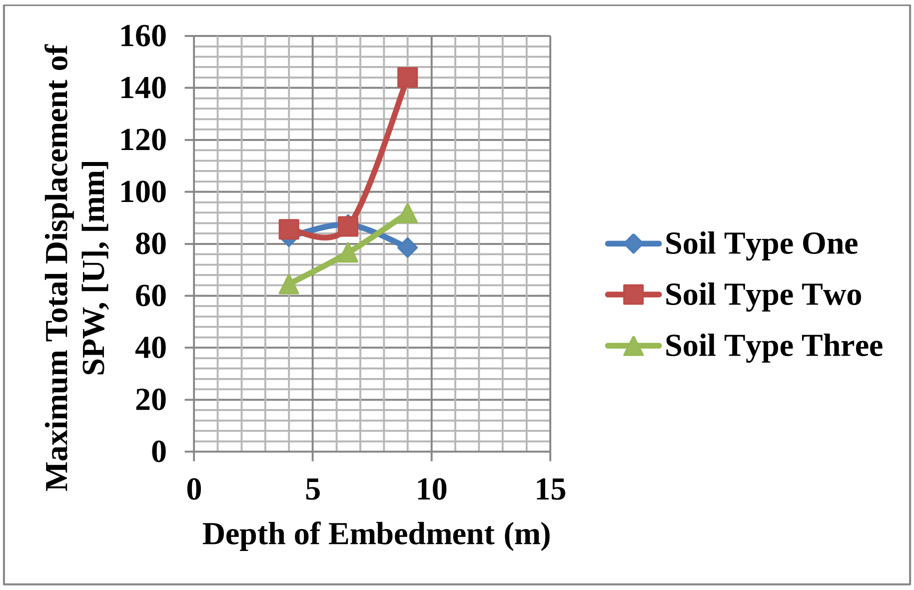


Figure 4.36 Maximum [U] of sheet pile wall with change in embedment depth

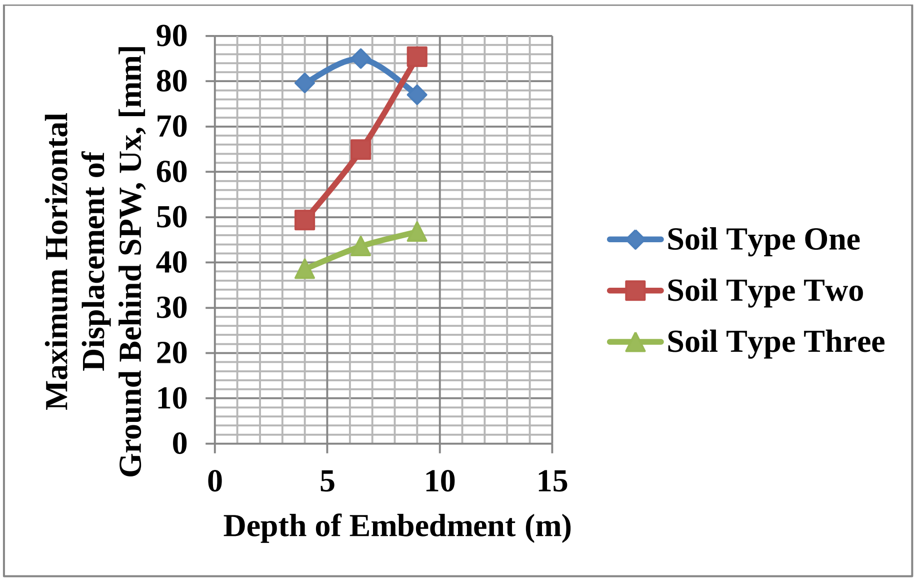


Figure 4.37 Maximum U_X of the ground behind SPW with change in embedment depth

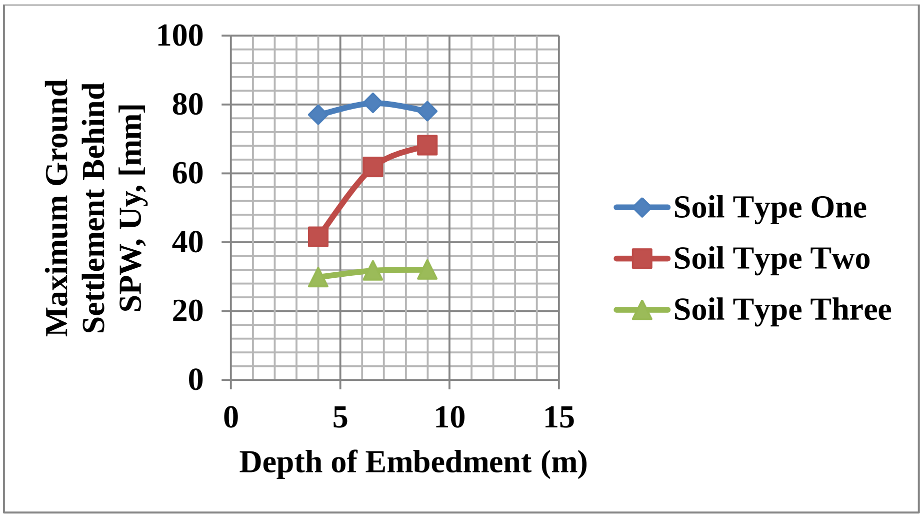


Figure 4.38 Maximum U_y behind the sheet pile wall with change in embedment depth

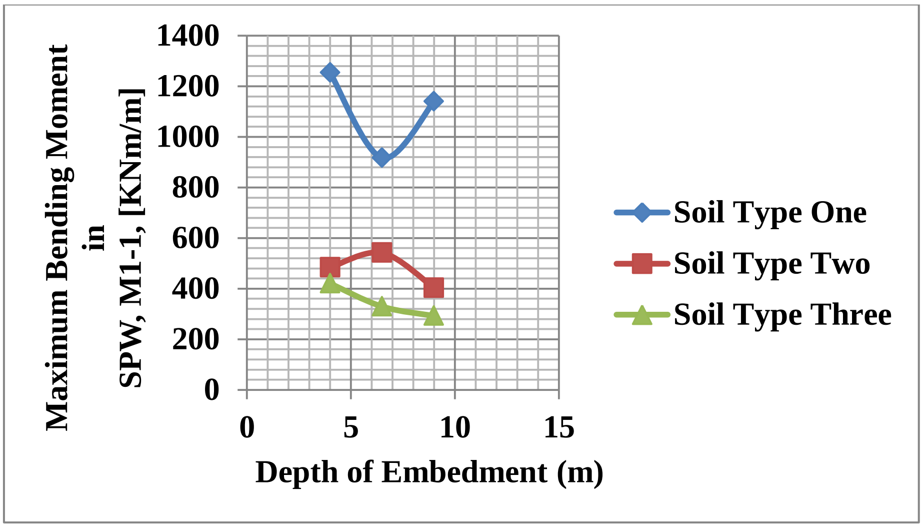


Figure 4.39 Maximum BM [M1-1] in the sheet pile wall with change in embedment depth

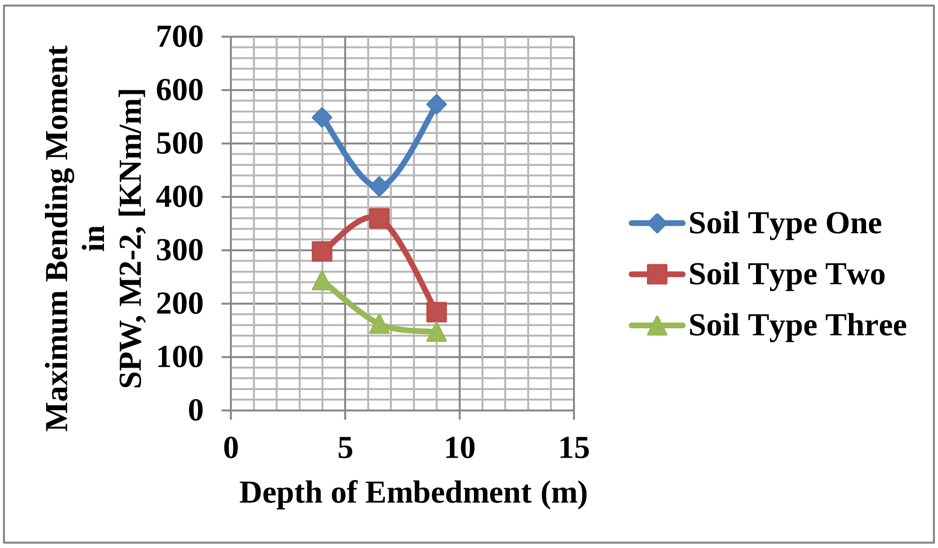


Figure 4.40 Maximum BM [M2-2] in the sheet pile wall with change in embedment depth

The wall and ground deformation

Figure 4.36-37 Observes that soil type one has the $U_{x,max}$ in sheet pile and the ground that increase with small variation to a considerable amount and then decrease it .The maximum $U_{x,max}$ in the SPW that results from braced deep excavation in soil type two is increase by very small change up to some extent and then extensively increases $U_{x,max}$ as the $U_{x,max}$ of the ground is extensively increase with increase in embedment depth. But the U_x in SPW and that of the ground is increase by small variation as D_{emb} is increases in soil type three.

Soil type one has very much more maximum bending moment (M1-1, M2-2) in the sheet pile wall as compared to others and provide significant decrease and increase of maximum BM (M1-1, M2-2) in the sheet pile, whereas the maximum BM (M1-1, M2-2) that results due to braced deep excavation in soil type two is increase and decrease significantly as D_{emb} is increases. Additionally the maximum bending moment (M1-1, M2-2) in the sheet pile wall is decrease with small variations as depth of embedment is increase.

Ground settlement

Figure 4.38 Observe that soil type one has much more vertical ground settlement behind the SPW than others and soil type one has an increase and then decrease in the vertical $U_{y,max}$ with very small variation whereas the vertical $U_{y,max}$ is significantly increase in soil type two. Finally the vertical U_y behind the SPW that results due to braced deep excavation in soil type three is increase by very small change as embedment depth is farther increase.

- **When Depth of Excavation (D_{exc}) = 20 m**

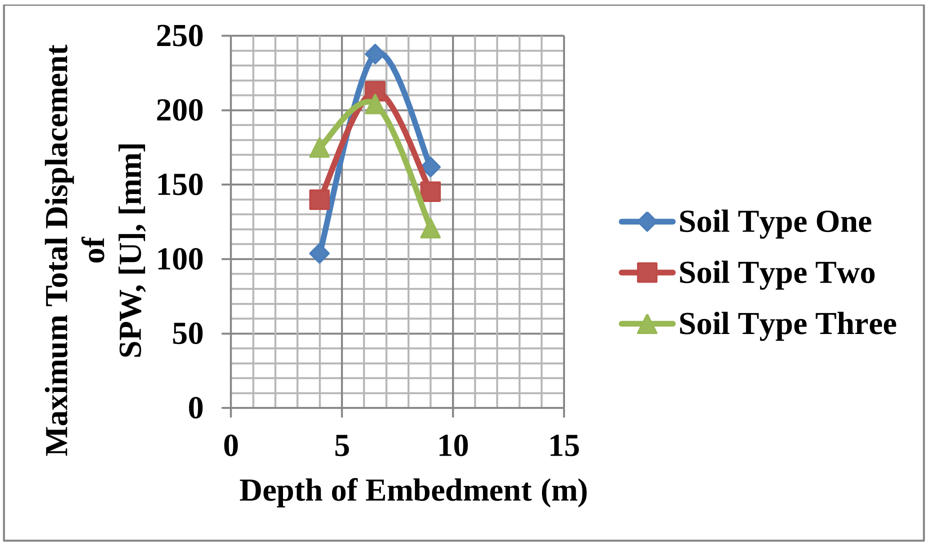


Figure 4.41 Maximum [U] of sheet pile wall with change in embedment depth

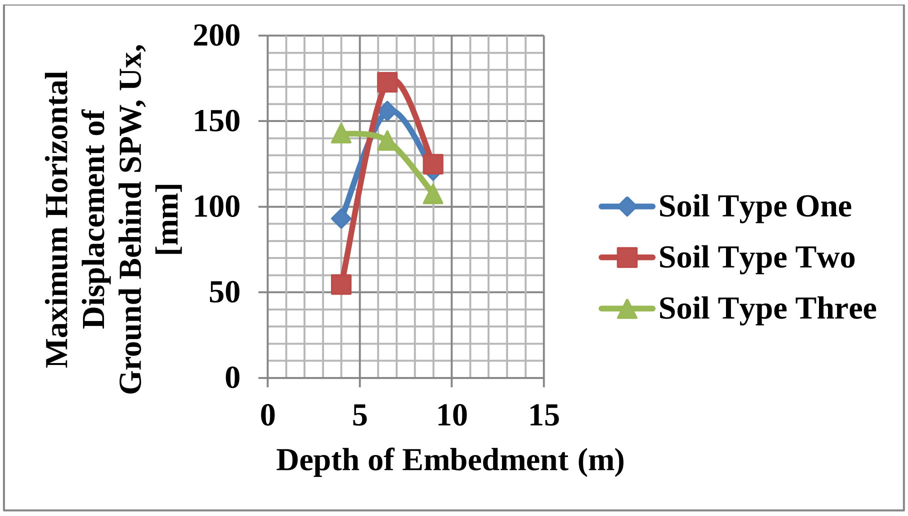


Figure 4.42 Maximum U_x of the ground behind SPW with change in embedment depth

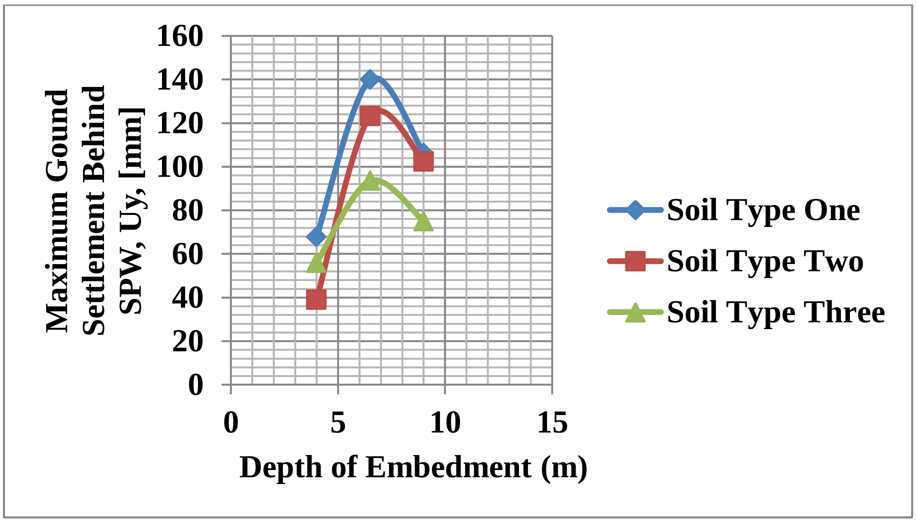


Figure 4.43 Maximum U_y behind the sheet pile wall with change in embedment depth

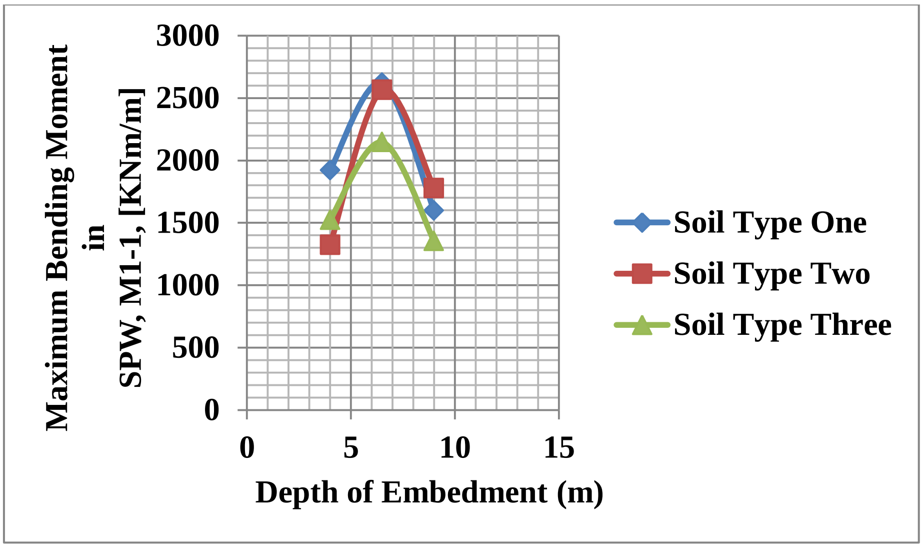


Figure 4.44 Maximum BM [M1-1] in the sheet pile wall with change in embedment depth

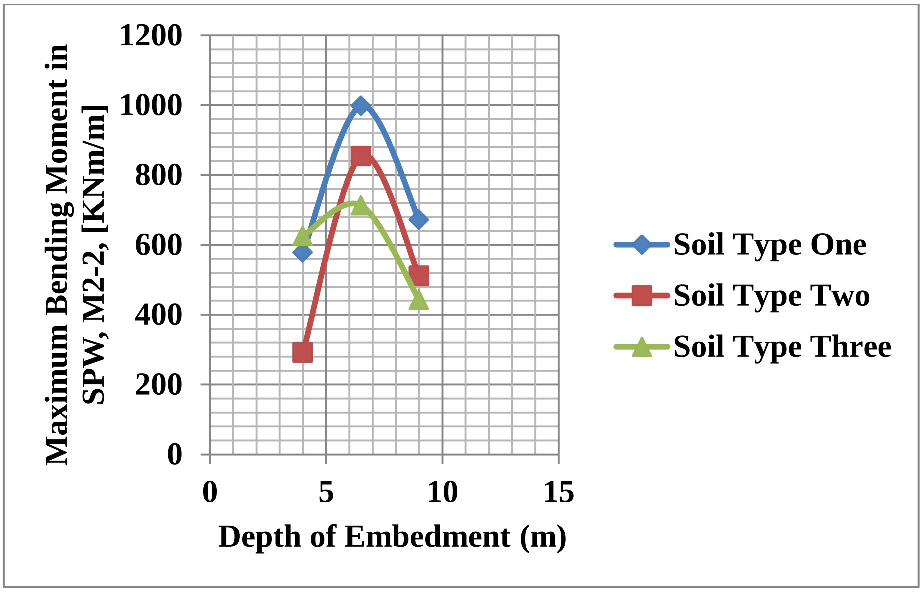


Figure 4.45 Maximum BM [M2-2] in the sheet pile wall with change in embedment depth

The wall and ground deformation

Soil type one, two and soil type three provide the extensively increase and decrease the $U_{x,max}$ in the sheet pile as Soil type one and two produce $U_{x,max}$ of the ground that increase and then decrease with increase in embedment depth. However as far as soil type three is concerned $U_{x,max}$ of the ground is increase slightly up to some extent and then decrease by medium variation as embedment depth is increase from the original value.

The maximum bending moment (M1-1) in the sheet pile wall is extensively increase up to some degree and then decrease it as a result of braced deep excavation in soil type one, two and three as maximum BM (M2-2) of sheet pile wall in soil type one and two is also significantly increase up to a considerable amount and then decrease with high variations. However when the maximum bending moment (M2-2) due to braced deep excavation in soil type three is slightly increase up to some extent and then highly decrease it. The percentage increment and decrement of the maximum BM (M1-1 and M2-2) in the SPW is 136.49%, 83.13% and 172.71%, 116.3% in soil type one, 193.95%, 134.5% and 293.14%, 175.6% in soil type two, 140.81%, 88.78% and 114.02%, 70.8% in soil type three as embedment depth of the sheet pile wall is increase from the original value.

Ground settlement

Figure 4.43 Observe that soil type one has much more vertical $U_{y,max}$ than other soil types and $U_{y,max}$ is significantly increase to a considerable amount and then highly decrease its $U_{y,max}$ due to braced deep excavation in soil type one and two. As far as soil type three is concerned the vertical $U_{y,max}$ is increase up to some extent and then decrease by medium variation as embedment depth of the sheet pile wall is increases as well.

4.1.1.5. Effect of Change in Stiffness of Struts and Walling

This part of analysis concentrates on how the change in stiffness of strut and walling affects the performance of braced deep excavations supported by tieback anchored SPW. The result of this study is presented for each soil types having varying stiffness of strut and walling, thickness = 0.6 m, D_{exc} = 16 m and D_{emb} = 4 m of the original model. The other parameters are same with that is written in section 4.1.1.1.

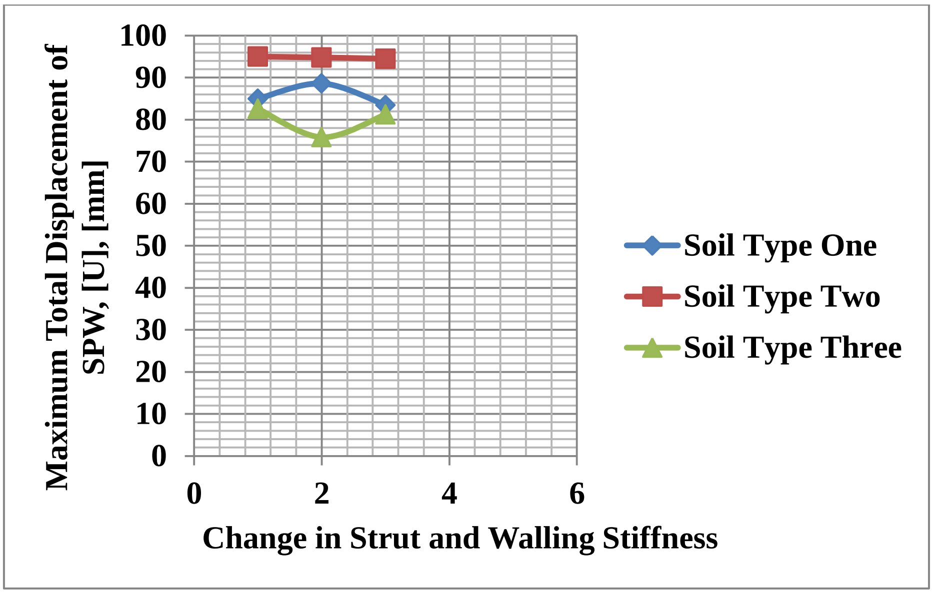


Figure 4.46 Maximum [U] of sheet pile wall with change in strut and walling stiffness

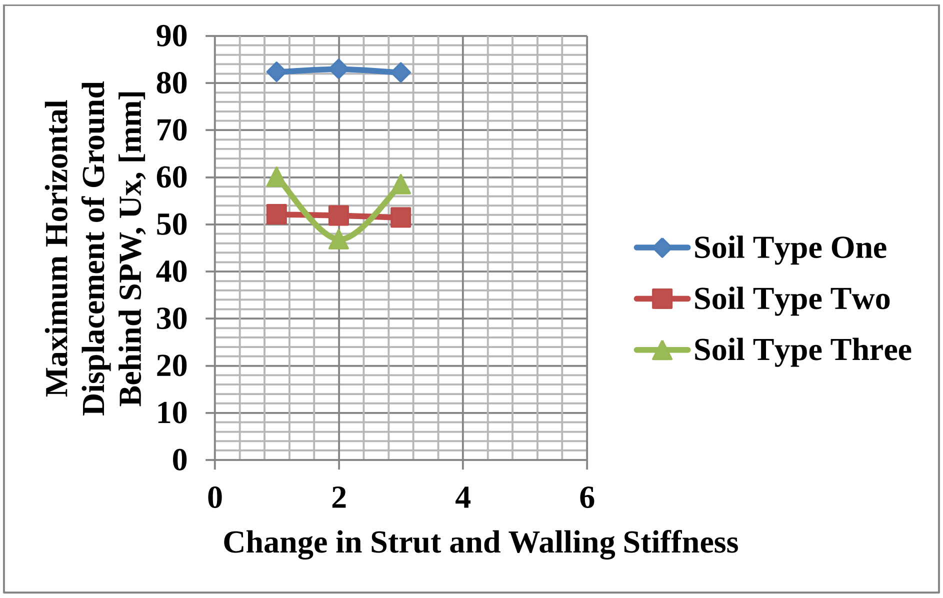


Figure 4.47 $U_{x,max}$ of the ground behind SPW with change in strut and walling stiffness

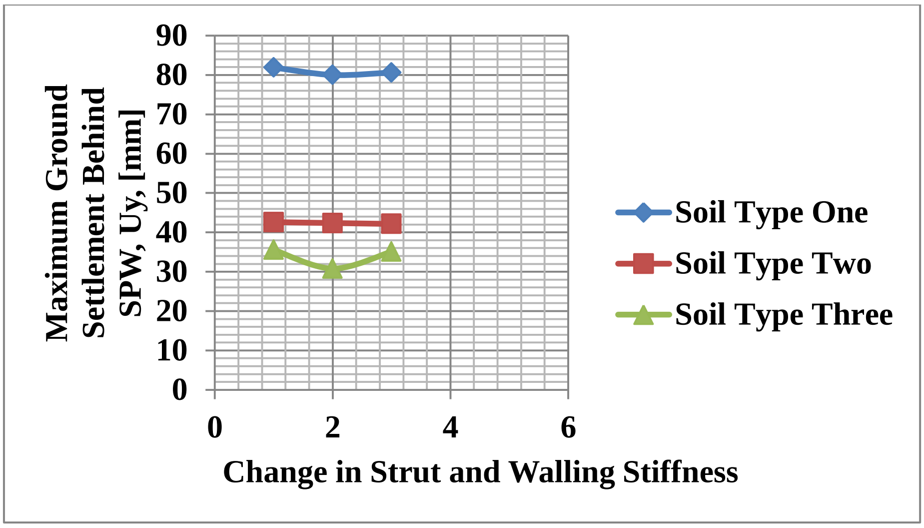


Figure 4.48 Maximum U_y behind the SPW with change in strut and walling stiffness

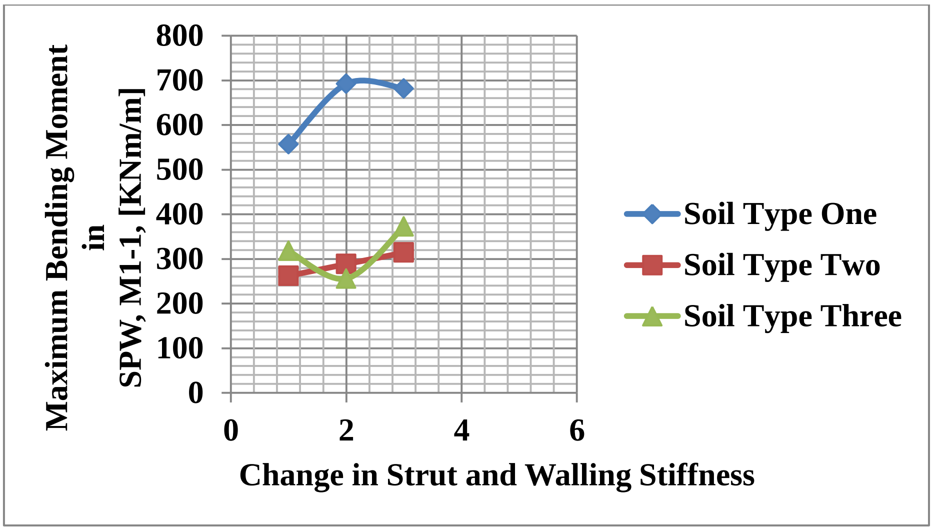


Figure 4.49 Maximum BM [M1-1] in the SPW with change in strut and walling stiffness

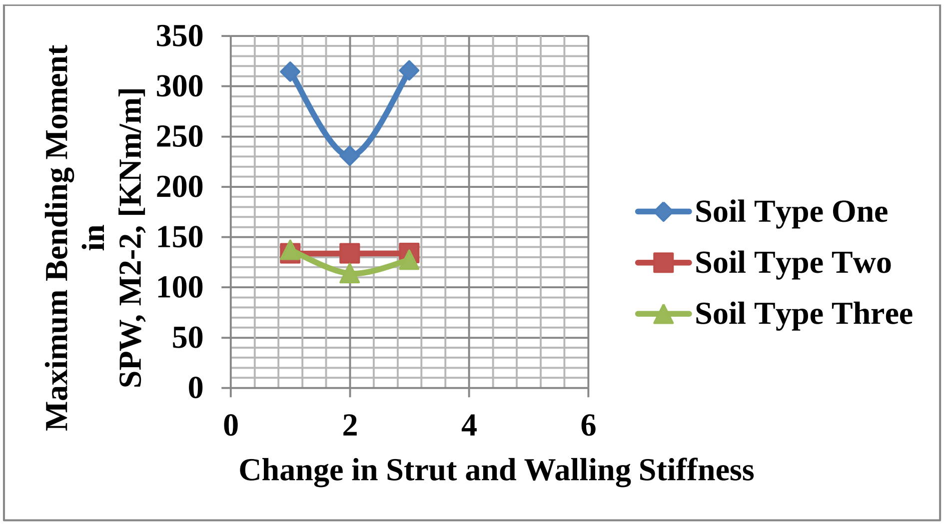


Figure 4.50 Maximum BM [M2-2] in the SPW with change in strut and walling stiffness

The wall and ground deformation

Figure 4.46-47 Observes that soil type one that provides the medium increase and decrease of $U_{x,max}$ in sheet pile and the $U_{x,max}$ of the ground which is almost constant. The $U_{x,max}$ in SPW and the ground around braced deep excavation in soil type two is almost constant, whereas a medium decrease and increase of $U_{x,max}$ in sheet pile wall and the ground due to excavation in soil type three is observed as stiffness of strut and walling is increase.

Soil type one has very much more maximum bending moment (M1-1, M2-2) in the SPW as compared to others and provides slight increase and decrease of the maximum BM (M1-1) and then extensive decrease and increase of the maximum BM (M2-2). However the maximum BM (M1-1) in the sheet pile that results from braced deep excavation in soil type two is increase by small variations and the maximum BM (M2-2) is almost constant as stiffness of strut and walling is increase. Besides this, the maximum BM (M1-1, M2-2) in the sheet pile wall due to braced deep excavation in soil type three is slightly decrease and increase by medium variation as stiffness of strut and walling is increase.

Ground settlement

Figure 4.48 Observe that soil type one has much more vertical $U_{y,max}$ than other soil types and the vertical $U_{y,max}$ is decrease and increase by a very small considerable amount as $U_{y,max}$ is approach constant for soil type two. Additionally the $U_{y,max}$ in soil type three is decrease and increase by medium variation as stiffness of strut and walling is increase.

Thus the above figures observed that increasing the stiffness of strut and walling only does not provide significant change in displacement of the ground and sheet pile, settlement and bending moment without increase in the stiffness of SPW and retained soil.

4.1.1.6. Effect of Change in Vertical Spacing of Struts and Walling

The specific objective of this paper in this section of study is concentrate on how the change in vertical spacing of struts and walling affects the performance of braced deep excavations. The result of this analysis is presented for each soil types having thickness of sheet pile wall = 0.9 m, D_{exc} = 20 m and struts and walling vertical spacing which is vary from 3 m→ 4 m→ 5 m during analysis. The other parameters are same with that is written in section 4.1.1.1.

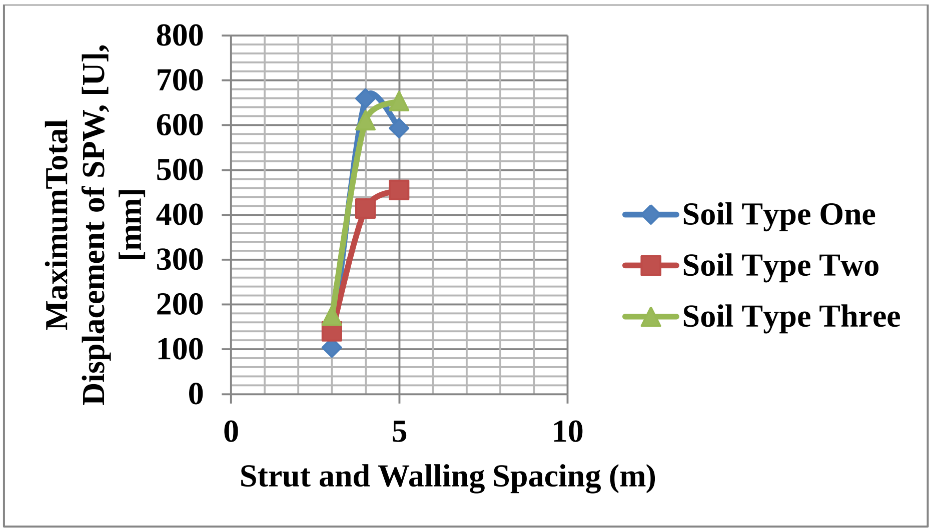


Figure 4.51 Maximum [U] of sheet pile with change in strut and walling vertical spacing

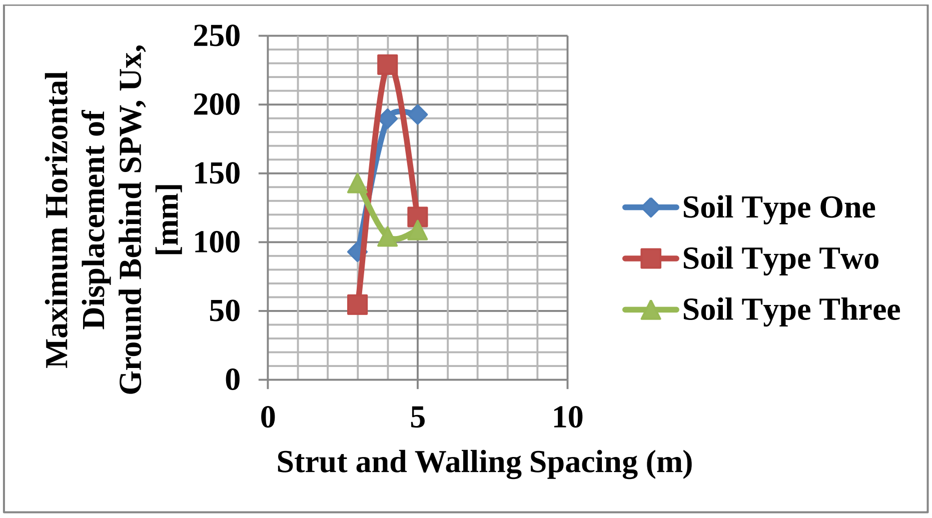


Figure 4.52 U_x of the ground behind SPW with change in strut and walling vertical spacing

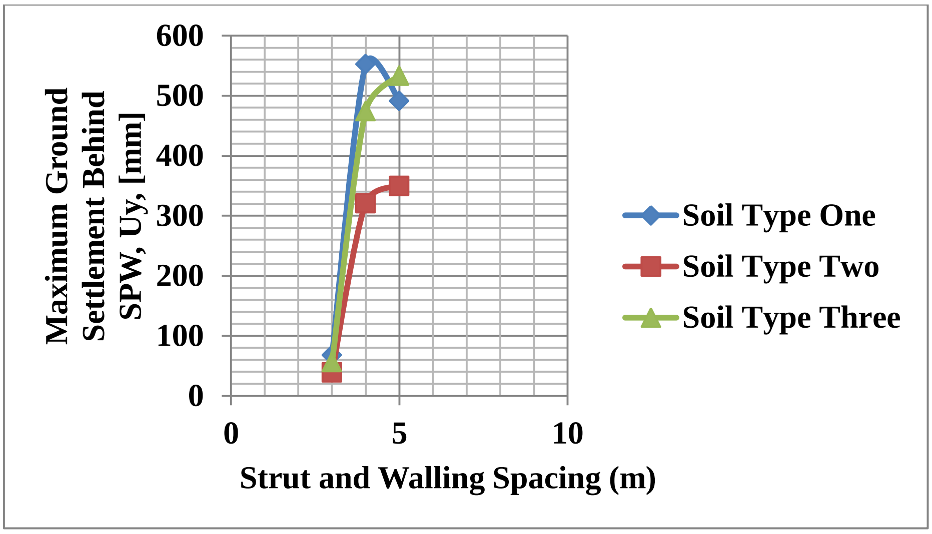


Figure 4.53 $U_{y,max}$ behind the SPW with change in strut and walling vertical spacing

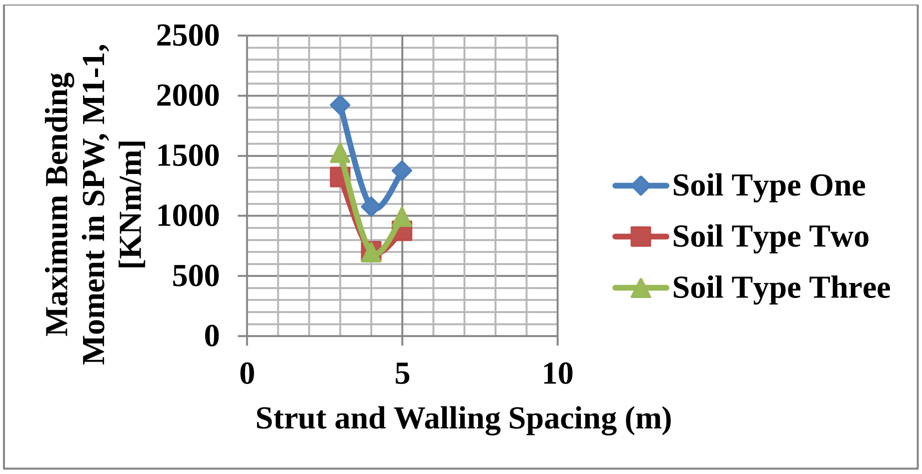


Figure 4.54 BM_{max} [M1-1] in the SPW with change in strut and walling vertical spacing

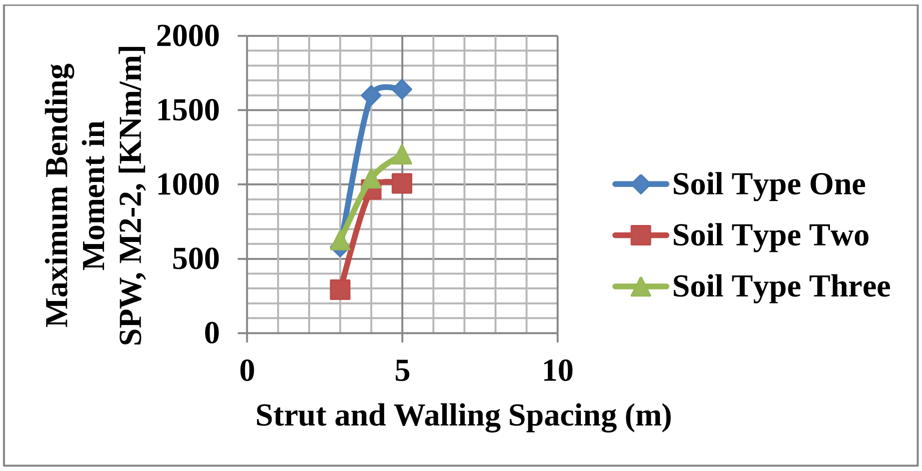


Figure 4.55 BM_{max} [M2-2] in the SPW with change in strut and walling vertical spacing

The wall and ground deformation

Figure 4.51-52 Observes that soil type one provides the extensive increase in the $U_{x,max}$ in the sheet pile up to some extent and then slightly decreases it as $U_{x,max}$ of the ground is increase extensively up to some degree and then increase by small variation. Apart from this, in soil type two $U_{x,max}$ of sheet pile is highly increase up to some extent and then increase slightly as $U_{x,max}$ of the ground is increase and decrease significantly. However the $U_{x,max}$ in the sheet pile that happen due to braced deep excavation in soil type three is significantly increase to a considerable amount and then slightly increase as the $U_{x,max}$ of the ground is slightly decrease and increase with increase in strut and walling spacing.

Soil type one, two and three has maximum bending moment (M1-1) in the sheet piles that highly decrease up to a considerable value and then increase slightly, whereas, maximum BM (M2-2) that results from excavation in soil type one and two is extensively increase up to some degree and then increase by small variation. However the maximum bending moment (M2-2) of sheet pile in soil type three is increase significantly.

Ground settlement

Figure 4.53 Observe that soil type two and three has $U_{y,max}$ that increase significantly up to a considerable degree and then increase by small variation as well. But the ground $U_{y,max}$ that happens due to braced deep excavation in soil type one is extensively increase up to some degree and slightly decrease it as strut and walling spacing is increases.

As it is observed above applying struts and walling with the required space is highly decreases the horizontal and vertical displacement of the ground and SPW. Thus as strut and walling spacing is increase the horizontal and vertical displacement of the ground and SPW and bending moment is also increase. But the reverse of the graph with increase in spacing is results from the stiffness of stratified soil found at that depth.

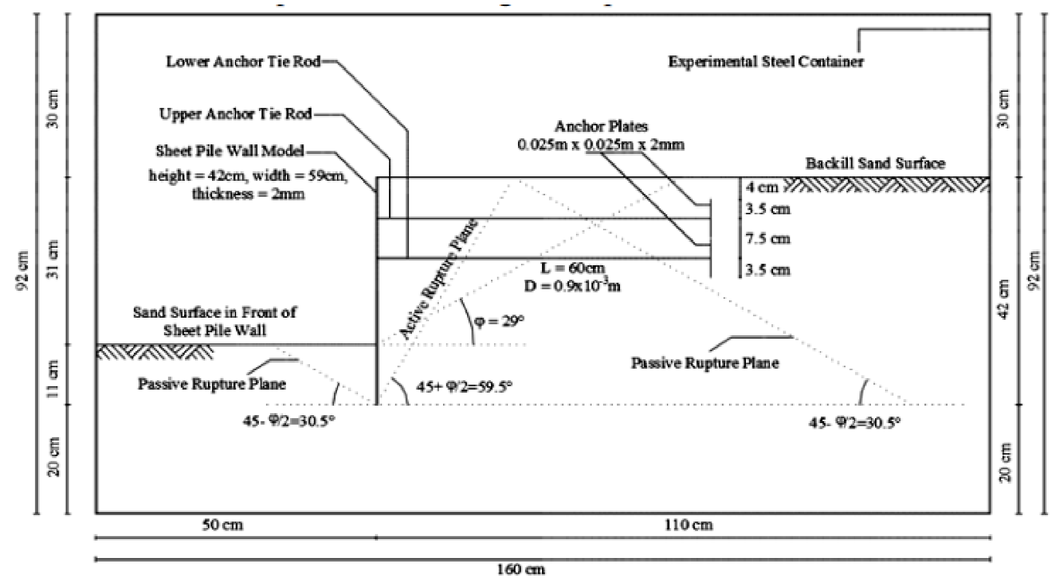
4.2. Validation

In this section of the paper, it will be conducted that whether PLAXIS-3D software is successfully applicable to assess the problem might be happen on the sheet pile wall or not.

4.2.1. Testing Apparatus and Wall Model

Experimental works were conducted at the laboratories of Faculty of Engineering, Cairo University. Figure 4.56 illustrates the apparatus and the sheet pile wall model used in the experimental work. The soil was used to be loose and fine sand and the container was polished internally and externally to minimize the side friction. The sheet pile model was designed to simulate the prototype sheet pile wall of Type Larsen II.

The surcharge loads in the experiments are applied via uniform rectangular steel sheet plates (Fig 4.57) having a length of 59 cm and a width of 16 cm. The surcharge loads were placed on the backfill surface such that the free distance between the edge of the steel plates and the sheet pile varied as h/6, h/3, 0.5 h, 2h/3, h, $1^1/_3$h, and 2h. The anchor rod diameter was selected to be $0.9*10^{-3}$m with 60 cm length. The horizontal spacing between anchor rods (S) was chosen to have a value of 8 cm, which simulates a distance of about 2.5 m in the field. In order to determine the distribution of bending moment along the total height of the sheet pile model, ten electrical strain gauges were installed along the wall height as shown in Figure 4.58.



(a) Anchored Sheet Pile Wall Model in Container

(b) Container Model

(c) Read-out Unit, Strain Transducer Set, Dial Gauges, Vertical and Horizontal Bars, Surcharge Loads

(d) Horizontal and Vertical Steel Bars Used for Hanging Dial Gauges

Fig 4.56 Detail of SP model in container and equipment used in experimental work

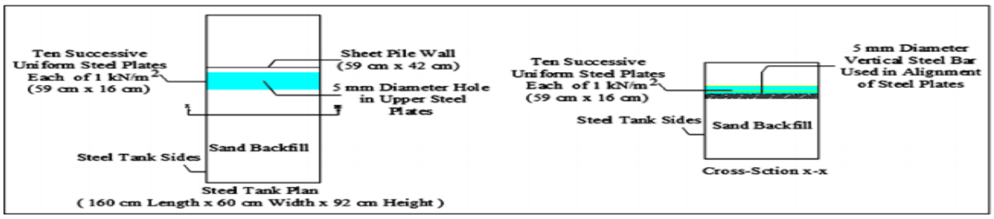


Fig 4.57 Uniform steel sheet plates used as surcharge loads acting on sand backfill

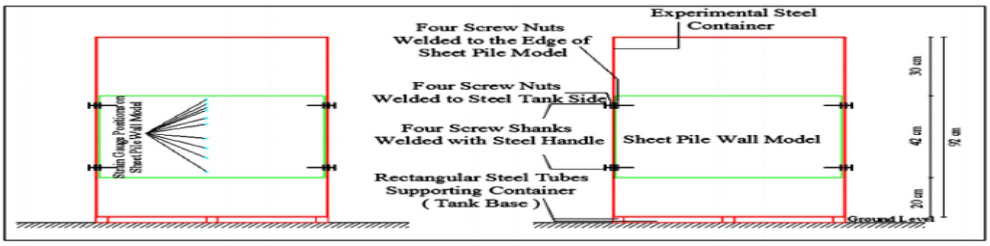


Fig 4.58 Positions of strain gauges

4.2.2. Critical dredging height of single anchored sheet pile

While the single anchored SP was standing in its stationary position, the filling process of the tank was executed. Upon reaching the upper level of sand on both sides of the sheet pile wall and leveling its upper surface, all screw shanks were simultaneously loosened. The developed stresses in the sheet pile wall, the induced anchor forces, and the horizontal wall displacements were monitored and compared with the permitted values. The stable dredging height was found to be 0.31 m as shown in Figure 4.56 (a). For convince we can consider the free height to be 0.3 m.

4.2.3. Numerical Analysis

A FE computer program PLAXIS-3D, has been used in this study to generate the model mesh that used to analyze the systems of double and single anchored SPW. An advanced constitutive soil model known as HSM was used to simulate the soil behavior around the excavation. The ground water is found at a great depth below the excavation depth. In this study, the SPW and anchor plates are simulated within the geometry model as beams.

Furthermore, soil and structural parameters that had been taken as an input for FEM (PLAXIS-3D,) were presented in tables bellow (From Cairo University experimental data).

Table 4.1 Loose fine sandy soil parameters for base model (own construction)

	Name	Loose fine sand
General		
Material model	Model	Hardening Soil Model
Drainage type	Type	Undrained A
Unit wt. of soil above phreatic line(KN/m^3)	γ_{unsat}	15
Unit wt. of soil below phreatic line(KN/ m^3)	γ_{sat}	15.5
parameters		
Secant modulus(KN/ m ^{2})	E_{50}^{ref}	18047.62
Oedometric stiffness (KN/ m ^{2})	E_{50}^{ref}	18047.62
Unloading-reloading stiffness (KN/ m ^{2})	E_{ur}^{ref}	54142.86
Power for stress level	m	0.5
Cohesion (KN/ m ^{2})	C'_{ref}	5
Friction angle of soil (^{o})	ϕ	37
Delatancy angle	ψ	3.5
Poisson's ratio	V'	0.25

Table 4.2 Material properties for beam (own construction)

Parameter	Name	Waling	Unit
Cross-section area	A	0.009782	m^2
Unit weight	γ	80.5	KN/m^3
Material behavior	Type	Linear	-
Young's modulus	E	$2.5*10^8$	KN/m^2
Moment of Inertia	I_3 I_2	$1.845*10^{-4}$ $4.05*10^{-4}$	m^4 m^4

Table 4.3 Material properties for anchors (own construction)

Parameters	Name	Node - to - node anchor	unit
Material type	Type	Elastic	-
Axial stiffness	EA	$6.9*10^5$	KN

Table 4.4 Material properties for embedded pile (grout body) (own construction)

Parameters	Name	Grout	Unit
Young's modulus	E	$3.2*10^7$	KN/m^2
Unit weight	γ	24.5	KN/m^2
Pile type	-	Pre-defined	-
Pre-defined pile type	-	Massive circular pile	-
Diameter	Diameter	0.2	m
Skin friction distribution	type	Linear	-
Skin resistance at the top	$T_{top\ max}$	200	KN/m
Skin resistance at the bottom	$T_{bot\ max}$	0.0	KN/m
Base resistance	$F_{bot\ max}$	0.0	KN

Table 4.5 Material properties for sheet pile wall (own construction)

Parameters	Name	Sheet pile wall	Unit
Thickness	d	0.2	m
Unit weight	γ	4.56	KN/m^3
Type or behavior	Type	Linear, non-isotropic	-
Young's modulus	E_1 E_2	$1.66*10^7$ $7.4*10^5$	KN/m^2 KN/m^2
Poisson's ratio	V	0.0	-

Shear modulus	G_{12} G_{13} G_{23}	$7.4*10^5$ $1.58*10^6$ $4.02*10^5$	KN/m^2 KN/m^2 KN/m^2

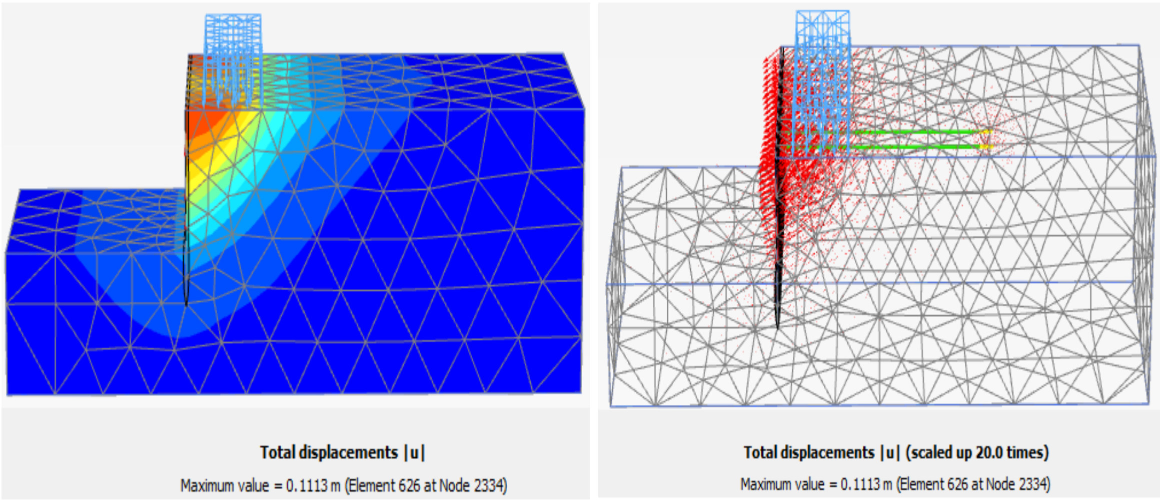


Fig 4.59 PLAXIS-3D model of double anchored SPW for maximum total displacement

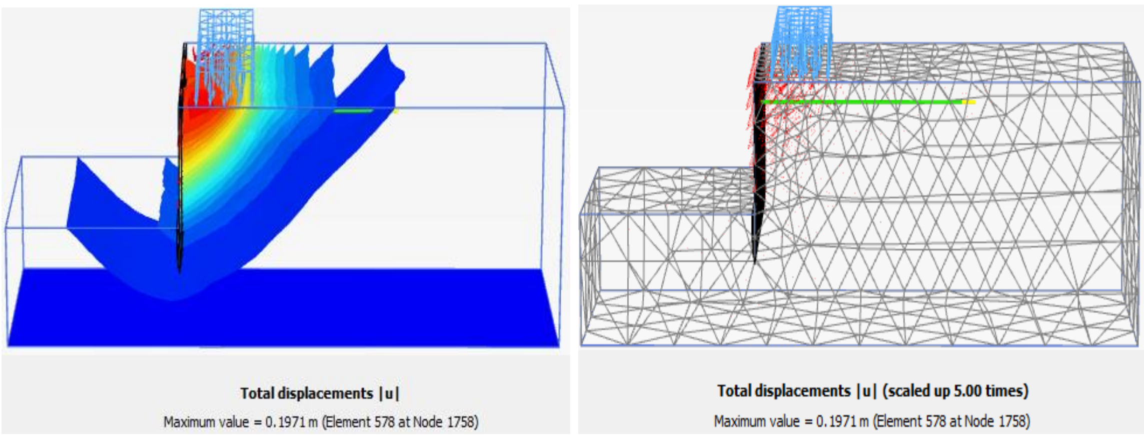


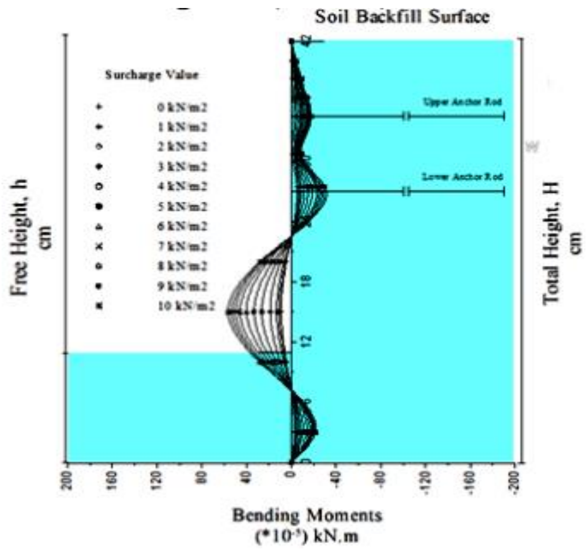
Fig 4.60 PLAXIS-3D model of single anchored SPW for maximum total displacement

4.2.4. Comparison of Experimental and Numerical Results

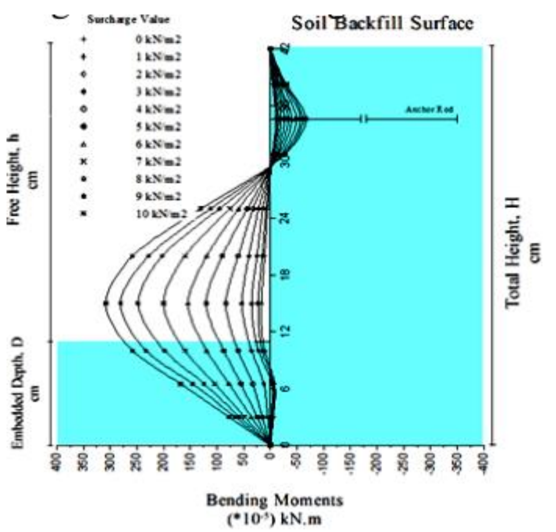
Figure 4.61 illustrates the distribution of bending moments developed experimentally and numerically in single and double anchored sheet pile wall after placing a surcharge of 10kN/m^2 at a free distance of h/6 (5 cm) from the wall. Figure 4.62 shows all maximum bending moment values obtained experimentally and numerically in single and double anchored SPWs. This figure indicates the similar general trends between the experimental and numerical results in case of double anchored system, especially when the surcharge load exceeds 5kN/m^2. Also, in case of single anchored system, it shows an agreement between the experimental and numerical results.

In case of single anchored SP, the maximum values of the BMs computed experimentally are higher than those resulting from the numerical work. However, after placing the surcharge loads at free distances of h (30 cm) or more, the numerical and experimental values became close to each other and the difference between them was insignificant. In case of double anchored SP, there is a high degree of similarity between the experimental and numerical results. In this SPW, the maximum values of BMs computed numerically and experimentally are almost identical.

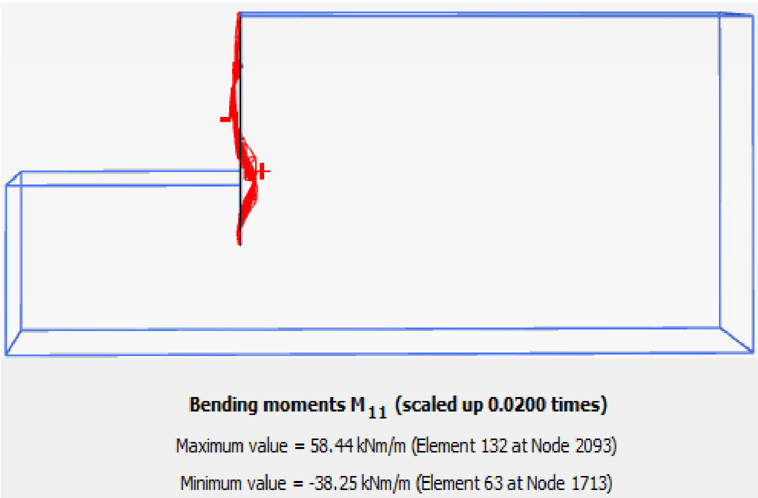
The achievement of a significant reduction in maximum bending moment values developed in the sheet pile wall has been proved experimentally and numerically by using a double anchored sheet pile system instead of a single anchored one.



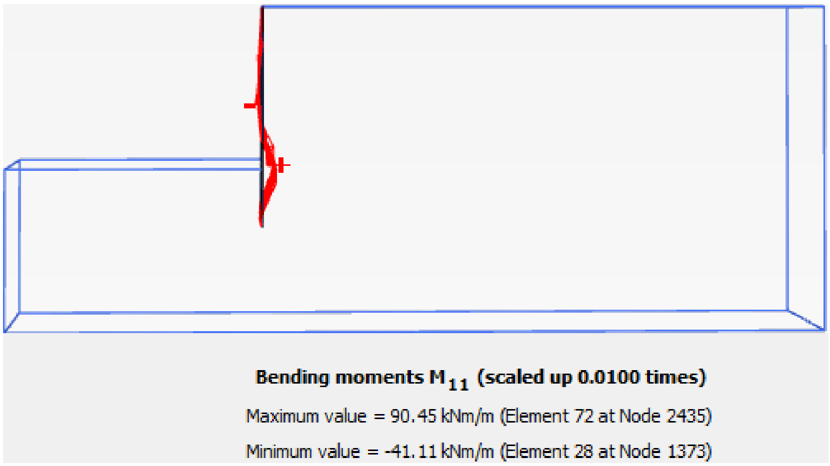
(a) Experimentally measured BMs of double anchored SPW (Surcharge = 10KN/m^2)



(b) Experimentally measured BMs of single anchored SPW (Surcharge = 10KN/m^2)

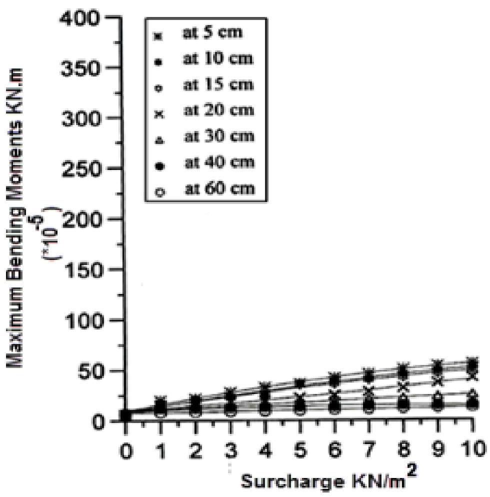


(c) Numerically measured BMs of double anchored SPW (Surcharge = 10 KN/m^2)

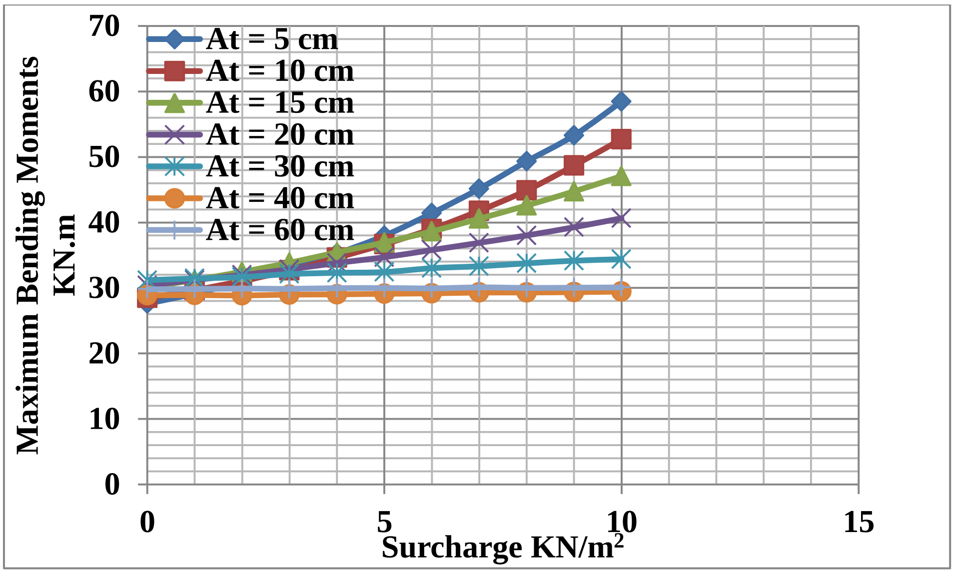


(d) Numerically measured BMs of single anchored SPW (Surcharge = 10 KN/m^2)

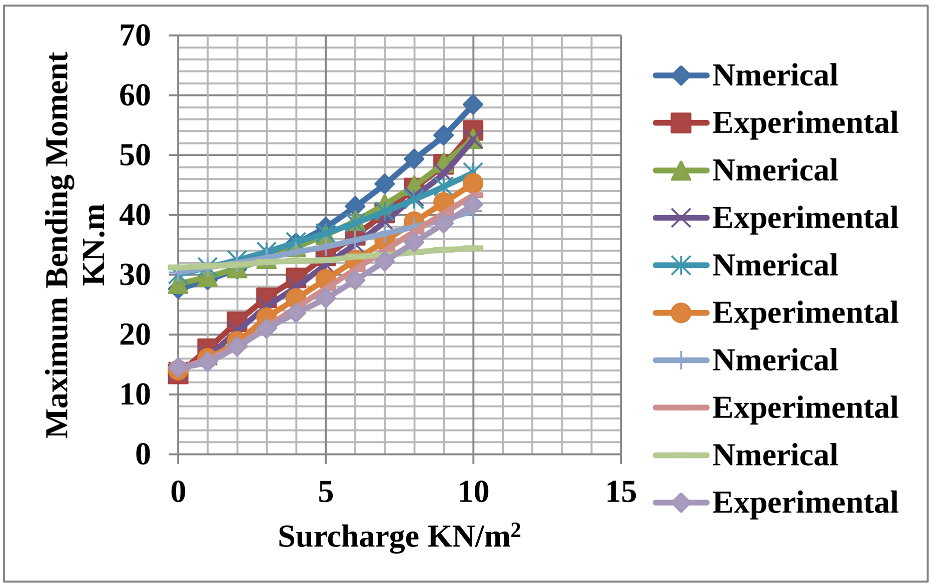
Fig 4.61 BMs measured experimentally and numerically in double & single anchored SP after placing surcharge at a free distance of 5cm from SPW (Surcharge = 10KN/m^2)



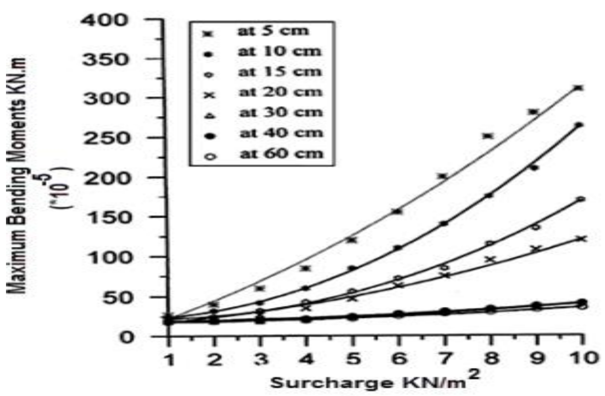
(a) Maximum BMs in Double Anchored Sheet Pile due to Experimental Analysis



(b) Maximum BM in Double Anchored Sheet Pile due to Numerical Analysis

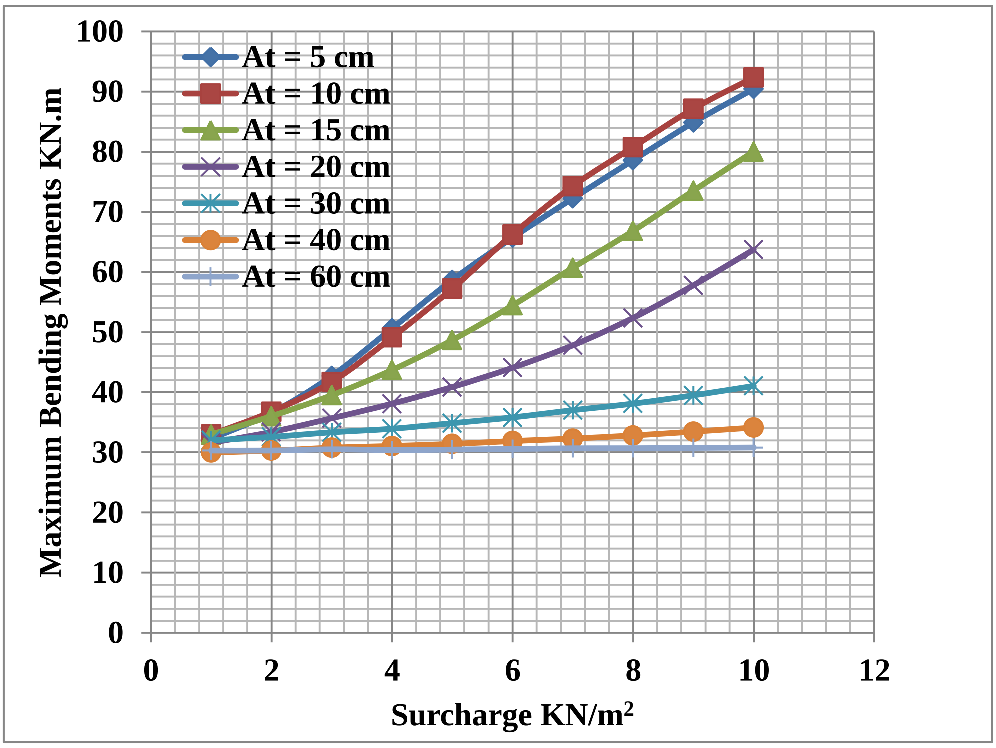


(c) Max- BM in Double Anchored SP for Both Experimental and Numerical Analysis

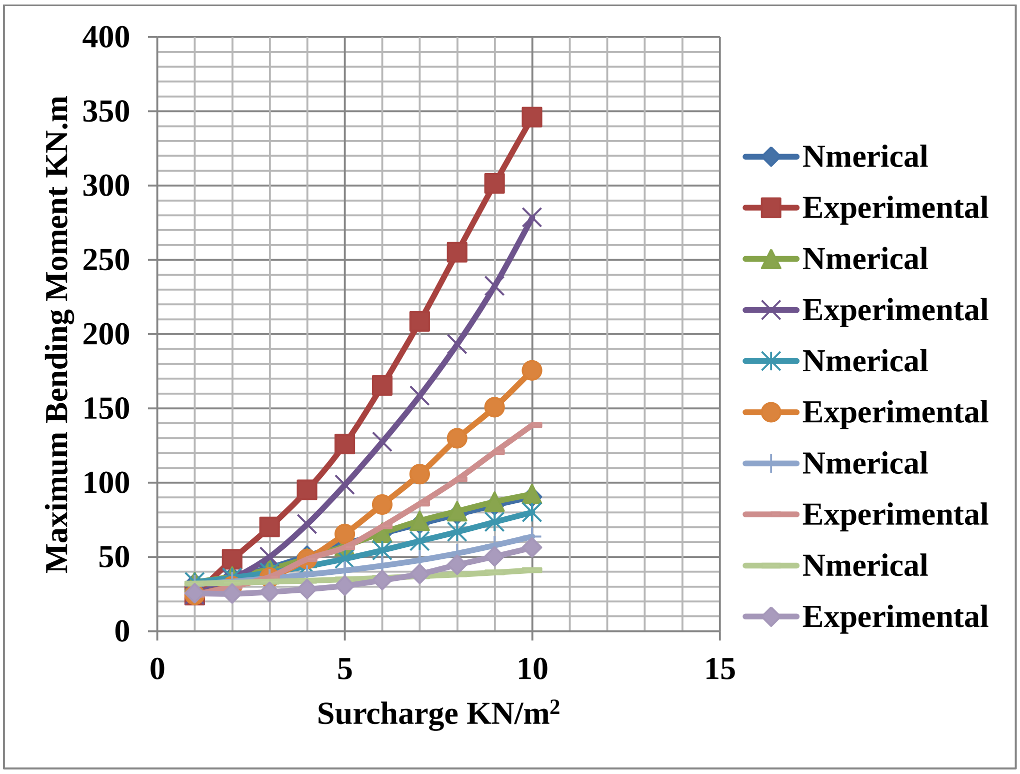


(a)
Maximum Bending Moments in Single Anchored Sheet Pile due to Experimental Analysis

(d) Maximum BM in Single Anchored Sheet Pile due to Experimental Analysis



(e) Maximum BMs in Single Anchored Sheet Pile due to Numerical Analysis



(f) Maximum Bending Moments in Single Anchored Sheet Pile for Both Experimental and Numerical Analysis

Fig 4.62 Comparison of the maximum bending moment values provided experimentally to that of numerical analysis in double and single anchored SPW (Surcharge = 0 - 10 KN/m^2).

From this comparison between the finite element (PLAXIS-3D) and experimental calculations, it can be concluded that PLAXIS-3D gives reliable results for the analysis of single and double anchored sheet pile walls.

4.2.5. Evaluation of the comparison

In the preceding sections the calculation results of PLAXIS-3D have been tested against one cases calculated by Experiment at Cairo University. The tests show that the maximum bending moment provided with PLAXIS-3D and Experiment correspond quite well.

Finally PLAXIS-3D is proven to be a useful tool to analyze tieback anchored sheet pile wall in order to determine the maximum bending moment, vertical ground settlement and any other geotechnical problem might be happen successfully and accurately as possible.

4.3. Empirical Methods of Analysis

It is very rare to find uniform deposits of sand or clay to a great depth. Many times layers of sand and clays overlying one another are found in nature. Based on field experience, empirical or semi-empirical procedures for estimating apparent pressure diagrams may be justified (Prof. V. N. S. Murthy, 958). From many empirical formulas that have been proposed in literature review, we only use the three most well-known and convenient one empirical formulas to predict the vertical ground settlement and horizontal displacement in the sheet pile wall on this paper. The three most well-known and convenient empirical formulas to predict vertical ground settlement and horizontal displacement in the sheet pile wall is going to proceed.

4.3.1. Peck's Method (1969)

Peck (1969) developed the first empirical method to predict the wall movement based on the actual vertical ground deformation data collected from temporary braced sheet pile wall and soldier pile walls with tie back anchor. He mainly employed the results of case histories in London, Oslo and Houston and established a relation curves between the ground surface settlement and the distance from braced wall or edge of excavation for three types of soil which are Soft clay, Stiff clay and Sand as shown in Figure 2.2.

Basically the apparent pressure envelopes available are for sand and clay only. Thus question may arise about how to treat silty clay, clayey silt, clayey silt with sand, and silty Sand. The apparent pressure envelopes proposed by Peck (1969) overcome this problem by considering silty clay and clayey silt as clay and clayey silt with sand, and silty Sand as Sand. According to this paper, soil type one represent silty clay and clayey silt, soil type two represent silty clay, clayey silt and clayey silt with sand and soil type three also represent silty clay, clayey silt, clayey silt with sand, and silty Sand. To overcome this soil stratification problem with 20 m depth, stiff clay is considered for soil type one and two, because, more of its' content is clay soil and sand is considered for soil type three as well. Therefore, let as assume soil type one and two is represented by soil type one new which is stiff clay and soil type three is represented by soil type two new which is sandy soil. Thus:

By using similar procedures the $\delta_{v,max}$ behind SPW and δH of SPW in stiff clay and sandy soil having maximum depth of excavation = 12 m, 16 m and 20 m is emphasized respectively in all empirical methods below.

Case One: Soil Type One (Stiff Clay Soil)

Table 4.6 Peck's method for stiff clay settlement envelope (own construction)

d_1 (m)	d_2(m)	d_3 (m)	$\frac{d_i}{H_{ei}}\%$	$\frac{\delta_v}{H_{ei}}\%$	δv_1 (m)	δv_2 (m)	δv_3 (m)
0.0	0.0	0.0	0.0	1.00	0.12	0.16	0.2
6.0	8.0	10	0.5	0.75	0.09	0.12	0.15
12	16	20	1.0	0.50	0.06	0.08	0.10
18	24	30	1.5	0.25	0.03	0.04	0.05
24	32	40	2.0	0.00	0.00	0.00	0.00

Case One: Soil Type Two (Sandy Soil)

Table 4.7 Peck's method for sandy soil settlement (own construction)

d_1(m)	d_2(m)	d_3(m)	$\frac{d_i}{H_{ei}}\%$	$\frac{\delta_v}{H_{ei}}\%$	δv_1 (m)	δv_2 (m)	δv_3 (m)
0.0	0.0	0.0	0.0	2.00	0.24	0.32	0.40
6.0	8.0	10	0.5	1.75	0.21	0.28	0.35
12	16	20	1.0	1.50	0.18	0.24	0.30
18	24	30	1.5	1.25	0.15	0.20	0.25
24	32	40	2.0	1.00	0.12	0.16	0.20
30	40	50	2.5	0.75	0.09	0.12	0.15
36	48	60	3.0	0.50	0.06	0.08	0.10
42	56	70	3.5	0.25	0.03	0.04	0.05
48	64	80	4.0	0.00	0.00	0.00	0.00

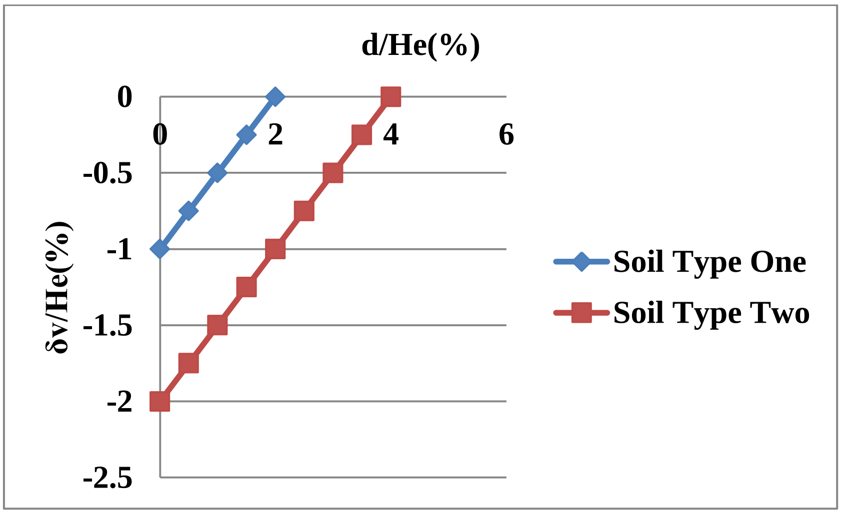


Figure 4.63 Pecks' ($\frac{\delta_v}{H_e}$ (%)).vs. d/He (%) for stiff clay and sandy soil

Case Two: Soil Type One (Stiff Clay Soil)

Table 4.8 Peck's horizontal displacement of sheet pile walls envelope (own construction)

z'$_1$ (m)	z'$_2$ (m)	z'$_3$ (m)	$\frac{z'_i}{H_{ei}}$ (%)	$\frac{\delta H}{H_{ei}}$	δH_1 (m)	δH_2 (m)	δH_3 (m)
6.0	8.0	10	0.50	0.17 %	0.0204	0.0272	0.034
9.0	12	15	0.75	0.25 %	0.03	0.04	0.05
12	16	20	1.00	0.33 %	0.0396	0.0528	0.066

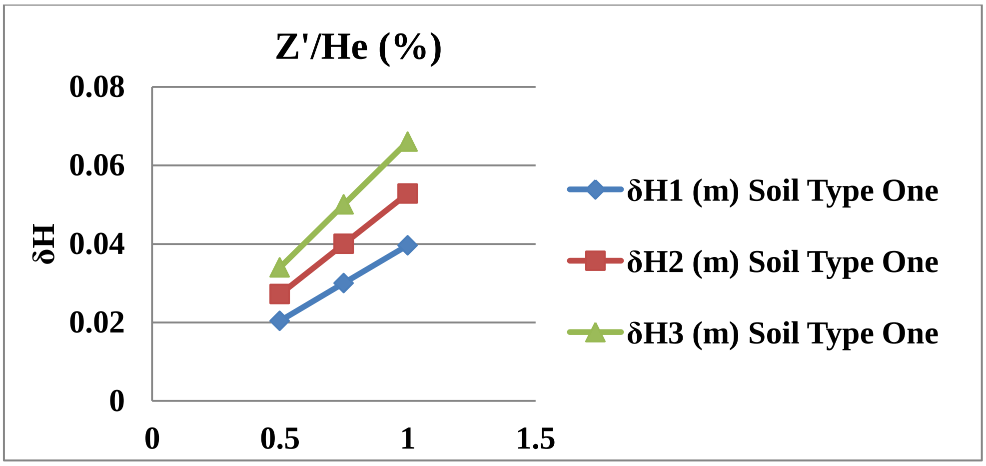


Figure 4.64 Pecks' maximum horizontal displacement in the SPW (δ_H).vs. Z'/He (%)

Case Two: Soil Type Two (Sandy Soil)

Table 4.9 Peck's horizontal displacement of sheet pile walls envelope (own construction)

z'$_1$ (m)	z'$_2$ (m)	z'$_3$ (m)	$\frac{z'_i}{H_{ei}}$ (%)	$\frac{\delta H}{H_{ei}}$	δH_1 (m)	δH_2 (m)	δH_3 (m)
6.0	8.0	10	0.50	0.18%	0.0216	0.0288	0.036
9.0	12	15	0.75	0.27%	0.0324	0.0432	0.054
12	16	20	1.00	0.36%	0.0432	0.0576	0.072

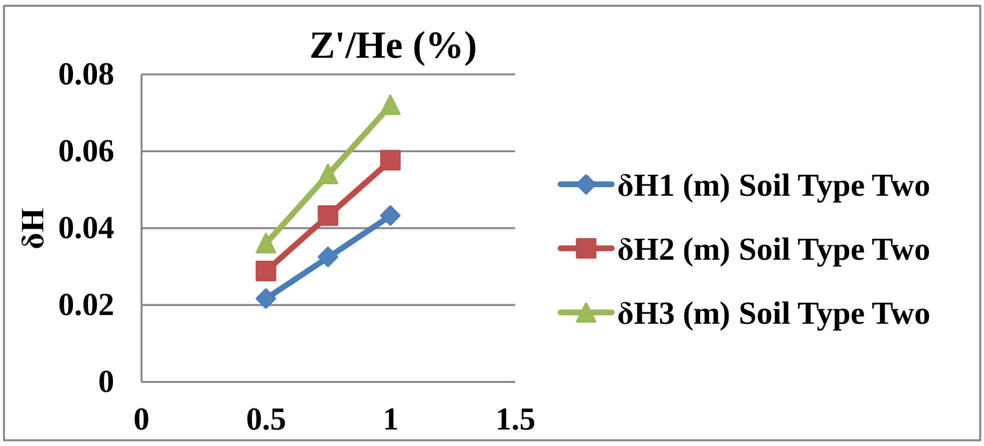


Figure 4.65 Pecks' maximum horizontal displacement in the SPW (δH).vs.Z'/He (%)

Figure 4.63 Observe that soil type two has much more vertical ground settlement than soil type one and the significant increase in the vertical ground settlement behind the sheet pile wall as distance from sheet pile wall is decrease. Figure 4.64-65 Observes that soil type two has much high horizontal displacement in the sheet pile wall that increase with increase in Z'/He than soil type one.

4.3.2. Clough and O'Rourke (1990)

Clough and O'Rourke (1990) presented the current state – of – the - art of empirical observations for estimating the lateral wall deflections and settlement in excavations. They are presented a dimensionless settlement profile for sand, stiff to very hard clays, and soft to medium clays to further support these finding. They also occluded in Figure 2.8 to Figure 2.9 about settlements adjacent to excavations in sandy soil and stiff to hard clay.

Case One: Soil Type One (Stiff Clay Soil)

Table 4.10 Clough and O'Rourke's settlement envelope for stiff clay (own construction)

d_1 (m)	d_2 (m)	d_3 (m)	$\frac{d_i}{H_{ei}}\%$	$\frac{\delta_v}{H_{ei}}\%$	δv_1 (m)	δv_2 (m)	δv_3 (m)
0.0	0.0	0.0	0.0	0.30	0.036	0.048	0.06
6.0	8.0	10	0.5	0.25	0.03	0.040	0.05
12	16	20	1.0	0.20	0.024	0.032	0.04
18	24	30	1.5	0.15	0.018	0.024	0.03
24	32	40	2.0	0.10	0.012	0.016	0.02
30	40	50	2.5	0.05	0.006	0.008	0.01
36	48	60	3.0	0.00	0.000	0.000	0.00

Case One: Soil Type Two (Sandy Soil)

Table 4.11 Clough and O'Rourke's settlement envelope for sandy soil (own construction)

d_1 (m)	d_2 (m)	d_3 (m)	$\frac{d_i}{H_{ei}}\%$	$\frac{\delta_v}{H_{ei}}\%$	δv_1 (m)	δv_2 (m)	δv_3 (m)
0.0	0.0	0.0	0.0	0.300	0.036	0.048	0.060
6.0	8.0	10	0.5	0.225	0.027	0.036	0.045
12	16	20	1.0	0.150	0.018	0.024	0.030
18	24	30	1.5	0.075	0.009	0.012	0.015
24	32	40	2.0	0.000	0.000	0.000	0.000

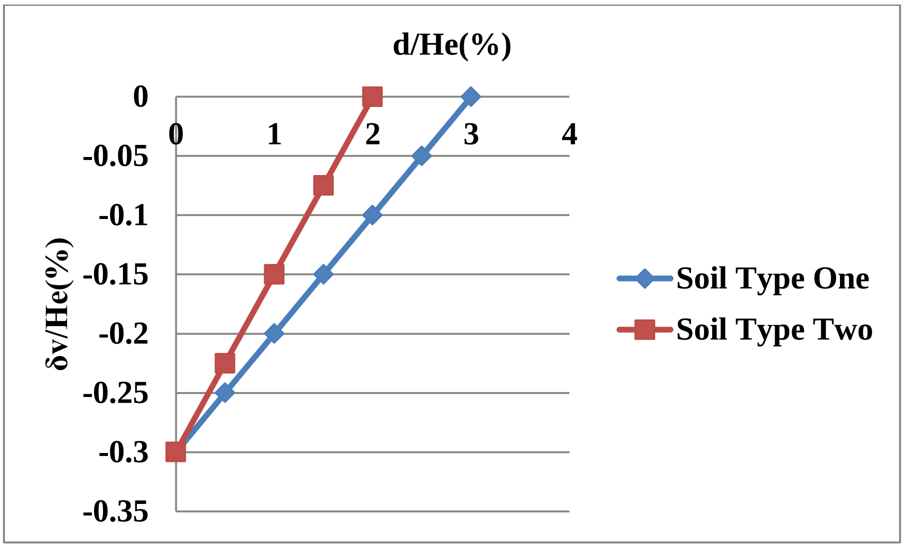


Figure 4.66 Clough and O'Rourke's ($\frac{\delta_v}{H_e}$ (%)).vs. d/He (%) for soil type one and two

Case Two: Soil Type One and Two (Stiff Clay and Sandy Soil)

By using figure 2.6 which is Clough and O'Rourke's horizontal displacement of SPW determination figure for both soil we can get the below results.

Table 4.12 Clough and O'Rourke's horizontal displacement of sheet pile walls envelope for stiff cay and sandy soil (own construction)

H_e (m)	$\frac{\delta H}{H_e}$	δH (m)
12	0.2%	0.022
16	0.2%	0.031
20	0.2%	0.040

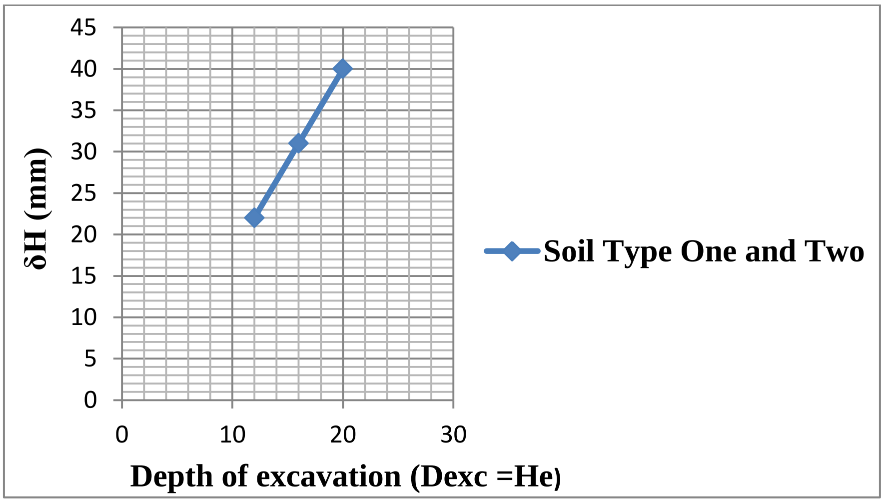


Figure 4.67 Clough and O'Rourke's horizontal displacement of sheet pile wall

We can observes from figure 4.66 that both soil type one and soli type two has a significant increase in the δ_v behind SPW having similar increment as distance from SPW is decrease farther.

Figure 4.67 observes that the horizontal displacement in the sheet pile that happen due to braced deep excavation in both soil type one and soil type two is significantly increase with the same manner as depth of excavation is increase from the original value.

4.3.3. Bowles's Method (1986)

Bowles (1986) proposed a method for estimating the spandrel-type settlement profile induced by excavation. Bowles is one of the researchers who suggested a procedure to estimate deep excavation-induced settlements. For additional reference see table 2.1.

Case One: Soil Type One (Stiff Clay Soil)

The vertical ground settlement behind sheet pile in stiff clay soil having maximum depth of excavation (He) = 12 m is emphasized below.

Solve for volume of soil or the amount of deformed soil from finite element analysis. V_s/meter of wall width = A_s = 0.25 m^2.

The influence zone (D) using the method suggested by Caspe (1966) is adopted

$D = (H_e + H_d) \tan(45 - \phi'/2)$

$D_1 = (12 \text{ m} + 16 \text{ m}) \tan(45 - 37.3^o/2) = 13.87 \text{ m}$

Where: - He = 12 m, $\phi = 37.3^o$, for cohesive soil, H_d = B, where B = 16 m = H_d

Maximum ground Settlement can be estimated by the following

$\delta_{vm} = 4Vs/D = 4*0.25/13.87 = 0.0721$ implies $\delta_{vm1} = 0.0721$ m

By using similar procedure the $\delta_{v,max}$ behind the SPW in stiff clay soil having maximum depth of excavation (He) = 16 m and 20 m is emphasized as below respectively.

Where: D_2 = 15.85 m and D_3 = 17.83 m. Thus:

δ_{vm2} = 0.0631 m and δvm_3 = 0.0561 m

Then the settlement (δ_v) at a distance (d) from the supported wall for depth of excavation (He) = 12 m, 16 m and 20 m can be calculated as

$\delta_{vi} = \delta_{vmi}(l_x/D)^2$

Table 4.13 Bowels's settlement envelope for stiff clay soil (own construction)

$l_{x,1}$	$l_{x,1}/D_1$	$(l_{x,1}/D_1)^2$	δv_1 (m)	δv_2 (m)	δv_3(m)
0	1.000	1.000	0.0721	0.0631	0.0561

2	0.856	0.732	0.0527	0.0482	0.0442
4	0.712	0.507	0.0365	0.0353	0.0338
6	0.567	0.322	0.0232	0.0244	0.0247
8	0.423	0.179	0.0129	0.0155	0.0170
10	0.279	0.078	0.0056	0.0086	0.0108
12	0.135	0.0182	0.0013	0.0037	0.0060
13.87	0.00	0.0000	0.0000	0.00088	0.0026
15.85	0.00	0.0000	0.0000	0.00000	0.00062
17.83	0.00	0.0000	.0.0000	0.00000	0.00000

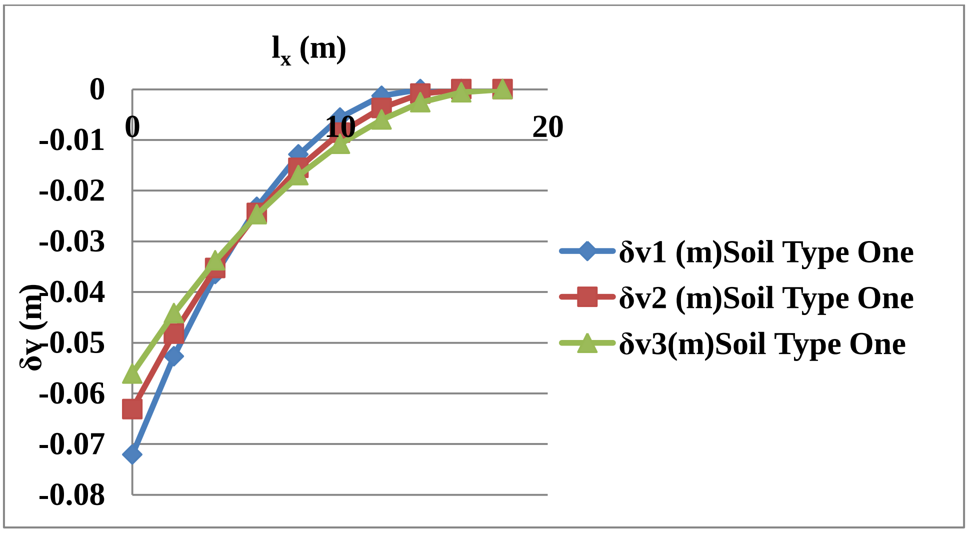


Figure 4.68 Bowels's maximum (δ_v (m)).vs. distance from excavation (l_x)

Case One: Soil Type Two (Sandy Soil)

The vertical ground settlement behind sheet pile walls in sandy soil having maximum depth of excavation (He) = 12 m is emphasized below.

Solve for volume of soil or the amount of deformed soil from finite element analysis

V_s/meter of wall width = A_s = 0.5 m^2

The influence zone (D) using the method suggested by Caspe (1966) is adopted

D = (He +H_d) tan (45 - ϕ'/2)

D = (12 m + 16.15 m) tan (45 - 37.3^o /2) = 13.94 m

Where: - He = 12 m, ϕ = 37.3^o

For cohesion less soil H_d = 0.5B tan (45+ ϕ'/2), H_d = 16.15 m = 0.5*16* tan (45+ 37.3'/2)

Maximum ground Settlement can be estimated by the following

δ_{vm} = 4Vs/D = 4*0.5/13.94 implies δ_{vm1} = 0.1435 m

By using similar procedure the maximum ground settlement behind SPW in sandy soil having maximum depth of excavation (He) = 16 m and 20 m is emphasized as below.

Where: D_2 = 15.92 m and D_3 = 17.91 m. Thus:

δ_{vm2} = 0.1256 m and δ_{vm3} = 0.1117 m

Then the settlement (δ_v) at a distance (d) from the supported wall with depth of excavation (He) = 12 m, 16 m and 20 m can be calculated.

$\delta_{vi} = \delta_{vmi} (l_x/D)^2$

Table 4.14 Bowels's settlement envelope for sandy soil (own construction)

$l_{x,1}$	$l_{x,1}/D_1$	$(l_{x,1}/D_1)^2$	δv_1 (m)	δv_2 (m)	δv_3 (m)
0.0	1.000	1.0000	0.1435	0.1256	0.1117
2.0	0.856	0.7327	0.1051	0.0959	0.0881
4.0	0.713	0.5084	0.0729	0.0705	0.0673
6.0	0.569	0.3244	0.0465	0.0487	0.0494
8.0	0.426	0.1816	0.0260	0.0310	0.0342
10	0.283	0.0801	0.0115	0.0173	0.0218
12	0.139	0.0193	0.0009	0.0077	0.0122
13.94	0.000	0.0000	0.0000	0.0018	0.0054
15.92	0.000	0.0000	0.0000	0.0000	0.0013
17.91	0.000	0.0000	0.0000	0.0000	0.0000

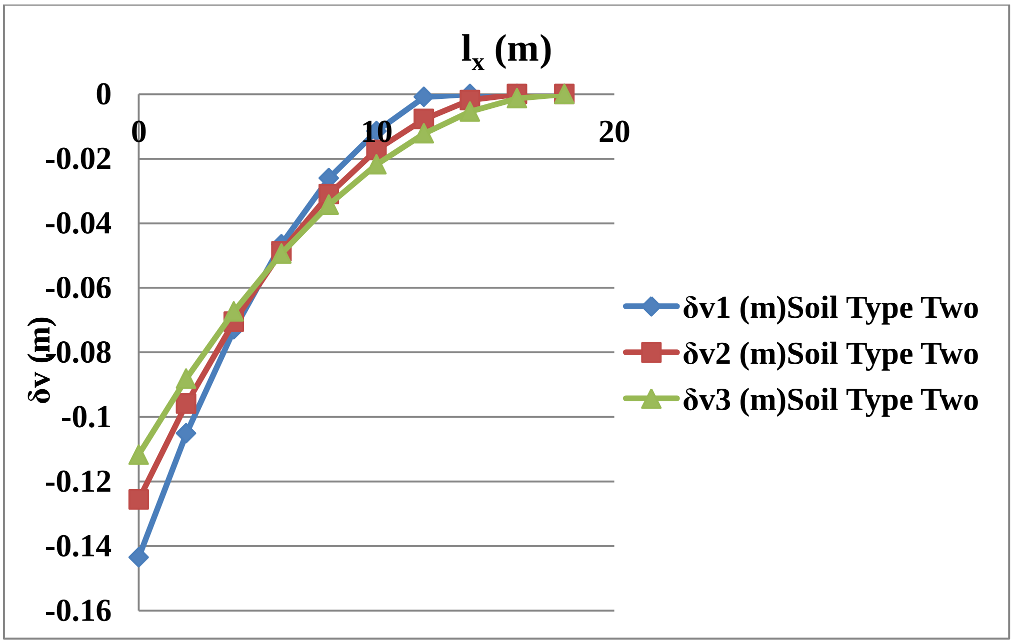


Figure 4.69 Bowels's maximum (δ_v (m)).vs. distance from excavation (l_x)

For reference see fig 2.13 from literature.

Case Two: Soil Type One and Two (Stiff Clay and Sandy Soil)

By using figure 2.6 which is the horizontal displacement of SPW determination figure for both soil we can get the below results.

Table 4.15 Bowles's horizontal displacement of SPW envelope for stiff clay and sandy soil (own construction)

H_e (m)	$\frac{\delta H}{H_e}$	δH (m)
12	0.2%	0.022
16	0.2%	0.031
20	0.2%	0.040

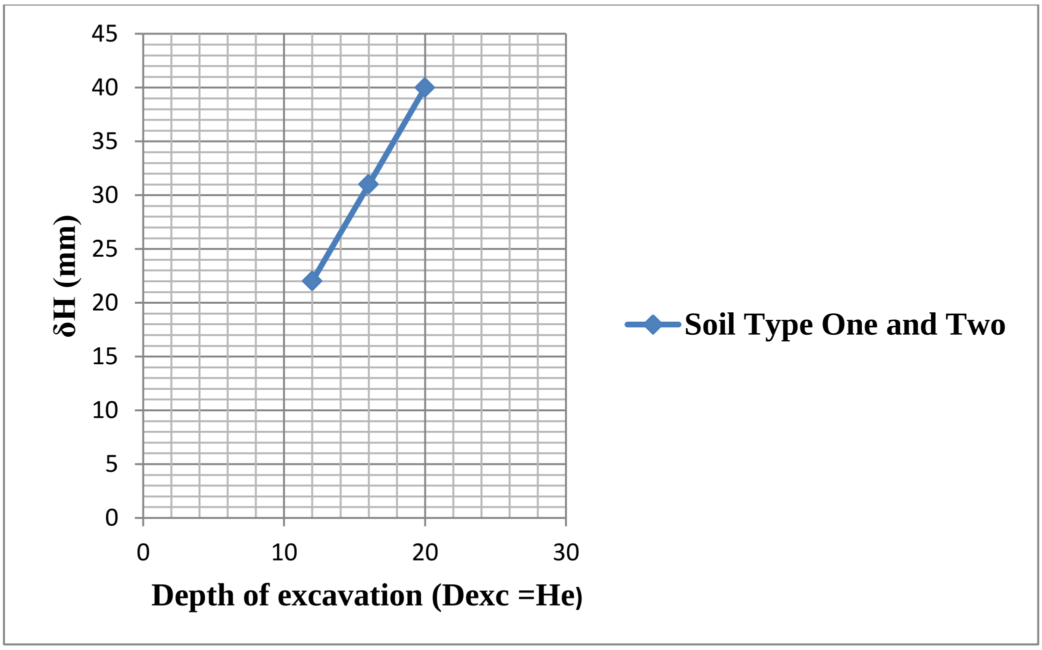


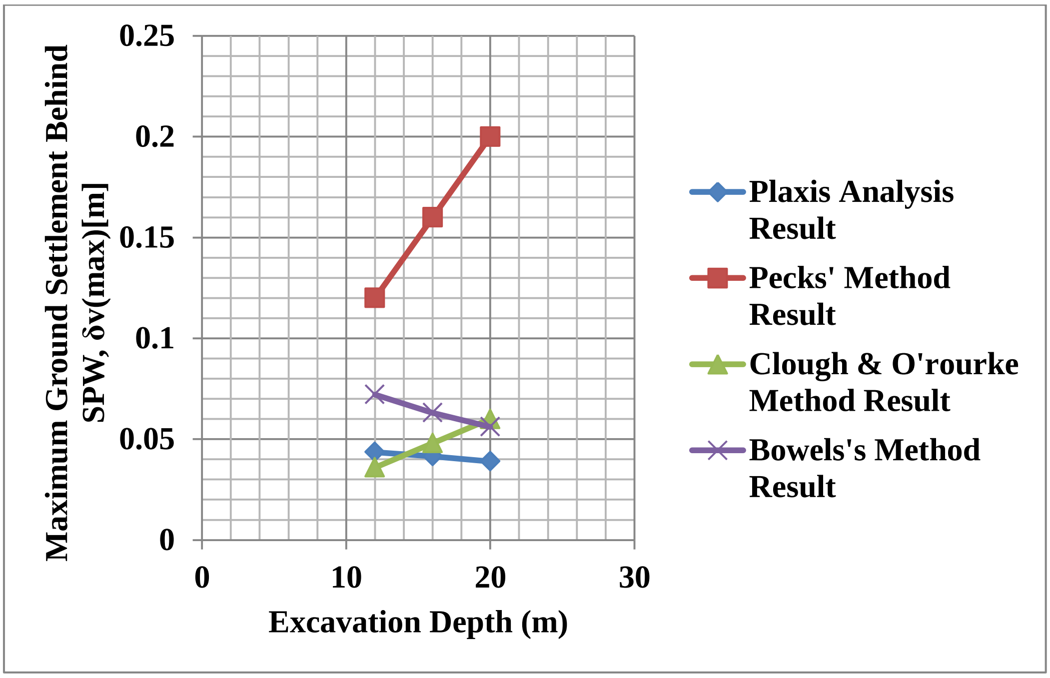
Figure 4.70 Bowels's maximum horizontal displacement of SPW (δH (mm)).vs. He (m)

We can observes from figure 4.68-69 that soil type two has much more vertical ground settlement behind SPW than soil type one. But in both soil type one and soil type two the vertical ground settlement is significantly decrease with increase in distance from SPW.

Figure 4.70 Observes that both soil type one and soli type two provides an extensive increase of the horizontal displacement in the sheet pile around braced deep excavation with increase in depth of excavation.

4.4. Comparison of Empirical Methods with Numerical Methods of Analysis

Here is the comparison of empirical method with PLAXIS-3D analysis to overcome the most important, economical and time bounded method for the analysis of displacement and settlement in the structure and the ground that results from braced deep excavation.



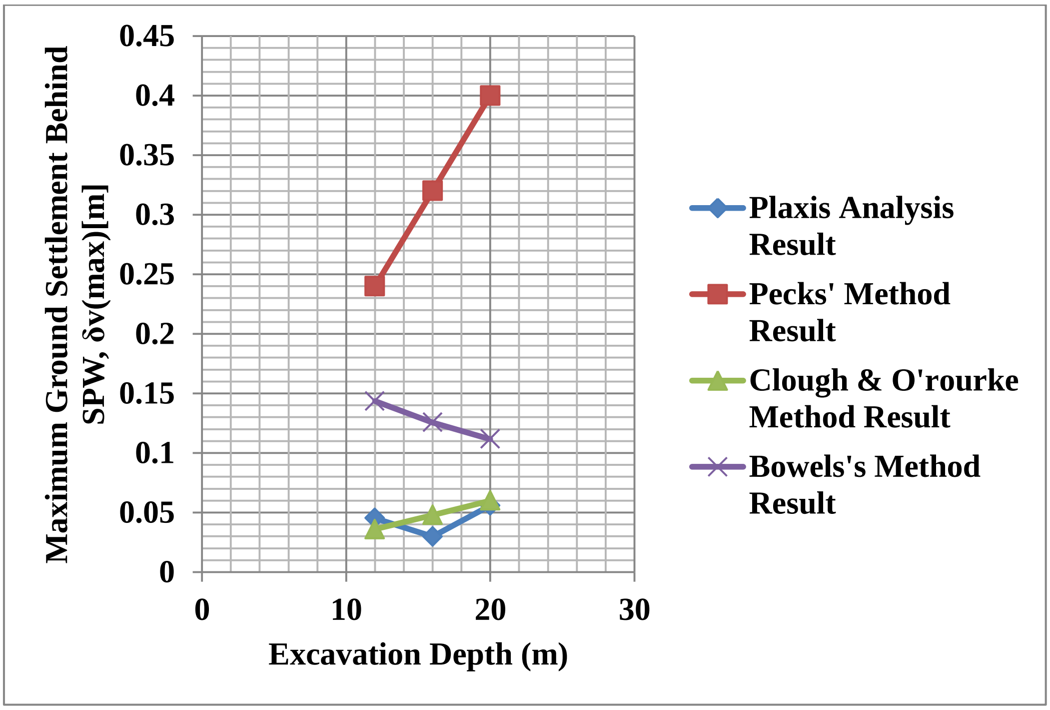
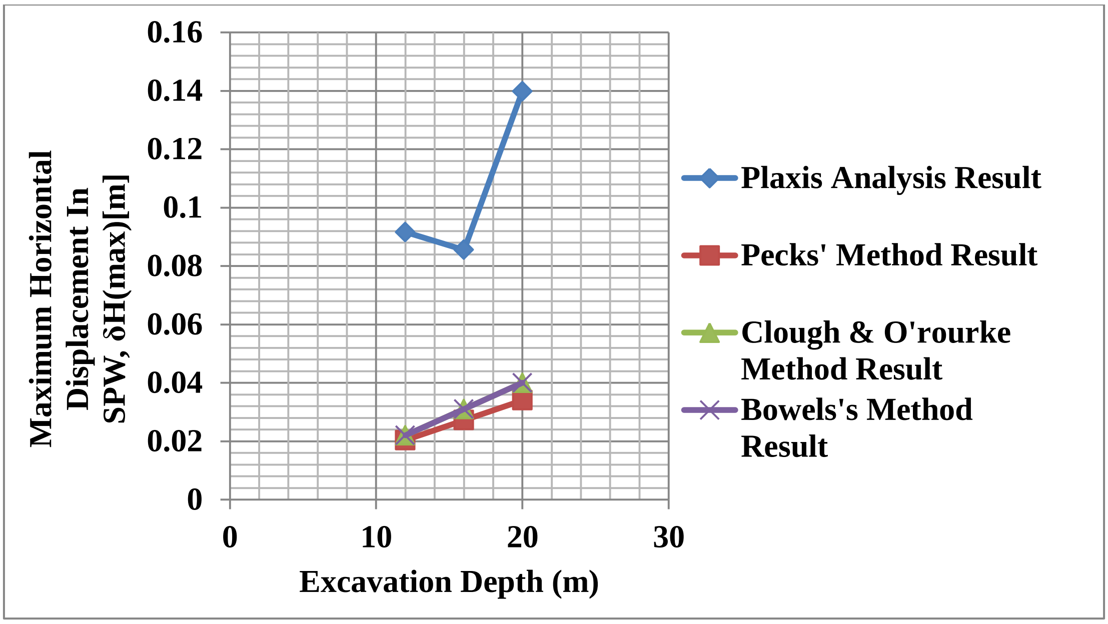


Figure 4.71 Comparison of Pecks's, Clough and O'rourkes and Bowels's $U_{y,max}$ behind the sheet pile in soil type one and two respectively with PLAXIS analysis results.



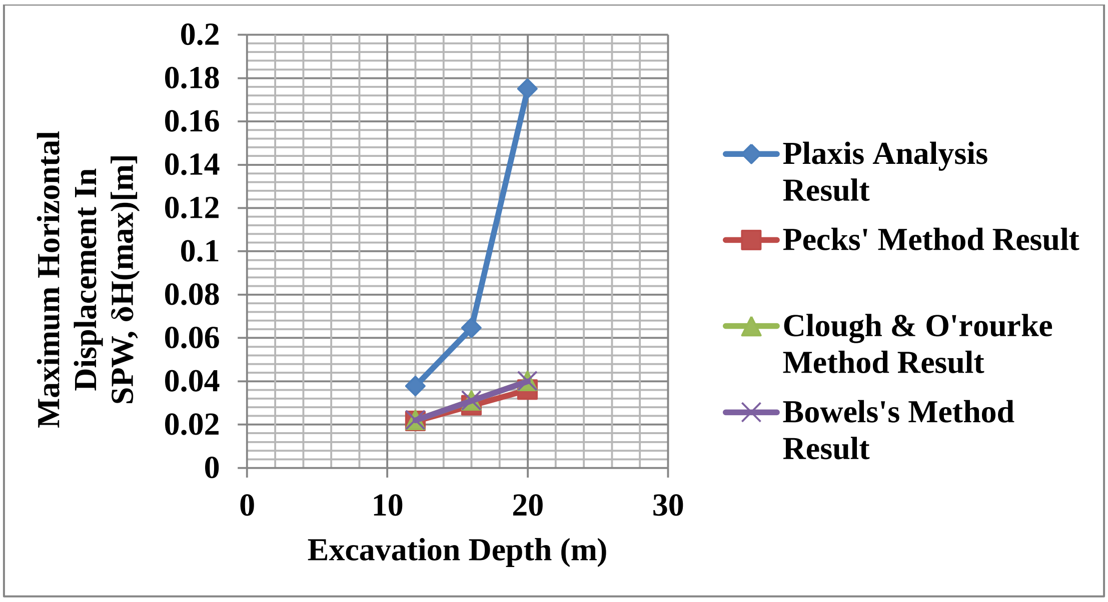


Figure 4.72 Comparison of Pecks's, Clough and O'rourkes and Bowels's max- horizontal displacement of SPW in soil type one and two respectively with PLAXIS analysis results.

Figure 4.72 Observes PLAXIS analysis has much more horizontal displacement in SPW around braced deep excavation in soil type one and two and extensively increases it as excavation depth is increases. However, Clough and O'rourkes and Bowels's horizontal displacement in the SPW is the second highest in soil type one and tow as compare to other methods. Generally the least result of the horizontal displacement in the sheet pile wall is Pecks' result as compare to other empirical methods of analysis.

5. SUMMARY AND CONCLUSION

5.1. Summary

The most extensive deep excavation-induced movements are a major concern for most underground construction projects in urban areas, because, it can cause possible collapse in adjacent ground and structures. Actually the main objective of this research is to investigate the settlement and total displacement in the ground and the displacement and bending moment in the SPW that might be happen due to braced deep excavation in three types of soil at Addis Ababa. However direct and quantitative predictions of ground movements are not easy tasks. This is because of the complexity of the system itself and the difficulty in modeling the wall installation and excavation processes. The 3D-finite element models are required for a realistic analysis of interaction between the soil and excavation retaining system. In this research, there are many different scenarios of braced deep excavations with different stiffness, excavation depth, embedment depth, strut and walling spacing, sheet pile wall thickness and in different soil types as well. Besides of these three empirical methods of analysis were used to provide the settlement and total horizontal displacement in the ground and sheet pile wall. In all cases the tieback anchored SPW is used as a retaining system. Finally as it is observed from both FE and empirical methods of analysis results it is decided as FEM is the most appropriate and accurate method for the analysis of braced deep excavation induced problem.

For the result of this study, the following conclusions can be made:

5.2. Conclusions

1. The sheet pile wall and ground deformations around braced deep excavations are extensively influenced by the soil type, embedment depth, excavation depth, thickness of sheet pile wall, strut and walling vertical spacing, sheet pile wall, strut and walling stiffness, tieback anchor including excavation techniques.

2. Excavation depth is significantly affects the performance of braced deep excavation in layered soil (Silty Clay, Clayey Silt Clayey Silt with Sand and Silty Sand soils) of Kirkos Sub City (Mexico and Lagahar area of Addis Ababa). Figure 4.16 - 17 observes that soil type one, two and three has an increase in horizontal displacement ($U_{x,max}$) in the SPW and ground as depth of excavation is increase from 12 m – 16 m – 20 m. The percentage increment and decrement of maximum BM (M1-1 and M2-2) in the SPW is 105.73%,

161.97% and 107.99%, 113.83% in soil type one, 37.27%, 101.93% and 59.83%, 58.68% in soil type two, 105.49%, 382.24% and 126.69%, 326.1% in soil type three as depth of excavation is increase as described above (Figure 4.19-20).

3. As it is observed from the results, a stiff sheet pile is required to control the $U_{x,max}$ in the SPW and the ground. Even though it is so, the increase in stiffness of sheet pile doesn't eliminate the $U_{x,max}$ completely; rather it is highly decreases as stiffness of SP wall is increases. Although the use of a stiff SPW results in reduction of $U_{x,max}$ in both SPW and the ground, it is virtually increase the maximum BM. The percentage increase of maximum BM (M1-1 and M2-2) in the SPW is 258.47%, 406.65% and 476.36%, 1046.55% in soil type one, 246.11%, 348.35% and 445.79%, 981.14% in soil type two, 209.78%, 255.36% and 323.17%, 559.39% in soil type three as stiffness of SPW is increases (0.3 – 0.6 – 0.9).

4. The horizontal displacement in both sheet pile and the ground is increase up to a considerable amount and then significantly decrease the $U_{x,max}$ of both SP wall and the ground which results from a consequence of braced deep excavation in Soil type one and two as embedment depth of SPW is increase. However the extensive increase and decrease in the $U_{x,max}$ of SPW and the decrease of $U_{x,max}$ in the ground by medium variation is observed due to braced deep excavation in Soil type three. Thus increasing of wall embedment depth decreases the wall and ground displacement and the ground settlement by decreasing the span moment while increasing the fixity at the bottom.

5. Applying tie-back struts and walling with the required space is significantly decrease the horizontal and vertical ground deformation and the sheet pile wall around braced deep excavation. However decreasing strut spacing beyond some limit doesn't reduce the deformation appreciably. Thus as strut and walling spacing is increase the horizontal and vertical deformation of the ground and sheet pile wall is also increase as well.

6. As compared with empirical methods which are developed based on many actual case studies, the PLAXIS software is underestimate the vertical ground settlement. This can be due to many reasons: thus are the accuracy of the modeling of the soil and supporting structure (which can have few drawbacks of their own), mesh coarseness, iterative precision of software and the different assumptions taken in staged construction process.

5.3. Recommendations

1. It is mostly important to obtain accurate estimates of soil parameters in order to obtain accurate estimates of horizontal and vertical ground deformation including bending moment and displacement in the sheet pile wall.

2. Prior to the commencement of any staged construction or deep excavation near adjacent ground or structures, it is recommended to record the existing condition of adjacent ground and properties before the construction of new project in order to perceive the problematic changes those results due to newly happen deep excavations and also to resolve any forwarding claims.

3. During deep excavation in stratified soils having low stiffness values, the structure used to support deep excavation must be stiff enough with adequate spacing of strut and walling in addition to tieback anchors as deep excavation in such type of soil would results a large deformations on adjacent grounds and properties.

4. The various investigations in this research could be better extend to other more advanced and more economical types of retaining systems and observes the different effects thereby. It could also be extend to the effects of lateral earth pressure from earth quake, dewatering and from any other natural hazards.

5. The use of Primary data's rather than secondary data's for the analysis process is most important for better understanding and improve the conclusions made on these research. Thus problem of braced deep excavation & its retaining system have further areas of study that will be investigate in the future.

6. If Triaxial tests will be conducted for the actual soil, installation of pressure gages and inclinometer on retaining structures are available, it will be very helpful to analyze in depth the behavior of retaining structures and that of adjacent ground besides better accuracy of the result by using any appropriate constitutive soil models

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Appendices

Appendices 1

A.1.The Hardening Soil Model (Isotropic Hardening)

In contrast to an elastic perfectly-plastic model, the yield surface of a hardening plasticity model is not fixed in principal stress space, but it can expand due to plastic straining. Distinction can be made between two main types of hardening, namely shear hardening and compression hardening. Shear hardening is used to model irreversible strains due to primary deviatoric loading. Compression hardening is used to model irreversible plastic strains due to primary compression in oedometer loading and isotropic loading. Both types of hardening are contained in the present model.

The Hardening-Soil model is an advanced model for simulating the behavior of different types of soil, both soft soils and stiff soils Schanz (1998). When subjected to primary deviatoric loading, soil shows a decreasing stiffness and simultaneously irreversible plastic strains develop. Such a relationship was first formulated by Kondner (1963) and later used in the well-known hyperbolic model (Duncan & Chang, 1970). The Hardening-Soil model, however, supersedes the hyperbolic model by far: First, by using the theory of plasticity rather than the theory of elasticity, second, by including soil dilatancy and third by introducing a yield cap. Some basic characteristics of the model are:

- Stress dependent stiffness according to a power law. Input parameter m
- Plastic straining due to primary deviatoric loading. Input parameter E_{50}^{ref}
- Plastic straining due to primary compression. Input parameter E_{oed}^{ref}
- Elastic unloading / reloading. Input parameters E_{ur}^{ref}
- Failure according to the Mohr-Coulomb model. Parameters c, ϕ and ψ

A basic feature of the present Hardening-Soil model is the stress dependency of soil stiffness. For Oedometer conditions of stress and strain, the model implies for example the relationship:

$$E_{oed} = E_{oed}^{ref}(\sigma/p^{ref})^{m} \quad \text{A.1}$$

$$\lambda^{*} = \frac{\lambda}{(1+e_o)} \quad \text{A.2}$$

In the special case of soft soils it is realistic to use m = 1. In such situations there is also a simple relationship between the modified compression index λ^{*}, as used in the PLAXIS-

-Soft Soil Creep Model and the oedometer loading modulus.

$$E_{oed}^{ref} = \frac{p^{ref}}{\lambda^*} \quad \text{A.3}$$

Where: p^{ref} is a reference pressure here it is considered that a tangent oedometer modulus at a particular reference pressure p^{ref}. Hence, the primary loading stiffness relates to the modified compression index λ^* or to the standard Cam-Clay compression index λ. Similarly, the unloading-reloading modulus relates to the modified swelling index κ^* or to the standard Cam-Clay swelling index κ. There is the approximate relationship:

$$E_{ur}^{ref} = \frac{3p^{ref}(1-2v_{ur})}{\kappa^*} \quad \text{A.4}$$

$$\kappa^* = \frac{\kappa}{(1+e_o)} \quad \text{A.5}$$

This relationship applies in combination with the input value m = 1.

Hyperbolic Relationship for Standard Drained Triaxial Test

A basic idea for the formulation of the Hardening-Soil Model is the hyperbolic relationship between the vertical strain, ε_1, and the deviatoric stress, q, in primary triaxial loading. Here standard drained triaxial tests tend to yield curves that can be described by:

$$-\varepsilon_1 = \frac{1}{2E_{50}}\frac{q}{1-q/q_a} \qquad \text{for } q < q_f \quad \text{A.6}$$

Where: q_a is the asymptotic value of the shear strength and E_i is the initial stiffness. E_i Is related to E_{50} by:

$$E_i = \frac{2E_{50}}{2-R_f} \quad \text{A.7}$$

This relationship is portrayed in Figure A.1.1. The parameter E_{50} is the confining stress dependent stiffness modulus for primary loading and is given by the equation:

$$E_{50} = E_{50}^{ref}\left(\frac{c\cos\phi - \sigma'_3\sin\phi}{c\cos\phi + p^{ref}\sin\phi}\right)^m \quad \text{A.8}$$

Where E_{50}^{ref} is a reference stiffness modulus, corresponding to the reference confining pressure p^{ref}. In PLAXIS, the default setting p^{ref} = 100 KN/m^2 stress units is used. The actual stiffness depends on the minor principal stress, σ_3' which is the confining pressure in a triaxial test. Please note that σ_3' is negative for compression. The amount of stress dependency is given by the power m. In order to simulate a logarithmic stress dependency, as observed for soft clays, the power should be taken equal to 1.0. The ultimate deviatoric stress q_f and the quantity q_a are defined as:

$$q_f = (c\cot\phi - \sigma'_3)\frac{2\sin\phi}{1-\sin\phi} \quad \text{and: } q_a = \frac{q_f}{R_f} \quad \text{A.9}$$

$$E_{ur} = E_{ur}^{ref}\left(\frac{c\cot\phi - \sigma'_3}{c\cot\phi + p^{ref}}\right)^m \quad \text{....A.10}$$

Where:- E_{ur}^{ref} is the reference Young's modulus for unloading and reloading.

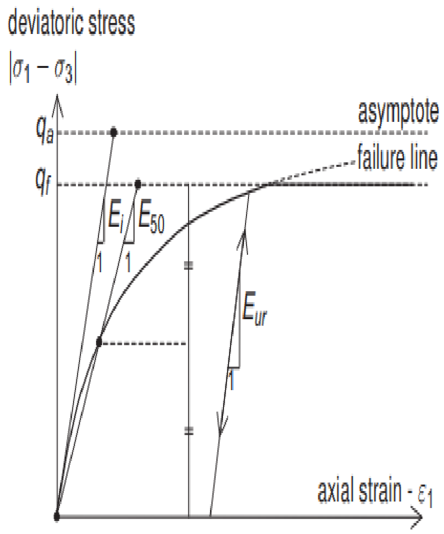

Figure A.1.1 Hyperbolic stress-strain relation in primary loading for a standard drained triaxial test (PLAXIS MATERIAL)

Parameters of the Hardening Soil Model

Some parameters of the present hardening model coincide with those of the non-hardening Mohr Coulomb Model. These are the failure parameters C, ϕ and ψ.

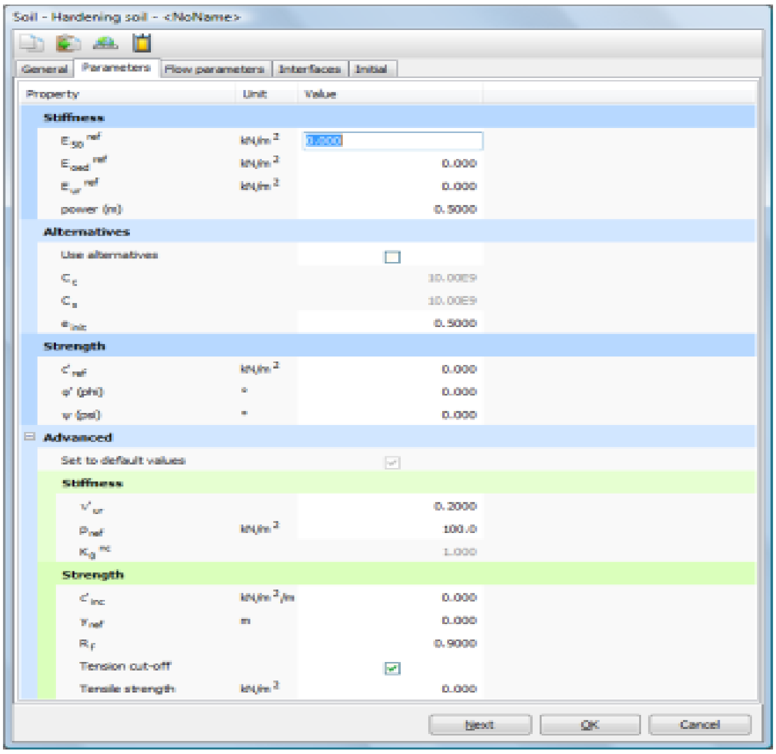

Figure A.1.2 Basic Parameters for the Hardening - Soil Model (PLAXIS MATERIAL)

Failure parameters as in Mohr-Coulomb Model:

C : (Effective) cohesion [KN/m^2]

Φ : (Effective) angle of internal friction [^{o}]

Ψ : Angle of dilatancy [^{o}]

Basic parameters for soil stiffness:

E_{50}^{ref} : Secant stiffness in standard drained triaxial test [KN/m^2]

E_{oed}^{ref} : Tangent stiffness for primary oedometer loading [KN/m^2]

E_{ur}^{ref} : Unloading / reloading stiffness (default E_{ur}^{ref} = $3E_{50}^{ref}$) [KN/m^2]

M : Power for stress-level dependency of stiffness [-]

Advanced parameters (it is advised to use the default setting):

v_{ur} : Poisson's ratio for unloading-reloading (default v_{ur} = 0.2) [-]

P^{ref} : Reference stress for stiffness's (default P^{ref} = 100 KN/m^2) [KN/m^2]

K_o^{nc} : K_o – value for normal consolidation (default K_o^{nc} = 1- sin ϕ) [-]

R_f : Failure ratio q_f/q_a (default R_f = 0.9) [-]

$\sigma_{tension}$: Tensile strength (default $\sigma_{tension}$ = 0 stress units) [KN/m^2]

C_{inc} : As in Mohr-Coulomb model (default C_{inc} = 0) [KN/m^3]

Instead of entering the basic parameters for soil stiffness, alternative parameters can be entered. These parameters are listed below:

C_c : Compression index [-]

C_s : Swelling index or reloading index [-]

e_{init} : Initial void ratio [-]

Stiffness Moduli E_{50}^{ref}, E_{oed}^{ref}, E_{ur}^{ref} and Power (m)

The advantage of the Hardening-Soil model over the Mohr-Coulomb model is not only the use of a hyperbolic stress-strain curve instead of a bi-linear curve, but also the control of stress level dependency. When using the Mohr-Coulomb model, the user has to select a fixed value of Young's modulus whereas for real soils this stiffness depends on the stress level. It is therefore necessary to estimate the stress levels within the soil and use these to obtain suitable values of stiffness. With the Hardening-Soil model, however, this cumbersome selection of input parameters is not required. Instead, a stiffness modulus E_{50}^{ref} is defined for a reference minor principal stress of $-\sigma_3' = p^{ref}$. As a default value, the program uses p^{ref}= 100 KN/m^2.

As some PLAXIS user's area familiar with the input of shear moduli rather than the above stiffness moduli, shear moduli will now be discussed. Within Hooke's theory of elasticity conversion between E and G goes by the equation E = 2(1+ν) G. As E_{ur} is a real elastic stiffness, one may thus write E_{ur} = 2(1+ ν)G_{ur}, where G_{ur} is an elastic shear modulus. PLAXIS allows for the input of E_{ur} and v_{ur} but not for a direct input of G_{ur}. In contrast to E_{ur}, the secant modulus E_{50} is not used within a concept of elasticity. As a consequence, there is no simple conversion from E_{50} to G_{50}. In contrast to elasticity based models, the elasto-plastic Hardening-Soil model does not involve a fixed relationship between the (drained) triaxial stiffness E_{50} and the oedometer stiffness E_{oed} for one-dimensional compression. Instead, this stiffness can be inputted independently. Having defined E_{50} by Eq. (A.11) it is now important to define the oedometer stiffness. Here we use the equation:

$E_{oed} = E_{oed}^{ref}\left(\frac{c\cot\phi - \sigma_1'}{c\cot\phi + p^{ref}}\right)^m$……………………………………………………………..A.11

Where E_{oed} a tangent stiffness modulus as is indicated in Figure (A1.3). Hence, E_{oed}^{ref} is a tangent stiffness at a vertical stress of .σ'$_1$ = $p^{ref} = \frac{-\sigma_3'}{K_o^{nc}}$ (PLAXIS MATERIAL). Note that we basically use σ'$_1$ rather than σ_3' and that we consider primary loading.

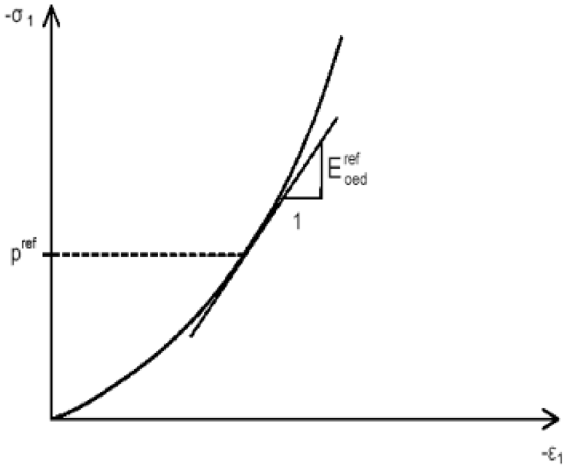

Figure A.1.3 Definition of E^{ref} in oedometer test results (PLAXIS MATERIAL)

When undrained behavior is considered in the Hardening Soil Model the drainage type should preferably be set to Undrained (A). Alternatively, Undrained (B) can be used in case the effective strength properties are not known or the undrained shear strength is not properly captured using Undrained (A). However, it should be noted that the material loses its stress-dependency of stiffness in that case. Undrained (C) is not possible since the model is essentially formulated as an effective stress model.

Appendix 2

A.2.1. Tables of Result Analysis

A.2.1.1. Table of Result Analysis for the Effect of Change in Soil Types (own construction)

Effect of Change in Soil Type																
Depth, Thickness and Deformation in m Whereas Bending Moment in KNm-m																
D_{ep}	Thick-ness	Soil type one					Soil type two					Soil type three				
		$U_{x,soil}$	$U_{x,wall}$	U_y	M1-1	M2-2	$U_{x,soil}$	$U_{x,wall}$	U_y	M1-1	M2-2	$U_{x,soil}$	$U_{x,wall}$	U_y	M1-1	M2-2
12	0.3	0.091	0.098	0.081	279.5	63.19	0.133	0.199	0.107	394.2	81.05	0.0488	0.05595	0.040	108.1	30.84
	0.6	0.083	0.0955	0.066	802	254.2	0.094	0.1285	0.063	957.4	272.2	0.0399	0.04475	0.045	240.7	104.4
	0.9	0.069	0.0805	0.054	1186	507.6	0.069	0.092	0.044	1297	496.6	0.034	0.03772	0.045	398.7	191.8
16	0.3	0.08675	0.0926	0.081	269.9	69.48	0.0607	0.1046	0.043	114.2	26.93	0.0611	0.09111	0.032	113.1	31.25
	0.6	0.0829	0.0887	0.08	692.2	230.6	0.05189	0.09481	0.042	288.4	133.7	0.0468	0.07582	0.031	255.9	113.8
	0.9	0.0796	0.08273	0.077	1254	548.2	0.0493	0.08554	0.041	483.4	297.1	0.0385	0.06453	0.029	420.6	243
20	0.3	0.1067	0.1199	0.078	472.4	55.21	0.082	0.1671	0.044	379.5	29.7	0.3196	0.3985	0.132	596.8	111.8
	0.6	0.101	0.1124	0.073	1221	263	0.0612	0.1519	0.041	934	132.4	0.218	0.267	0.064	1252	361.3
	0.9	0.0929	0.1038	0.068	1921	577.8	0.0544	0.1398	0.039	1322	291.4	0.143	0.175	0.056	1524	625.4

A.2.1.2. Table of Result Analysis for the Effect of Change in D_{exc} (own construction)

Effect of Change In Depth of Excavation															
Depth, t = 0.9 and Deformation in m Whereas Bending Moment in KNm-m															
D_{exc}	Soil type one					Soil type two					Soil type three				
	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}
12	0.0691	0.0805	0.0537	1186	507.6	0.06944	0.0916	0.0436	1297	496.6	0.034	0.03772	0.04523	398.7	191.8
16	0.0796	0.08273	0.0769	1254	548.2	0.0493	0.0855	0.0415	483.4	297.1	0.03854	0.06453	0.02982	420.6	243
20	0.093	0.1038	0.0678	1921	577.8	0.0544	0.1398	0.039	1322	291.4	0.1428	1.175	0.05585	1524	625.4

A.2.1.3. Table of Result Analysis for the Effect of Change in Stiffness (own construction)

Effect of Change In Stiffness of Sheet Pile Wall															
D_{exc} = 16, t = 0.9 and Deformation in m Whereas Bending Moment in KNm-m															
Stiffness	Soil type one					Soil type two					Soil type three				
	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}
1	0.07958	0.08273	0.077	1254	548.2	0.0493	0.0855	0.0415	483.4	297.1	0.03854	0.06453	0.029	420.6	243
2	0.0667	0.06974	0.066	3561	1549	0.0453	0.07543	0.0483	1281	718.9	0.0379	0.05463	0.035	1100	606.8
3	0.06373	0.06037	0.066	4237	2061	0.04243	0.0707	0.0482	1564	925.1	0.03169	0.04973	0.035	1143	789.8

A.2.1.4. Table of Result Analysis for the Effect of Change in D_{emb} (own construction)

Effect of Change In Wall Embedment Depth

D_{emb}, D_{emb}, t and Deformation in m Whereas Bending Moment in KNm-m																
D_{exc},t	D_{emb}	Soil type one					Soil type two					Soil type three				
		$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}
16m, t = 0.3	4	0.0867	0.09256	0.081	269.9	69.48	0.061	0.1046	0.043	114.2	26.93	0.06115	0.09111	0.0321	113.1	31.25
	6.5	0.1059	0.1092	0.086	274.7	50.21	0.0961	0.1218	0.066	279	47.81	0.06379	0.09567	0.03365	95.84	15.38
	9	0.0851	0.08918	0.082	268.9	32.98	0.1074	0.1622	0.078	223.2	28.25	0.06578	0.1035	0.0355	85.55	14.40
20m, t = 0.3	4	0.1067	0.1199	0.078	472.4	55.21	0.0823	0.1671	0.044	379.5	29.7	0.3196	0.3985	0.1316	596.8	111.8
	6.5	0.2923	0.473	0.257	594.1	126	0.3643	0.4891	0.317	783.2	138.9	0.2127	0.2226	0.07076	314	50.47
	9	0.2127	0.2364	0.151	501.2	67.29	0.2672	0.2902	0.15	635.5	62.49	0.2424	0.2596	0.1285	564.8	76.12
16, t = 0.9																
	4	0.0796	0.08273	0.077	1254	548.2	0.0493	0.08554	0.042	483.4	297.1	0.03854	0.06453	0.0298	420.6	243
	6.5	0.849	0.08735	0.080	916.5	418.7	0.0648	0.08667	0.062	541.8	359	0.04357	0.07665	0.0317	329.2	161.7
	9	0.07697	0.07856	0.078	1140	572.5	0.0854	0.1441	0.068	402.8	183.4	0.04677	0.09187	0.03197	292.5	146.8
20m, t = 0.9	4	0.0929	0.1038	0.068	1921	577.8	0.0544	0.1398	0.039	1322	291.4	0.143	0.175	0.05585	1524	625.4
	6.5	0.1558	0.2376	0.14	2622	997.9	0.1727	0.2127	0.123	2564	854.2	0.1385	0.2041	0.09353	2146	713.1
	9	0.121	0.1618	0.106	1597	672	0.125	0.1452	0.1024	1778	511.6	0.1074	0.1208	0.07496	1353	442.6

A.2.1.5. Table of Result Analysis for the Effect of Change in Strut and Walling Stiffness (own construction)

Effect of Change In Strut and Walling Stiffness															
D_{exc} = 16, t = 0.6 and Deformation in m Whereas Bending Moment in KNm-m															
Strut and Walling Stiffness	Soil type one					Soil type two					Soil type three				
1	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}
	0.08234	0.08496	0.082	556.8	314.3	0.05211	0.0951	0.043	262.1	133.6	0.0601	0.08261	0.035	317.9	137.1
2	0.08295	0.08868	0.08	692.2	230.6	0.05189	0.09481	0.042	288.4	133.7	0.0468	0.07582	0.031	355.9	113.8
3	0.08221	0.08349	0.81	681.6	315.5	0.05142	0.09451	0.042	314.1	133.8	0.0585	0.08126	0.035	372.7	127.4

A.2.1.6. Table of Result Analysis for the Effect of Change in Strut and Walling Vertical Spacing (own construction)

Effect of Change In Strut and Walling Spacing															
D_{exc} = 20, t = 0.9 and Deformation in m Whereas Bending Moment in KNm-m															
Strut and Walling Spacing	Soil type one					Soil type two					Soil type three				
3	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}	$U_{x,soil}$	$U_{x,wall}$	U_y	M_{1-1}	M_{2-2}
	0.0929	0.1038	0.068	1921	577.8	0.05437	0.1398	0.039	1322	291.4	0.1428	0.175	0.056	1524	625.4
4	0.1897	0.6589	0.552	1075	1599	0.229	0.4139	0.321	730.4	964.5	0.104	0.6103	0.047	697.7	1040
5	0.1926	0.5928	0.491	1374	1640	0.118	0.4551	0.035	875.4	1007	0.1087	0.6532	0.053	992.2	1202

A.2.2. Procedure Used For Simulation

The following procedure is the major stapes has used for the analysis of tieback anchored sheet pile wall problem that might be happen as a result of braced deep excavation in Addis Ababa soil. Thus after the arrangement of all dimensions, Soil and Structural element data, it has been proceeding like below:

Open PLAXIS-3D Input version 2013→ Start New Project→ select Model→ Check units and input the excavation dimension→ Create borehole at (0, 0, 0) → Open the material set window→ Add four soil layers having different colors with bottom levels at -4 m, -8 m, -14 m and -40 m for when excavation depth is 20 m→ Set the head in the borehole column to Z = -25 m→ After completing the whole soil model save it→

Start with Structural element→ Open material window→ Select Beam element→ Select Hardening Soil Model→ change color→ Input the Strut and Walling parameters→ Select Node to Node Anchors→ Select Hardening Soil Model→ change color→ Input the parameters→ Select Embedded Pile→ select grout body→ Select Hardening Soil Model→ change color→ Input the parameters→ Select sheet pile walls→ Select Hardening Soil Model→ change color→ Input the parameters→ Save it→

Create a surface between (30 m, 26 m , 0), (30 m, 42 m, 0), (60 m, 42 m, 0), (60 m, 26 m, 0)→ Extrude the surface to Z = -3 m, -8 m, -16 m and -24 m→ Save it→

draw Walling b/n: first (30 m, 26 m, -3 m), (30 m, 42 m, -3 m), (60 m, 42 m, -3 m), (60 m, 26 m, -3 m) and (30 m, 26 m, -3 m) second (30 m, 26 m, -6 m), (30 m, 42 m, -6 m), (60 m,42 m, -6 m), (60 m, 26 m, -6 m) and (30 m, 26 m, -6 m) third (30 m, 26 m, -9 m), (30 m, 42 m, -9 m), (60 m, 42 m, -9 m), (60 m, 26 m, -9) and (30 m, 26 m, -9 m) fourth (30 m, 26 m, -12 m), (30 m, 42 m, -12 m), (60 m, 42 m,- 12 m), (60 m, 26 m, -12 m) and (30 m, 26 m, -12 m) fifth (30 m, 26 m, -15 m), (30 m, 42 m, -15 m), (60 m, 42 m, -15 m), (60 m, 26 m, -15 m) and (30 m, 26 m, -15 m) sixth (30 m, 26 m, -18 m), (30 m, 42 m, -18 m), (60 m, 42 m, -18 m), (60 m, 26 m, -18 m) and (30 m, 26 m, -18 m)→

Draw Struts b/n: first (35 m, 26 m, -3 m) and (35 m, 42 m, -3 m) second (35 m, 26 m, -6 m) and (35 m, 42 m, -6 m) third (35 m, 26 m, -9 m) and (35 m, 42 m, -9 m) fourth (35 m, 26 m, -12 m) and (35 m, 42 m, -12 m) fifth (35 m, 26 m, -15 m) and (35 m, 42 m, -15 m) sixth (35 m, 26 m, -18 m) and (35 m, 42 m, -18 m)→ Draw Node to Node Anchor b/n: first (30 m, 30 m, -3 m) and (21 m, 30 m, -18 m) second (60 m, 30 m, -3 m) and (69 m, 30 m, -

18 m)→ then by using array add another anchor on both side using 4 m space between them→

Draw grouted part between: first (21 m, 30 m, -18 m) and (19.8 m, 30 m, -20 m) second (69 m, 30 m, -18 m) and (70.1 m, 30 m, -20 m)→ then by using array add another grout on both side using 4 m space between them→ Save it→

Create plate on the whole surfaces→ select sheet pile on the whole surface→ Create positive interface on the whole surfaces→ Create negative interface on the whole surfaces → Click on geometry and decompose the whole geometry→ Generate Mesh→ coarse→ Ok→ View Mesh→ Select the Geometry→ Water levels→ Save it→

Staged Construction→ select the shoring face at the top of geometry→ Add phase One→ Double click on phase one→ General→ Installation of Pile→ Ok→ Activate Plate and Interface→ Add phase two→ Double click on phase two→ General→ Excavation one→ Ok→ Select shoring part and inactivate soil→ Add phase three→ Double click on phase three→ General→ Place structure element→ Ok→ Activate beam, embedded pile and node to node anchor→ Double click on node to node anchor→ Make true→ Adjust pressure→ Add phase four→ Double click on phase four→ General→ Excavation two→ Ok→ Select shoring part and inactivate CS→ Click on shoring face again→ Make dry water condition in place of global level→ by using similar procedure we can add excavation and finally save it→

Calculate→ View calculation results→ Open PLAXIS-3D output→ we can get the needed output of deformation and settlement in the soil and structure by simple technic.

Appendix 3

A.3. PLAXIS - 3D Models Output for Deformation of the Ground and Sheet Pile Wall

A.3.1.When depth of excavation (**D**$_{\mathbf{exc}}$ = 20 m), t = 0.9 m and **D**$_{\mathbf{emb}}$ = 4 m for soil type one

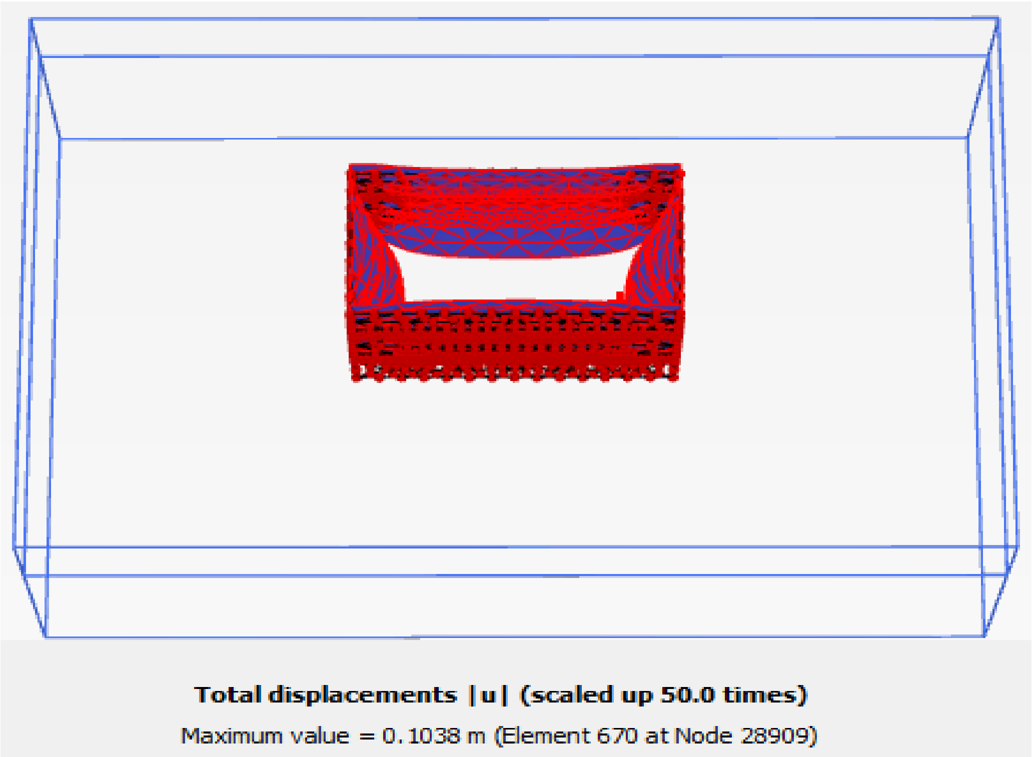

Figure A.3.1 [U] of the sheet pile wall due to braced deep excavation in soil type one

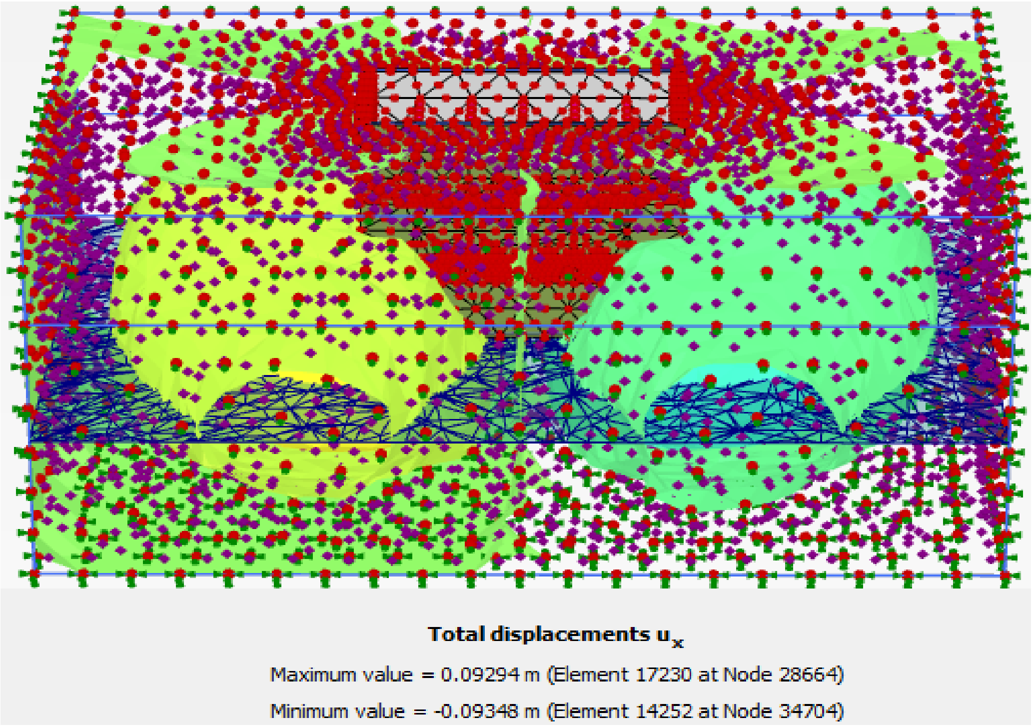

Figure A.3.2 $U_{x,max}$ of the ground around braced deep excavation in soil type one

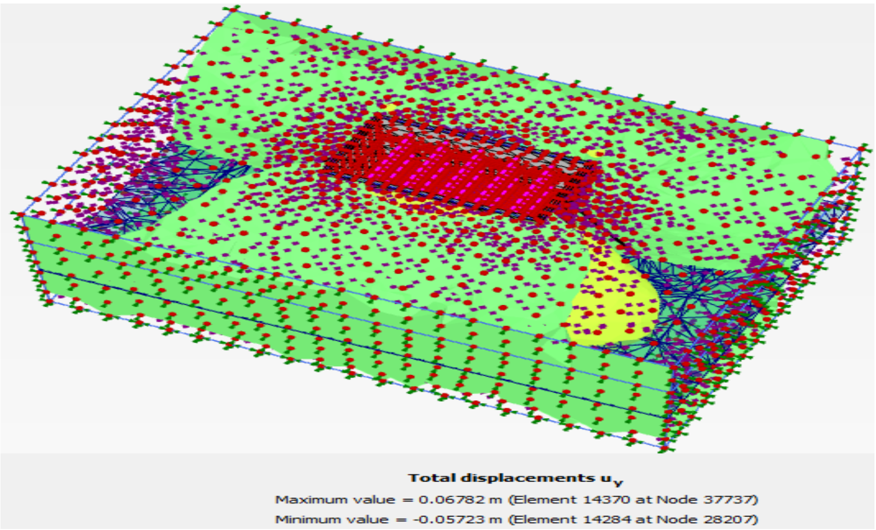

Figure A.3.3 $U_{y,max}$ behind SPW that results from braced deep excavation in soil type one

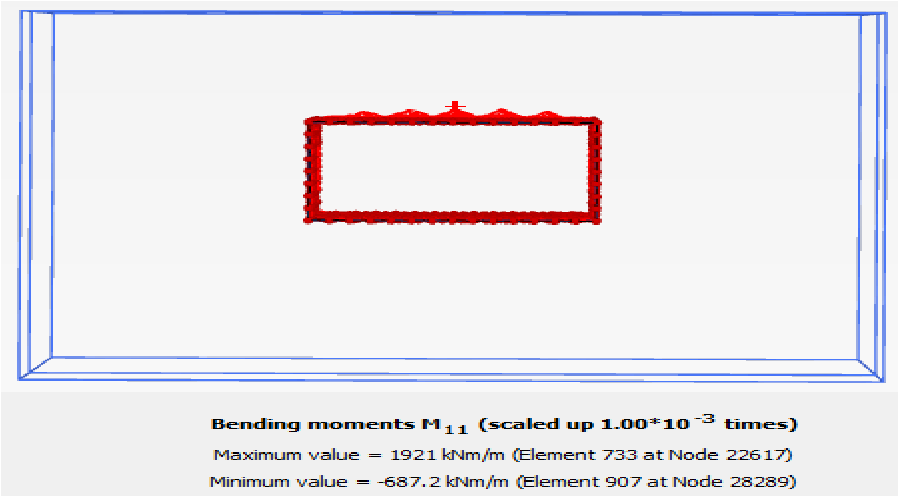

Figure A.3.4 BM_{max} (M1-1) in the SPW due to braced deep excavation in soil type one

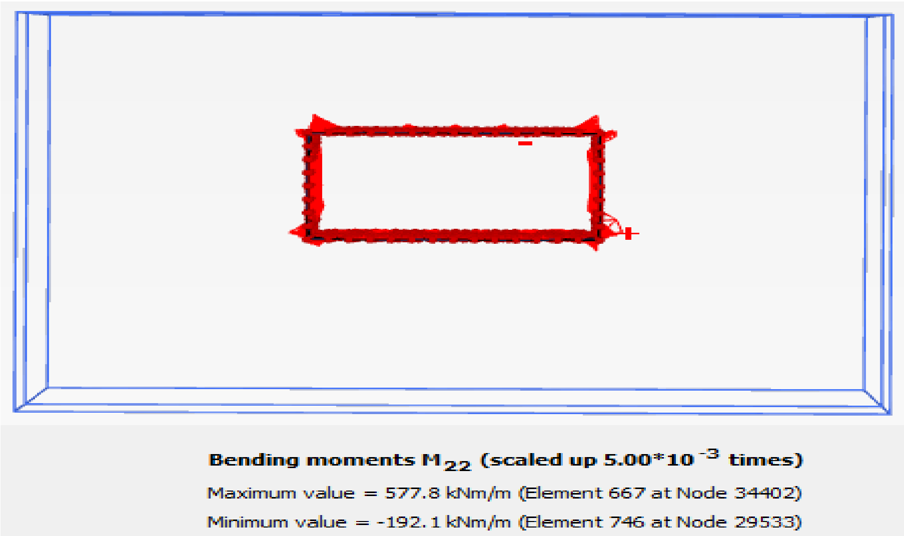

Figure A.3.5 BM_{max} (M2-2) in the SPW due to braced deep excavation in soil type one

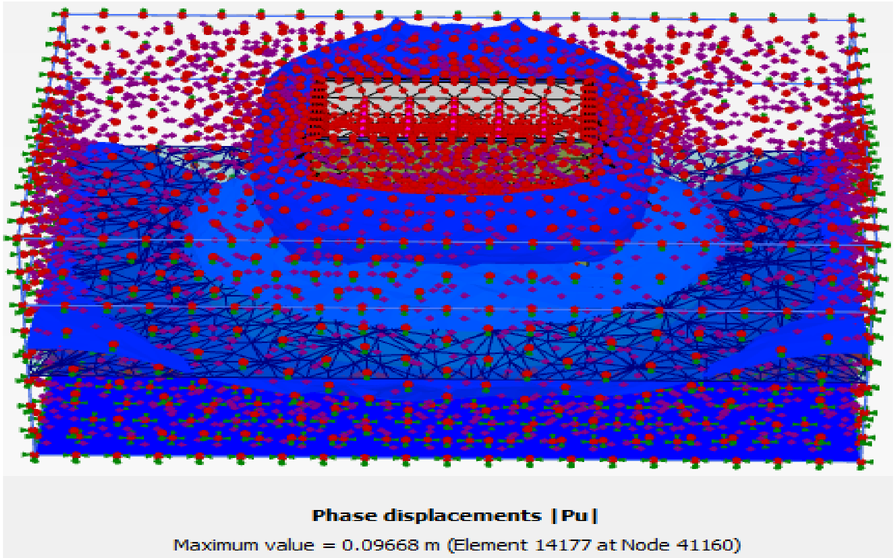

Figure A.3.6 $P_{u,max}$ of the ground due to braced deep excavation in soil type one

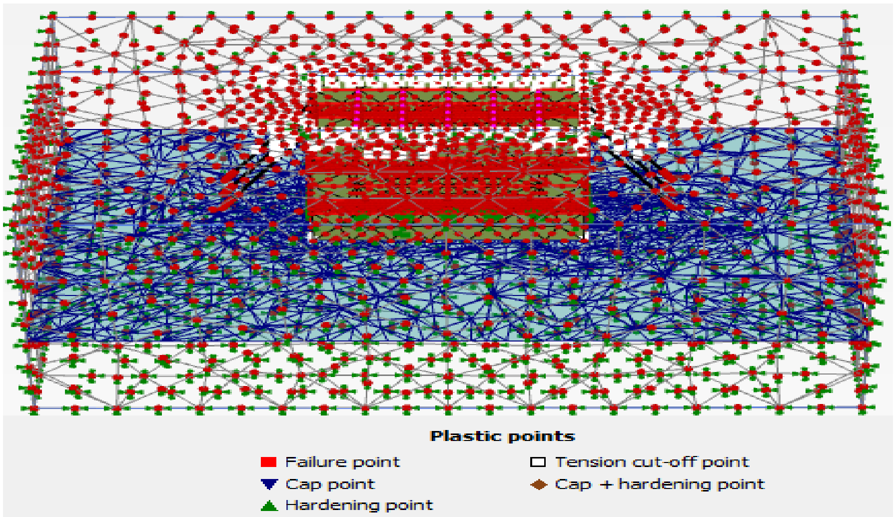

Figure A.3.7 Plastic points that result from braced deep excavation in soil type one model

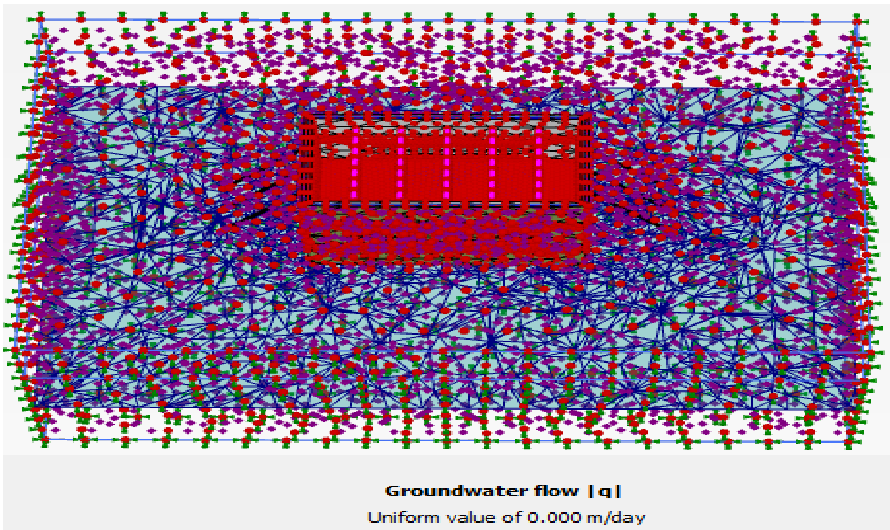

Figure A.3.8 Ground water that happen from braced deep excavation in soil type one