ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

Comparison of Existing Lateral Earth Pressure Theories with FEM Software for Braced Deep Excavations and Its Design Implication

By

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A Thesis Submitted to the School of Graduate Studies, Addis Ababa University, in partial fulfillment of the requirements for the Degree of

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Advisor

Dr.-Ing HENOK FIKERE

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Declaration

I declare that the work in the project entitled "Comparison of Existing Lateral Earth Pressure Theories with FEM Software for Braced Deep Excavations and Its Design Implication" has been performed by me in the Department of Civil Engineering, Faculty of Technology, under the supervision of Dr. Ing Henok Fikere .The information derived from literature has been dully acknowledged in the text and list of references provided. No part of this project was previously presented for another degree at any university**.**

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Abstract

This thesis presents the comparison of existing lateral earth pressure theories and results of earth pressures found from finite element software Plaxis (2D) for braced cuts for different soil samples. In addition to this, the comparison of deformation, ground surface settlement, bending moment and factor of safety of braced cut structures for different samples are studied.

To study the behavior of braced cuts, different models were developed and analyzed using existing theories and Plaxis software. This was accomplished by parametric study conducted on a number of alternative arrangements of variables under consideration.

The parametric study indicates that increasing the spacing of struts and decreasing the stiffness of struts increases the lateral pressure. On the other hand, increasing or decreasing depth of embedment and stiffness of sheet pile do not have significant influence on lateral pressure.

Finally, the comparison of existing lateral earth pressure theories and Plaxis (2D) indicates that existing pressure theories especially Terzaghi's and Peck's theories overestimate lateral pressure. In addition to this, strut loads, bending moments, sheet pile wall deformation and ground surface settlement calculated from existing theories are larger than these obtained from Plaxis (2D).

Key words: Deep excavation, Finite element, Plaxis 2D, strut loads, ground surface settlement.

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Symbols and Abbreviations

E Young modulus of soil

FEM Finite Element method

FHWA Federal highways Authority

G Shear modulus of building component

Ls Strut spacing

MCM Mohr coulomb method

R_{intr} interface element parameter (parameter reduction factor to account the wall friction and adhesion)

$\boldsymbol{\delta}_{\mathbf{V\,(max)}}$ Maximum ground settlement

$\boldsymbol{\delta}_{\mathbf{H\,(max)}}$ Maximum horizontal deformation

SPT Standard penetration test

H height of strut

B width of strut

t_1 flange thickness

t_2 web thickness

1. Introduction

1.1 Background of the study

Deep excavation is conventionally defined as an excavation in soil deeper than 6m. Excavation for underground construction can be done either with side slope or with vertical side and bracings. In urban areas, where sufficient space is not available, vertical cuts without spacing should be provided. To design braced excavation, it will be very useful for geotechnical engineers to estimate the lateral earth pressure.

Rankine's and Coulomb's theories of earth pressure cannot be directly used for the computation of lateral earth pressure on flexible walls, as those theories are applicable to rigid retaining walls rotating about their bases. The sheeting and bracing system is somewhat flexible, and rotation takes place at top of the wall (Arora, 2004).

The general wedge theory also does not provide the relationships required for estimating the lateral pressure variation with depth. The method of earth pressure calculation has been developed by Terrzaghi based on observation of actual struts in full scale excavation.

The lateral earth pressure on a braced cut is dependent on the type of soil, construction method and type of equipment used. Each strut should be designed for the maximum load to which it may be subjected. Therefore, the braced cuts should be designed using apparent pressure diagrams that are envelopes of all the pressure diagrams determined from measured strut loads in the field (Das, 2007).

The results of field studies can be used for the design of deep excavation supporting system. Apparent pressure envelopes are simple to use 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).

With the application of finite element methods, geotechnical aspects such as the soil properties, construction sequences, surcharge loads, water pressure and details of structures of braced excavation can be simulated in a more realistic manner.

In this study, the existing lateral earth pressure theories of deep excavations for different types of soils in comparison with finite element analysis will be presented. The program used in this study is PLAXIS and a program that has in the last decades gained high acclaim in finite element analysis of geotechnical problems. The conventional hand calculation methods will be evaluated with PLAXIS and the results studied.

1.2 Statement of the problem

Nowadays a number of multi-story buildings are being constructed in Addis Ababa with basements for parking, storage, etc. Any deep excavation in congested urban areas leads to some movement of the surrounding ground and any structure situated within the zone of influence will be affected.

These excessive ground movements can also affect the functionality of structures in terms of efficiency and durability. The cause of this excessive ground movement around the excavation area might be due to the lateral earth pressure, soil structure interaction, water pressure, etc.

The design of supporting systems of braced cuts mainly has been made based on the existing theories of earth pressure. These theories have been developed based on field observation of full scale excavations. This method by itself may exaggerate or undermine the lateral earth pressure distribution. However, there are many factors which will affect the lateral pressure distribution such as staged construction, surcharge load, etc.

1.3 Objectives

1.3.1 General objectives

- To compare lateral earth pressure calculated using empirical formulas of braced excavations with that of finite element methods.

1.3.2 Specific objectives

- To study the main factors which have major influences on lateral earth pressure on deep excavation such as constitutive models, surcharge load, water pressure, etc.
- To show in detail the effects of staged construction on the lateral earth pressure distribution
- To study the implications of the Finite element and empirical methods on the design of braced cuts.

1.4 Methodology

The methodology employed consists of the following:

- The existing theories in the area of deep excavation were examined for different types of soils.
- Representative deep excavation models which are used for the FEM analysis such as types of soil, depth of staged construction, strut stiffness, wall thickness, strut arrangement, depth of excavation and the embedded depth of the wall were determined.
- Analysis of models using existing theories and finite element software performed independently.

- Comparison of results found from the two approaches done. Based on this result the implications on the design of braced cut studied.

1.5 Scope of the study

No partial factors have been used for soil strength or surcharges. This is motivated by the fact that this will lead to the same percentile change of the resultant forces for the different methods and does not help in the evaluation of them.

The earth pressure that is being evaluated is the lateral earth pressure that act upon the vertical sheet pile wall. In addition to this, the ground surface is horizontal and the geometry of excavation is symmetrical and plain strain. This simplification has been made to reduce the risk of low quality of mesh in finite element software.

1.6 Significance of the study

The results of this study will be used in design of braced cuts for deep excavations. Especially for geotechnical engineers who are in the design practice help to analyze factors which affect braced cuts of deep excavation using the classical theories and FEM.

Besides, the study will increase the awareness of geotechnical engineers how lateral earth pressures are affected by staged construction, soil condition, types and approaches of analysis.

1.7 Research Question

The two important questions in this thesis are:

- How much deviation is there between calculation of earth pressure with existing earth pressure theories and finite element methods in braced deep excavations?
- Does this deviation have significant implication on design of supporting systems of braced deep excavation?

1.8 Structure of the Thesis

This thesis has six chapters that discuss various aspects of lateral earth pressure. Chapter one explains the background of the research and spells out what the research intends to achieve. Chapter two deals with literature review that provides a general understanding of previous studies and theories related to the research. Chapter three discusses the description of the cases and conditions that are used in the research. Chapter four discusses results of the study using Plaxis and empirical formulas. The last chapter draws conclusion of the research and provides recommendations on outstanding issues.

2. Literature review

2.1 Introduction

During design of earth retaining structures whether the structure is flexible or rigid determination of lateral earth pressure is a key factor. Lateral earth pressure on a retaining structure depends on number of factors. These are: the physical properties of the soil, drainage problems, the time-dependent nature of the soil, amount of surcharge load, the interaction between the soil and retaining structure, location of ground water table, etc.

When the soil is on the verge of failure as it is defined by Mohr rupture envelopes lateral earth pressures will be developed. The stress induced in the soil is progressive so that it is difficult to produce a plastic equilibrium state in a soil mass.

Lateral earth pressure classified into three different categories based on the movement of soil behind the retaining structure. Earth pressure at rest refers to lateral earth pressure caused by unyielding wall preventing earth from any lateral movement. When a wall is allowed to move away from the retained soil, the soil will expand laterally and shearing resistance developed within the soil mass and act opposite to the direction of expansion and results decrease in lateral pressure refers to Active earth pressure. When the wall move to the retained soil, the soil will be compressed laterally and shearing resistance acting opposite to the lateral compression refers to Passive earth pressures.

2.2 Lateral Earth Pressures Theories

2.2.1 Lateral Earth Pressures at Rest

Any type of soil whether it is sand or clay, normally consolidated in the ground under the natural condition of no lateral deformation (i.e., vertical compression only), no friction between the retaining structures and soil under an incremental application of vertical load experiences a condition known as the earth pressure at rest. The value of the coefficient of the earth pressure at rest, K_o, can be estimated by Jacky's equation (2.1) for both cohesive and cohesion less soils.

$$K_o = \frac{\sigma_h^{'}}{\sigma_v^{'}} = 1 - \sin\phi^{'} \quad \text{...} \quad (2.1)$$

For normally consolidated clay, K_o is typically in the range of 0.55 to 0.65; for sands, the typical range is 0.4 to 0.5. For lightly over consolidated clays $(OCR \leq 4)$, K_o may reach a value up to 1; for heavily over consolidated clays (OCR>4), K_o values may range up to or greater than 2. (FHWA, 1999)

2.2.2 Active and Passive Lateral Earth Pressure

Rankine (1857) proposed a solution for lateral earth pressure in retaining walls based on theory of plastic equilibrium. He assumed that the soil is isotropic and homogeneous; the soil is dry and cohesion less. According to Rankine the active and passive failure zone will be Mohr coulomb failure theory and the angle for active failure surface is at (45+ϕ/2) and for the passive failure surface is at (45-ϕ/2).

Active case:

$$\sigma_a' = \sigma_v' K_a - 2C' \sqrt{K_a} \quad \text{..} \quad (2.2)$$

Where:

$$K_a = \tan^2\left(45 - \frac{\phi'}{2}\right)$$

Passive case:

$$\sigma_p' = \sigma_v' K_p + 2C' \sqrt{K_p} \quad \text{..} \quad (2.3)$$

Where:

$$K_p = \tan^2\left(45 + \frac{\phi'}{2}\right)$$

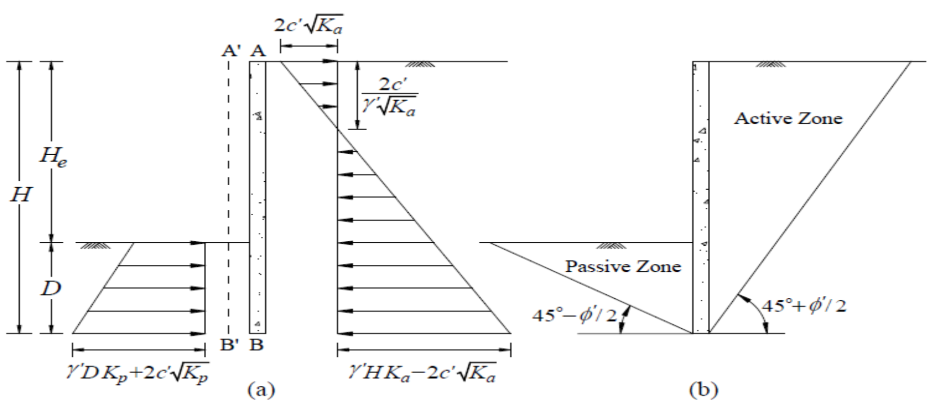


Figure 2-1 : (a) Rankine's pressure distributions, and (b) Passive and Active Zones

Coulomb (1776) presented a theory for active and passive earth pressure by assuming the existence of wall friction, failure surface is plane, cohesion less soil and homogenous and the wedge between the wall and failure surface is rigid material, and the weight of the wedge, the reaction of the soil and the reaction of the wall are in equilibrium.

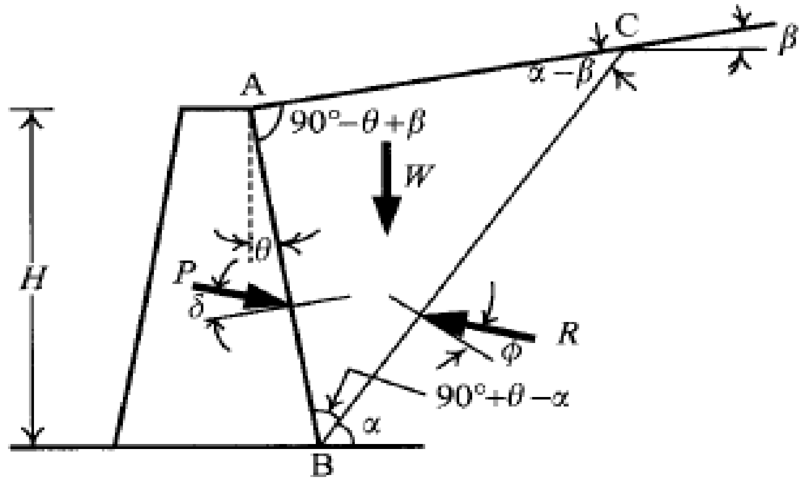


Figure 2-2: Coulomb's pressure distributions

Rankine's earth pressure theory cannot consider friction or adhesion between the retaining wall and soil on the other hand Coulomb's earth pressure theory can do this. Though the two theories are differently based, the obtained earth pressure is the same when both assume a vertical and smooth wall back. Both theories assume the failure surfaces as planes, which do not conform reality and as the result of this these theories cannot predict real earth pressures.

Caquot and Kerisel (1948) assumed that the failure was an elliptical curved surface, James and Bransby (1970) assumed it to be log spiral, Terzaghi and peck (1967) assumed it to be another log spiral. Basically speaking, the better assumed function of the curved surface represents the actual failure surface, the closer the derived earth pressure will be to the real earth pressure. None of these theories can be expressed in a simple equation. Among them, Caquot and Kerisel (1948) assumed failure surface is quite close to the actual one and the derived coefficients of earth pressure are expressed by many tables to be readily applied. (OU, 2006).

For problems of excavation, considering that the active earth pressure is usually the main force leading to the failure of the retaining walls. Caquot and Kerisel's active earth pressure should be adopted for analysis and design since it is regarded as most close to the real active earth pressure. Columb's coefficient of active earth pressure smaller than Caquot and Kerisel's but can be used for analysis and design. For conservative reasons, Rankine's coefficient of active earth pressure is recommended for it is the largest among the three without significant difference from Caquot-Kerisel's (Ou,2006).

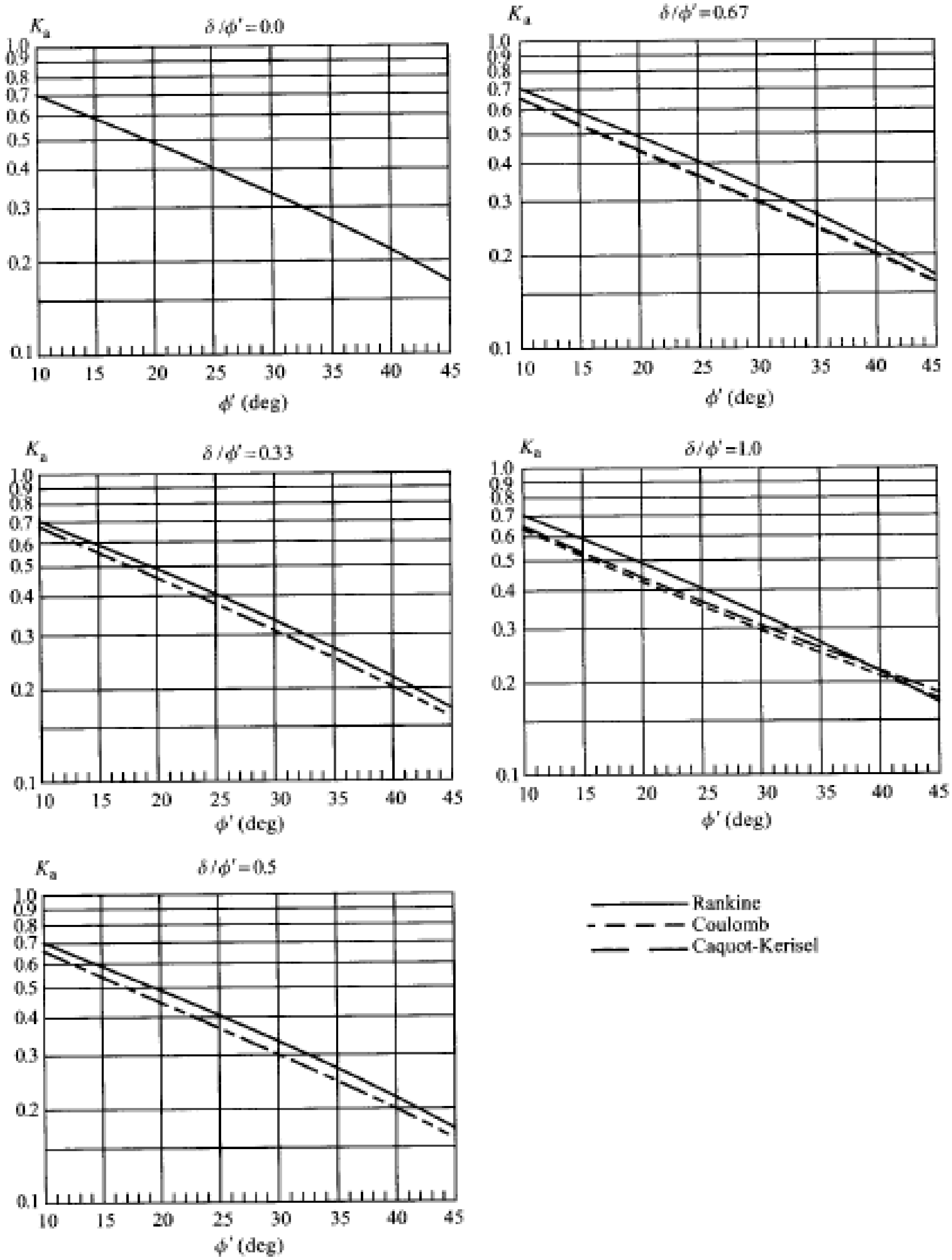


Figure 2-3 : coefficients of Rankin's, coulomb's and caquot-Kerisel's active earth pressure (horizontal component $K_{a,h}$=$K_a$cosδ)

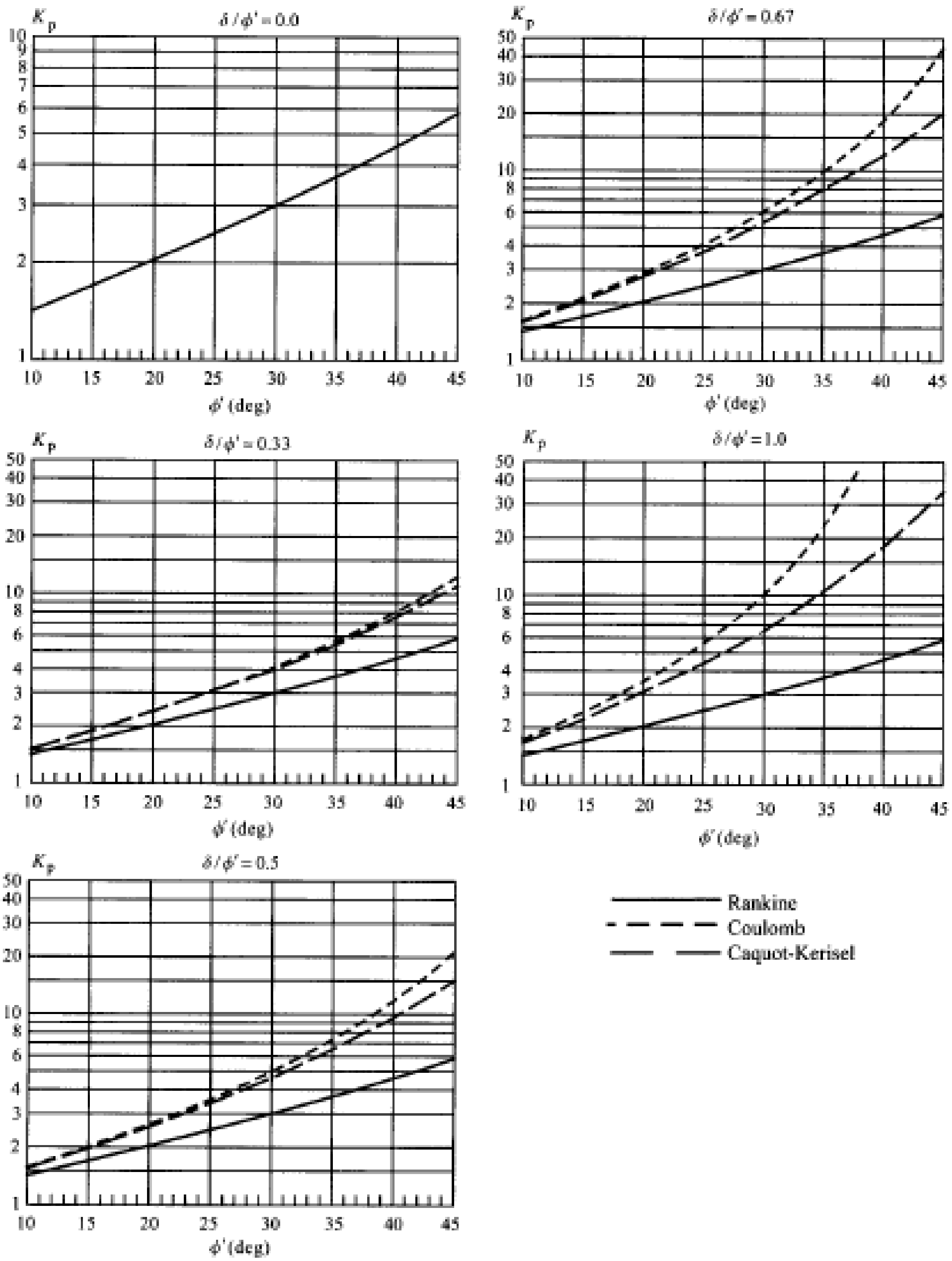
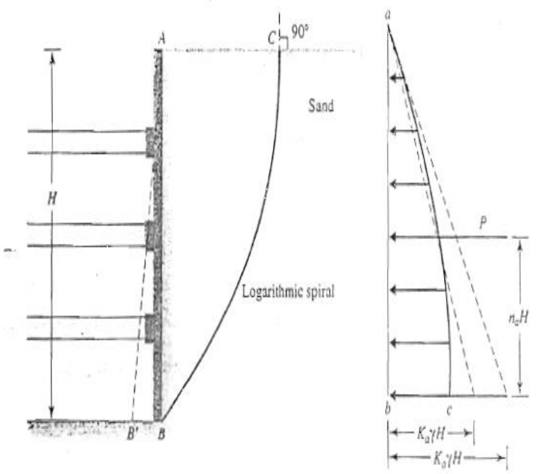
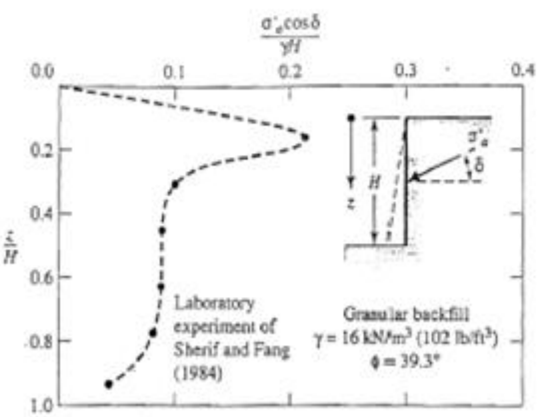


Figure 2-4: coefficients of Rankin's, coulomb's and caquot-Kerisel's passive earth pressure (horizontal component $K_{p,h}$=Kpcosδ)

However, in the case of the braced cut wall is rotating about the top of the excavation, thus the earth pressure is somewhat parabolic rather than increasing linearly from soil surface and it cannot be predicted using Rankine's or Coulomb's theory. This difference is due to the arching effect, preloading, and incremental excavation and strut installation. Following this there are many field measurements of earth pressure in braced excavation done and different authors prepared semi empirical formulas **(**Das, 2007).





(a) (b)

Figure 2-5: (a) Earth pressure in braced Excavation (Das, 1999): b) Field Measurement of Earth Pressure in Braced Excavation (Sheriff and Fang, 1984)

Figure 2-5: (a) shows a typical comparison between at rest earth pressure, active earth pressure, and the earth pressure in braced excavation. The earth pressure in braced excavation is not at at-rest, neither active condition, and is parabolic in shape. Figure 2-5: (b) demonstrates the non-hydrostatic earth pressure distribution behind braced wall based on the laboratory experiments by Sherif and Fang (1984).

2.3 Semi empirical formulas of Lateral Earth pressure

2.3.1 Terrzaghi and Peck (1967), Tschebotarioff's pressure envelopes

The apparent earth pressure diagrams developed by Terrzaghi and Peck (1967) and Peck (1969), provide the framework for the diagrams that will be recommended in subsequent sections. These diagrams represent the envelope of pressures back-calculated from field measurements of strut loads in internally braced excavations. These diagrams produce conservative design loads, implying that if a strut load would be equivalent to the calculated load from the apparent pressure diagram at that location, the other strut loads would necessarily be less than that calculated from the apparent pressure diagram.

The Terrzaghi and Peck apparent earth pressure envelopes are rectangular or trapezoidal in shape and were developed based on the following factors:

- The excavation is assumed to be greater than 6 m deep and groundwater is assumed to be below the base of the excavation for sands, and for clays, its position is not considered important. Specifically, loading due to water pressure was not considered in these analyses.
- The soil mass is assumed to be homogeneous and soil behavior during shearing is assumed to be drained for sands and undrained for clays.
- The loading diagrams apply only to the exposed portion of the wall and not the portion of the wall embedded below the bottom of the excavation.

Peck (1969) provided the envelope apparent lateral pressure diagrams for design of cuts in sand.

$$\sigma_a = 0.65\gamma H K_a \quad \text{(2.4)}$$

Peck (1969) introduced a term, called stability number γH/cu, where γ and cu are the unit weight and un drained shear strength of the soil adjacent to the excavation. In case of soft-to-medium clay is applicable when the stability number exceeds 4, and a plastic zone is expected to develop near the bottom of the excavation. The pressure is the larger of

$$\sigma_a = \gamma H\left(1 - \frac{4C}{\gamma H}\right) \text{ and } \sigma_a = 0.3\gamma H \quad \text{(2.5)}$$

In the case of stiff clay where the stability number is less than 4 and the clay is strong enough to resist the load transferred from the structure, most of the soil is in the elastic zone and the case of stiff clay is applicable. The pressure envelope will be

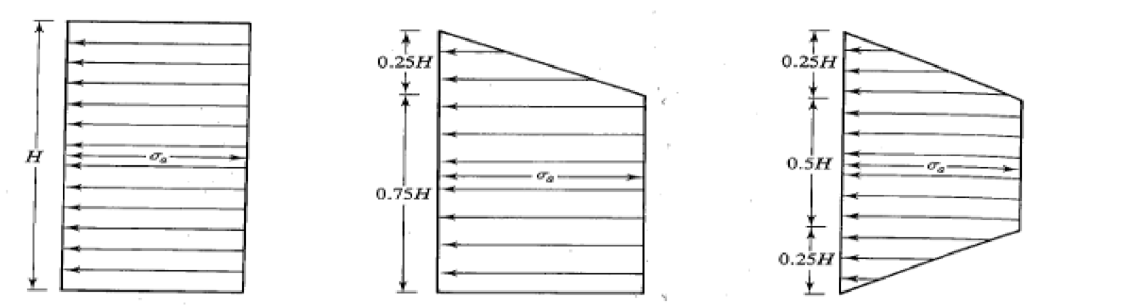


Fig2-6 (a) Peck's (1969) apparent pressure envelope for cuts sand

Fig2-6 (b) Peck's (1969) apparent pressure envelope for in soft to medium clay

Fig2-6 (b) Peck's (1969) apparent pressure envelope for cutsin stiff clay

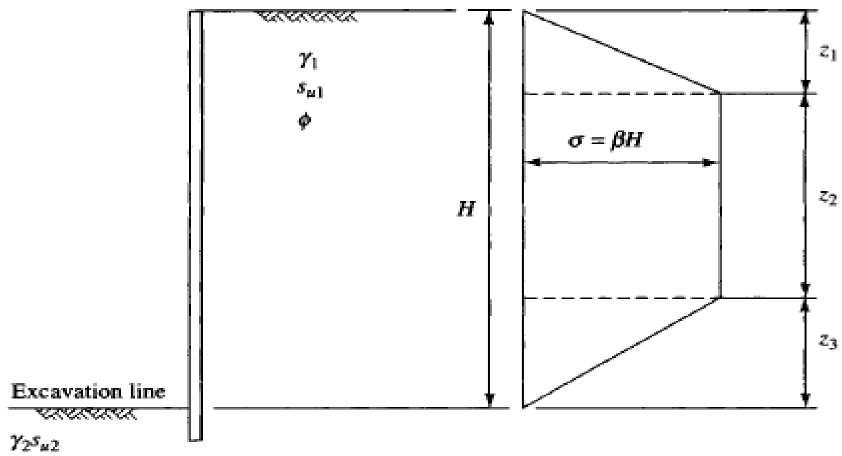
Figure 2-6 : Apparent pressure envelope of Peck (1969)

$$\sigma_a = 0.2\gamma H \text{ to } \sigma_a = 0.4\gamma H \text{ .... (With an average of } \sigma_a = 0.3\gamma H\text{)} \quad \text{(2.6)}$$

$$C_a = \frac{1}{n}\left(C_1 H_1 + C_2 H_2 + \text{........} C_n H_n\right) \quad \text{(2.7)}$$

$$\gamma_a = \frac{1}{n}\left(\gamma_1 H_1 + \gamma_2 H_2 + \text{........} \gamma_n H_n\right) \quad \text{(2.8)}$$

Tschebotarioff et. al. (1973) modified the Peck pressure diagram for certain combinations of $c_u/\gamma H$ of clay since Peck's method could produce $K_a=0$ which was not realistic. Tschebotarioff observed that for most cohesion less soils $0.65K_a=0.25$ for all practical purposes, since Φ is usually approximated (Boweles, 1997).



Soil Type	Author	z_1	z_2	z_3	β
Sand	P	0	1.0	0	$0.65_\lambda K_a$
Sand	T	0.	0.7	0.2	0.25_λ
Soft-to-Medium Clay	P	0.25	0.75	0	λK_{ap}*
Temp. Support Medium Clay	T	0.6	0	0.4	0.3λ
Stiff fissured Clay	P	0.25	0.50	0.25	K_{ap}†
Perm. Support Medium Clay	T	0.75	0	0.25	0.375λ

* $K_{ap} = 0.4$ to 1.0
† $K_{ap} = 0.2$ to 0.4
Source: P = Peck (1969); T = Tschebotarioff (1973).

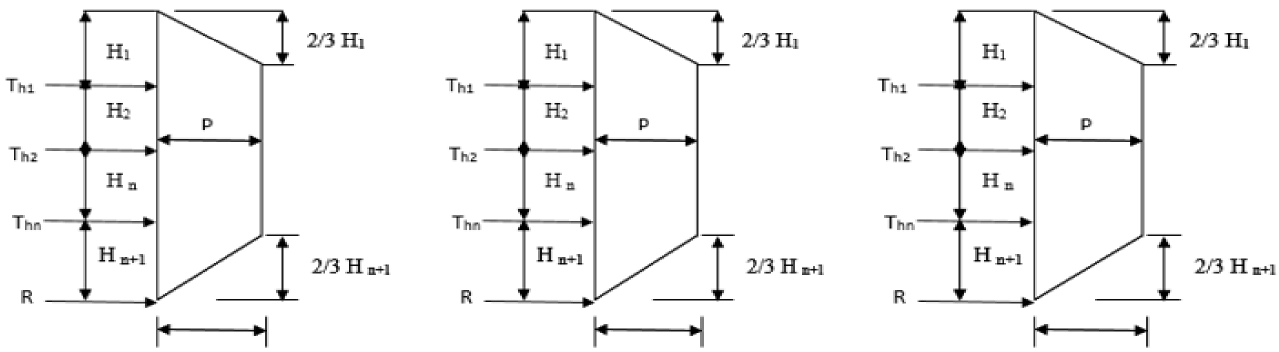
Figure 2-7: Comparison of Tschebotarioff and peck empirical formulas (Boweles, 1997)

2.3.2 USA Federal highways Administration (FHWA)

Unlike the Terzaghi and Peck envelopes, the diagrams recommended for sand here in require that the location of the upper and lower struts are known in order to construct the apparent earth pressure diagram. The trapezoidal diagram is more appropriate than the rectangular diagram for the following reasons:

- earth pressures are concentrated at the anchor locations resulting from arching;
- earth pressure of zero at the ground surface is appropriate for sands (provided no surcharge loading is present);
- earth pressures increase from the ground surface to the upper ground anchor location; and
- for medium dense to very dense sands, earth pressures reduce below the location of the lowest anchor owing to the passive resistance that is developed below the base of the excavation.

This diagram is appropriate for both short-term (temporary) and long-term (permanent) loadings in sands. Water pressures and surcharge pressures should be added explicitly to the diagram to evaluate the total lateral load acting on the wall. (FHWA, 1999)



$P = 0.2\gamma H - 0.4\gamma H$

$P = K_a \gamma H$

$$P = \frac{Total\ load}{H - \frac{1}{3}H_1 - \frac{1}{3}H_{n+1}}$$

$$Total\ Load = 0.65 K_a \gamma H^2$$

Figure 2-8 (a): Apparent pressure diagram for sand

Figure 2-8 (b): Apparent pressure diagram for stiff to hard clay

Figure 2-8 (c): Apparent pressure diagram for soft to medium clay

Figure 2-8: Apparent pressures of different types of soil (FHWA, 1999)

In the case of soft to medium clay soils the Terzaghi and Peck (1967) diagrams did not account for the development of soil failure below the bottom of the excavation. Observations studies have demonstrated that soil failure below the bottom of the excavation can lead to very large movements for temporary retaining walls in these soft clays.

For N_s values greater than 6, relatively large areas of the retained soil near the base of the excavation are expected to yield significantly as the excavation progresses resulting in large movements below the excavation, increased support loads on the exposed portion of the wall, and potential instability of the excavation base. Henkel (1971) equation should be used directly to obtain K_a for use in evaluating the maximum pressure ordinate for the soft to medium clay apparent earth pressure diagram. (FHWA, 1999)

$$K_a = 1 - \frac{4S_u}{\gamma_s H} + 2\sqrt{2}\,\frac{d}{H}\left(\frac{1 - 5.14S_{ub}}{\gamma_s H}\right) \geq 0.22 \quad \ldots\ldots\ldots\ldots.\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.(2.9)$$

Where

S_u=Undrained shear strength of retained soil

S_{ub}=Undrained shear strength of soil below design grade in front of wall

γ_s=total unit weight of retained soil

d=depth of potential base failure surface below the design grade in front of wall

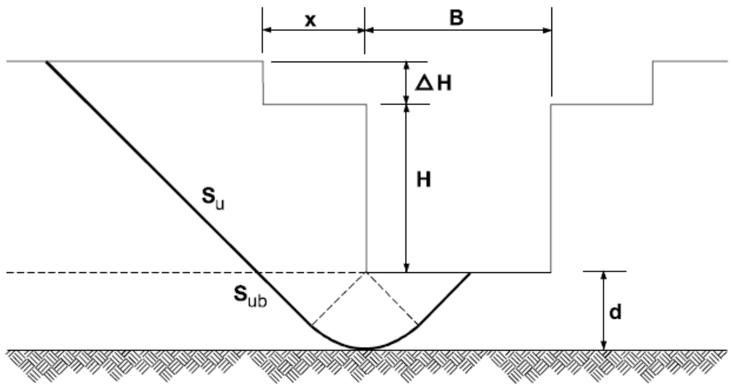


Figure 2-9: Henkel's mechanism of base failure.

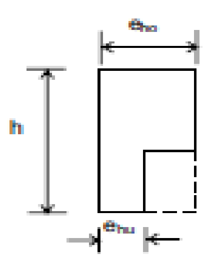
2.3.3 German recommendation for excavation (EAB)

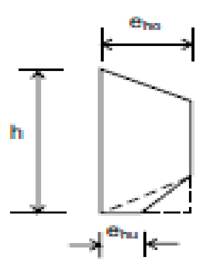
The working group "Excavations"(EAB) of the German Geotechnical Society (DGGT) has extended the known standard recommendation for excavations to include a new recommendation for excavations in soft soils. (EAB, 2008)

The apparent pressure diagram by Terzaghi and Peck was developed for flexible structures and does not include the effect of the position of the struts or anchors and number of the struts. On the other hand, the German working group "Excavation" (EAB) recommends apparent pressure diagrams that incorporates the influence of the above factors. The shape of the diagram varies from simple rectangular to trapezoidal distribution according to the position of the struts, the number of the struts, and type of the structure. (Kempfert & Gebreselassie, 2006)

Table 2-1: Apparent pressure diagrams for supported excavations in sand and stiff clays according to the recommendation of (EAB, 2008)

Support condition	Location of the support from wall top	Shape of the apparent pressure diagram	Location of the change in the apparent pressure diagram	e_{ho} / e_{hu}	
				Soldier piles	Sheet piles and cast in-situ walls
single support	1) ≤ 0.1·h	Rectangle	uniform	1.0	1.0
	2) between 0.1·h and 0.2·h	Two rectangles	0.5·h	>1.5	>1.2
	3) between 0.2·h and 0.3·h·	Two rectangles	0.5·h	>2.0	>1.5
double support	1) when the first support is near the top of the wall and the second support is within the top half of the excavation dept	Two rectangles	at the second support positions	2.0	1.5
	2) when the first support is near the top of the wall and the second support is about half of the excavation depth	Trapezoid	at the first and the second support positions	∞	2.0
	3) when both struts are located at lower position	Rectangle or trapezoid	uniform for soldier piles; at the second support position for sheet pile walls and cast in-situ walls	1.0	1.0
three or more supports	equally spaced supports	trapezoid	at the second and the third support positions for walls with three supports; at the second and the forth support positions for walls with more than three supports	∞	2.0





NB: 1) The hidden lines show the possible shapes of the apparent pressure diagram for different loading and/or wall types.
2) The ordinates e_{ho} and e_{hu} are determined in such a way that the resultant force from theoretical pressure distribution will be equal to the resultant force from the apparent pressure diagram.

2.3.4 Other Empirical methods of calculating lateral pressure

Twine and Roscoe (1999) proposed another empirical method by considering wall stiffness to estimate the pressure distribution behind the wall based on 60 cases of flexible wall and 21 cases if stiff wall. The excavation depth ranged from 4m to 27m in several soil conditions, i.e. soft and firm clays, stiff and very stiff clays, and coarse grained soils.

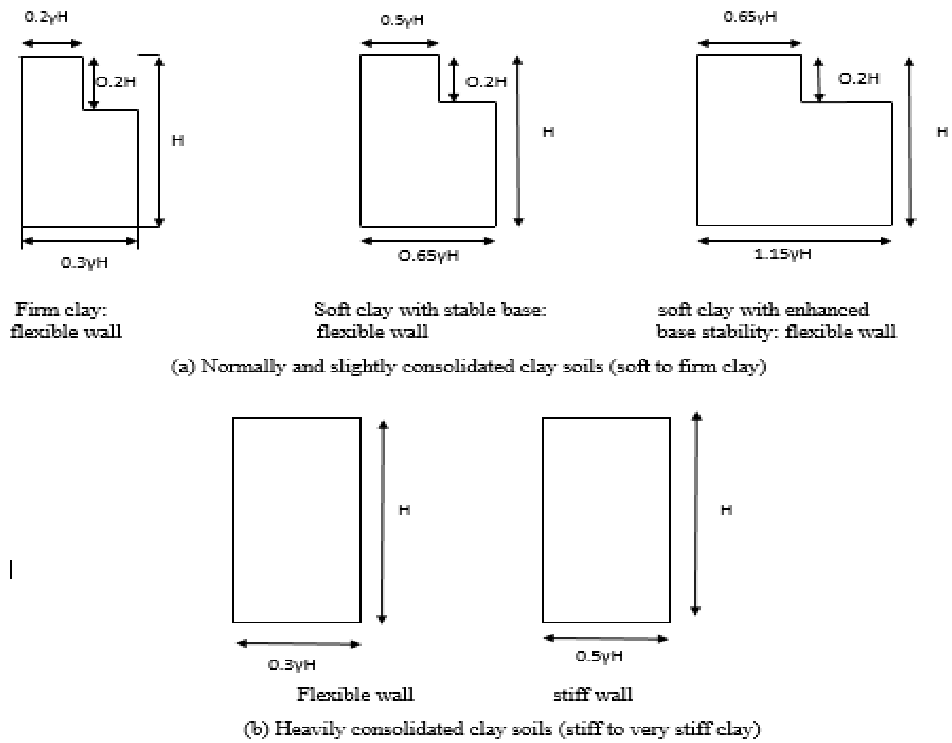


Figure 2-10: Earth pressure distribution on flexible and rigid walls for clay soils (Twine and Roscoe, 1999)

Since the Apparent Pressure Diagram was established some time ago, when the excavation dimensions and depth were relatively smaller as compared to the cases nowadays, the reliability of apparent pressure diagram is questionable. It is suggested that the method is still useful for excavation depths not exceeding 10m, while more studies are needed for excavation depth exceeding 20 m. Moreover, Apparent Pressure Diagram concentrated on the cases of flexible wall, which were popular at that time. Nowadays, higher stiffness wall systems are being used.

In the determination of strut loads from apparent pressure envelopes, a reaction at the base of the excavation is assumed to exist. This reaction is provided by the passive resistance of the soil beneath the excavation method given by Goldberg et al (1976) for determining the depth of penetration in 'competent soils (e.g. medium dense to dense granular soils and stiff to hard clays), which are capable of developing adequate passive resistance.

In weak soils, such as soft clays, the passive resistance may never reach the value of the active pressure on the retained side, no matter how deep the penetration is. This may also apply to a deep excavation in competent soil where the pressure on the wall below the lowest strut level is in an active state due to a high lateral pressure resulting from surcharge loading. In such cases, Goldberg et al (1976) have recommended that the wall should be driven to a depth required to prevent bottom heave or piping, and be designed as a cantilever about the lowest strut level.

There are also other semi-empirical methods of obtaining horizontal soil stresses and strut loads, such as Teng (1962), Armento (1972), Tschebotarioff (1973) and the Japan Society of Civil Engineers (1977). The apparent pressure envelopes given in these references are generally conservative, but they are less widely used than those of Terzaghi & Peck.

Swatek et al (1972), however, found reasonable agreement with the Tschebotarioff pressure envelopes in designing the support system for a 21 m deep excavation in Chicago clay. It appears that the Tschebotarioff method may be more appropriate when the excavation depth exceeds about 16 m.

2.4 Structural Parts of Braced Excavation

2.4.1 Estimation of Strut Forces in Braced Excavation

The struts in braced excavation are placed horizontally to resist the earth pressure on the back of the wall and most of the time I-section or circular hollow sections are used. The distribution of pressures against the wall cannot be accurately predicted from theory. Therefore, many field measurements were taken and envelope of probable distributions was drawn through Apparent Pressure Diagram.
The calculated strut loads might be approached but would not be exceeded in the actual excavation and it can be concluded that only the strut forces that can be obtained effectively from the Apparent Pressure Diagram can be calculated by assuming the wall as a simply supported beam.

The strut force is calculated by assuming the load in each strut to be equal the total earth pressure acting on the sheeting over a rectangular area extending horizontally half the distance to the next vertical row of struts on each side, and vertically half the distance to the horizontal sets of struts immediately above and below (Terrzaghi, Peck, and Mesri, 1996) as shown in Figure 2-11. This method is called the Tributary Area Method. It doesn't take account the effects of toe extending below the excavation level due to this it usually leads to a conservative design.

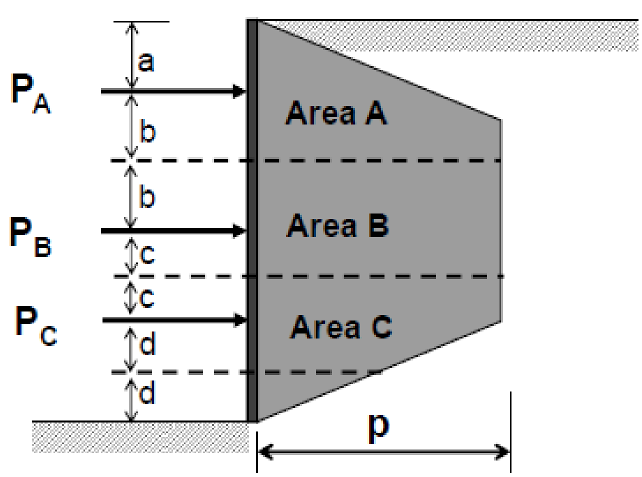


Figure 2-11: Calculation of struts using Tributary Area Method

Another method to calculate the strut forces from the Apparent Pressure Diagram is using Simple Beam Method. The retaining wall is considered as a continuous beam. The strut forces are calculated by dividing the beam into several simply supported beams with the struts acting as the supports. Figure 2.14 describes Simple Beam Method. Simple equilibrium calculation is then used to obtain each strut force. However, these two methods yield slightly different results.

Lambe et al. (1970) and Golder et al. (1970) verified that Peck's Apparent Pressure Diagram might overestimate the strut force in normally consolidated soils up to 50% greater than the actual measured loads. However, different soil conditions may lead the error to be on the unsafe side.

The Distributed Propped Load (DPL) Method proposed by Twine and Roscoe (1999) is adopted by CIRIA to calculate the strut forces. Figure 2-12 describes the Distributed Propped Load Method. Both Tributary Area and Simple Beam Method may be used to calculate the strut forces. This method is an update of Peck's Apparent Pressure Diagram, which doesn't consider the stiffness of the wall. It is stated that the strut forces calculated using this method provide conservative estimation to be expected in the field of normal circumstances.

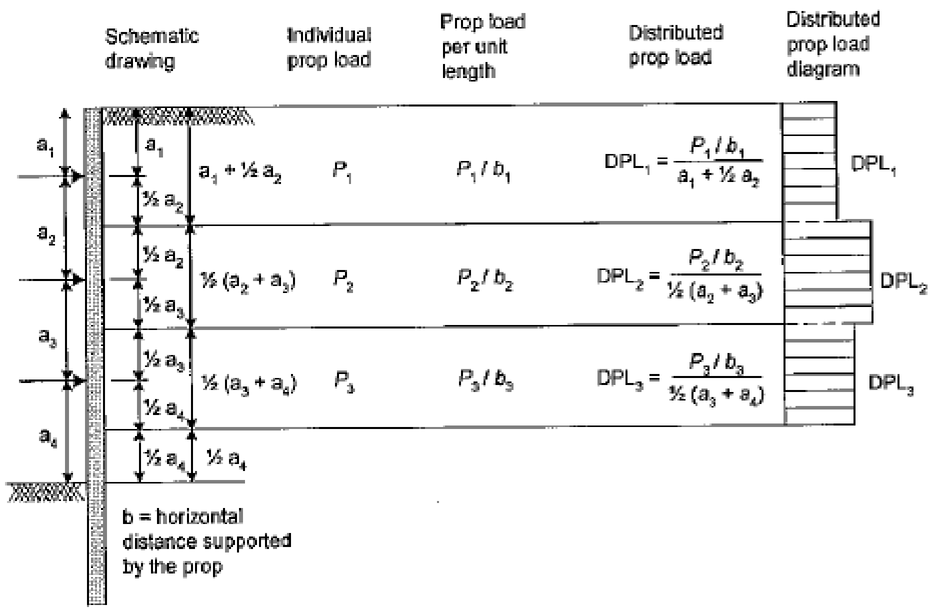


Figure 2-12: Example of Strut Force Calculation by DPL Method using Tributary Area Method

The structural design of the struts complying with the Design Code is based on the calculation of the strut forces. However, the soil-structure interaction is not included in the empirical formula although it may model the stress redistribution more realistically. Moreover, although Apparent Pressure Diagram and Distributed Prop Load Method represent the envelopes of the strut forces throughout the entire excavation stages, the staged construction needs to be simulated in order to achieve more effective and economical design.

2.4.2 Ground surface settlements induced by excavation

Many empirical formulas have been proposed by different authors to predict ground surface settlement and characteristics of soil movement. In this thesis only four of most widely used ones presented and discussed briefly.

2.4.2.1 Peck's Method

Peck (1969b) was first to propose method to predict excavation induced ground surface settlement, based on field observations. He established the relation curves between the ground surface settlements, the distance from wall for different types of soils as shown in Figure 2-13 (Ou, 2006).
Type I. Sand and soft to stiff clay, average workmanship
Type II. Very soft to soft clay
Type III. Very soft to soft clay to significant depth below the excavation bottom

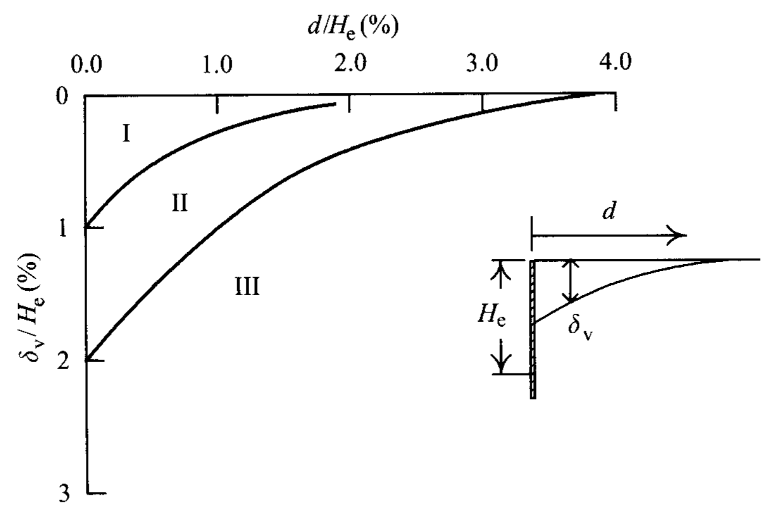


Figure 2-13: Peck's method (1969) for estimating ground surface settlement

2.4.2.2 Bowles' Method

Bowles (1986) proposed a method for estimating the spandrel-type settlement profile induced by 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.

$$D = (H_e + H_d)\tan(45 - \varphi'/2) \quad (2.10)$$

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 $H_d = 0.5B\tan(45 + \varphi'/2)$

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

$$\delta_{vm} = 4V_s/D \qquad (2.11)$$

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

$$\delta_v = \delta_{vm}(x/D)^2 \qquad (2.12)$$

Where: *D & x* are the distance from the wall.

2.4.2.3 Clough and O'Rourke's Method

Based on several case histories, Clough and O'Rourke (1990) suggested that the settlement profile is triangular for an excavation in sandy soil or stiff clay. The maximum ground surface settlement will occur at the wall. The non-dimensional profiles are given in Figure2-14 and which shows that the corresponding settlement extends to about $2H_e$ and $3H_e$ for sandy soil and stiff to very hard clays, respectively. For an excavation in soft to medium clay, the maximum settlement usually occurs at some distance away from the wall. The trapezoidal shape of the settlement tough is proposed as indicated in Figure 2.14 (c).The influence zone extends up to 2 times the maximum excavation depth. If the δ_{vm} is known, the settlement at various locations can be estimated.

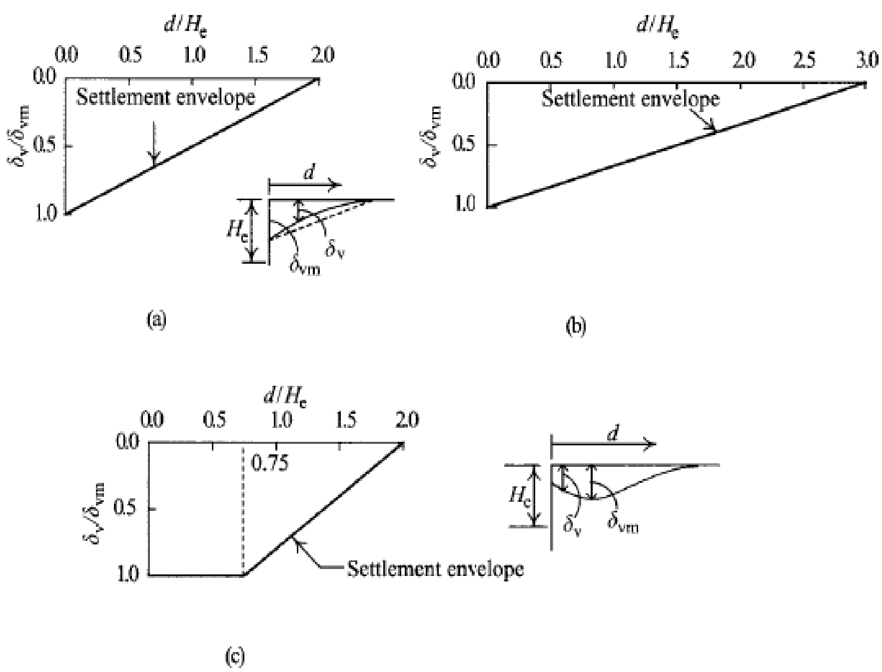


Figure 2-14: Method of Clough and O'Rourke (1990) for estimating ground surface settlement (a) Sand (b) Stiff to very stiff clay, and (c) soft to medium soft clay

2.4.2.4 Ou and Hsieh's method

Ou and Hsieh's method developed a method to predict the ground surface settlement on the basis of studies of the type of ground surface settlement as shown in Figure 2-15, the location of influence zone and the location of maximum settlement. According to Ou and Hsieh's the ground settlement can be predicted as follows

1. Estimate the maximum lateral displacement of the wall.
2. Determine the type of ground surface settlement.
3. Estimate the value of δ_{vm} on relationship between the maximum settlement and maximum lateral displacement δ_{hm}.
4. According to the type of settlement determine at the second step for various settlements occurring in different positions in back of the wall.

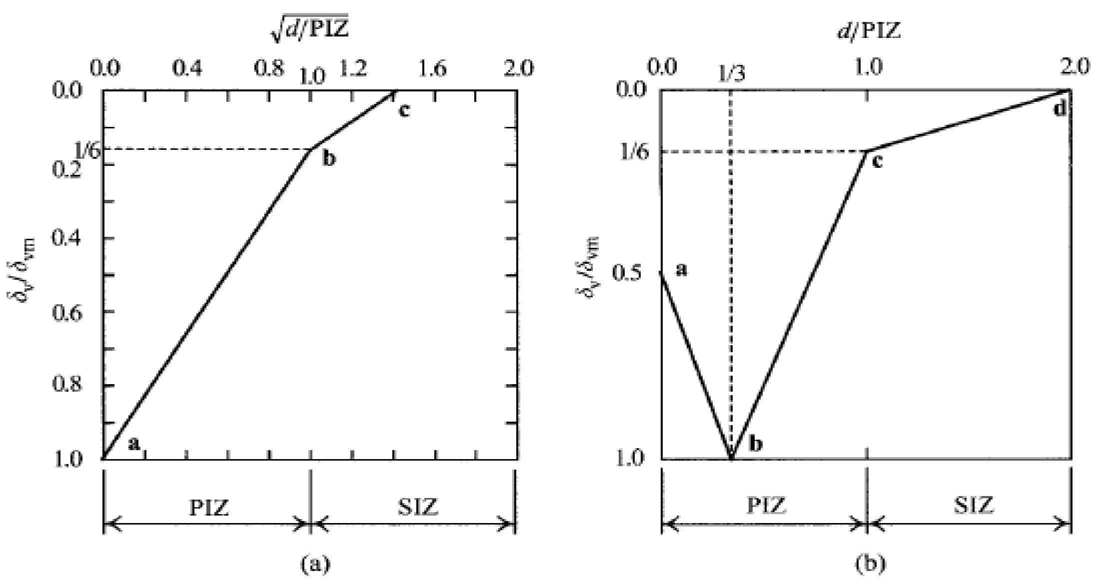


Figure 2-15: Ou and Hsieh's method

2.4.3 Excavation heave

Bottom heave in an excavation in normally consolidated soft soils is primarily caused by the elastic swelling of the bottom of excavation due to the relief of vertical stresses during the excavation process, the deflection of the foot of the wall which pushes the soil inwards, and the plastic deformation of the soil below the excavation level due to the change of the principal stresses.
The factors that affect the heave at bottom of the excavation include the depth of excavation, the stiffness (primarily) and strength (secondarily) of the ground, and the depth to the firm layer below bottom of excavation.

Several basal heave analysis methods have been suggested in literature, however, only four of them will be presented here, because most of them are based on similar principles. Terzaghi 1943 was the first to develop a method for bottom heave analysis for shallow or wide excavation (*h/b < 1*) as shown in Fig. 2-16. For length of the failure surface, r > *b*, the factor of safety against basal heave is given by

$$F.S. = \frac{5.7C_u}{h\left(\gamma - \frac{C_u\sqrt{2}}{b}\right)} \quad \text{………………………………….………………..(2.13)}$$

and for r<0.7b

$$F.S. = \frac{5.7C_u}{h\left(\gamma - \frac{C_u}{r}\right)} \quad \text{…………………..…………………………….…...(2.14)}$$

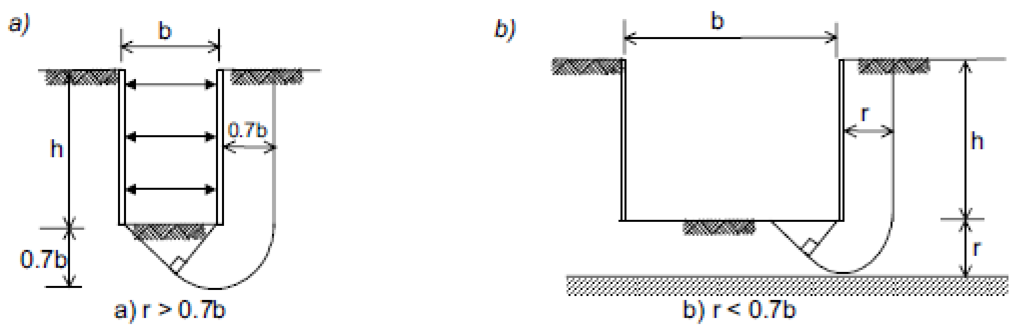


Figure 2-16: Bottom heave analysis for deep excavations (h/b < 1) (after Terrzaghi 1943)

2.5 Numerical modeling and FEM

When the complexity of a geotechnical problem is increasing, it could be preferable to use a numerical model. This is based on calculations that use algorithms to solve differential equations which are functions of several variables. These variables are material parameters, stresses, and strains etc. which in geotechnical engineering often show very complex correlation. The Finite Element Method (FEM) is a technique to find the solutions to this kind of equation systems. This is done by collecting the variables into matrices and vectors in computer programs and by numerical methods solve the system.

The use of this kind of tools does not only require knowledge about the theory behind numerical methods and skills to handle computer software, but also good knowledge and experience about the subject. In this case it is important to understand the soil mechanics and structural engineering and to be able to see the limitations of the model (Potts et al., 2002). Generalization and simplifications of the reality are often needed to build the model, and to do this, understanding and experience in geotechnical engineering is essential. This is also needed in order to evaluate the calculation results. If this is not done properly, the output will be misleading, which can get serious consequences.

3 . Modeling of excavation

3.1 Soil properties

The soil parameters considered for the study are obtained from Bowels (1997) and modeled using Mohr coulomb (MC) constitutive relation. The detail properties of the soil model are presented in Appendix A.2.

A total stress undrained soil condition for soft, medium and stiff clays soils are adopted whereas for sandy soil drained soil conditions adopted for analysis.

From the four cases of excavation one of them has ground water. Therefore, to model correctly the pore pressure change resulting from stage excavation a fully coupled stress pore pressure type analysis is performed. This means hydraulic boundary conditions as well a stress strain boundary conditions must be applied at each stage of excavations.

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 will be at the base of excavation. This is accommodated in the analyses with the specification of zero pressure at the original water table on the front side of wall and zero water pressure along the base of excavations.

The detailed soil properties which are taken as input for Plaxis 2D software are presented in Table 3-1 to Table 3-4.

Table 3-1 : Soil properties of case a

Parameter	Unit	Name	Sandy soil
Material model	-	Model	MC
Material behavior	-	Type	Drained
Soil unit weight above phreatic level	kN/m^3	$\gamma_{unsaturated}$	14.5
Soil unit weight below phreatic level	kN/m^3	$\gamma_{saturated}$	18.85
Horizontal Permeability	m/day	K_x	0.5
Vertical Permeability	m/day	K_y	0.5
Young’s modulus	kN/m^3	E_{ref}	25,562
Increase of stiffness	kN/m^3	E_{incr}	4,519
Reference Level	m	Y_{ref}	2
Poisson’s ratio	-	u	0.25
Cohesion	kN/m2	C	0
Friction angle	degree	Ø	30
Dilatancy angle	degree	Ψ	0
Interface strength	-	R_{inter}	0.65

Table 3-2 : Soil properties case b

Parameter	Unit	Name	Soft clay soil
Material model	-	Model	MC
Material behavior	-	Type	Un drained
Soil unit weight above phreatic level	kN/m^3	$\gamma_{unsaturated}$	13
Soil unit weight below phreatic level	kN/m^3	$\gamma_{saturated}$	18.2
Horizontal Permeability	m/day	K_x	1E-09
Vertical Permeability	m/day	K_y	1E-09
Young's modulus	kN/m^3	E_{ref}	7469
Increase of stiffness	kN/m^3	E_{incr}	1072
Reference Level	m	Y_{ref}	4
Poisson's ratio	-	u	0.375
Cohesion	kN/m2	C	10
Friction angle	degree	Ø	17.5
Dilatancy angle	degree	Ψ	0
Interface strength	-	R_{inter}	0.5

Table 3-3 Soil properties case c

Parameter	Unit	Name	Sandy soil	Soft clay soil
Material model	-	Model	MC	MC
Material behavior	-	Type	Drained	Un drained
Soil unit weight above phreatic level	kN/m^3	$\gamma_{unsaturated}$	14.5	13
Soil unit weight below	kN/m^3	$\gamma_{saturated}$	18.85	18.2
Horizontal. Permeability	m/day	K_x	0.5	1E-09
Vertical Permeability	m/day	K_y	0.5	1E-09
Young's modulus	kN/m^3	E_{ref}	25562	7469
Increase of stiffness	kN/m^3	E_{incr}	4519	1072
Reference Level	m	Y_{ref}	2	4
Poisson's ratio	-	ν	0.25	0.375
Cohesion	kN/m^2	C	0	10
Friction angle	degree	Ø	30	17.5
Dilatancy angle	degree	Ψ	0	0
Interface strength	-	R_{inter}	0.65	0.5

Table 3-4 : Soil properties case d

Parameter	Unit	Name	Sandy soil	Soft clay soil	Soft to medium clay soil	Stiff clay
Material model	-	Model	MC	MC	MC	MC
Material behavior	-	Type	Drained	Un drained	Un drained	Un drained
Soil unit weight above phreatic level	kN/m^3	$\gamma_{unsaturated}$	14.5	13	15	17
Soil unit weight below phreatic level	kN/m^3	$\gamma_{saturated}$	18.85	18.2	19.58	20.57
Horizontal Permeability	m/day	K_x	0.5	1E-09	1E-11	1E-11
Vertical Permeability	m/day	K_y	0.5	1E-09	1E-11	1E-11
Young's modulus	kN/m^3	E_{ref}	25,562	7469	14586	25559
Increase of stiffness	kN/m^3	E_{incr}	4,519	1072	1090	1096
Reference Level	m	Y_{ref}	2	1	1	2
Poisson's ratio	-	u	0.25	0.375	0.325	0.3
Cohesion	kN/m2	C	0	10	25	20
Friction angle	degree	Ø	30	17.5	25	20
Dilatancy angle	degree	Ψ	0	0	0	0
Interface strength	-	R_{inter}	0.65	0.5	0.5	0.5

3.2 Structural parameters

The structural parameters include the struts and wall properties. For two-dimensional analysis, the wall and the struts are assumed to be isotropic and plane strain conditions are assumed. Each strut level consists of primary struts and secondary struts, spaced by 2.5m vertically to represent the real construction condition. Horizontally, the struts are spaced at 5m interval .Sheet pile and strut parameters described for every cases of excavation in Table 3-5.

The system stiffness is used to represent the flexibility of the wall. The equation of system stiffness, S, is as follows: (Ou, 2006)

$$S = \frac{EI}{\gamma(h_{avg})^4} \quad \text{(3.1)}$$

Where E = modulus of elasticity of sheet pile

I= moment of inertia of sheet pile

ɣ= unit weight of the soil, and

h_{avg} =vertical strut spacing, respectively.

Table 3-5 : Sheet pile and strut Parameters

Parameters	Unit	Case 1		Case 2		Case 3		Case 4	
		Sheet pile	Strut	Sheet pile	Strut	Sheet pile	Strut	Sheet pile	Strut
Type	-	Elastic	Elastic	Elastic	Elastic	Elastic	Elastic	Elastic	Elastic
EA	KN/m	9.30E+06	1.27E+06	1.22E+07	1.27+06	1.53E+07	1.27+06	9.30E+06	9.30E+06
EI	KN/m	7.64E+04	-	1.74E+05	-	1.01E+06	-	7.64E+04	-
Thickness	m	0.012	-	0.012	-	0.012	-	0.012	-
Weight	KN	0.9045	-	1.1772	-	1.4715	-	0.9045	-
Ls (Strut spacing)	m	-	5	-	5	-	5	-	5

3.3 Modeling using Plaxis 2D software

Two dimensional finite element models may be either plain strain or Axis symmetric. Plain strain model is used for geometries with a uniform cross section and corresponding stress state and loading scheme over certain length perpendicular to the cross section, whereas an axis symmetry model is used for circular structures with a uniform radial cross section and loading scheme around central axis. (Brinkgreve,2002)

In this study a plane strain analysis is adopted using 15 node triangular elements which provide a fourth order interpolation for displacements and numerical integration involves twelve gauss points (stress points).The 15 node triangle is very accurate element that has produced high quality stress for difficult problems specially collapse calculation for incompressible soils.

The Mohr-Coulomb Model (MCM) is used to simulate the behavior of the soils in all the layers and to simulate the contact behavior. The structural elements are assumed to behave elastically

The first step in any finite element analysis of geotechnical problem is to convert the data from the geotechnical reports to a simplified soil profile, idealize the structural elements and determine the extent of the model.

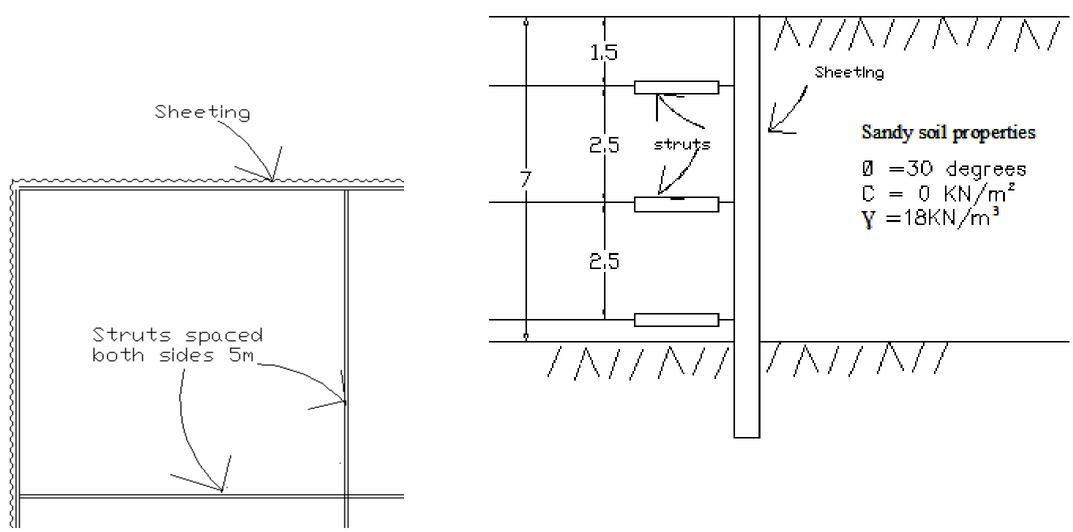
3.3.1 Case a

3.3.1.1 Type of soil and geometry of excavation

The type of soil used in this case is sandy soil and its depth of excavation is 7m. The geometry of excavation is rectangular in shape with width of 20 m and length of 50m.

3.3.1.2 Supporting system

The supporting the system is braced sheet pile. The struts are installed 5m spacing horizontally both sides. The first strut placed 1.5m from top surface of the ground and the next strut placed at 2.5m interval down to the bottom of excavation.



a) Horizontal spacing of struts b) positions of struts and soil profile

Figure 3-1: supporting system of braced excavation for sandy soil

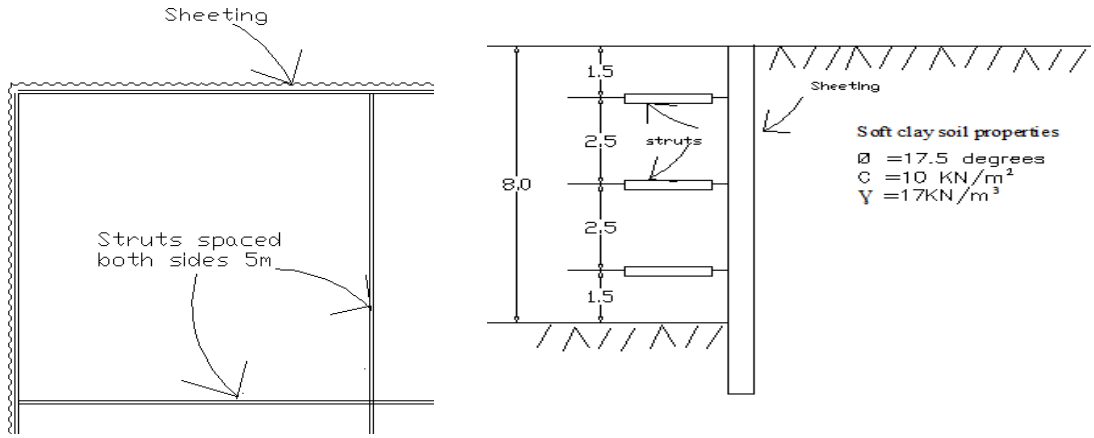
3.3.2 Case b

3.3.2.1 Type of soil and geometry of excavation

The type of soil used in this case is soft clay soil and its depth of excavation is 8m.The geometry of excavation is rectangular shape with width of 26 m and length of 40m.

3.3.2.2 Supporting system

The excavation supporting the system is braced sheet pile. The struts are installed 5m spacing horizontally both sides. The first strut placed 1.5m from top surface of the ground and the next strut placed at 2.5m interval up to bottom of excavation.



a) Horizontal spacing of struts b) positions of struts and soil profile

Figure 3-2: supporting system of braced excavation for soft clay soil

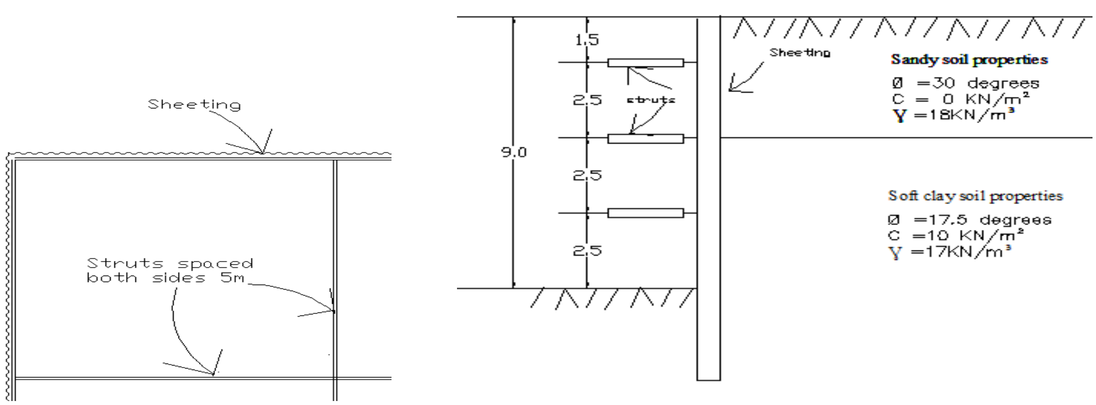
3.3.3 Case c

3.3.3.1 General Description of site

The type of soil used in this case is sandy soil from 0m up to 4m, soft clay soil is from 4m down ward and its depth of excavation is 9m.The geometry of excavation is rectangular shape with width of 32 m and length of 60m.

3.3.3.2 Supporting system

The excavation supporting the system is braced sheet pile. The struts are installed 5m spacing horizontally both sides. The first strut placed 1.5m from top surface of the ground and the next strut placed at 2.5m interval up to bottom of excavation.



a) Horizontal spacing of struts b) positions of struts and soil profile

Figure 3-3: supporting system of braced excavation for stratified soil type 1

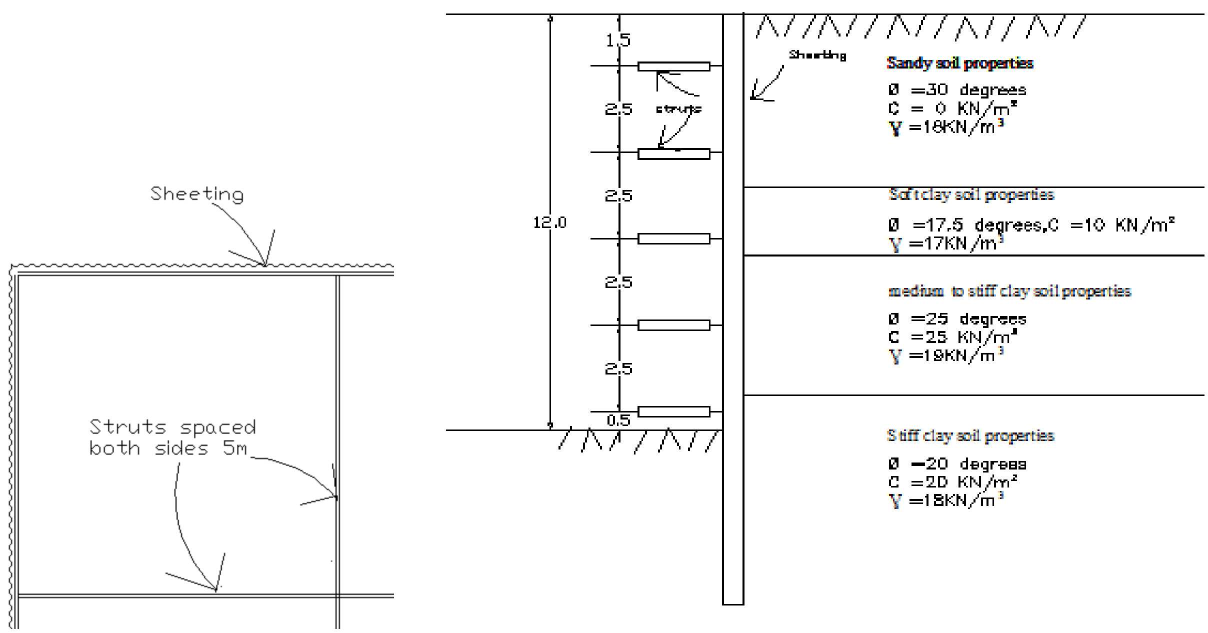
3.3.4 Case d

3.3.4.1 Geometry of excavation

The type of soil used in this case is sandy soil from 0m up to 4m, soft clay soil is from 4m up to 6m, medium stiff clay soil from 6m up to 8m, stiff clay soil from 8m down ward and its depth of excavation is 12m.The geometry of excavation is rectangular shape with width of 20 m and length of 70m.

3.3.4.2 Supporting system

The excavation supporting the system is braced sheet pile. The struts are installed 5m spacing horizontally both sides. The first strut placed 1.5m from top surface of the ground and the next strut placed at 2.5m interval up to bottom of excavation.



a) Horizontal spacing of struts b) positions of struts and soil profile

Figure 3-4: supporting system of braced excavation for stratified soil type 2

3.4 Deformation properties of soil

Deformation of a soil is one of the most important physical aspects in geotechnical problems. Many investigators have revealed that the soil deformation modulus was found to have the greatest influence on deformation behavior of geotechnical structures.

It is necessary to take account of the fact that soil stiffness is likely to increase with increasing effective stress. This effect is often modeled by assuming that the soil stiffness is proportional to the depth. According to DIN 4094-1

$$E_s = \nu * P_a * \left[\frac{\sigma_z + 0.5 * \Delta\sigma_z}{P_a}\right]^w \qquad \text{……………………………………….. (3.2)}$$

Where u = stiffness coefficient

w = stiffness exponent, which has a value of 0.5 for non-cohesive soil

σ_z = overburden pressure at a depth z below the foundation level

$\Delta\sigma_z$ = additional vertical stress due to the loads from the superstructure at a depth z under the foundation level

Pa = average atmospheric pressure, taken as 100 kN/m²

3.5 Concepts of staged excavations

In finite element terminology staged excavation means adding or removing elements from the mesh. The mesh remains the same for all analyses but regions can activate or deactivate to simulate, for example, the placement of fill or the removal of soil to create an excavation. PLAXIS 2D can string together a series of analyses.

All the analyses can be sequential by assigning each analysis a duration in time to form a continuous time line. The start time of an analysis is the ending time of the previous analysis. By assigning each analysis time duration and making the time line continuous, the results can be viewed simultaneously for multiple analyses. The geometry and mesh must be the same for all analyses but boundary conditions, material properties and structural components can change for each analysis.

Basically the geometry of excavation is taken as symmetrical and the typical stages of the excavation which are assumed to be carried out an interval of 3m are**:**

Phase 01: Generate the initial stresses

Phase 02: Activate the surcharge and traffic loads

Phase 03: wall installation

Phase 04: 1st stage excavation

Phase 05: strut installation

Phase 06: 2nd stage excavation

Phase 07: strut installation

Phase 08: 3rd stage excavation

Phase 09: strut installation

4. Analysis and results

4.1 Analysis using Plaxis software

4.1.1 Lateral earth pressure at different stages of excavations

In this part of the analysis, the study concentrates on how lateral earth pressure varies in every stage of excavations supported by Sheet pile wall for 7m, 8m, 9m and 12m depth of excavation. The output of this analysis is presented for each cases of excavation from Figure 4-1 to Figure 4-4 as follows.

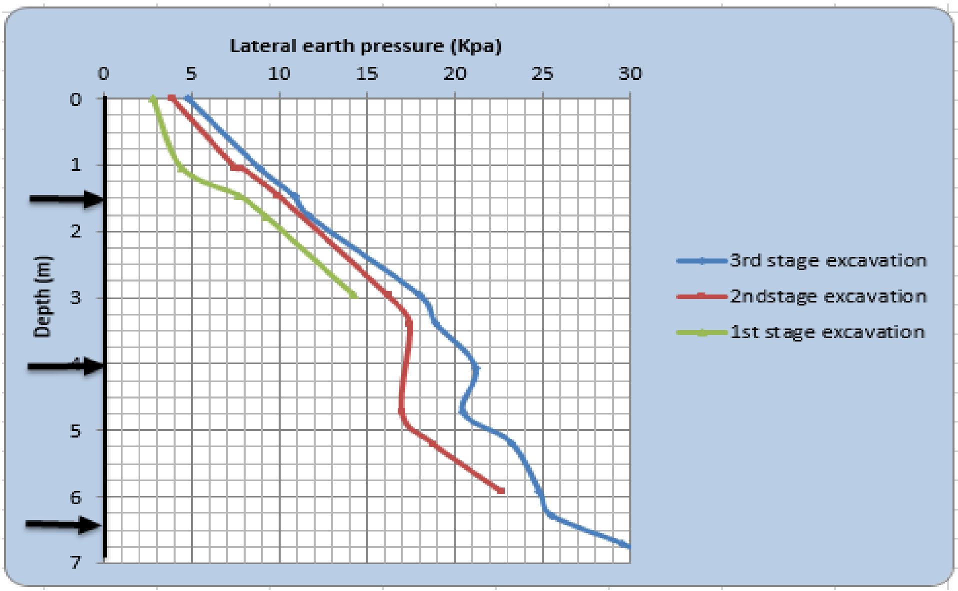


Figure 4-1: Lateral earth pressure at different stages of excavation for sandy soil

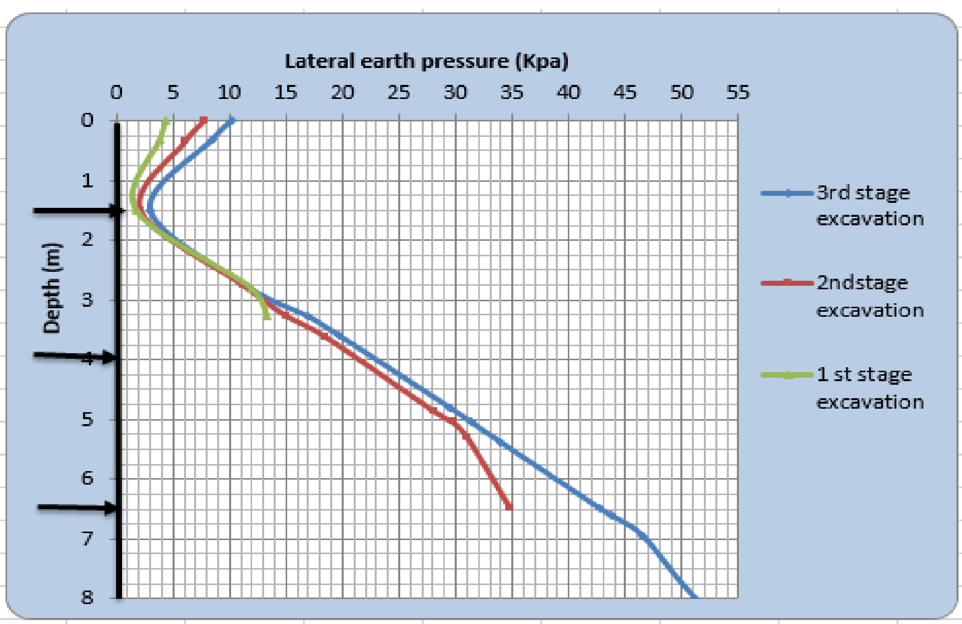


Figure 4-2: Lateral earth pressure at different stages of excavation soft clay soil

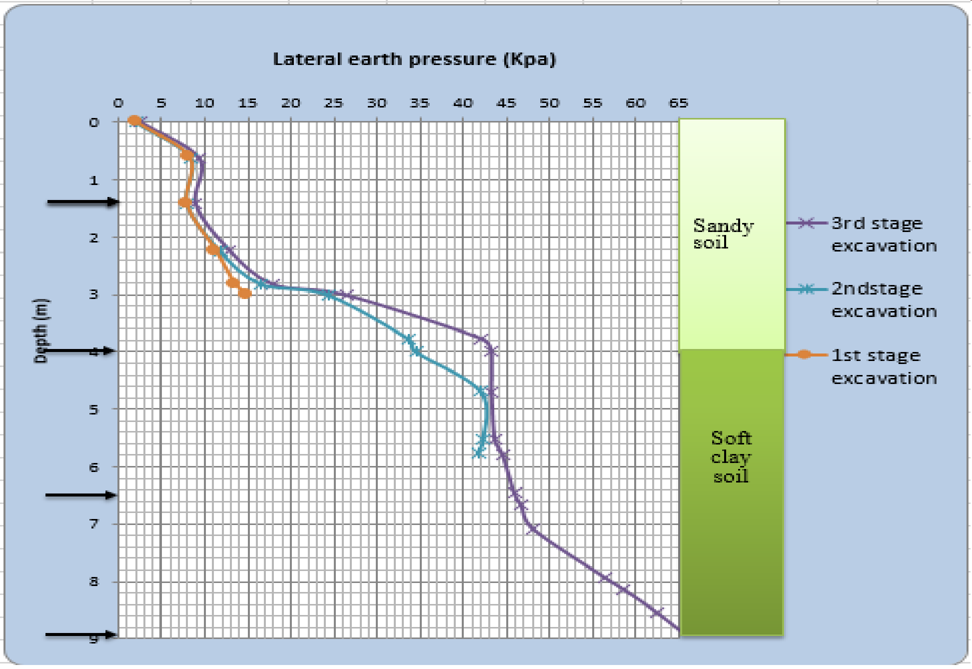


Figure 4-3 : Lateral earth pressure at different stages of excavation for stratified soil type 1

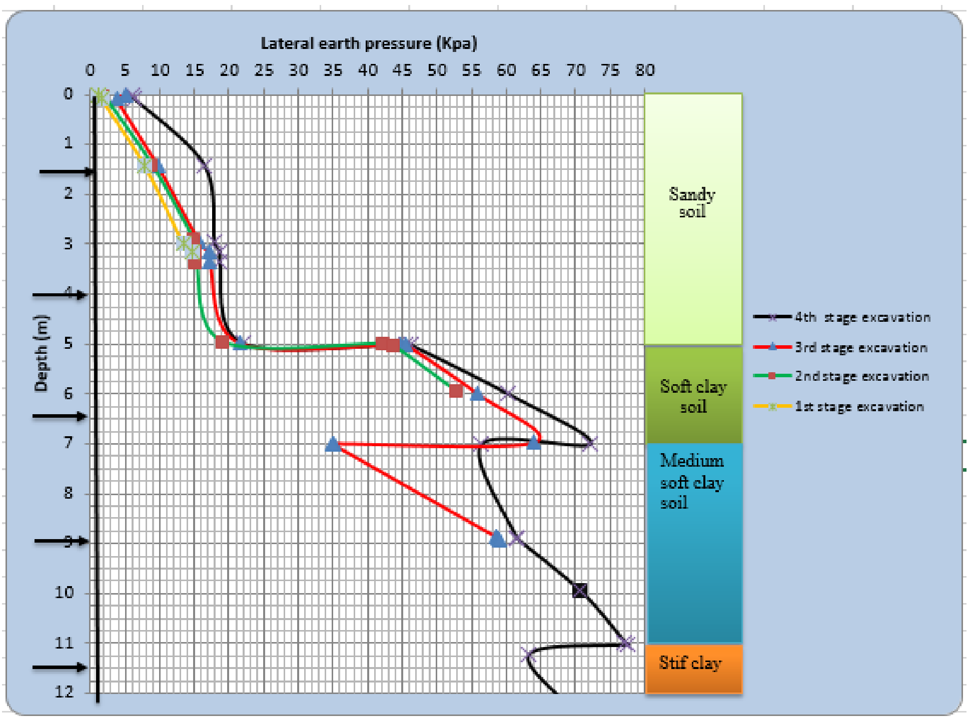


Figure 4-4: Lateral earth pressure at different stages of excavation for stratified soil type 2

From Figures 4-1 up to Figure 4-4, the lateral earth pressure increases as the stage of excavation increases and the shapes of the lateral pressure found from finite elements software are not linear. In addition to this, a change in lateral earth pressure can be visualized as soil profile changes as shown in Figure 4-3 and Figure 4-4.

4.1.2 Parametric study of lateral earth pressure

In this section, the details and results of a total of 4 parametric studies are presented. The parametric study conducted a number of alternative arrangements of variables under consideration. Accordingly, the effect of varying a parameter is manifested by obtaining the variation in lateral earth pressure**.**

4.1.2.1 Effect of change of depth of embedment

In this section, the effect of change in wall embedment depth on the lateral earth pressure of braced sheet pile wall will be studied in four cases. The study conducted by increasing the depth of wall embedment by 2.5m from existing position. The output of this analysis is presented from figure 4-5 up to figure 4-8.

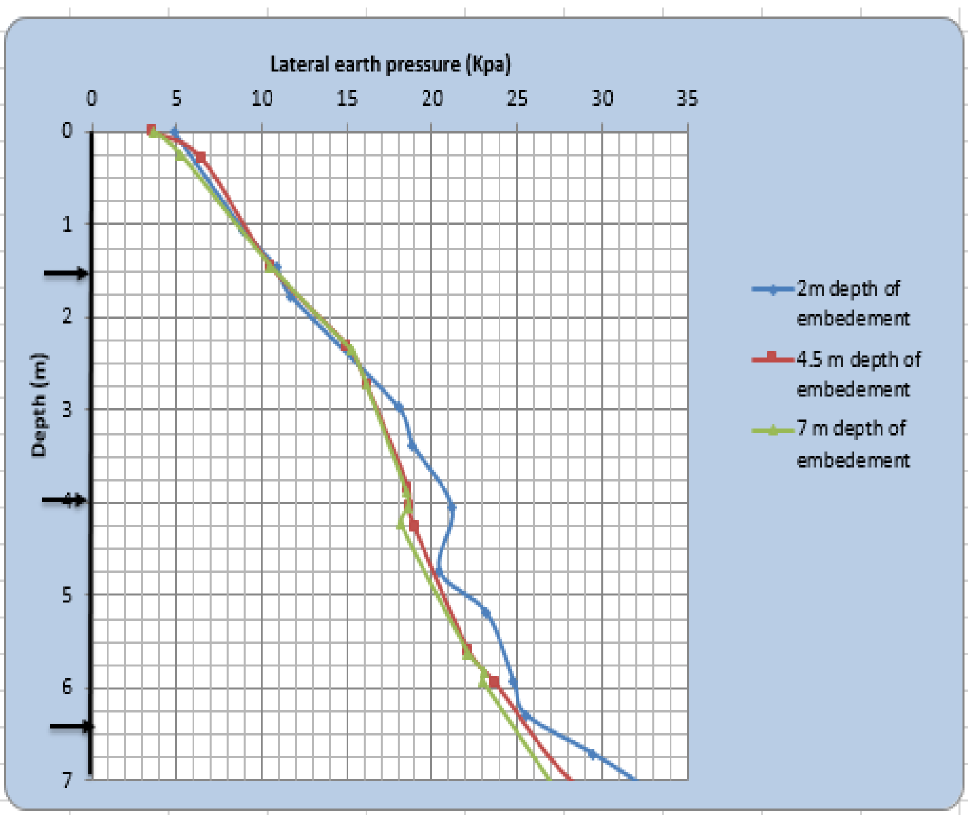


Figure 4-5: Lateral Earth pressure variation due to depth of embedment of sandy soil

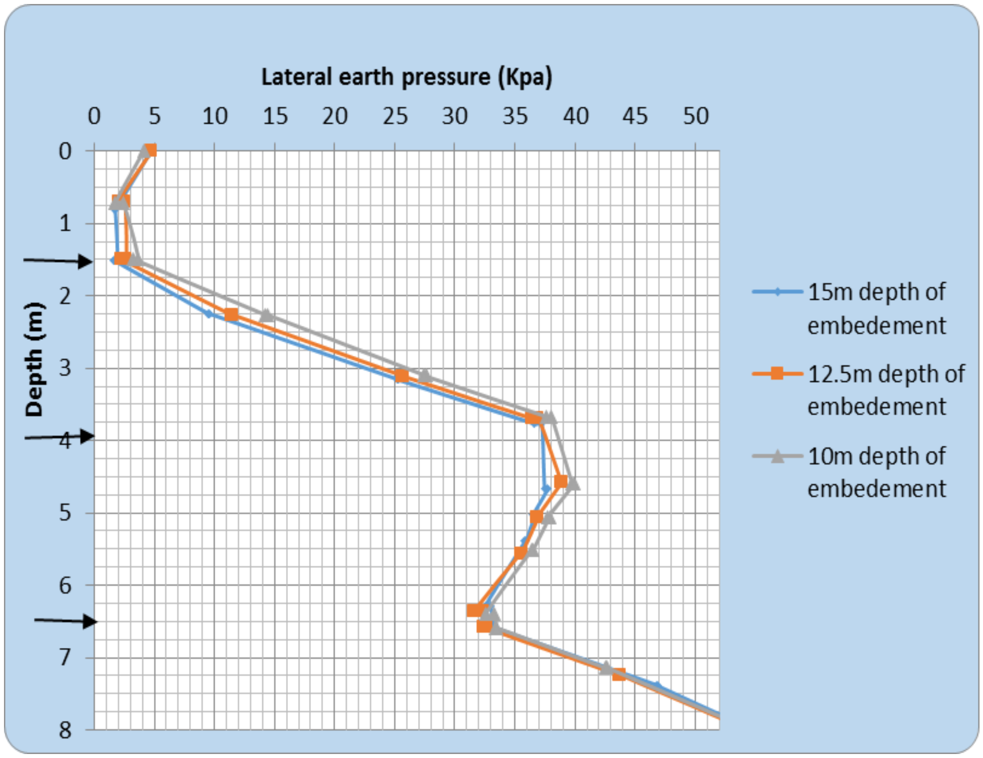


Figure 4-6: Lateral Earth pressure variation due to depth of embedment of soft clay soil

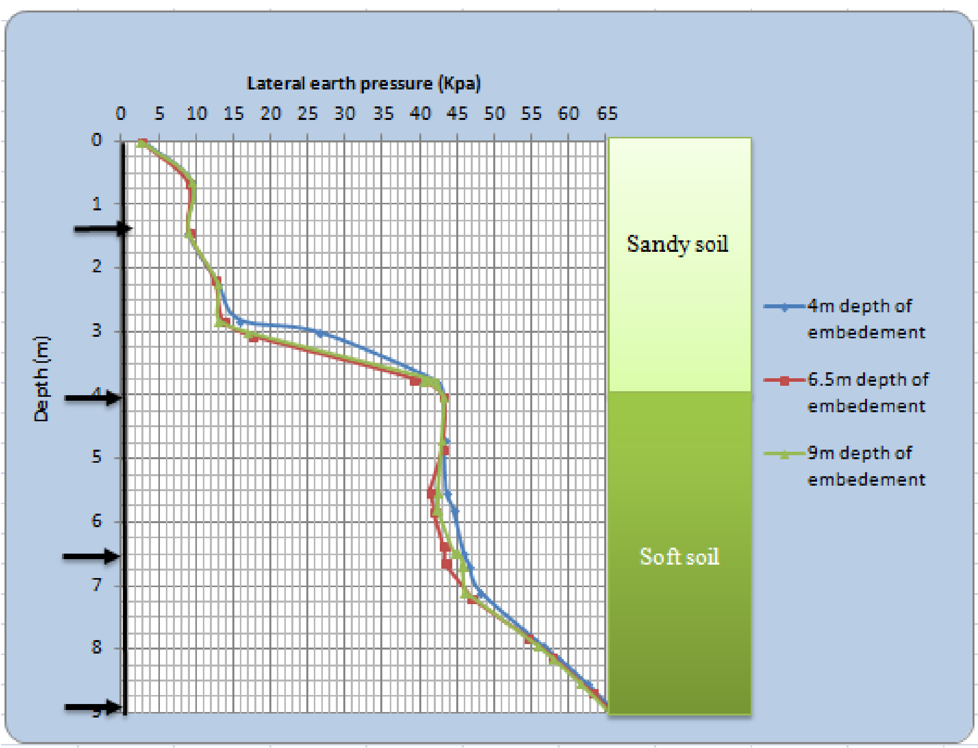


Figure 4-7: Lateral Earth pressure variation due to depth of embedment of stratified soil type 1

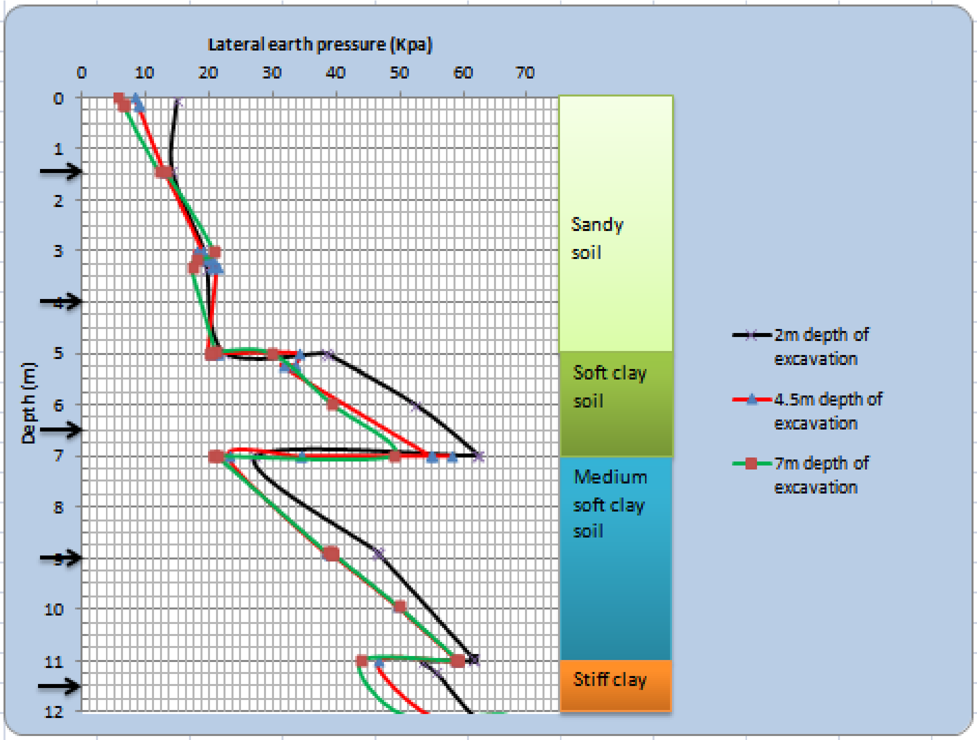


Figure 4-8: Lateral Earth pressure variation due to depth of embedment of stratified soil type 2

From figures 4-5 up to figure 4-8, the lateral earth pressure does not have significant change when the depth of embedment increases.

4.1.3 Effect of change of stiffness of sheet pile

In this section, the effect of change in stiffness of sheet on the lateral pressure of braced sheet pile wall is presented for 7m, 8m, 12m and 16m depth of excavation. The stiffness of sheet pile and output from this analysis is presented below.

Table 4-1: Properties of sheet pile sections chosen for analysis (Ou, 2006)

Description	EA (KN/m)	EI (KN-m^2)	Weight (KN/m)
Type 1	9.3E+08	7.64E+06	0.922
Type 2	1.22E+09	1.74E+07	1.20
Type 3	1.53E+09	3.28E+07	1.50

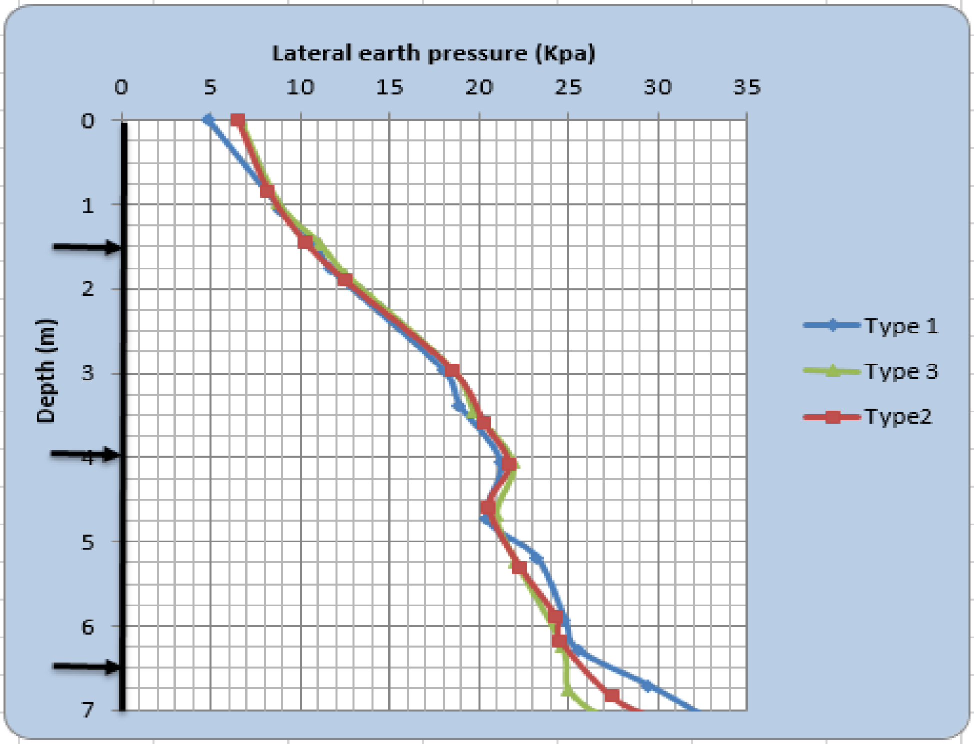


Figure 4-9: Lateral Earth pressure variation due to stiffness of sheet pile of sandy soil

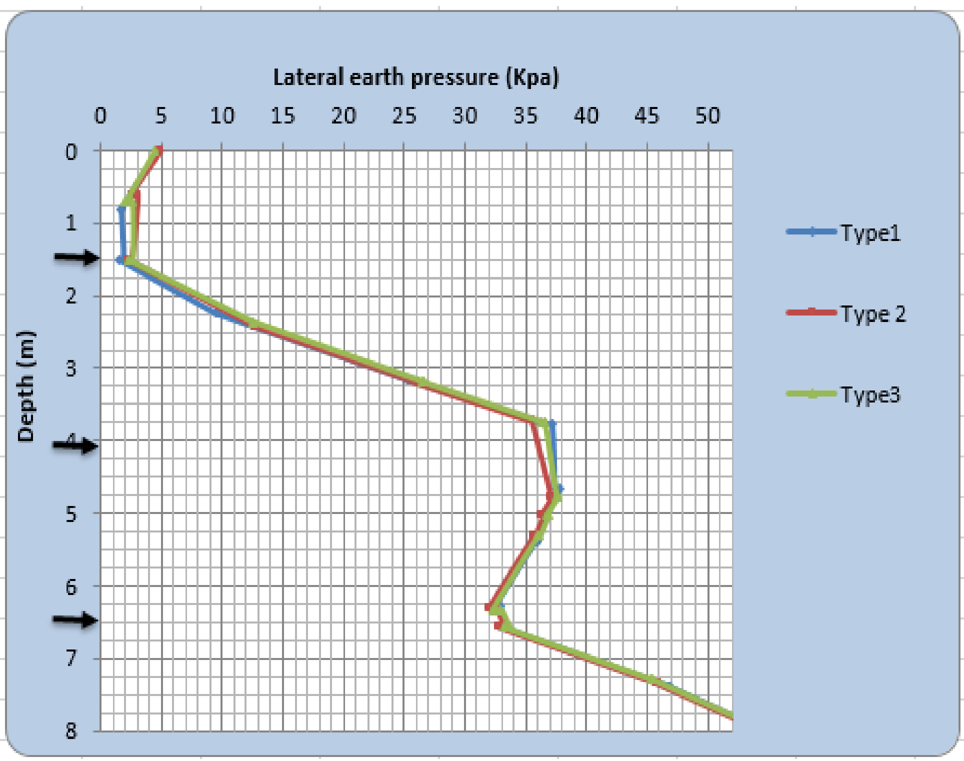


Figure 4-10: Lateral Earth pressure variation due to stiffness of sheet pile of soft clay soil

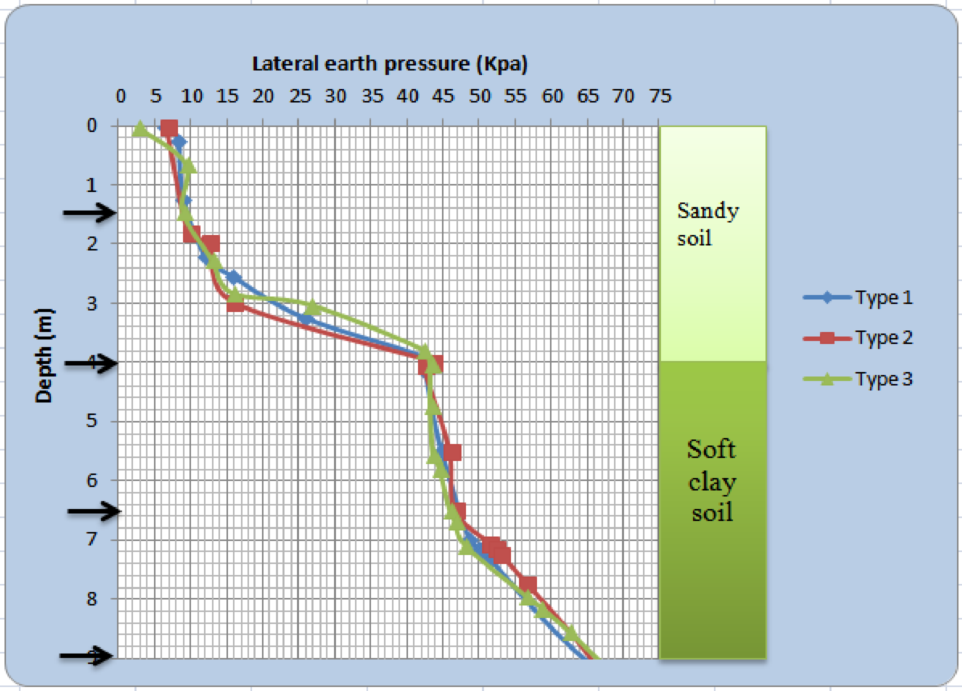


Figure 4-11: Lateral Earth pressure variation due to stiffness of sheet pile of stratified soil type 1

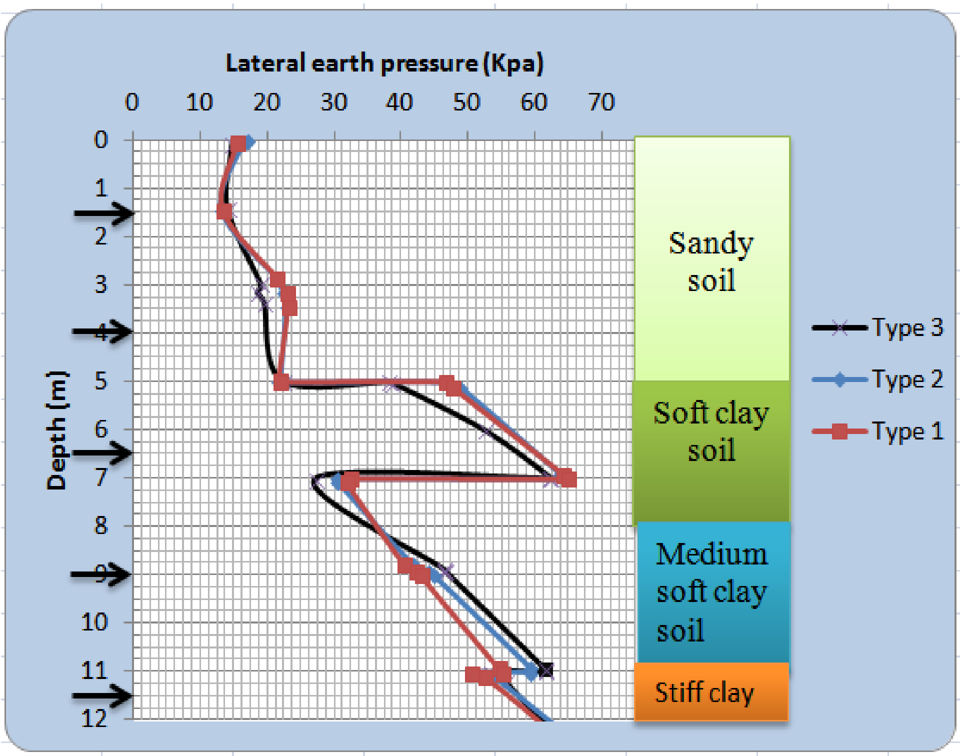


Figure 4-12: Lateral Earth pressure variation due to stiffness of sheet pile of stratified soil type 2

From Figures 4-9 up to Figure 4-12, it can be notified that the lateral earth pressure does not have significant change when the stiffness of sheet pile increases or decreases.

4.1.4 Effect of change of spacing of struts

In this section, the effect of change in spacing of struts on the lateral pressure of braced sheet pile wall is presented for 7m, 8m, 12m and 17m depth of excavation. The arrangement of vertical spacing and output from this analysis is shown Table4-2.

Table 4-2: Positions of struts for every excavation cases

Sites		Vertical spacing of struts (m)							
Case a	Type 1	1.5	2.5	2.5	-	-	-	-	-
	Type 2	2	3	-	-	-	-	-	-
	Type 3	2.5	3.5	-	-	-	-	-	-
Case b	Type 1	1.5	2.5	2.5	-	-	-	-	-
	Type 2	2	3	3	-	-	-	-	-
	Type 3	2.5	3.5	-	-	-	-	-	-
Case c	Type 1	1.5	2.5	2.5	2.5	2.5	-	-	-
	Type 2	2	3	3	3	-	-	-	-
	Type 3	2.5	3.5	3.5	-	-	-	-	-
Case d	Type 1	1.5	2.5	2.5	2.5	2.5	2.5	2.5	-
	Type 2	2	3	3	3	3	3	-	-
	Type 3	2.5	3.5	3.5	3.5	3.5	-	-	-

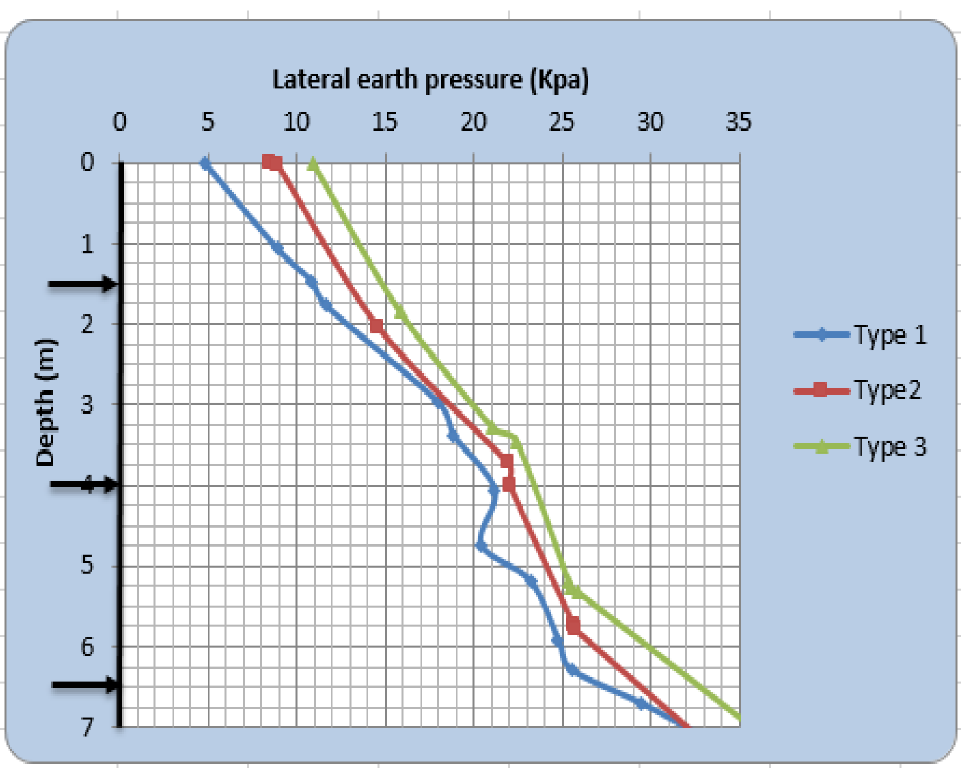


Figure 4-13: Lateral Earth pressure variation due to spacing of struts of sandy soil

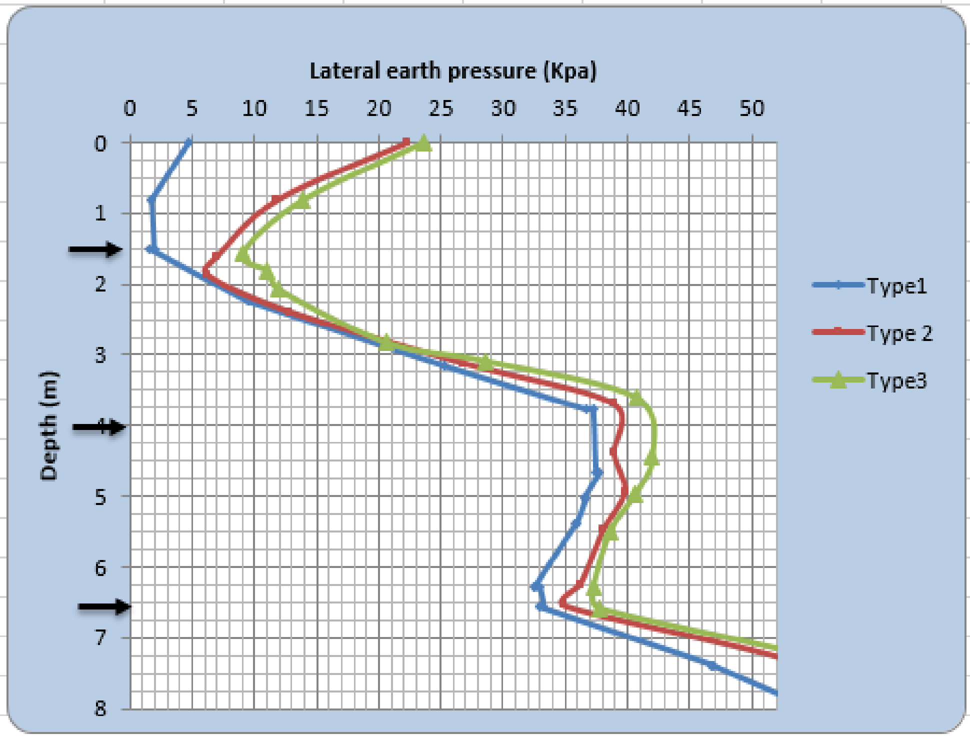


Figure 4-14: Lateral Earth pressure variation due to spacing of struts of soft clay soil

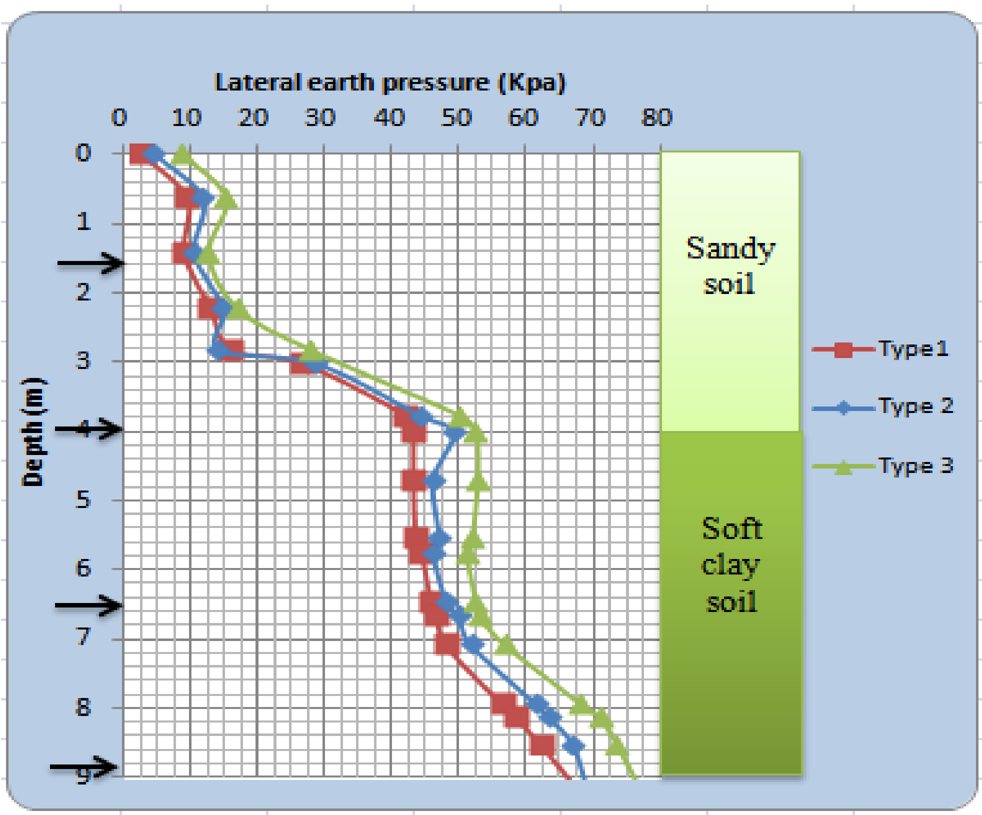


Figure 4-15: Lateral Earth pressure variation due to spacing of struts of stratified soil type 1

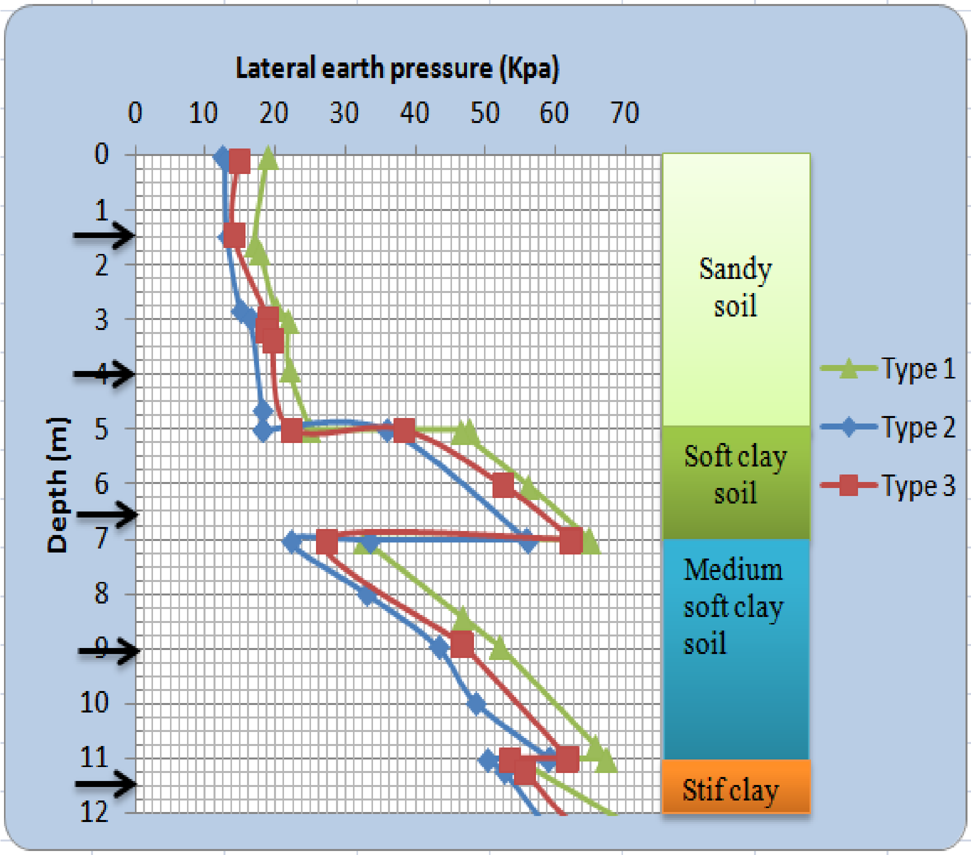


Figure 4-16: Lateral Earth pressure variation due to spacing of struts of stratified soil type 2

From Figures 4-13 up to Figure 4-16, the lateral earth pressure increases when the spacing of struts increases.

4.1.5 Effect of change of stiffness of struts

In this section, the effect of change in stiffness of struts on the lateral pressure of braced sheet pile wall is presented for 7m, 8m, 12m and 17m depth excavation. The sectional properties of struts and output from this analysis are shown in table 4-9.

Table 4-3: properties of I sections struts

Description	Dimension (mm)				Area (cm^2)	Weight(Kg/m)	EA(KN/m)
	HXB	t_1	t_2	R			
Type 1	200X200	8	12	13	63.53	49.9	1.27E+06
Type 2	250X250	9	14	16	92.18	72.4	1.84E+06
Type 3	300X300	10	15	18	119.78	94	2.4E+06

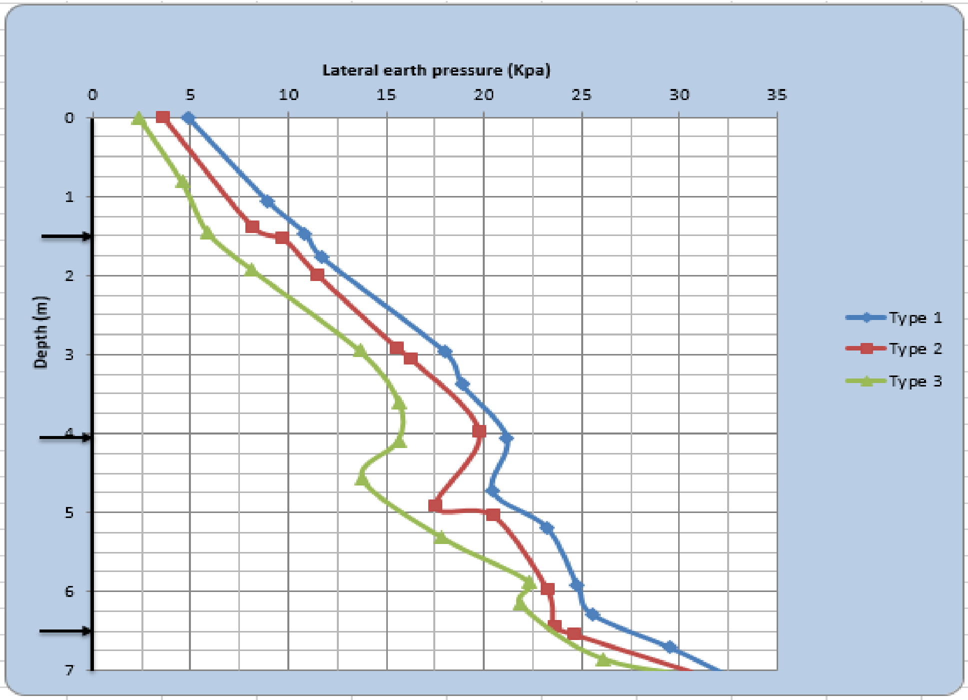


Figure 4-17: Lateral Earth pressure variation due to stiffness of struts of sandy soil

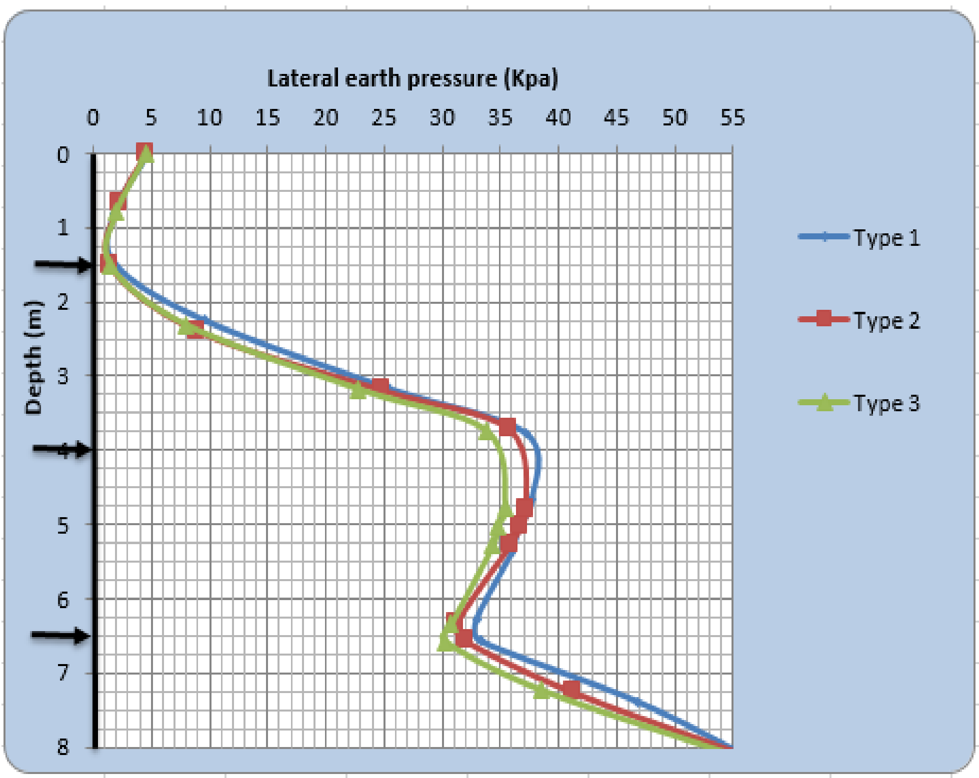


Figure 4-18: Lateral Earth pressure variation due to stiffness of struts of soft clay soil

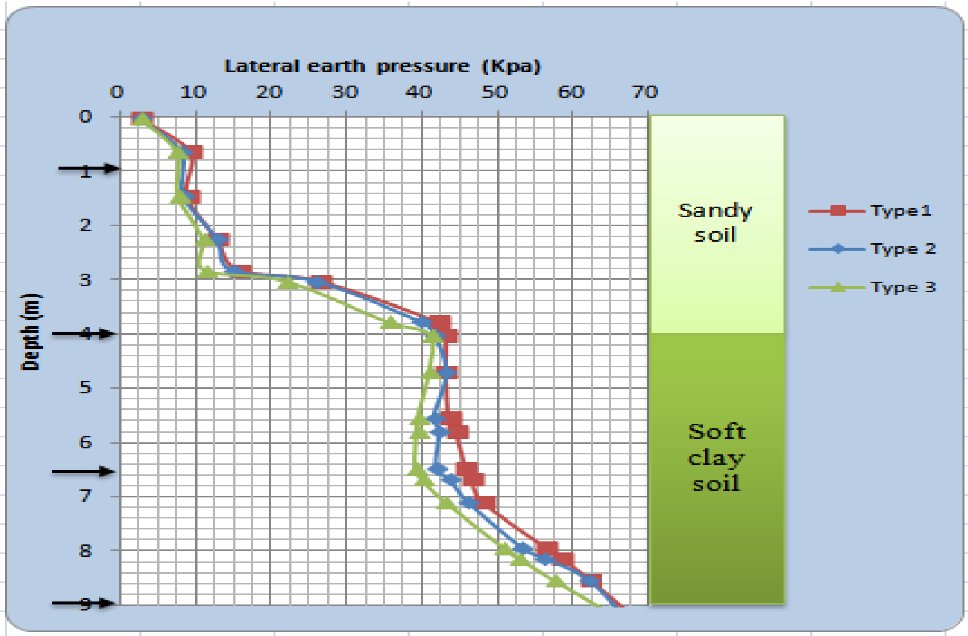


Figure 4-19: Lateral Earth pressure variation due to stiffness of struts of stratified soil type 1

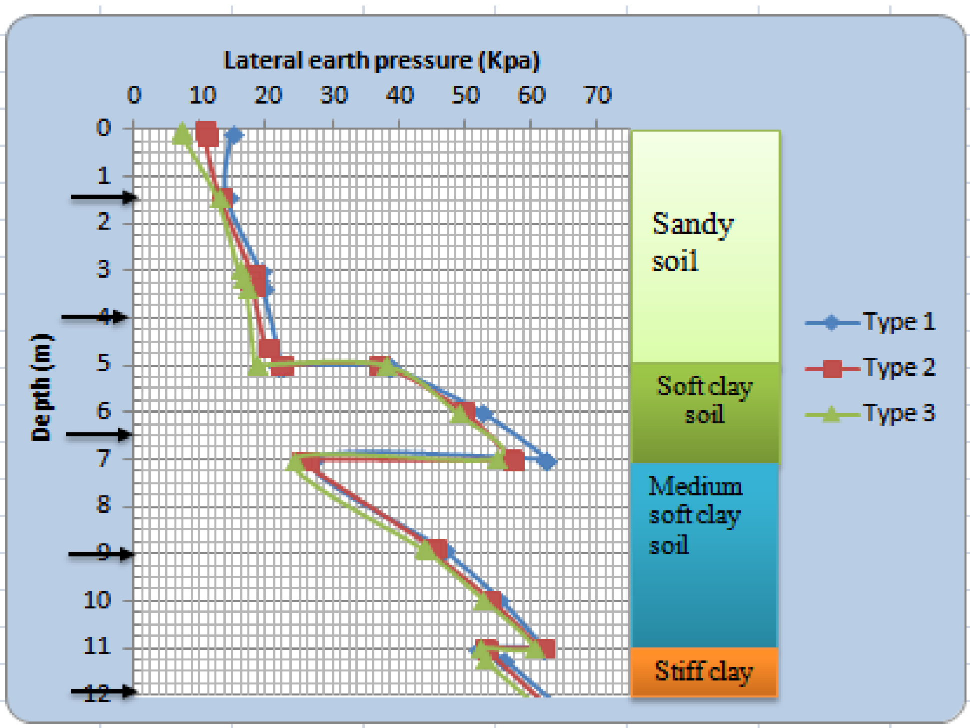


Figure 4-20: Lateral Earth pressure variation due to stiffness of struts of stratified soil type 2

From Figures 4-17 up to Figure 4-20, it can be notified that the lateral earth pressure increases when the stiffness of struts increases. Especially, in the case of sandy soil has significant change of lateral pressure.

4.1.6 Strut loads at final excavation stage

Table 4-4 shows the total strut load as computed from Plaxis output at the end of the excavation. It is evident from these values that the summation of the loads in the struts does reflect the pressure on the wall.

Table 4-4 strut loads at final excavation stage

Strut Levels	Unit	Case a	Case b	Case c	Case d
A	KN/m	59.92	162.16	64.99	100.404
B	KN/m	89.74	93.34	201.36	249.38
C	KN/m	75.48	2.782	269.04	347.875
D	KN/m	-	-	276.57	313.282
E	KN/m	-	-	-	278.694

4.1.7 Phi reduction method

In the software PLAXIS, the shear strength reduction procedure is called *phi-c reduction*, and is used to compute safety factors.

The total multiplier *Σ Msf* is used to define the value of the soil strength parameters at a given stage in the analysis:

$$\sum msf = \frac{\tan \emptyset input}{\tan \emptyset reduced} = \frac{c_{input}}{c_{reduced}} \qquad (4.1)$$

A phi-c reduction calculation is performed using the Load advanced number of steps procedure. The incremental multiplier *Msf* is used to specify the increment of the strength reduction of the first calculation step.

$$\mathrm{Sf} = \frac{available\ strength}{strength\ at\ failure} \qquad (4.2)$$

If a failure mechanism has not fully developed, then the calculation must be repeated with a larger number of additional steps. To capture the failure of the structure accurately, the use of Arc-length control in the iteration procedure is required and results of factor of safety for excavation case presented in Table 4-5.

Table 4-5 Results of Plaxis factor of safety for every excavation

Description	Sandy soil	Soft clay soil	Stratified soil type 1	Stratified soil type 2
Factor of safety	2.424	2.736	2.101	2.163

4.2 Analysis using Empirical formulas

4.2.1 Properties of soil

The unit weight γ s, un-drained shear strength, S_u, effective friction angle, φ' for each layer of soil in every cases has been taken from bowels and soil engineering books. According to Terzaghi when there is more than one layer of soil type, the weighted average system is used to find the soil design parameters (C_{avg} and γ_{avg}) and the soil is categorized accordingly.

4.2.2 The apparent earth pressure envelope

The detailed calculation of the apparent earth pressure presented below for each cases of excavation using Terrzaghi and peck, Terbachove theory, FHWA and EAB (German code).

During calculation process in the case of layered soil ,the dominant layer of soil within the deep of the excavation will be identified and use those properties for design, or apply Peck's (1943) equivalent un-drained shear strength and unit weight parameters. The surcharge load used in the study is 10 KN/m^2

The apparent earth pressures diagrams proposed by Peck (1969) are used in this study to size the struts and wales because they are more conservative than other methods.

Case a: Sandy soil

A.Terzaghi and Peck method

According to Terzaghi and peck (1969) for sandy soil type the maximum probable pressure calculated using the following equation

$$P = 0.65 * K_a * (\gamma * H_e + q_s) \quad (4.3)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

B.Terbachoves method

According to Terbachoves for sandy soil type the maximum probable pressure calculated using the following equation

$$P = 0.25 * (\gamma * H_e + q_s) \quad (4.4)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

C.USA Federal highway Administration (FHWA)

According to FHWA for sandy soil type the maximum probable pressure calculated using the following equation

$$P = \frac{Total\ Load}{H - \frac{1}{3} * H_1 - \frac{1}{3} * H_{n+1}} \quad (4.5)$$

$$P = 0.65 * H_e * K_a * (\gamma * H_e + q_s) \quad (4.6)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

D. German Code (EAB)

The pressure diagram according to Lehmann may be assumed a realistic for triple or multiple supported sheet pile walls and ratio of $eh_{ok:ehu,k}$=2 .The ordinates $eh_{o,k}$ and $eh_{u,k}$ are determined in such a way that the resultant force from Rankine theoretical pressure distribution will be equal to the resultant force from the apparent pressure diagram.

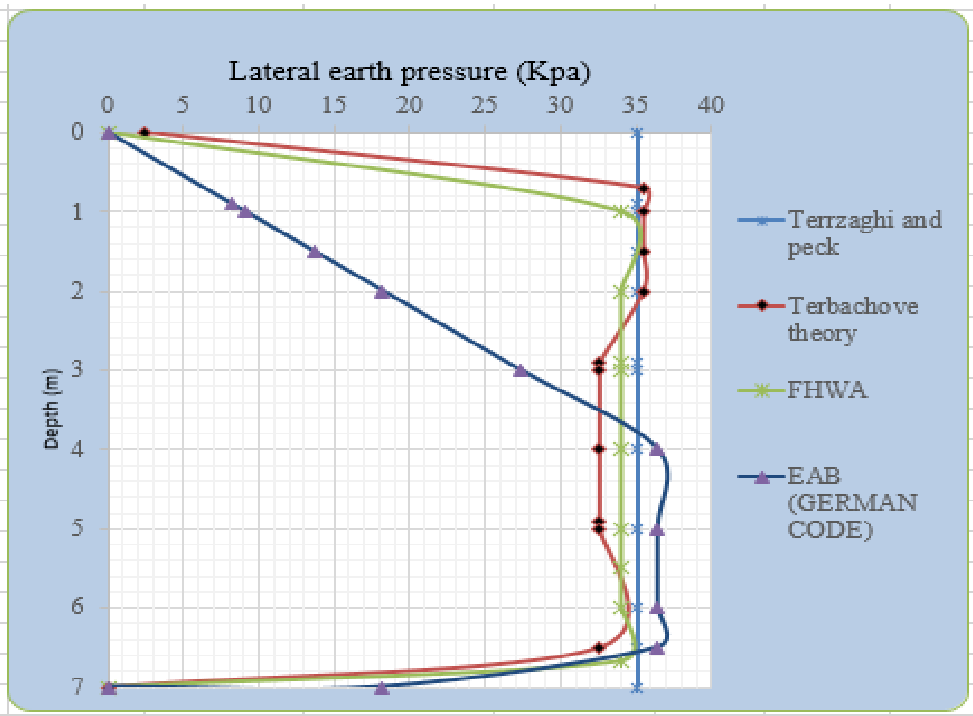


Figure 4-21: Apparent pressure using different approaches sandy soil

As we can see from Figure 4-21, the pressure envelope of Terzaghi, FHWA and EAB very similar maximum ordinate. On top and bottom of excavation Terzaghi's approach estimates 43.26KN/m^2 but the other three approaches estimates the pressure envelope to nil. In general, Terrzaghi's lateral pressures envelopes all other approaches.

Case b: Soft clay soil

A.Terzaghi and Peck method

According to Terzaghi and peck (1969) for soft to medium soil type the maximum probable pressure calculated using the following equation

$$P = (\gamma * H_e + q_s) * \left[1 - m\left(\frac{4c}{\gamma * H_e}\right)\right] \qquad (4.7)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

m=0.4

B.Terbachoves method

According to Terbachoves for soft to medium soil type the maximum probable pressure calculated using the following equation

$$P = 0.375 * (\gamma * H_e + q_s) \qquad (4.8)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

C.USA Federal highway Administration (FHWA)

According to FHWA for soft to medium soil type the maximum probable pressure calculated using the following equation

$$P = \frac{Total\ Load}{H - \frac{1}{3} * H_1 - \frac{1}{3} * H_{n+1}} \qquad (4.9)$$

$$K_a = 1 - \frac{4 * c}{\gamma * H} + 2\sqrt{2} * \frac{d}{H} * \left(1 - \frac{5.14 * c}{\gamma * H}\right) \qquad (4.10)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

D. German Code (EAB)

The pressure diagram according to Lehmann may be assumed a realistic for triple or multiple supported sheet pile walls and ratio of $eh_{ok:ehu,k}$=2 .The ordinates $eh_{o,k}$ and $eh_{u,k}$ are determined in such a way that the resultant force from Rankine theoretical pressure distribution will be equal to the resultant force from the apparent pressure diagram.

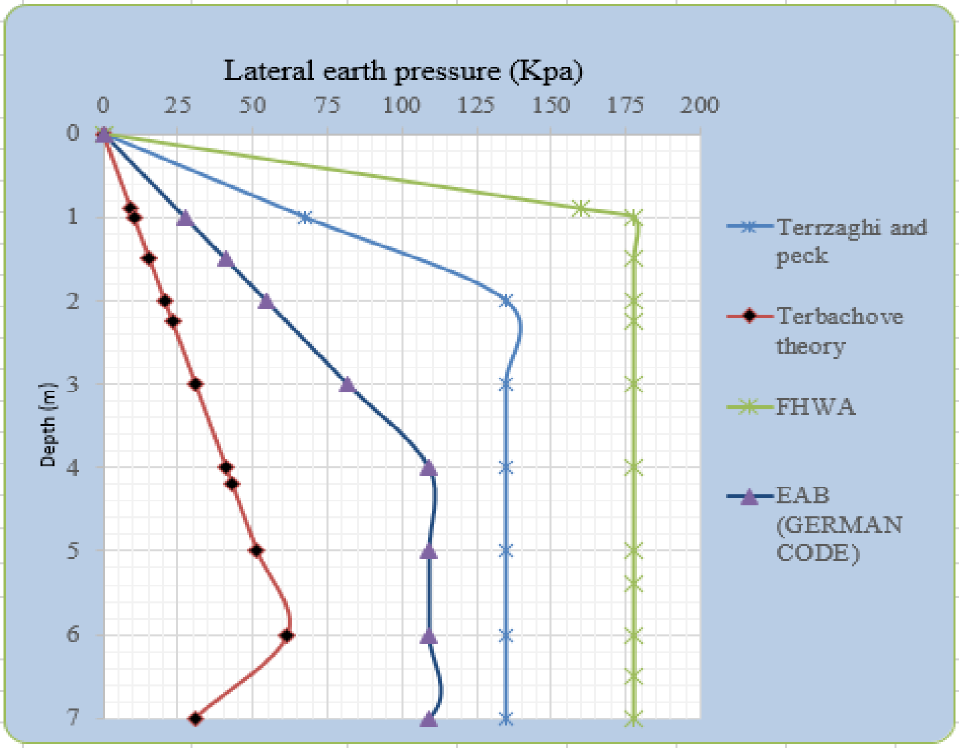


Figure 4-22: Apparent pressure using different approaches soft clay soil

As we can see from Figure 4-22, the pressure envelope of all approaches has very similar trend except at the bottom of excavation. The maximum pressure at bottom of excavation is180KN/m^2.

Case c: Layered soil type 1

The soil property used for this type of stratified soil shown below in Table 4-6 and average values of γ and c calculated as per Terzaghi theory.

Table 4-6: Soil property of each layer

Soil parameters	Unit	Sandy soil	Soft clay soil
Ø	degrees	30	17.75
C_{avg}	KN/m2	0	10
γ_{avg}	KN/m3	18.85	18.2
Depth of layer	m	4	5
He	m	9	
C_{avg}	KN/m2	7.39	
γ_{avg}	KN/m3	18.49	
γ_{avgc} *He/C_{avg}	-	23.87	
Soil property	-	Soft to medium clay	

A.Terzaghi and Peck method

According to Terzaghi and peck (1969) for sandy soil type the maximum probable pressure calculated using the following equation

$$P = (\gamma * H_e + q_s) * \left[1 - m\left(\frac{4c}{\gamma * H_e}\right)\right] \ldots\ldots (4.11)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

m=0.4

B.Terbachoves method

According to Terbachoves for sandy soil type the maximum probable pressure calculated using the following equation

$$P = 0.375 * (\gamma * H_e + q_s) \ldots\ldots (4.12)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

C.USA Federal highway Administration (FHWA)

According to FHWA for sandy soil type the maximum probable pressure calculated using the following equation

$$P = \frac{Total\ Load}{H - \frac{1}{3} * H_1 - \frac{1}{3} * H_{n+1}} \ldots\ldots (4.13)$$

$$K_a = 1 - \frac{4*c}{\gamma * H} + 2\sqrt{2} * \frac{d}{H} * \left(1 - \frac{5.14*c}{\gamma * H}\right) \ldots\ldots (4.14)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

D. German Code (EAB)

The pressure diagram according to Lehmann may be assumed a realistic for triple or multiple supported sheet pile walls and ratio of $eh_{ok:ehu,k}$=2 .The ordinates $eh_{o,k}$ and $eh_{u,k}$ are determined in such a way that the resultant force from Rankine theoretical pressure distribution will be equal to the resultant force from the apparent pressure diagram.

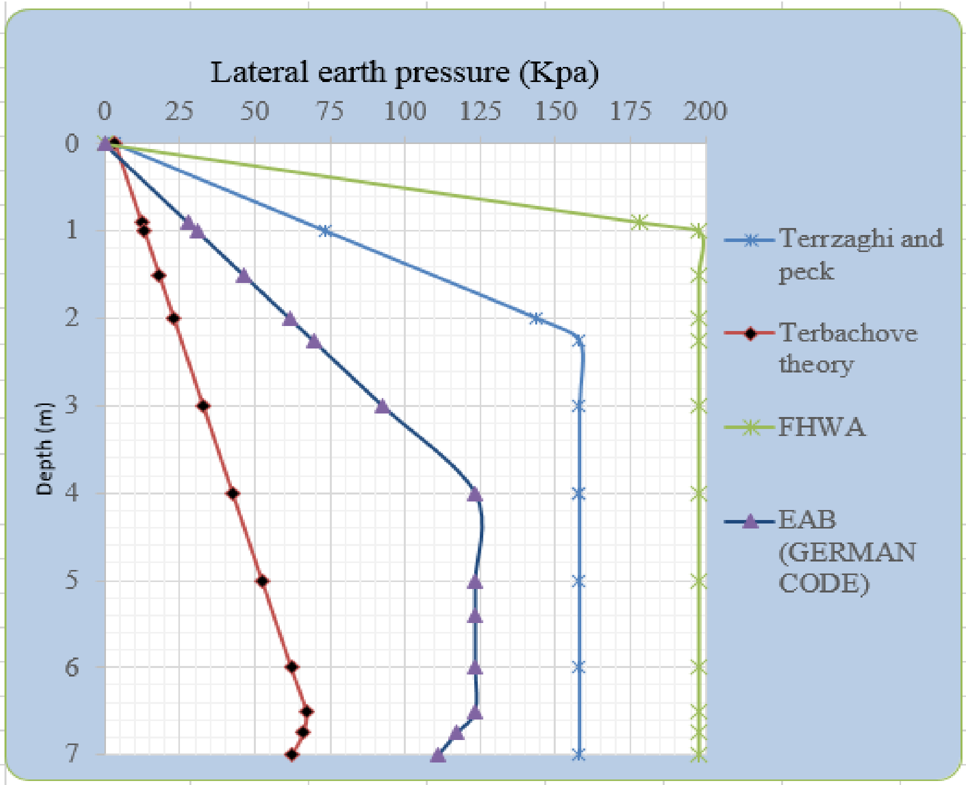


Figure 4-23: Apparent pressure using different approaches stratified soil type 1

As we can see from Figure 4-23, the pressure envelope of all approaches has very similar trend except at the bottom of excavation. The maximum pressure at bottom of excavation is197 KN/m^2.

Case D: Layered soil type 2

The soil property used for this type of stratified soil shown below in Table 4-7 and average values of γ and c calculated as per Terzaghi theory.

Table 4-7: Soil property of each layers

Soil parameters	Unit	Sandy soil	Soft clay soil	Soft to medium clay soil	Stiff clay soil
Ø	degrees	30	17.75	25	20
C_{avg}	KN/m2	0	10	25	20
γ_{avg}	KN/m3	18.85	18.2	19.58	20.57
Depth of layer	m	5	2	4	3.5
H_{avg}	m	12			
C_{avg}	KN/m2	12.67			
γ_{avg}	KN/m3	20.15			
γ_{avgc} *He/C_{avg}	-	19.87			
Soil property	-	Soft to medium clay			

A.Terzaghi and Peck method

According to Terzaghi and peck (1969) for sandy soil type the maximum probable pressure calculated using the following equation

$$P = (\gamma * H_e + q_s) * \left[1 - m\left(\frac{4c}{\gamma * H_e}\right)\right] \ldots\ldots (4.15)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

m=0.4

B.Terbachoves method

According to Terbachoves for sandy soil type the maximum probable pressure calculated using the following equation

$$P = 0.375 * (\gamma * H_e + q_s) \ldots\ldots (4.16)$$

Where P=lateral pressure

γ=unit weight of soil

K_a=coefficient of active earth pressure

qs=surcharge load

H=depth of excavation

C.USA Federal highway Administration (FHWA)

According to FHWA for sandy soil type the maximum probable pressure calculated using the following equation

$$P = \frac{Total\ Load}{H - \frac{1}{3} * H_1 - \frac{1}{3} * H_{n+1}} \ldots\ldots (4.17)$$

$$K_a = 1 - \frac{4*c}{\gamma * H} + 2\sqrt{2} * \frac{d}{H} * \left(1 - \frac{5.14*c}{\gamma * H}\right) \ldots\ldots (4.18)$$

Where P=lateral pressure

γ=unit weight of soil

K_a =coefficient of active earth pressure

q_s = surcharge load

H =depth of excavation

D. German Code (EAB)

The pressure diagram according to Lehmann may be assumed a realistic for triple or multiple supported sheet pile walls and ratio of $eh_{ok:ehu,k}$=2 .The ordinates $eh_{o,k}$ and $eh_{u,k}$ are determined in such a way that the resultant force from Rankine theoretical pressure distribution will be equal to the resultant force from the apparent pressure diagram.

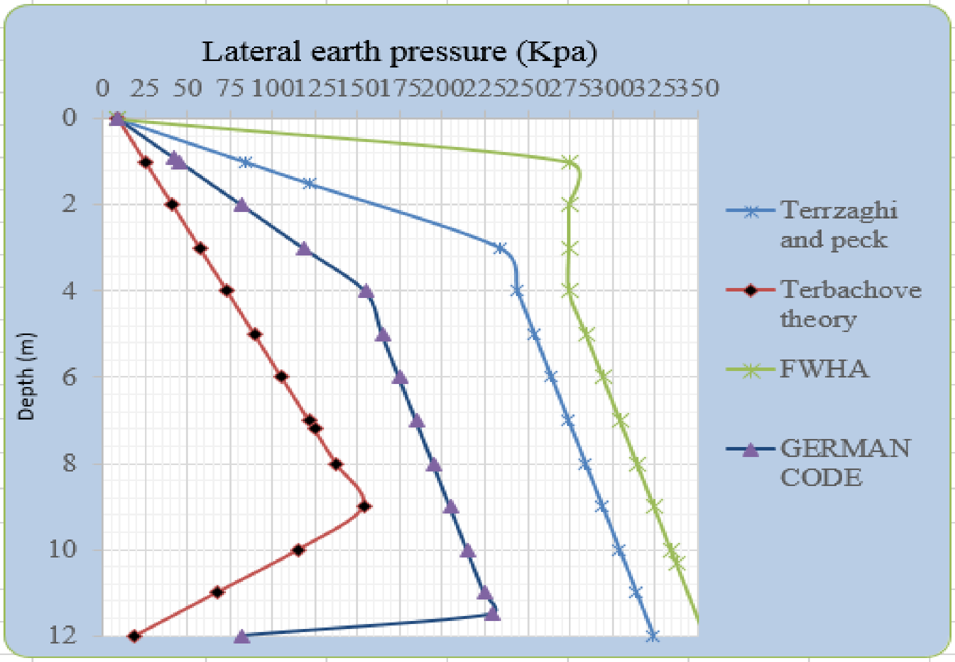


Figure 4-24: Apparent pressure using different approaches stratified soil type 2

As we can see from Figure 4-24, except Terbachoves's method the pressure envelope of the others three approaches has similar trend. The maximum pressure at bottom of excavation is 345 KN/m^2.

4.2.3 Determination of strut position and calculation of strut loads

Once the number of support levels and their respective locations determined, the first support level shall be installed at a depth below the ground surface less than the depth of the tensile crack. The positions and Young's modulus of each strut will be shown in the Table 4-8 for each case.

Table 4-8: Position of struts

Description	Unit	Sandy soil	Soft clay soil	Stratified soil type 1	Stratified soil type 2
Length of horizontal spacing of strut=	m	2.5	2.5	1.5	1.5
Depth of Vertical spacing of strut 1st level=	m	1.5	1.5	1.5	1.5
Depth of Vertical spacing of strut 2nd level=	m	2.5	2.5	2.5	2.5
Depth of Vertical spacing of strut 3rd level=	m	2.5	2.5	2.5	2.5
Depth of Vertical spacing of strut 4th level=	m	-	-	2.5	2.5
Depth of Vertical spacing of strut 5th level=	m	-	-	-	2.5
Depth of Vertical spacing of strut 6th level=	m	-	-	-	2.5
Depth of Vertical spacing of strut 7th level=	m	-	-	-	2.5
Young's modulus of strut (E)	Gpa	200	200	200	200

Table 4-9 Strut Loads calculated for each case

Strut Levels	Unit	Sandy soil	Soft clay soil	Stratified soil type 1	Stratified soil type 2
1st level	KN/m	117.621	258.53	237.75	315.46
2nd level	KN/m	75.35	373.38	326.69	553.56
3rd level	KN/m	55.14	294.31	342.09	641.29
4th level	KN/m	-	-	171.04	707.96
5th level	KN/m	-	-	-	410.42

Table 4-9 shows the strut loads at the each case of the excavation. It is evident from these values that the summation of the loads in the struts does reflect the pressure on the wall

4.2.4 Design of sheet pile wall

In this study, the length of sheet pile wall is determined by balancing the moment at the bottom strut level due to active and passive earth pressure on either side of the wall using Rankine earth pressure theory.

The maximum moment on sheet pile calculated based on Terrzaghi and peck's (1969) apparent earth pressure envelope diagram .The results of maximum moment, depth of embedment, properties of sheet pile according to Ou, 2006 shown in Table 4.10.

Table 4-10 Calculated results of sheet pile sections

Case	Maximum moment (KN-m)	Depth of embedemen t(m)	EA	EI	Weight (KN/m)
a. Sandy soil	41.35	2	9.3+E06	7.64+E04	0.904
b. Clay soil	101.44	10	1.22+E07	1.74+E05	1.177
c. Stratified soil type 1	106.99	3.7	1.53+E07	1.10+E06	1.472
d. Stratified soil type 2	171.48	2	1.53+E07	1.10+E06	1.472

4.2.5 Factor of safety against basal heave

Ukritchon et al. (2003) proposed a modified version of the Terzaghi (1943a) factor of safety against basal heave for including the wall embedment factor. The expression is given by:

$$\mathbf{FS_{heave}} = \frac{S_u N_c + \sqrt{2}\left(\frac{H}{B}\right) + 2S_u\left(\frac{D}{B}\right)}{\gamma_s H_e} \quad \text{(4.19)}$$

Where Su= cohesion

D =depth of excavation

B=width of excavation

S_u, N_c= the shear capacity

($\sqrt{2}$)*(H/ B) = the shear resistance of the soil mass

$2s_u$ *(D /B) = the adhesion along the inside faces of the wall assuming a rough surface.

It is advisable to have a factor of safety against basal heave, FS, higher than 1.5. The factor of safety calculation using different depth of embedment for each cases of excavation summarized as shown below in Table 4-11.

Table 4-11: Factor of safety against basal heave

Case	Depth of embedement (m)	Factor of safety according to Terrzaghi	Factor of safety according to Bjerrum and Eide	Factor of safety according to Ukritchon et al.
Sandy soil	2	Not applicable	Not applicable	Not applicable
Soft clay soil	10	1.04	0.90	1.11
Stratified soil type 1	3.7	1.02	1.00	1.04
Stratified soil type 2	2	1.04	0.89	1.05

4.2.6 Prediction of the maximum horizontal wall deformation, $\delta_{H\,(max)}$

Lateral wall movements and ground settlements are influenced by several factors including wall installation, soil conditions, factor of safety against basal heave, support system stiffness, and methods of support system installation.

The stiffness of an excavation support system is a function of the flexural rigidity of the wall element; the vertical and horizontal spacing of the supports; and the structural stiffness of the support elements and the type of connections between the wall and supports.

Clough et al. (1989) presented a design chart for clays which allows the user to estimate lateral movements in terms of effective system stiffness and the factor of safety against basal presented by Terzaghi (1943a).

The system stiffness combines the wall stiffness (EI), unit weight of water (γ_w) and h_{avg} the average spacing of the struts. Figure 4-49 was created from parametric studies using plane strain finite element analyses of sheet piles and slurry walls and it illustrates the influence of basal stability on movements and can be used to estimate maximum lateral wall movements in circumstances where displacements are primarily due to the excavation and support process .The result tabulated as shown in Table 4-12.

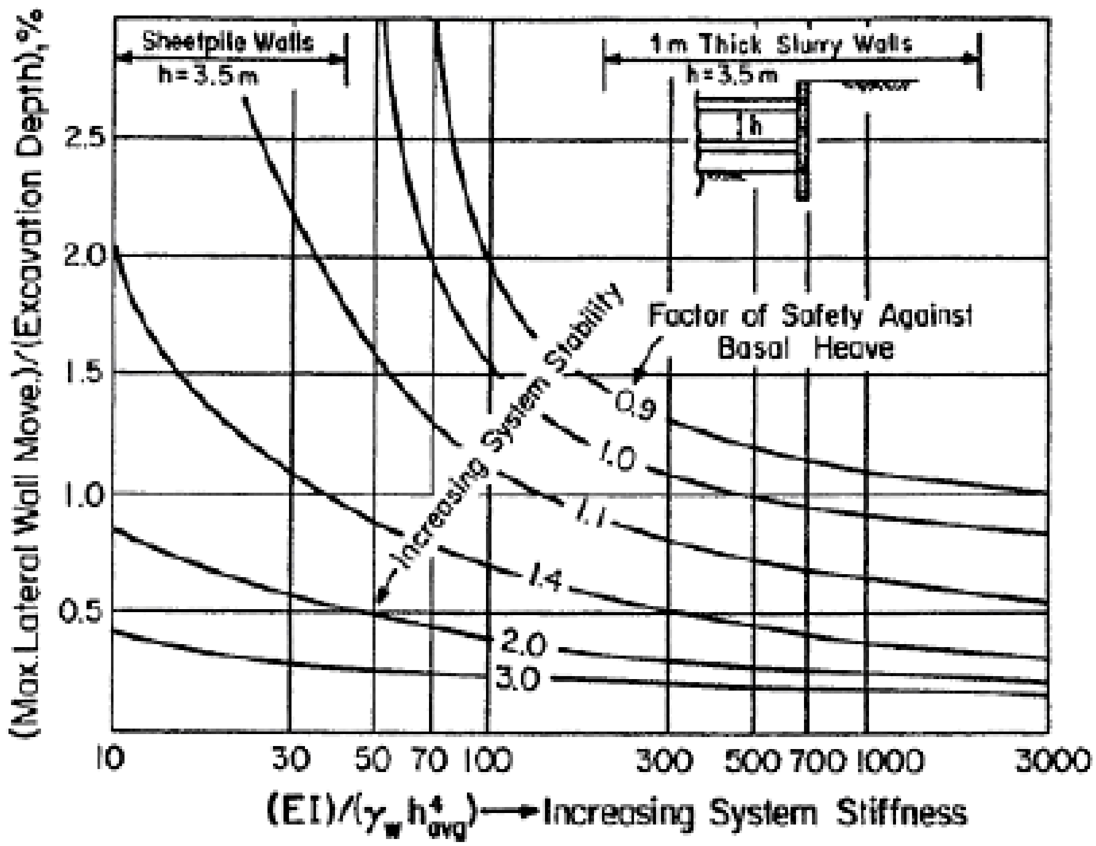


Figure 4-25: Correlation for Maximum Lateral Wall Deflection with Factor of safety Against Basal Heave and System Stiffness [Clough and O'Rourke, 1990]

Table 4-12: Maximum wall deflection against factor of safety

Case	$EI/\gamma_w h^4_{avg}$	He (m)	Max wall deflection/dep of excavation (%)	Factor safety against basal heave	Δhmax (m)
Sandy soil	4370	7	0.4	1.2	0.028
Soft clay soil	8690	8	0.55	1.11	0.044
Stratified soil type 1	4967	9	0.85	1.04	0.076
Stratified soil type 2	3820	12	0.85	1.05	0.102

4.3 Results and Discussion

4.3.1 Comparison of the Empirical and Plaxis Results

4.3.1.1 Earth pressure

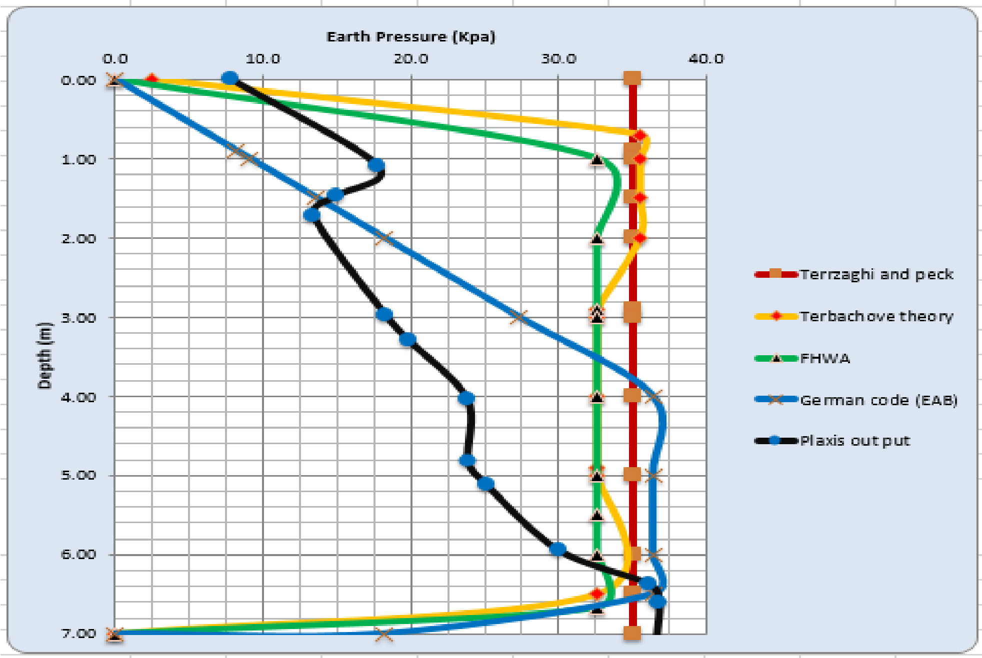


Figure 4-26: Comparison of different lateral earth pressure theories with Plaxis for sandy soil

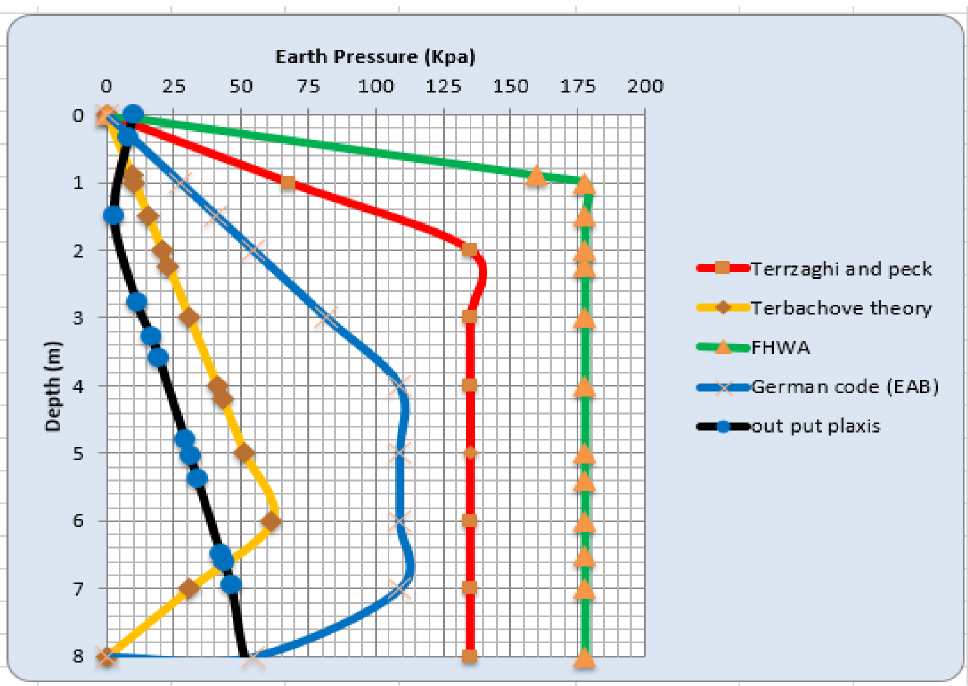


Figure 4-27: Comparison of different lateral earth pressure theories with Plaxis for soft clay soil

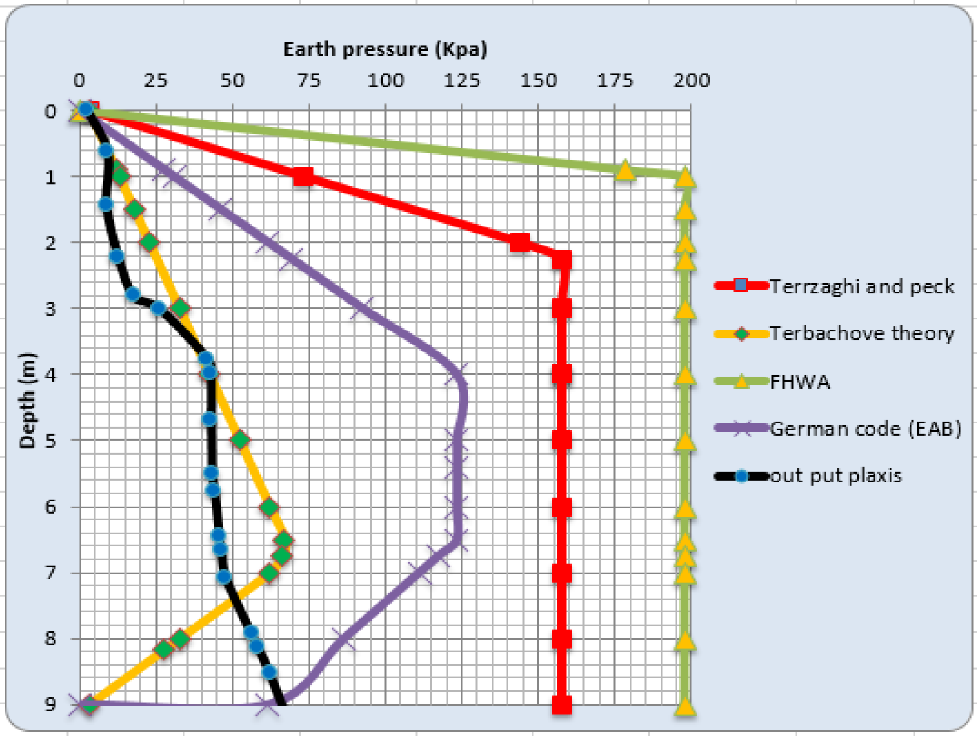


Figure 4-28 : Comparison of different lateral earth pressure theories with Plaxis for stratified soil type 1

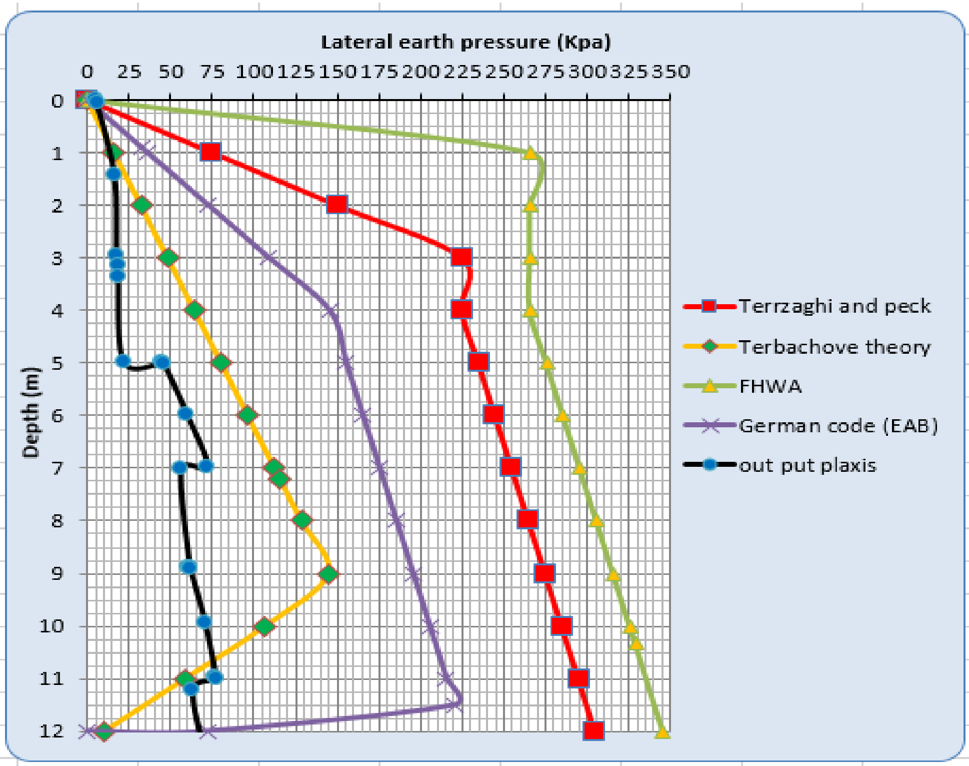


Figure 4-29 : Comparison of different lateral earth pressure theories with Plaxis for stratified soil type 2

As shown from the above graph in case of sandy soil the envelope of earth pressure suggested by EAB (German Recommendation) has better agreement with Plaxis than Terrzaghi, FHWA and Terbachoves estimation.

When we see in the case of soft clay soil, stratified soil type 1 and type 2 the envelope of earth pressure suggested by all existing theories and Plaxis out have better agreement at top surface of excavation.

Except in the case of sandy soil, Plaxis out has good agreement with Terbachoves estimation at middle of excavation. On the other hand, in the case of soft clay soil, stratified soil type 1 and type 2 the envelope of earth pressure suggested by EAB (German Recommendation) has better agreement with Plaxis at bottom of excavation.

4.3.1.2 Ground surface settlement

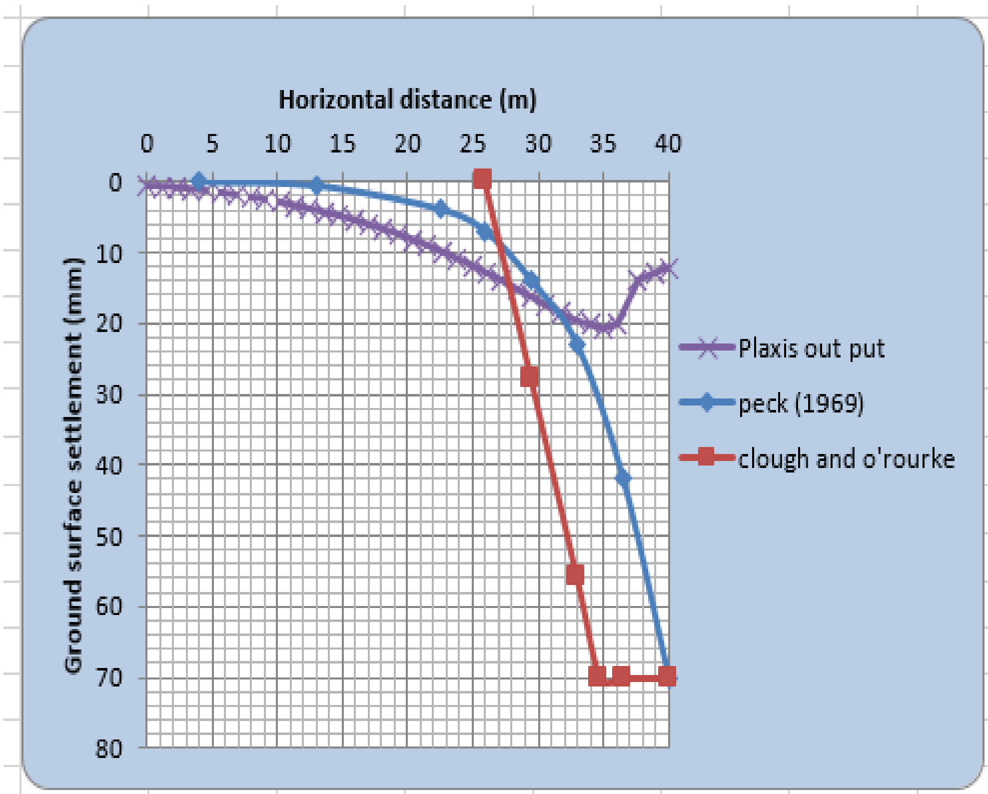


Figure 4-30: Ground surface settlement for sandy soil

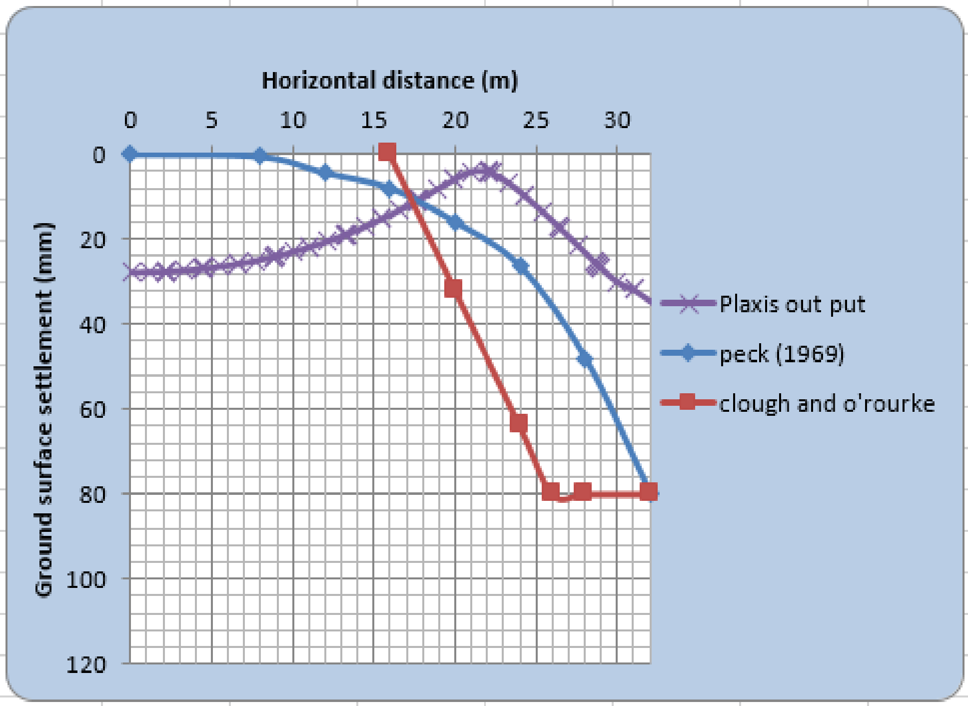


Figure 4-31: Ground surface settlement for soft clay soil

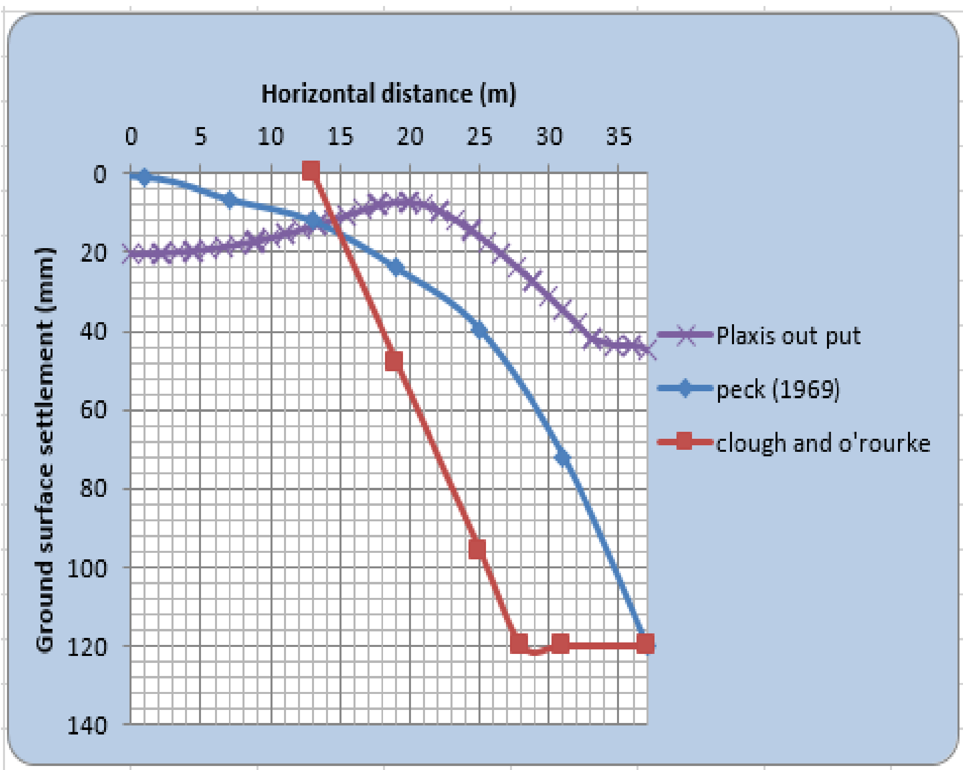


Figure 4-32: Ground surface settlement for stratified soil type 1

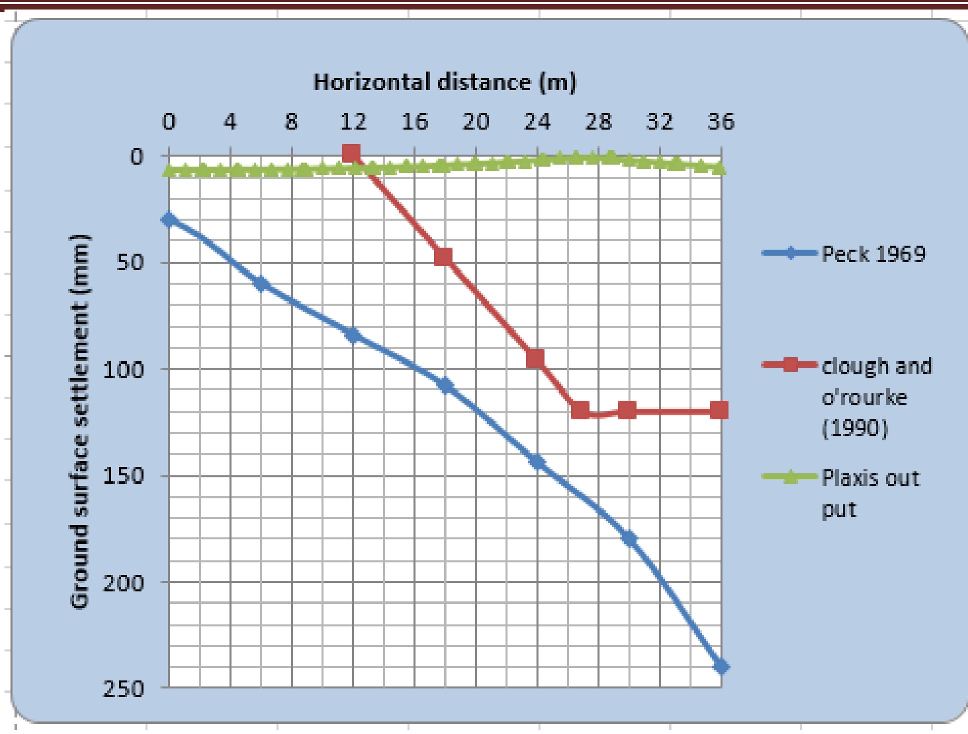


Figure 4-33: Ground surface settlement for stratified soil type 2

As it can be seen from Figure 4-30 up to Figure 4- 33 the maximum ground surface settlement according to peck'1969 and Clough and O'Rourke (1990) occurs near the sheet pile but in the case of Plaxis out this is not necessarily true.

4.3.1.3 Deformation of sheet pile wall

From the comparison Table 4-13 below it is clearly shown that the maximum wall deformation suggested from empirical formulas somehow higher than the Plaxis output. This is because of the envelope for lateral pressure suggested by existing theories are somehow higher than the Plaxis output.

Table 4-13 : Comparison of horizontal deformations

Description	Maximum horizontal deformation (mm)	
	Empirical calculations	Plaxis out put
Sandy soil	28	8.54
Soft clay soil	44	80.19
Stratified soil type 1	76	49.69
Stratified soil type 2	102	11.42

4.3.1.4 Strut loads at final excavation stage

From the comparison Table 4-14 below it is clearly shown that the summation of strut loads at final excavation stage suggested by Terrzaghi deviates from Plaxis result by 10.2% in case of sandy soil, by 73.55% in case of soft clay soil, by 32.71 % in case of stratified soil type 1 and by 103.83 % in case of stratified soil type 2.

Table 4-14: Comparison of strut loads

Strut Levels	Unit	Sandy soil		Soft clay soil		Stratified soil type 1		Stratified soil type 2	
		Empirical	Plaxis	Empirical	Plaxis	Empirical	Plaxis	Empirical	Plaxis
A	KN/m	117.621	59.92	258.53	162.16	237.75	64.99	315.46	100.404
B	KN/m	75.35	89.74	373.38	93.34	326.69	201.36	553.56	249.38
C	KN/m	55.14	75.48	294.31	278.20	342.09	269.04	641.29	347.875
D	KN/m	-		-		171.04	276.57	707.96	313.282
E	KN/m	-		-		-		410.42	278.694

4.3.2 Bending moment of the sheet pile wall

From the Figure 4-34 up to Figure 4-37 below it is clearly shown that the maximum bending moment of wall calculated from Terzaghi apparent earth pressure diagram greater than PLAXIS output in the case of sandy soil by 180 %, in case of soft clay soil by 112.50%, in case stratified soil type 1 by 73.33 % and in case d by 75%..This implies that the section modulus of sheet pile estimated by Terrzaghi method will be greater than Plaxis output.

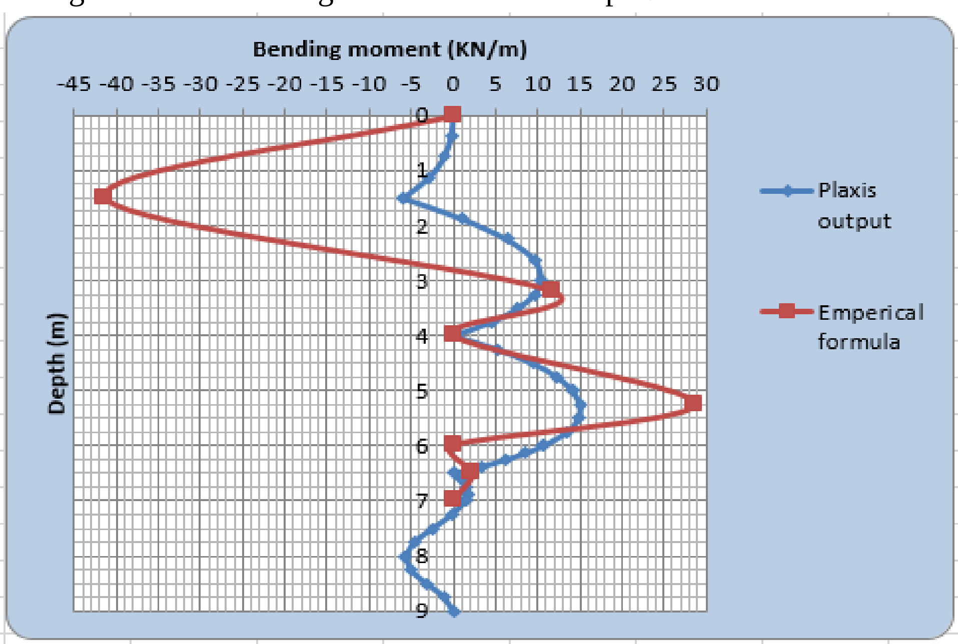


Figure 4-34: Bending moment for sandy soil

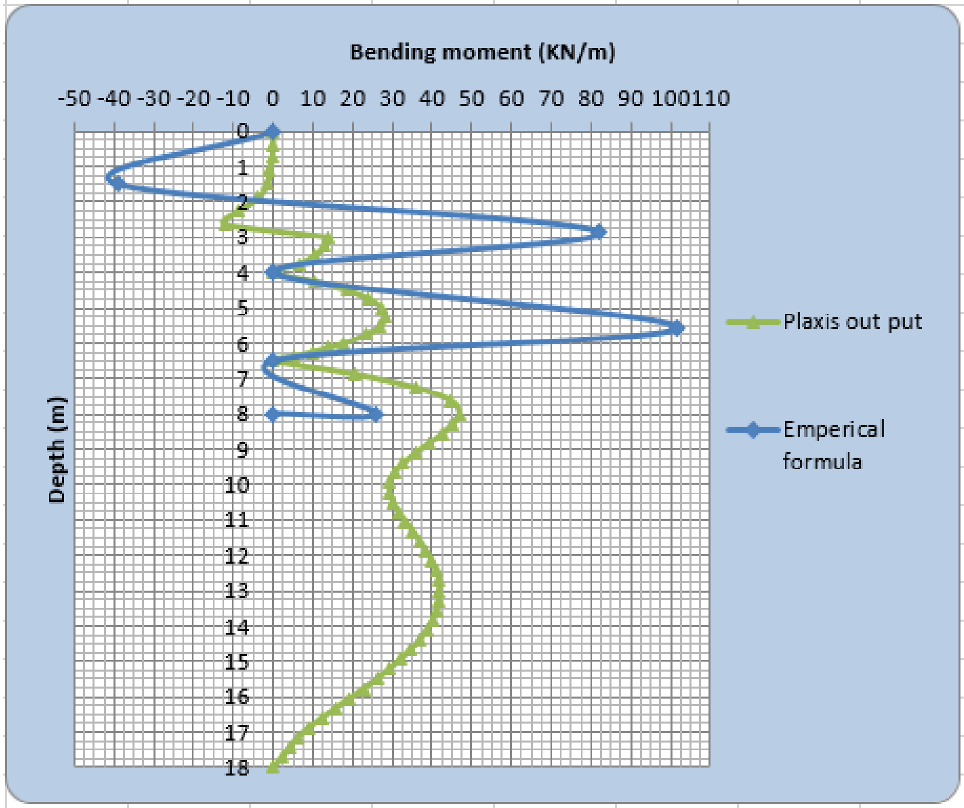


Figure 4-35: Bending moment for soft clay soil

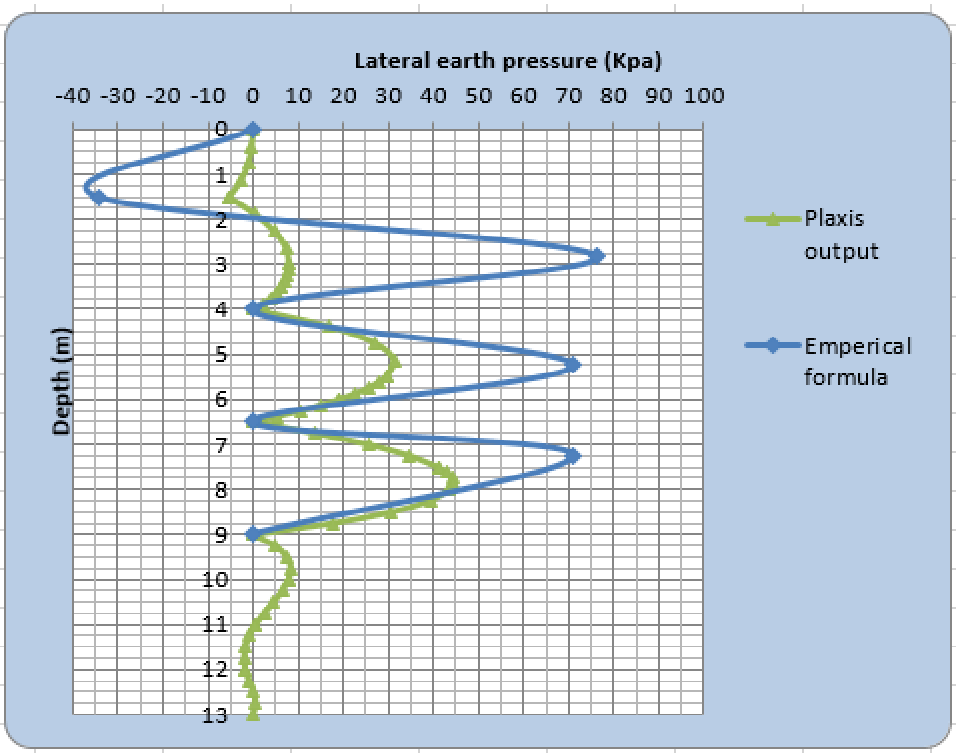


Figure 4-36: Bending moment for stratified soil type 1

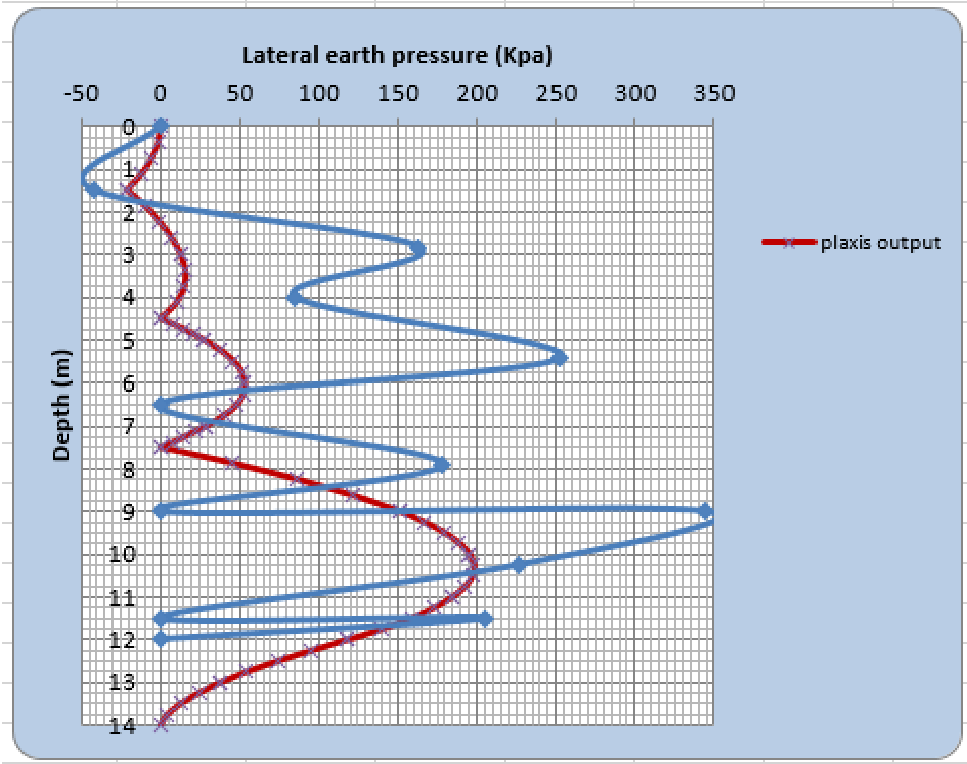


Figure 4-37: Bending moment for stratified soil type 2

4.3.2.1 Factor of safety

According to the comparison shown in Table 4-15 most of the empirical formulas do not consider the embedment depth contribution to failure against basal heave. Plaxis considers the shear failure surfaces of soil and gives a higher safety factors than empirical formulas as shown below.

Table 4-15: Comparison of factor of safety

Description	Factor of safety	
	Empirical formulas	Plaxis out put
Case a	-	2.424
Case b	1.11	2.736
Case c	1.04	2.101
Case d	1.05	2.163

5. Conclusion and Recommendations

Based on the study, the following conclusions can be made:

1. The lateral earth pressure of a deep excavation is significantly influenced by the type of soil, spacing of struts, stiffness of strut, and stages of excavation. Therefore, it is important to have accurate estimates of soil parameters in order to give better estimates of lateral pressure for deep excavations.

2. The summation of the strut loads computed by PLAXIS is 10.20% to 103.83% lower than the Terzaghi method for excavation cases studied.

3. The maximum moments computed by PLAXIS are considerably lower than from 73.33% to 180% those by Terrzaghi method, which gives us lesser section modulus of sheet pile.

4. It has been determined that the factor of safety calculated by PLAXIS is greater than other empirical methods because it considers the most shear failure surfaces of soil.

5. The envelope of lateral earth pressure should be used as a preliminary determination of structural parts of deep excavation because it does not consider stages of excavation, the pressure distribution of lateral pressure under the required level, surcharge load and the stiffness of the retaining structures.

6. The lateral pressure increases significantly when the spacing of strut increases and stiffness of struts decreases but the depth of embedment and stiffness of sheet pile do not have a significant influence on lateral pressures.

7. The effects of stiffness and depth of embedment of sheet pile have a great impact on deformation of ground surface rather than lateral earth pressure.

8. To consider the effect of the lateral pressure at corner points using 3D PLAXIS software will be preferable and will give us better results.

9. If triaxial tests conducted for actual soil, installation of pressure gages and inclinometer on retaining structures are available, it will be very helpful to analyze in more depth the behavior of retaining structures of deep excavation using other constitutive soil models.

6 .Bibliography

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Appendices

A.1.Procedures for design of braced sheet piles using empirical methods

A proposed method that allows the designer to predict final horizontal wall displacements and vertical ground settlements,given data about soil and support system is presented in this section. The necessary steps for the design of the excavation support system and the determination of the ground movements are numbered as follows:

1. **Define soil properties and excavation geometry**: for each layer of soil determine unit weight, γs ; undrained shear strength, su ; effective friction angle, φ' ; and reference secant modulus, Es . For multiple layers, make a weighted average to find the soil design parameters for the excavation. Also, define the plan dimensions of the excavation(i.e., width, B , and length, L) and the final excavation depth, H e .
2. **Define support system parameters**: based on the plan dimensions and the required construction equipment to use in the excavation, define the average vertical and horizontal support spacing respectively to allow for enough space for accommodation. In addition, define the wall Young's modulus and an initial guess value for the wall moment of inertia per unit length. For sheet pile walls, both parameters are listed by the manufacturer.
3. **Determined the lateral earth pressure envelope**: determine the shape of the lateral earth pressure diagram. For a layered soil profile, determine which layer of soil is the dominant within the deep of the excavation and use those properties for design, or apply Peck's (1943) equivalent undrained shear strength, and unit weight
4. **Define strut levels:** based on the average vertical support spacing defined in Step 2, define the number of support levels and their respective locations. It is advisable to have the first support level installed at a depth below the ground surface less than the depth of the tensile crack.
5. **Calculate strut loads:** the two most commonly used methods for calculating the loads in the struts are the internal hinge and the tributary area methods. The internal hinge method assumes a pivot generally located at the midpoint of the excavation depth in order to obtain a statically determinate structure. If it is necessary, more pivot locations can be assumed in order to satisfy statically determinate conditions. The strut load equations obtained after applying equilibrium to

the three strut excavation system illustrated in Figure A-1 (a) are presented in Figure A-1(b). The tributary area method is a much more simplified approach where no equilibrium conditions are satisfied. Figure A-1(c) illustrates its procedure and presents the necessary equations for calculating the strut loads using this approach.

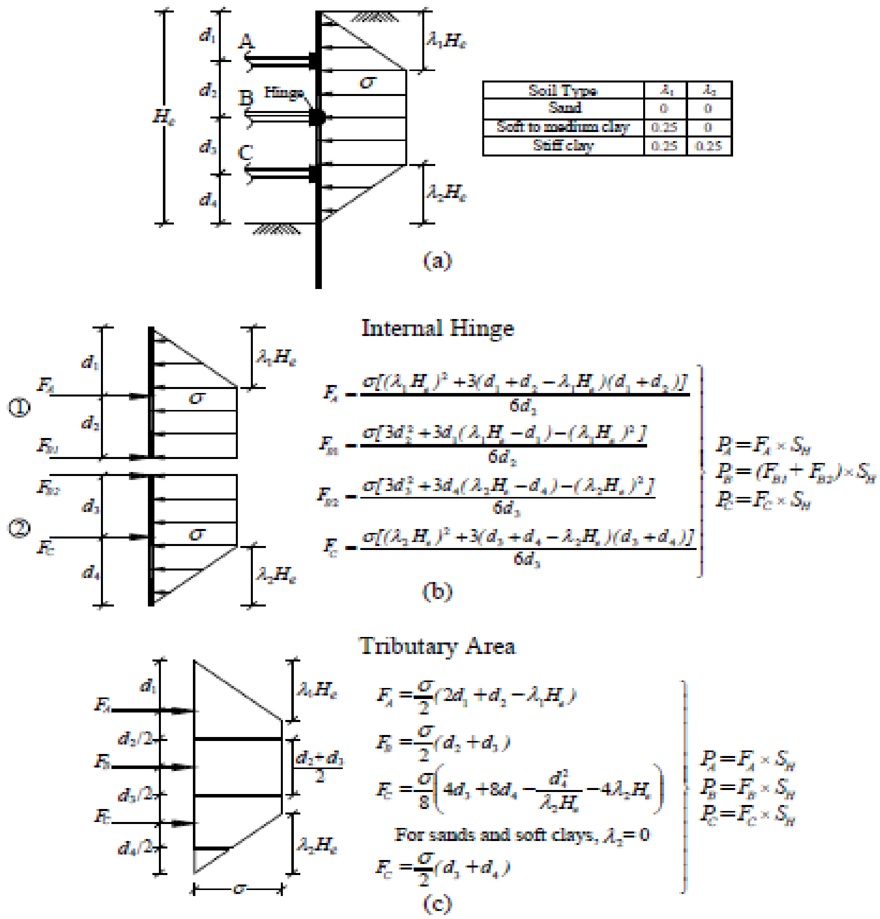

Figure A-1: calculation of strut loads

6. **Select proper struts sections**: commonly, circular steel pipes are used as horizontal supports in deep excavations because of their symmetry cross section and simplified design. In this step, the struts are sized based on the load and resistance factor design specification for I steel or hollow structural sections presented by the Manual of Steel.
7. **Calculate the maximum moment in the wales**: the wales may be treated as a continuous horizontal member if they are spliced properly. They may also be treated as though they are pinned at the struts, but this is a very conservative approach.

8. ***Calculate the required wale section modulus***: the required section modulus of the wales is calculated as: where *max* is the maximum bending moment at the wale (calculated in Step 7) and σ_{all} is the allowable flexural stress of the wale material.
9. ***Size the wales*****:** to size the wales, just choose a steel section such as *req. wale* $S \geq S$
10. **Determine the wall design earth pressure**:
11. **Calculate the required wall embedment depth**: once the strut loads and the wall design earth Pressure is determined from Steps 5 and 10, respectively, find the required wall embedment, *D,* This equation is found by applying moment equilibrium at the last strut at the bottom of the system.
12. ***Calculate the maximum wall bending moment:*** Once *D* is determined, find the maximum bending moment in the wall, *max M,* by applying static equilibrium to the system.
13. **Calculate the required wall section modulus**: the required section modulus of the wall is calculated in the same form as for the wales: where *max* is the maximum bending moment at the wall (calculated in Step 12) and *Mall* σ is the allowable flexural stress of the wall material.
14. ***Size the wall:*** from the sheet pile wall section properties tables provided by the fabricant, choose a sheet pile wall such as r_{eq}. To size this type of wall, first, assume the thickness, *t,* of the wall.
15. **Calculate the factor of safety against basal heave**: use which includes the wall embedment depth below the excavation level.
16. **Check the factor of safety value:** it is advisable to have a factor of safety against basal heaves, higher than 1.5. If the computed *FS* is less than 1.5, go back to Step 11 and increase the wall embedment depth below the excavation level until an adequate factor of safety is obtained.
17. **Calculate relative stiffness ratio, R**:.
18. **Predict the maximum horizontal wall deformation, H (max) δ**: calculate *H (max)* δ.
19. **Predict the maximum vertical settlement, V (max) δ**: calculate *V (max)* δ.

A.2.Plaxis output

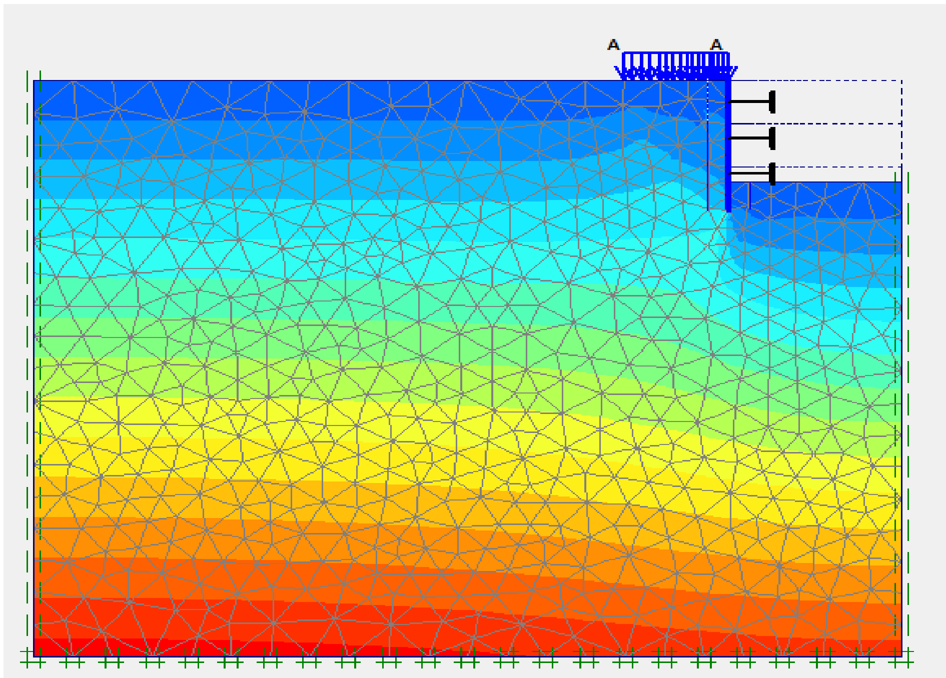

Figure A-2: Deformed shape of the geometry in sandy soil

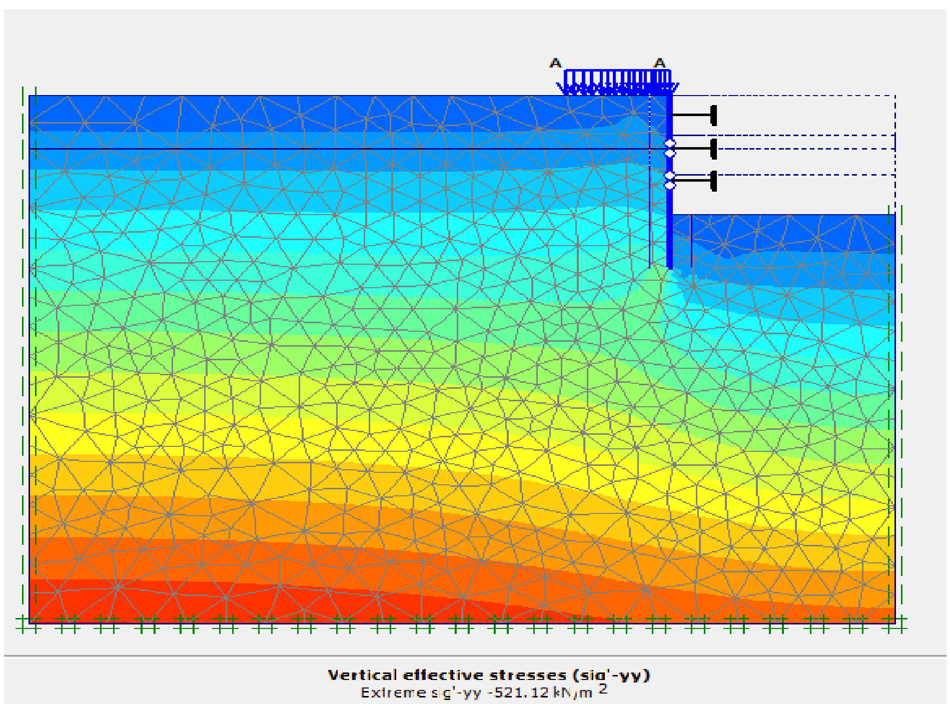

Figure A-3 : Deformed shape of excavation in soft clay soil

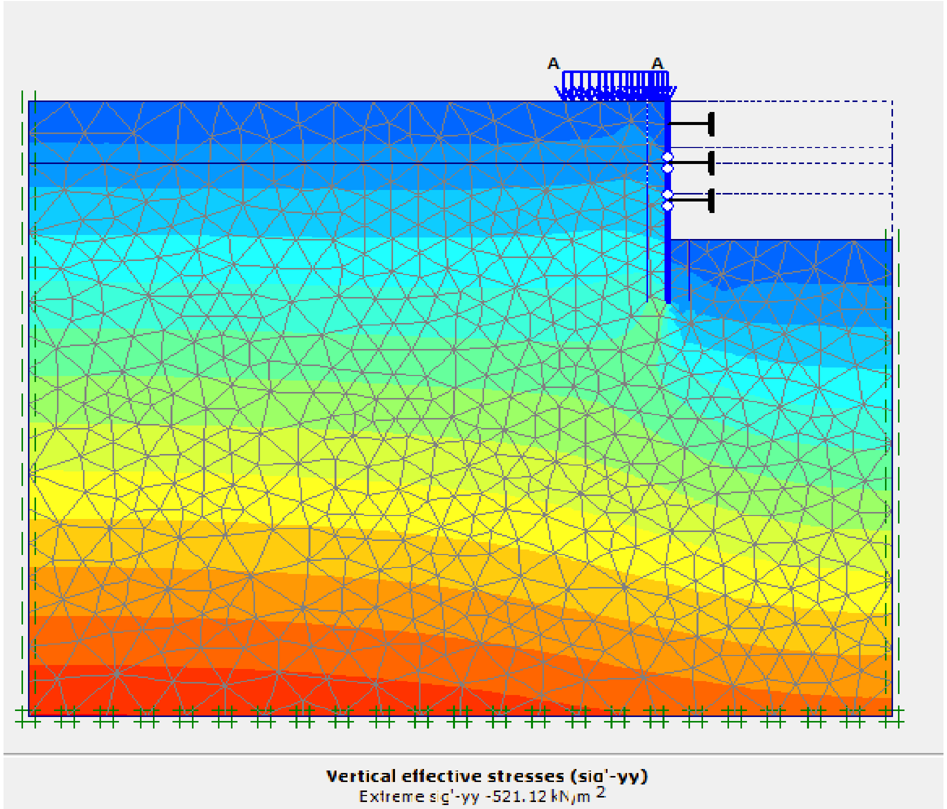

Figure A-4: Deformed shape of the geometry in stratified soil type 1

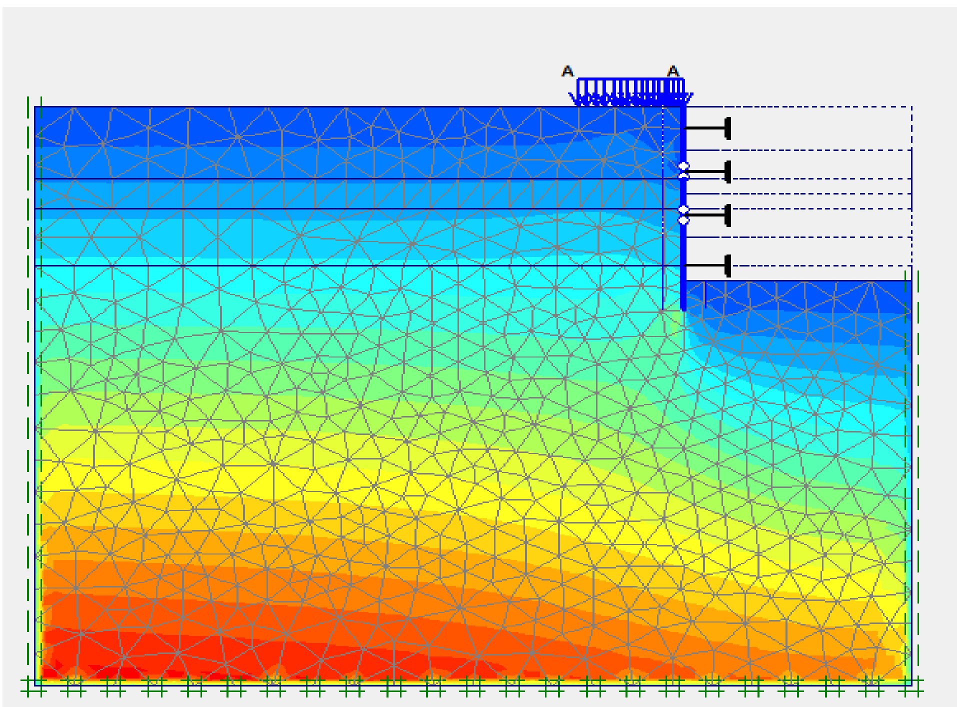

Figure A-5: Deformed shape of the geometry in stratified soil type 2

A.3.Soil model

Mohr-Coulomb Model

The Mohr-Coulomb model coded in Plaxis version 8.0 is based on an elastic perfectly-plastic mode which is constitutive law with a fixed yield surface, i.e. a yield surface that is fully defined by model parameters and not affected by (plastic) straining. For stress state represented by points within the yield surface, the behavior is purely elastic and all strains are reversible. There is no hardening or softening law required for the Mohr-Coulomb model as it is assumed to be perfectly plastic. A plastic yield function, *f*, is introduced as a function of stress and strain that can often be presented as a surface in principal stress space as shown in Figure below.

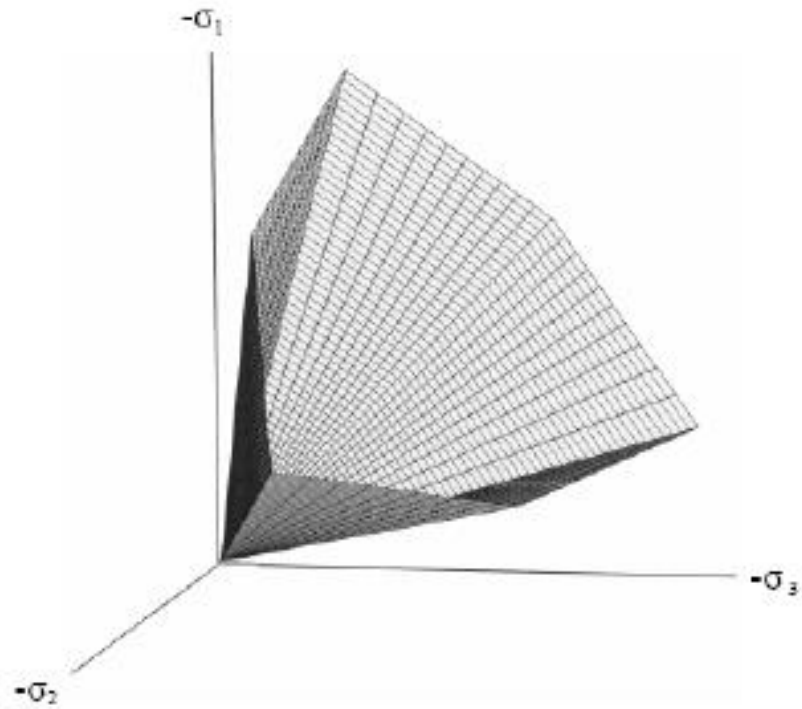

Figure A-6: The Mohr-Coulomb yield surface in principal stress space (c = 0)

The strains and strain rates are decomposed into an elastic part and a plastic part in theory of elastoplasticity:

$$\varepsilon = \varepsilon e + \varepsilon p = + \quad \text{......} \quad (A.1)$$

The Mohr-Coulomb model requires five parameters. Parameters related to elastic behavior are E and ν, whereas parameters related to plastic behavior are c and φ, and ψ, angle of dilatancy. These parameters with their standard units are listed below:

Table A-1: Mohr coulomb parameters

PARAMETER		UNIT
E	Young's Modulus	[kN/m^2]
ν	Poisson's ratio	[-]
Φ	Friction angle	[°]
C	Cohesion	[kN/m^2]
ψ	Dilatancy angle	[°]

PLAXIS uses the Young's modulus as the basic stiffness modulus in the Mohr-Coulomb model. E_{ur} which tend to increase with the confining pressure is needed for unloading modeling such as in the case of excavations. This behavior in turn results in the deep soil layers tend to have greater stiffness than shallow layers. Hence, when using a constant stiffness modulus to represent soil behavior, a value that is consistent with the stress level and the stress path development should be chosen.

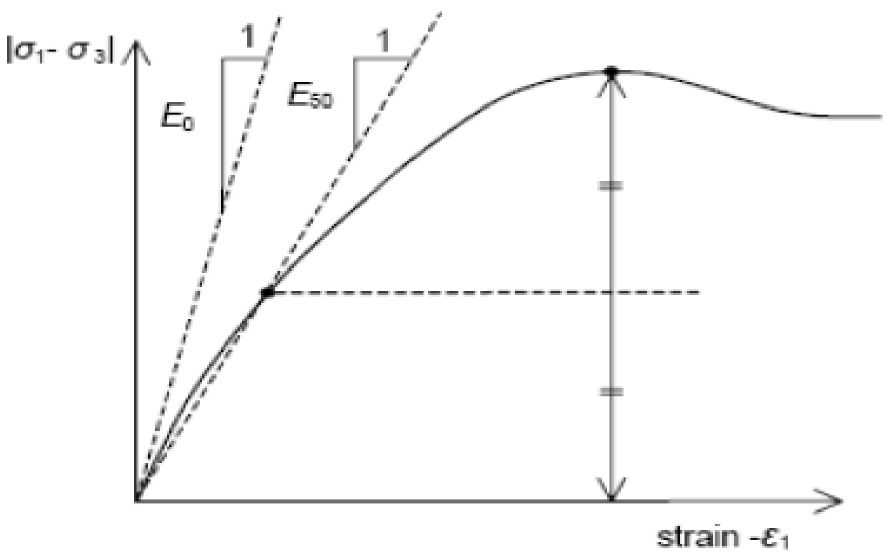

Figure A-7: Definition of E_0 and E_{50} for standard drained triaxial test results

Although standard drained triaxial tests may yield a significant rate of volume decrease at the very beginning of axial loading, but PLAXIS recommended the use of a high value initial value of Poisson's ratio (ν0) when using the Mohr-Coulomb model. In general, for unloading conditions, however, PLAXIS suggested to use values in the range between 0.15 and 0.25. PLAXIS can handle cohesion less sands (c = 0), but some options will not perform well. To avoid complications, non-experienced users are advised to enter at least a small value (use c > 0.2 kPa). The friction angle, Φ, is entered in degrees. High friction angles, as sometimes obtained for dense sands, will substantially increase plastic computational effort. PLAXIS suggested using this model as first analysis of the problem considered by approximating a constant average stiffness for each layer of soil to obtain a first impression of deformations.

A.4.Deformation of sheet pile wall at different stages of excavations

In this part of analysis, the study concentrates on how deformation of sheet pile wall differ in every stage of excavations for 7m, 8m,12m and 17m depth of excavation. The output of this analysis is presented for each cases of excavation from Figure A-8 to Figure A-11 as follows and the arrows indicate the positions of struts.

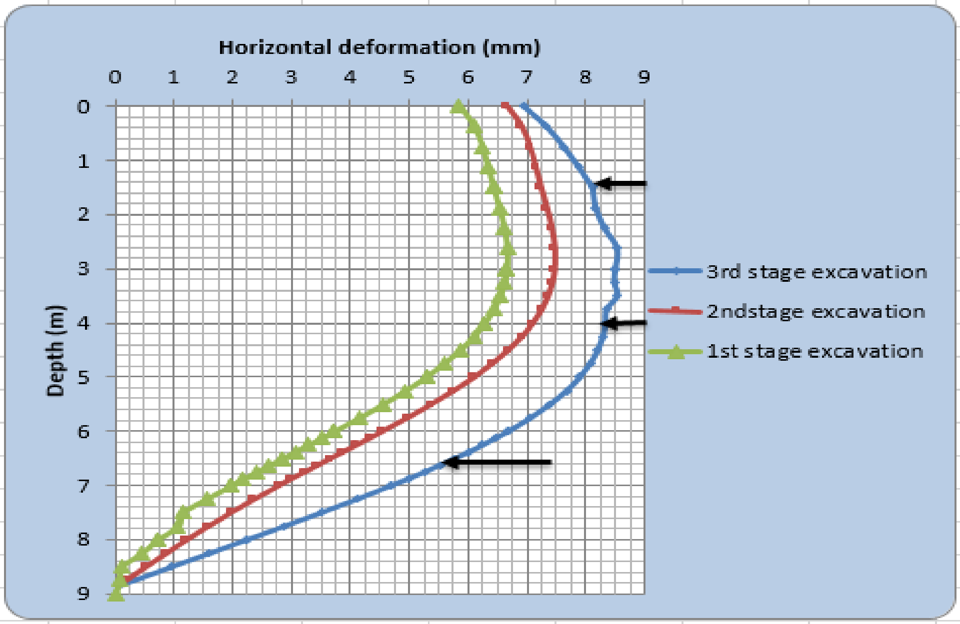

Figure A-8 Deformation of sheet piles at different stages of excavation of sandy soil

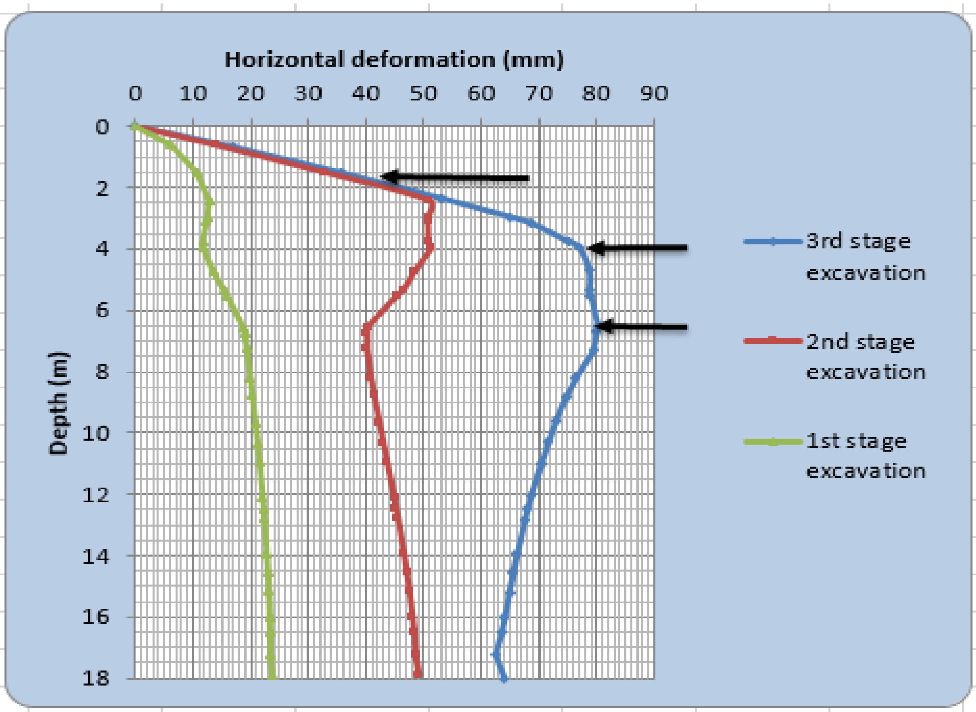

Figure A-9: Deformation of sheet piles at different stages of excavation soft clay soil

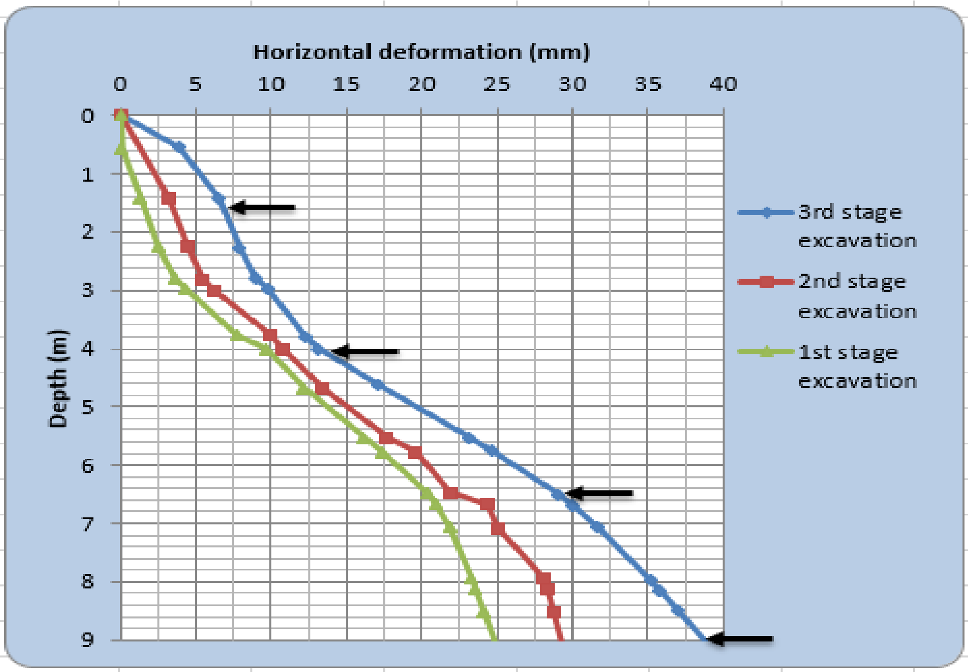

Figure A-10 Deformation of sheet piles at different stages of excavation stratified soil type 1

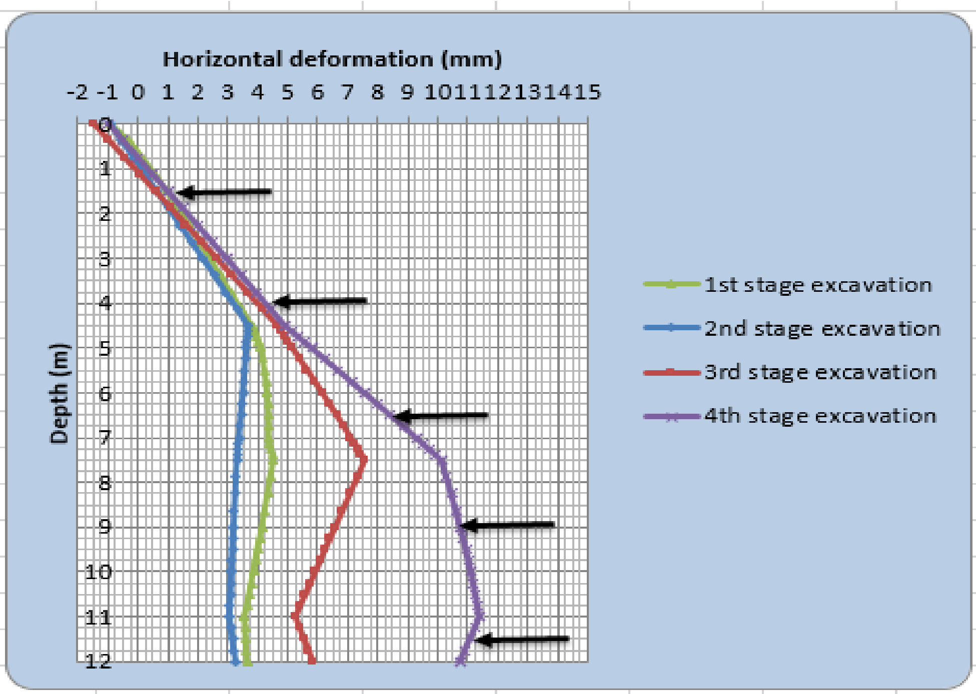

Figure A-11. Deformation of sheet piles at different stages of excavation stratified soil type 2

In Figures A-8 and Figure A-9, the lateral maximum wall deformation occurs at depth of 3m and 4m respectively whereas in Figure A-10 and Figure A-11 maximum wall deformation occurs at the bottom of excavation.

A.5.Bending moment of the sheet pile wall at different stages of excavation

One of the significant benefits of Finite element modeling structures braced excavations is that it is possible to investigate the bending moments and structural forces which can be used to check that they have sufficient capacity to withstand the resulting stresses.

In this part of analysis, the study concentrates on bending moment diagram of sheet pile wall in every stage of excavations for 7m, 8m, 9m and 12m depth of excavation. The output of this analysis is presented for each cases of excavation from Figure A-12 to Figure A-15 as follows.

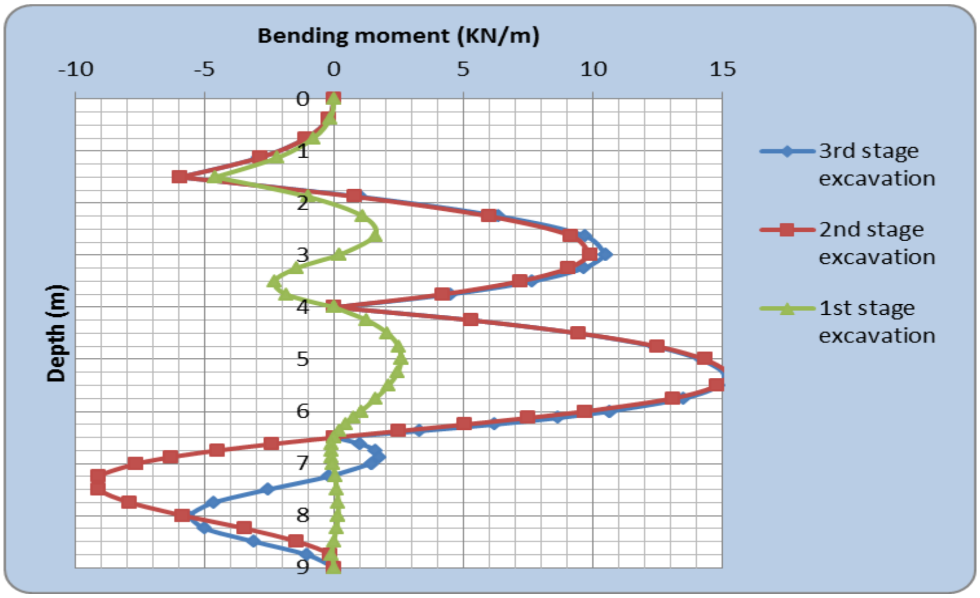

Figure A-12: Bending moment of the sheet pile wall at stages of excavation of sandy soil

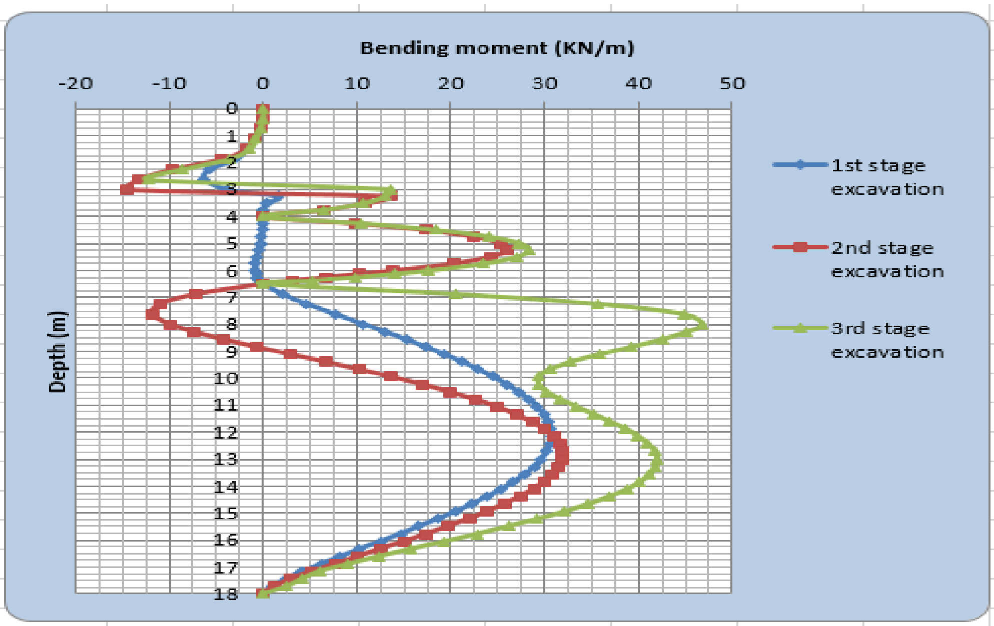

Figure A-13: Bending moment of the sheet pile wall at stages of excavation soft clay soil

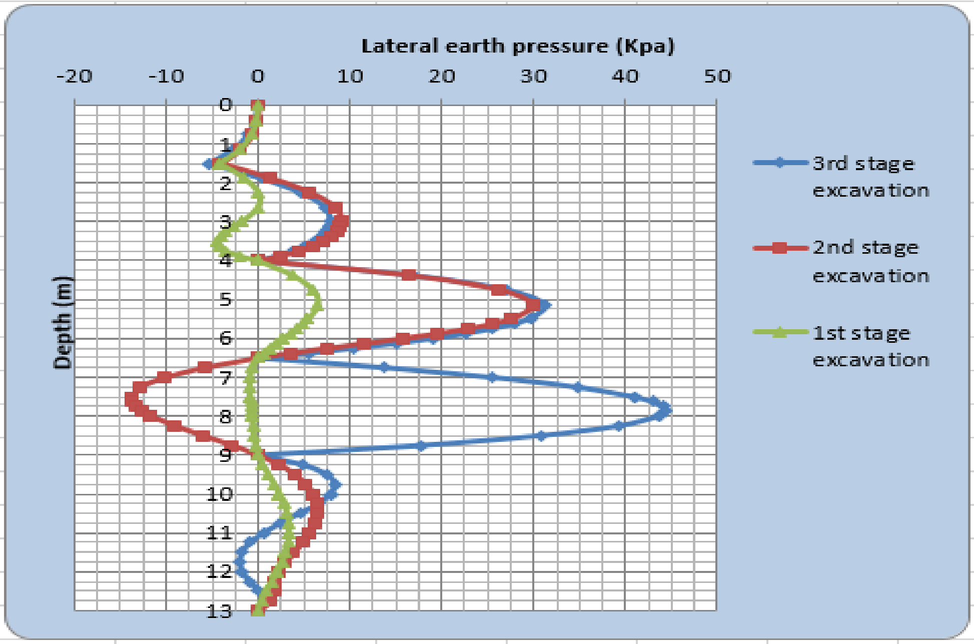

Figure A-14 Bending moment of the sheet pile wall at stages of excavation stratified soil 1

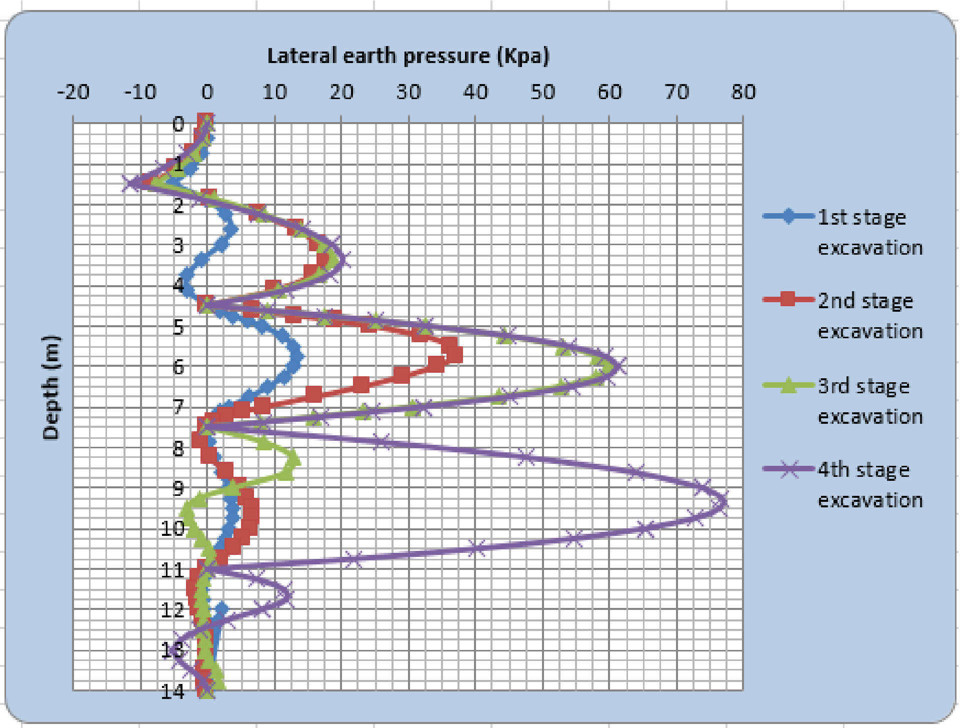

Figure A-15: Bending moment of the sheet pile wall at stages of excavation stratified soil 2

The maximum bending moments as shown from Figure A-13 to Figure A-16 are in sandy soil 15 KN-m, in soft clay soil is 157.5 KN-m, in stratified soil type (1) is 122.5KN-m and in stratified soil type (2) is 72.5KN-m.

A.6.Ground surface settlement at different stages of excavations

The settlement behind a braced excavation wall is always a key issue. A Plaxis 2D software analysis makes it possible to obtain a picture of the possible magnitude and trend. The surface settlements for the four cases in every stage of excavation are shown from Figure A-16 up to figure A-19.

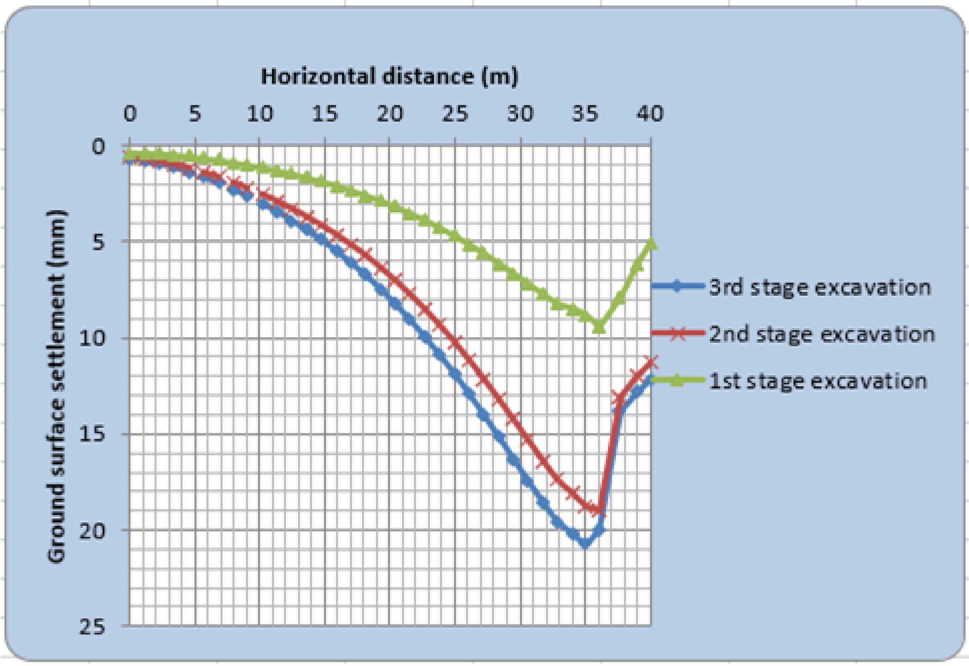

Figure A-16: Ground surface settlement of sandy soil

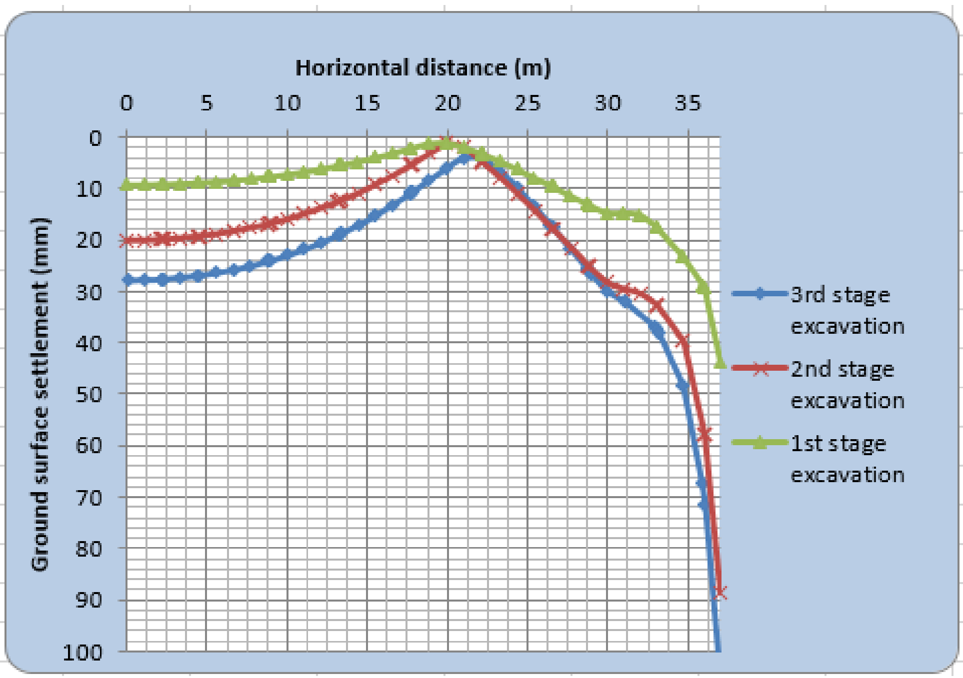

Figure A-17: Ground surface settlement softy clay soil

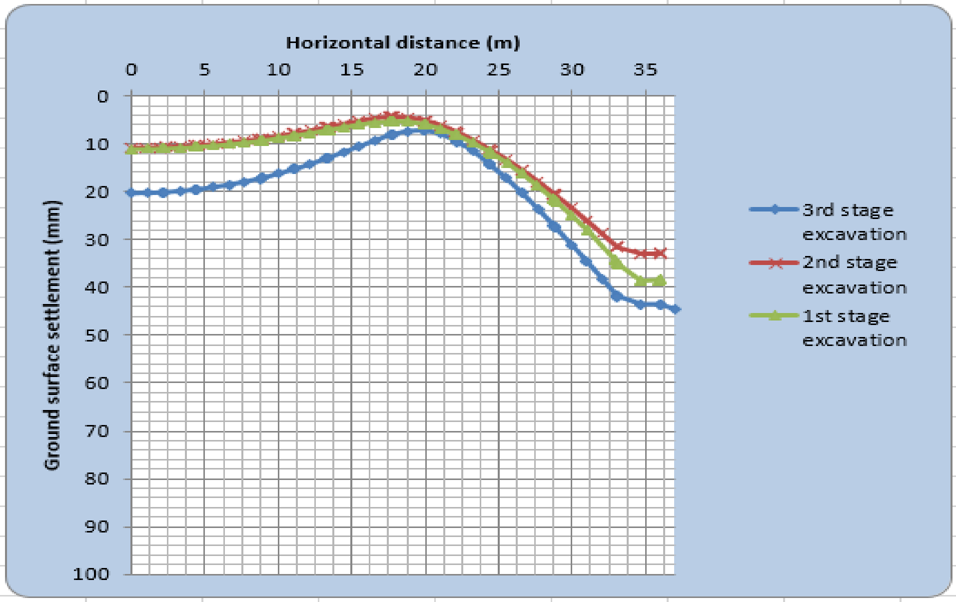

Figure A-18: Ground surface settlement stratified soil type 1

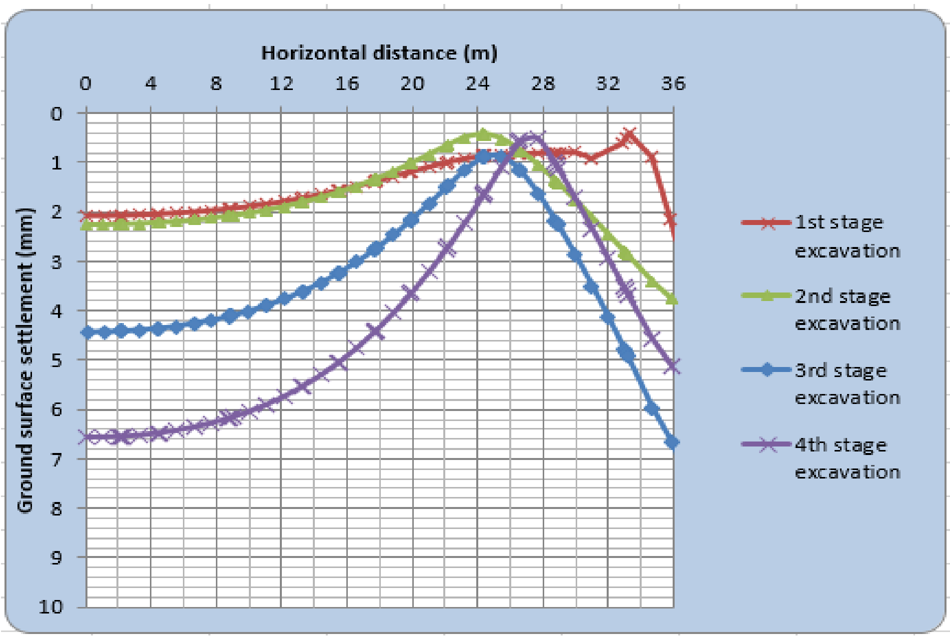

Figure A-19: Ground surface settlement stratified soil type 2

The maximum ground surface settlement as shown in the above graphs are in sandy soil 21mm at a distance of 5m from the sheet pile, in soft clay soil is 100mm at position of sheet pile, in stratified soil type (1) is 45 mm at position of sheet pile and in stratified soil type (2) is 6.7mm at position of sheet pile. In general, the shape of ground surface settlement can be viewed in conjunction with deformation of sheet pile to get a better picture about the situation.

Probably the most significant observation is that the maximum settlement takes place some distance behind the wall and not immediately behind the wall. This in essence causes the wall and soil wedge behind the wall to rotate about the top strut.

A.7. Heave analysis

In this part of analysis, the thesis concentrates on the amount of heave for 7m, 8m, 9 m and 12m depth of excavation. The output of this analysis is presented for each cases of excavation from Figure A-20 to Figure A-23 as follows.

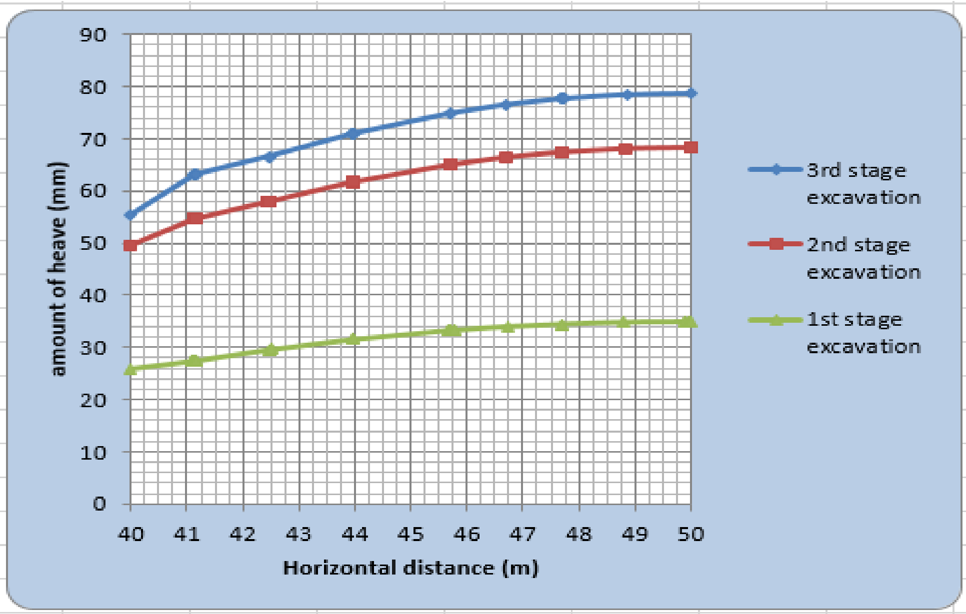

Figure A-20: Heave at bottom of excavation sandy soil

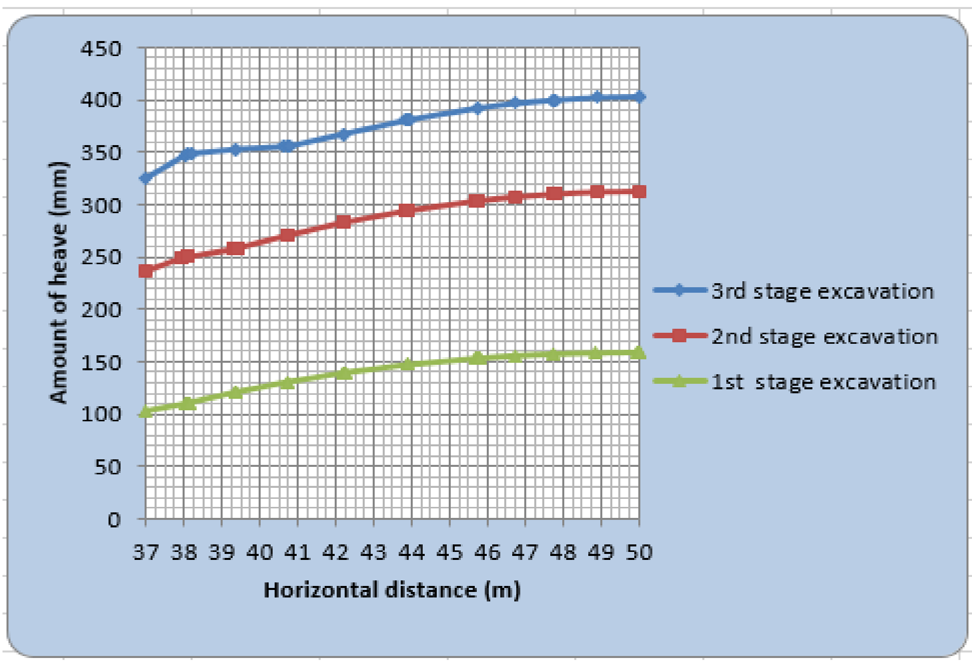

Figure A-21: Heave at bottom of excavation soft clay soil

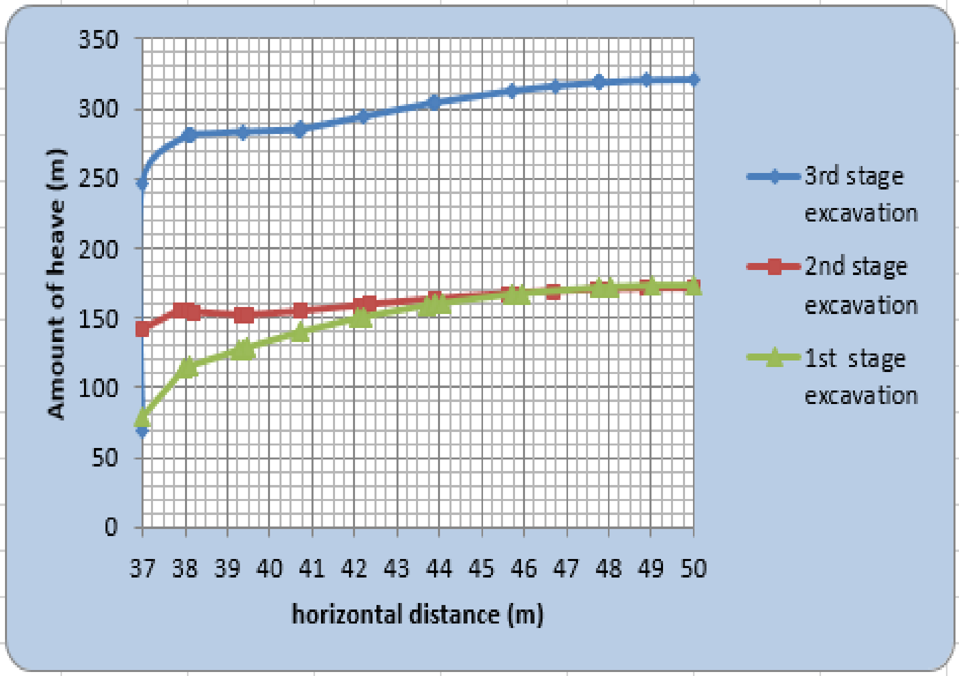

Figure A-22: Heave at bottom of excavation stratified soil type 1

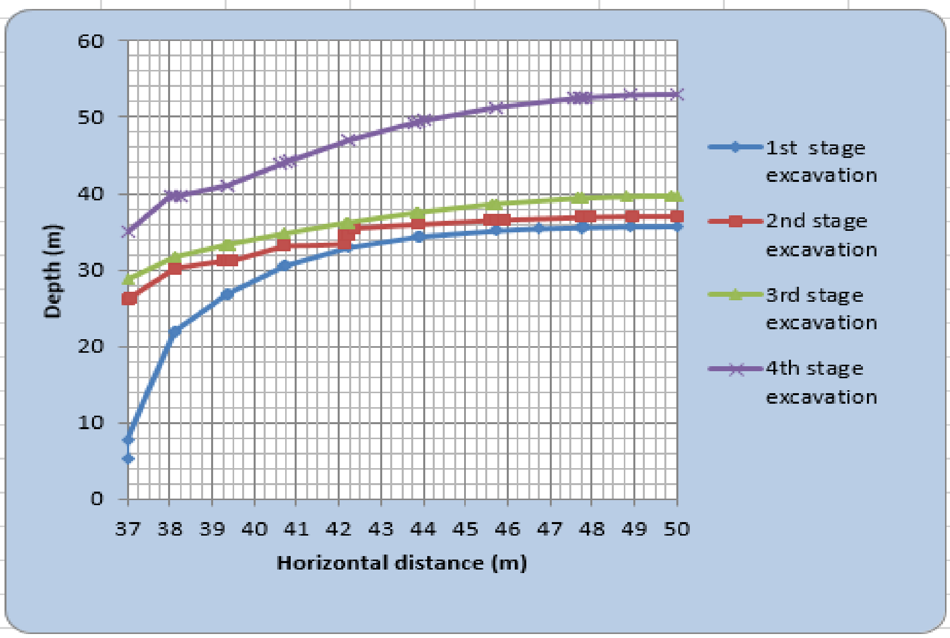

Figure A-23: Heave at bottom of excavation stratified soil type 2

As shown from above graphs, heave increases when stages of excavations proceeds down wards, from this we can understand that the heave occurs at bottom of excavation not only because of swelling nature of soil but also due to relief of soil due to excavation.