

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIROMENTAL ENGINEERING

SIMPLE MODEL FOR WALL DEFLECTION CAUSED BY BRACED EXCAVATION IN CLAYS.

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UNDERTAKING

I certify that research work titled “Simple Model for Wall Deflection Caused by Braced Excavation in Clays” is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged referred.

Firehun Matebo

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ABSTRACT

Due to space limitations in urban areas, underground construction has become a common practice worldwide. When using deep excavations, excessive lateral movements of wall and adjacent structure are a major concern because they can lead to significant displacements in adjacent structures. Therefore, an accurate predictions of lateral wall deflections were a major concern for design and construction of excavation support system. The objective of this thesis is to develop a simple polynomial regression model for predicting maximum wall deflection due to deep excavation, in clays. The excavation width, excavation depth, soil shear strength, wall embedment depth and wall thickness were varied to generate maximum wall deflection and to study the wall deflection behavior. Eight excavation case histories of deep excavation in thick layers of clays was adopted for generation of data used for development of intended model as a basis of numerical analyses.

This research used extensive plane strain (2D) analysis to examine the wall deflection for excavation-induced ground deformation in clay deposits supported by diaphragm wall and bracing, using the Finite Element Method (FEM).

For the development of the Simple Polynomial Regression models for estimating the maximum wall deflection, a database of excavation case histories are compiled and to complement the database of excavation case histories, a large number of data was generated using FEM solutions, by varying the principal parameters considered in the study including excavation depth, excavation width, soil shear strength, embedment depth and wall stiffness with Mohr coulomb soil model. Based on the established database, a Simple Polynomial Regression model is developed for predicting the maximum wall deflection, caused by braced excavations in clays. The proposed method provides a fairly good prediction of wall deflection as compared to FEM.

A Simple Polynomial Regression models has been proposed to estimate the lateral deflection of wall braced excavation in clay with average to good workmanship. Computed values by SPRM (simple polynomial regression model) agreed well with results from observed maximum wall deflection from different excavation case histories which are not used in the development of simple model.

Keywords: Wall deflection, Case histories braced excavations, Polynomial regression, FE analysis.

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LIST OF ABBREVIATIONS AND SYMBOLS

B	Width of the Excavation
E_u	Unloading stiffness
E_r	Reloading stiffness
E_{50}	Secant modulus
D	Domain area
FEA	Finite Element Analysis
FEM	Finite Element Method
H_e	Depth of the excavation
MC	Mohr-Coulomb Model
R_{inter}	Interface reduction factor
ΔH_{max}	Horizontal maximum deflection
I	Moment of Inertia
C	Cohesion
C_u	Undrained cohesion
fc’	Compressive strength of concrete
m	meter
mm	millimeter
kN	kilo Newton
kPa	kilo Pascal
ν	Poisson’s ratio
w	relative weight
γ	Unit weight
Φ	Internal angle of friction
ψ	Dilatancy angle
δ_{hmax}	maximum horizontal deflection
k_o	initial stress
ε	Axial stress
σ_1	deviatoric stress
σ_3	confine stress

CHAPTER 1 INTRODUCTION

1.1 Background

One of the problem in constructing an underground facility within a built-up environment is the impact of the associated ground movements on adjacent buildings. This is primarily because space for construction activities in urban areas is typically constrained by the proximity of adjacent infrastructure. Deep excavations for high-rise buildings, tunnels, and subway stations in poor subsoil can cause large lateral displacements of the ground around deep excavation. Excessive ground deformation induced by deep excavation will inevitably damage the surrounding near-by surface and sub-surface structures, resulting in delays, dispute or even litigation and cost overrun. Especially when the excavation is close to adjacent infrastructure, e.g. buildings, tunnels, buried pipelines, piled foundations, the excavation induced ground movements must be carefully monitored and controlled to be within an acceptable range to avoid any potential damage to these classes of infrastructure. The failure of excavation may have catastrophic consequences, and special care must be taken to avoid such failure. To reduce the excavation-induced deformations, an appropriate retaining wall and support system should be designed, and employing adequate construction method to ensure the serviceability limit state is satisfied. Therefore, accurate predictions of lateral wall deflections where important design criteria in the analysis and design of excavation support systems (Ou, 1954).

Conventionally, excavation support systems are designed based on structural limit equilibrium. Although these approaches will prevent structural failure of the support wall, they may result in excessive wall deformations and ground movements.

The finite element method and the empirical/semi-empirical method are two common approaches for estimating wall deflections induced by excavation. The finite element method is widely employed to model complex soil-structure interaction problems (Zhang et al. 2015).

Empirical and semi-empirical methods involve interpolating from a published empirical database or numerical analyses using finite elements. Several empirical and semi-

empirical methods are available for estimating the excavation-induced maximum wall deflection (Zhang, et al., 2015).

A relevant journals has been reviewed on deep excavation case histories in clay soil and parametric studies are carried out to examine the influence of each input parameters on lateral deformation and generation of maximum wall deflection data to develop intended model using plane strain finite element Plaxis 2D software with Mohr-Coulomb (MC) soil model, i.e. analyses was carried out to evaluate the behavior of excavation in clay based on braced excavation and for generating maximum wall deflection at each fraction depth (Dang , et al., 2014)

Finally, a simple polynomial Regression model was developed to estimate the maximum wall deflection to five input parameters which are excavation depth, excavation width, undrained shear strength of soil, wall thickness and wall embedment depth. Computed deflections have been compared with a number of case records from wall inclinometer data (Zhang, et al., 2015).

1.2 Statement of the Problem

Currently, deep excavations are anticipated when the need to utilize the underground space for different uses in urban areas. Now a day's deep excavation is widely practiced in the world. Lately, there are some failures related to the retaining structures in braced excavation that are used for deep excavation and basement work. The failures of walls can be catastrophic in addition to affecting the serviceability of adjacent structures for excavations in the ground that comprises of thick soft clays overlying stiff clay. For this reason, the exact quantification of maximum wall deflection for braced excavation is needed. To ensure the serviceability limit state is satisfied, a common design criterion is to limit the maximum wall deflection. Unnecessarily severe restrictions may lead to uneconomic design. Therefore, reliable estimates of wall deflections under working conditions are essential. There are two method used to estimate the maximum wall deflection induced by deep excavation. The finite element method and the empirical/semi-empirical method. But some reason these approaches are not easily available for users to estimate maximum wall deflection for design and construction of deep excavation.

1.3 Objectives

1.3.1 General Objective

The objective of this thesis is to develop a simple polynomial regression model to estimate maximum wall deflection caused by braced excavation in clays.

1.3.2 Specific Objective

The specific objectives of this thesis are to examine factors that induce unbalanced force for wall deflection and establish a database for developing the intended model. The database consists of collected case histories and those generated maximum wall deflection from finite element method due to factors that induced unbalanced force to wall deflection and which are used to develop the simple polynomial regression model. The input parameters are: -

- Undrained shear strength, (Su).
- Depth of excavation, (He)
- Width of excavation, (B)
- Wall thickness (t)
- Wall embedment depth, (D)

1.4 Significance of the Study.

It is generally difficult to predict with certainty building damage potential caused by wall deflection of adjacent to deep excavation. As a result of this difficulty, engineers often face the problem of over-design or under-design. These simple polynomial regression model is developed based on multi-order approximation which is easy to use, requires less computational work, and provides a practical tool to estimate the maximum wall deflection caused by the uncertainty in input parameters. When buildings are in close proximity to excavation, there is a potential for damage to the structures. The current design procedures, however, are less than adequate for the evaluation of excavation-induced building damage due to wall deflection and often lead to the over-design or under-design of braced

excavations, which subsequently results in additional costs in the construction of braced excavations.

1.5 Scope of the Study

This thesis is concerned on the development of a simple polynomial regression model to estimate maximum wall deflection caused by braced excavation in clays. And a detail examination of input parameters for the accurate determination of horizontal displacements of the excavation wall when deep excavation on the area of clay found. To take advantage of real field measured data, the actual site found in a different country around the world considered. Since this thesis is literature based, stiffness of the wall and stiffness of the soil was directly taken from journals.

1.6 Methodology of the Work

The general concern of this study focuses on the development of simple polynomial regression model for the analysis of diaphragm walls supporting deep excavations. As this study was literature-based, the deep excavation project used as a benchmark was selected on the bases of soil type and construction methods. The possible factors like; wall thickness, depth of excavation, excavation width, shear strength of soil, and wall embedment depth are believed to influence the maximum wall deflection in braced excavation in clays, described previously input parameter variation in the FEM numerical analysis for generating secondary wall deflection to develop the intended model.

For this purpose, wall deflection data was generated using Mohr- Coulomb constitutive model implemented in finite element software, PLAXIS. The chosen geometries, soil profiles, and supporting systems are representatives of typical excavations in the respected benchmark projects.

Finally, the calculated deformations using proposed model and finite element results are compared with the observed field data to assess the bias of the proposed model.

1.7 Structure of the Thesis

This research work consists of six chapters. In the first Chapter, description of the general overviews of deep excavation, methods of deep excavation and types of lateral support system. This Chapter also discusses the objectives, significance and the scope of the research are presented.

In Chapter 2, gives the review of previous works, deep excavation systems and previous works related to the issue at hand are reviewed.

In Chapter 3, the detailed methodology is stated.

In Chapter 4, analysis and parametric study of excavation case histories and generation of wall deflection data which are used to develop the intended model in chapter five. A sequence of parametric studies was conducted to investigate the effects parameter variation on lateral wall deflection on the two-dimensional ground movements caused by excavation in clay soils.

In Chapter 5**,** based on generated maximum wall deflection results, on pervious chapter a simple polynomial regression model is developed, relating the maximum wall deflection for five input parameters. For the development of intended model excel analysis is used to calculate coefficient's for input parameter's. Validations of the proposed PR model were also carried out through comparisons with well-documented excavation case histories on observed wall deflection by inclinometer from sites not used for development of simple model.

In Chapter 6, the conclusion and recommendations.

In Appendix A, a detailed calculation of the wall input parameters are discussed.

In Appendix B, Excel regression output.

CHAPTER 2 LITERATURE REVIEW

2.1 Deep Excavations

Due to space limitations, underground construction in urban areas has become a common practice worldwide. In recent years, the demand for underground space, for use as underground parking, railway tunnels/stations, road tunnels, redevelopment of buildings, etc., has increased in many heavily urbanized areas. Deep excavations are required to meet the demand, and, in many cases, excavation sites are in close proximity to existing structures/facilities and problems related to deep excavations and their effects on surrounding structures (Zhang, et al., 2017).

Deep excavations are required to meet the demand, and, in many cases, excavation sites are in close proximity to existing infrastructures, buildings, etc. and problems related to deep excavations and their effects on surrounding infrastructures, buildings are developing.

In underground excavation, sufficient space is not available for the stability of the excavated slope because this type of excavation is done in congested urban areas. Thus, this type of excavation is made vertically, below the ground surface, and the excavated soil surface is retained with the help of a wall, which may be in the form of a sheet pile, diaphragm, or soldier piles, with timber lagging so that soil does not fall within the construction zone. In urban areas, many buildings and roads are located near construction areas. As the excavation progresses, the retaining wall is supported by horizontal members known as struts to increase the performance of lateral support to resist deformation due to unbalanced induced force. However, lateral displacement of the wall and vertical displacement of the ground surface occur simultaneously, which may, in turn, directly affect the stability of the surrounding infrastructures. Thus, one of the critical aspects of underground excavation is the deformation of the adjacent infrastructure (Wong, et al., 1989).

Performance of deep excavation is a complex subject in geotechnical engineering and has been studied using various methods, e.g. theoretical and empirical methods, laboratory tests, field measurements, and more sophisticated numerical analysis. However, all these

methods have their limitations, although they have contributed in various degrees to the understanding of the performance of deep excavations. Different methodologies and processes are employed to limit the impact of deep excavation on surrounding structures. The performance of deep excavation is related to both stability and deformation. Deep excavations are designed to be stable and to limit deformations to acceptable levels. However, deformations is more difficult to quantify and analyses are often used for the purpose when ground movements are particularly important.

The excavation of soil for a deep excavation has two main effects. The first is that the removal of overburden pressure which results a decrease in the vertical stress in the soil beneath the excavation. And the second one is a loss of lateral support for the soil around the excavation.

The purpose of a deep excavation lateral support system is to provide lateral support for the soil around an excavation and to limit movement of the surrounding soil. Support systems for deep excavation consist of two main components. The first is a retaining structure. The second component is the supports provided for the retaining structure. Many types of supports have been used in deep excavations. The common types of supports are diaphragm wall (structural slurry), sheet pile, soldier piles, tangent piles, and contiguous piles. The principal types of supports are struts (braces), and tieback anchors (Ou, 1954).

2.2 The Behavior of Deep Excavations

In soil mechanics, there are two common failure type occur due to: -

I. Shear failure of the soil, leading to excessive distortion of a structure or a disruption of highway.
II. Excessive displacement of the soil, inducing unacceptably high stresses in a structure as a result of differential movement.

For retaining structures, failure is a performance problem, related to either strength or deformation. A retaining structure can fail to perform in a satisfactory way for a number of reasons, associated with the failure of the structure itself, failure of the soil or because of unacceptable deformation (Clayey, 1993).

In general, the design of a retaining structure should consider the following points:

Overstress of any part of the structure (bending or shear), and the general stability of the soil around the structure (slope failure, overall stability, basal stability). The stability of the structure should be satisfactory both in short-term and in the long-term condition. Because many retaining structures are associated with decreased level of total stress, it is normal to carry out long-term analysis in terms of effective stresses and effective strength parameters. On the other hand, a retaining structure may perform unsatisfactorily because of the excessive displacement due to unbalanced induced force. It is seldom possible to predict such movements of the retained ground with any degree of confidence analytically. To limit the excessive displacement, it is common to apply a large factor of safety against failure to the critical area. For example, a total factor of safety of 1.5 to 2.0 ((EAB, 1994)) is applied on the passive resistance to reduce tilt and lateral displacement of the wall in soft clays.

Common types of braced excavation failure: -

I. Buckled wall. II. Bottom heave.



Figure 2-1 Braced Excavation Failure type (ou, 1954)

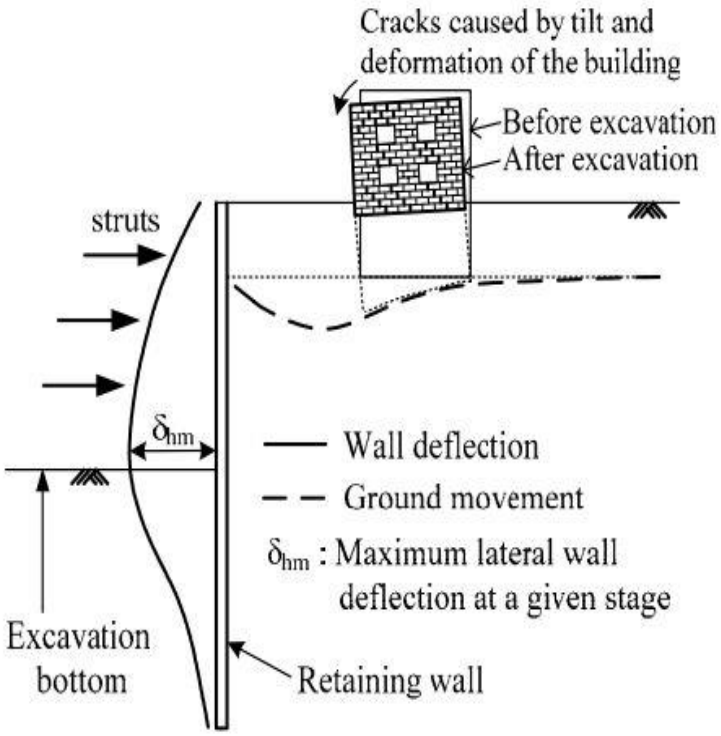

Figure 2-2 Effect Caused by Braced Excavation in Clays, Wall Deflection and Ground Movement, Adjacent Structure Response (Hsiao, 2007)

2.3 General Deflection Behavior of an Excavation Support System

Lateral wall deformations represent the performance of excavation support systems. These are closely related to the stiffness of the supporting system, geometry of excavation, the shear strength of soil and groundwater conditions, the earth and water pressures, and the construction procedures.

Excavation activities generally include three main stages:

(I) Installation of wall

(II) Excavation of soil mass and

(III) Installation of lateral support elements.

2.4 Excavation Method and Lateral Supporting System

Deep excavation or the construction of basement includes excavation ,the construction of retaining walls, installation of struts, the construction of foundation and floor slabs, etc. with the great variety of excavation methods and lateral supporting systems to come to the most appropriate design we have to consider in combination the local geological conditions, the environmental conditions the allowable construction period the budget, and the available construction equipment and make an overall accordingly. The most commonly used excavation methods are full open cut method, the braced excavation method, the island excavation method, the anchored excavation method, the top down excavation method, and the zoned excavation methods. Among those excavation methods braced excavation method is the most commonly used excavation method.

2.4.1 Braced Excavation Methods

Installing horizontal struts in front of retaining walls to resist the earth pressure on the blacks of walls is called the braced excavation method. Figure 2-3, shows the typical arrangement of horizontal struts on braced excavation. The bracing system of the braced excavation method include struts, wales, end braces corner braces and center posts. The function of wales is to transfer the earth pressure on the back of retaining walls on to horizontal stress. End or corner braces can help shorten the span of wales without increasing the number of struts (ou, 1954)**.**

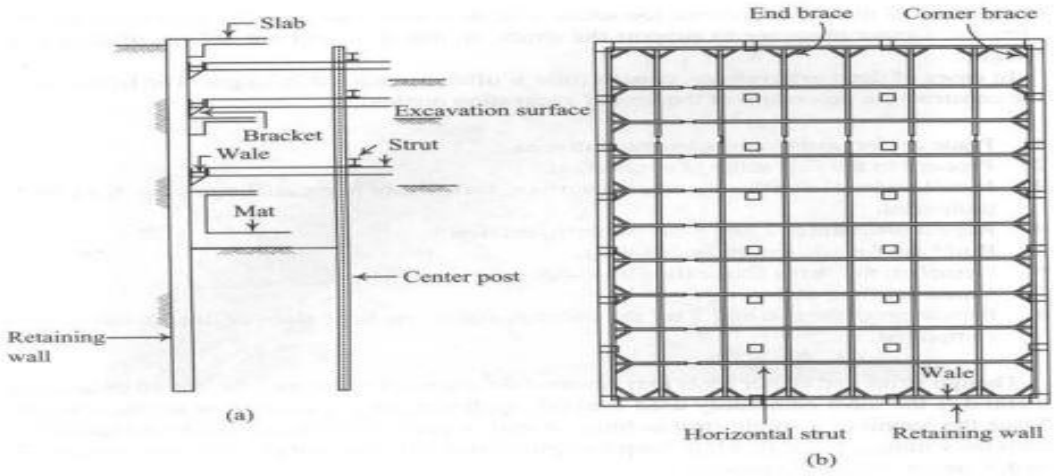

Figure 2-3 Braced Excavation Method (a) Profile and (b) Plan (ou, 1954)

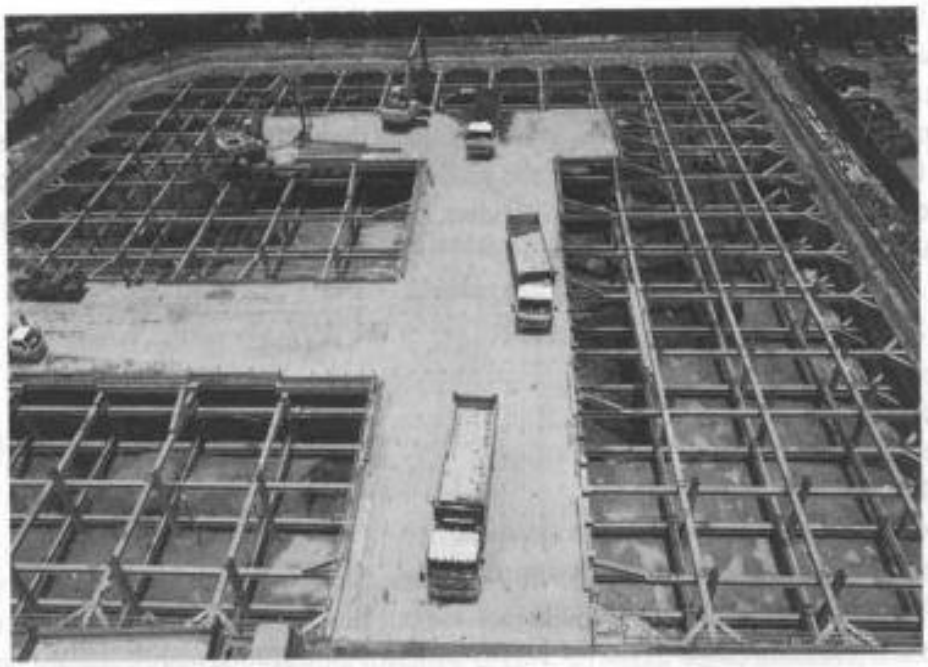

Figure 2-4 Photo of the Braced Excavation Method (ou, 1954)

In cases of deep excavations, construction is often carried out in stages. The following is the construction procedure of braced excavation method:

1. Place center posts in the construction area.

2. Proceed to the first stages of excavation.

3. Install wales above the excavation surface, then install horizontal struts and have them preloaded.

4. Repeat procedure 2 and 3 till the designed depth.

5. Build the foundation of the building.

6. Demolish the struts above the foundation.

7. Construct floor slabs.

8. Repeat procedure 6 and 7 till the construction of floor slabs of the ground floor is completed (ou, 1954).

2.4.2 Lateral Supporting System

2.4.2.1 Diaphragm Walls

Diaphragm walls are also called slurry walls. Since first adopted in Italy in the 1950s they have been widely used around the world. With technological advances more and more news methods and construction equipment have been developed. The basement walls (BW) methods and Impresa construzioni opere specializzate (ICOS) method, designed separately by Japanese company and an Italian company are commonly used in some Asian countries. Many improvement measures have been developed, some of which require jet grouting near the joints to prevent the leaking of diaphragm walls during excavation. To ensure the quality of diaphragm walls, many other construction details need to be taken care of. Concerning the construction details of diaphragm walls, interested readers are advised to read (ou, 1954)**.**

Construction procedure of diaphragm wall panel.

a) Construction of the guided wall.
b) Excavation of the trench.
c) Placement of reinforcements.
d) Concrete casting.

The merits of diaphragm wall method include: -

1. Low vibration, low noise, high rigidity, and relatively small wall deflection.

2. Adjustable thickness and depth of the wall

3. Good sealing capability.

4. May be used as permanent structure.

5. The diaphragm wall and foundation slabs form a unity such that former can serve as pile foundation.

6. Perform in multiple functions simultaneously, earth retention, groundwater flow control, and load bearing.

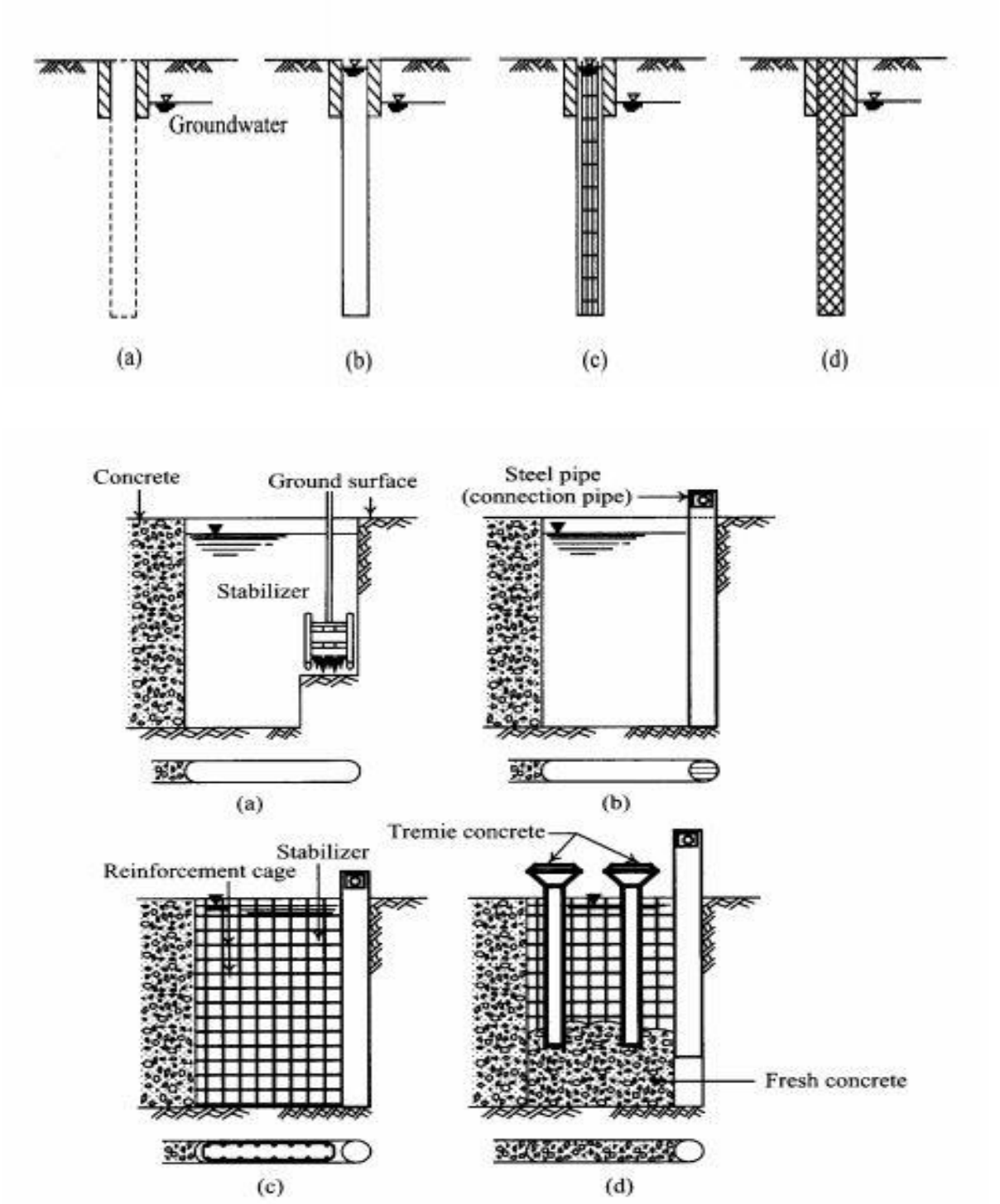

Figure 2-5 Procedure for Diaphragm Wall Installation (Ou 2006).



Figure 2-6 Diaphragm Wall Construction (ou, 1954).

2.5 Factors that Influence Lateral Wall Deflection in Braced Excavation

The magnitude of wall movement is determined by the excavation included unbalanced forces the stiffness of the retaining-strutting system, the excavation stabilities. The unbalanced forces are synthetic results of many factors such as the excavation depth, excavation width, wall penetration depth, shear strength, and wall thickness

2.6 Methods for Predicting Deformation of Excavation

One of the main concerns in a braced excavation in an urban area is the risk of damage to adjacent infrastructures caused by the excavation-induced wall deflections and ground movements. Damage to the adjacent set-ups caused by ground movements is the serviceability failure in a braced excavation. In projects the controlling agencies establish the limiting wall and ground responses as a means of preventing excavation failure and damage to adjacent structures. As described in earlier the deformation behavior of a braced excavation may be affected by many factors. Thus, it is necessary to have the method that

able to precisely predict the maximum wall deflection and ground settlement during the design of braced excavations.

2.6.1 Empirical Methods

Empirical methods are routinely used by engineers to estimate excavation induced ground movements. Such methods have been successful in the past ensuring base stability and accurate prediction of retaining wall deflection and the resulting ground surface settlement are important design criteria in the analysis and design of excavation support systems.

2.6.1.1 Peck's method (1969)

Peck's developed the first empirical method to predict the wall movement based on actual ground deformation data collected from temporary braced sheet pile and soldier pile walls with tie back support as shown in Figure below. The case histories used in the development of the figure are prior and the excavations are supported by sheet pile or soldier piles with lagging. Also, Peck (1969) summarized the experiences with performance of deep excavation support system, and the factors that are most important in controlling the performance. These factors include the type and strength of the soil around and beneath the excavation, the excavation and supporting procedure, and workmanship. Beside the above factors, the ground water condition, the flexibility or rigidity of the components used for construction, the time taken for construction are also important factors that influence the performance of excavations.

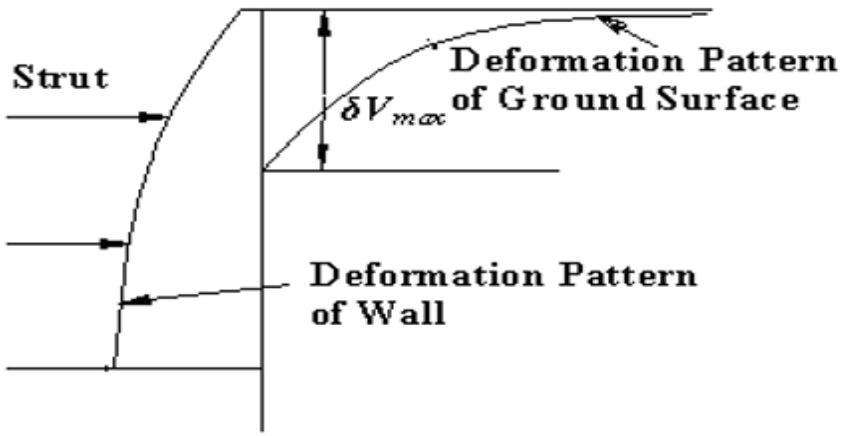

Figure 2-7 The movement Pattern of Wall and Ground Surface for Braced Excavation (Peck, 1969)

2.6.1.2 Lambe (1970)

Lambe's 1970 paper on braced excavation focused on design and analysis of deep excavation and their support systems. Lambe reviewed factors influencing the movement of soils due to excavation and the engineering of deep excavations. He included three case histories of excavation for the MBTA subway in Boston, and applied the state of art in design and analysis to each of the three cases and then compared predictions to measured performance. Lambe concluded that the state of art for the design and analysis of braced excavation was far from satisfactory, since support system loads and ground movement could not be predicted with confidence.

2.6.1.3 Goldberg et al. (1976)

In 1976 Goldberg et al. wrote a three-volume report for the FHWA on design recommendation, design consideration and construction techniques for lateral support systems. The report is a comprehensive source of information on the state of practice in 1976. The authors used information from the measurements and performance of 63 case histories as a basis for estimating maximum horizontal wall movement.

2.6.1.4 Mana and Clough 1981

In the studies of Mana and Clough (1981), 11 case histories were examined. The maximum observed movements for case histories are normalized by the excavation depth and correlated with the factor of safety against basal heave set out by Terzaghi (1943) as shown in Figure 2-9.

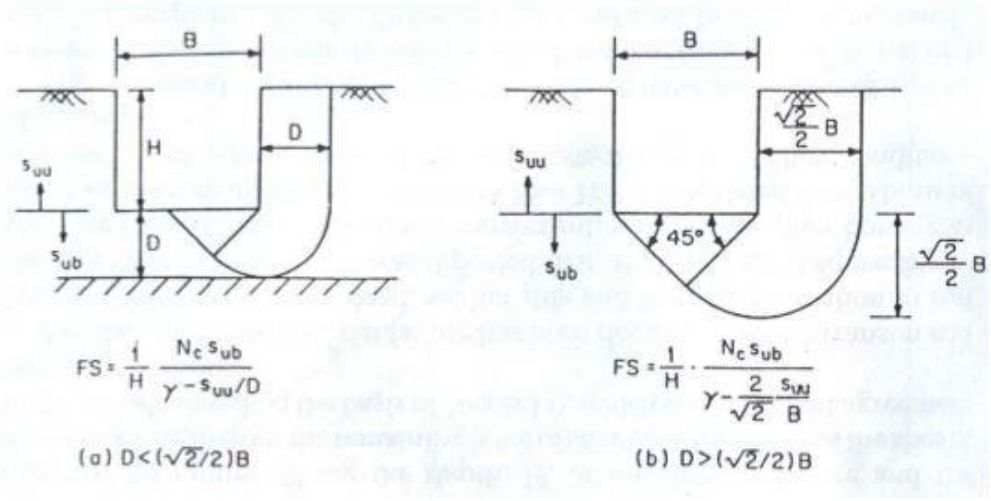

Figure 2-8 Excavation Geometry and Soil Strength Parameter for Factors of Safety

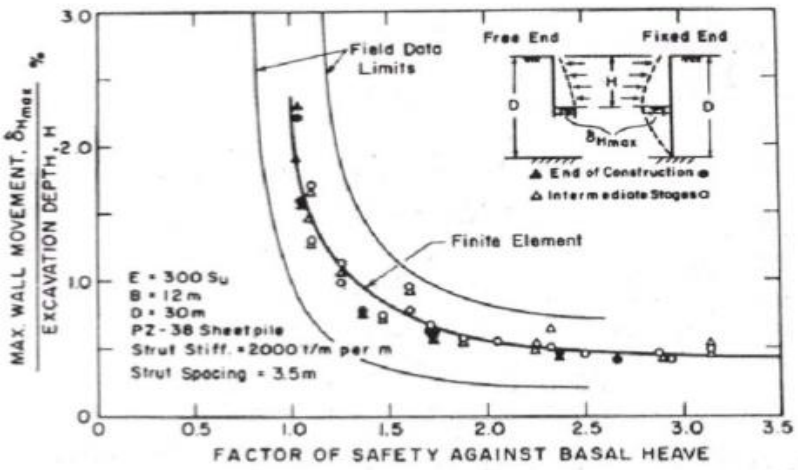

Figure 2-9 Analytically Defined Relationship b/n Factor of Safety Against Basal Heave and Non-dimensional Maximum Lateral Wall Movement (after mana and Clough 1981).

2.6.1.5 Hashash (1992)

Hashash (1992) developed a new capability to model deep excavations supported by diaphragm walls in clay deposits using accurate numerical techniques and an advanced soil model. Hashash (1992) was motivated by the practical problem of predicting the performance of deep excavations (excavation depth up to 30m) in clays associated with the construction of the Central Artery/Tunnel project in Boston and the Taipei Rapid Transit system in Taiwan. In these situations, accurate predictions of lateral wall deflections and surface settlements are important design criteria to avoid damages to adjacent structures/facilities.

Hashash (1992) carried out analyses for L = 12.5m, 20.0m, 40.0m and 60.0m long walls. The results show two distinct failure mechanisms. For short walls (L = 12.5m and 20.0m) there is a basal failure of the soil beneath with the embedded length of the walls deflects in a cantilever mode with the largest deflections occurring at the toe ("kick-out" mode). For long walls (L = 40.0m and 60.0m), the failure mechanism is constrained by bending of the embedded wall length ("bulging mode") and the failure may initiate with the formation of a plastic hinge in the wall.

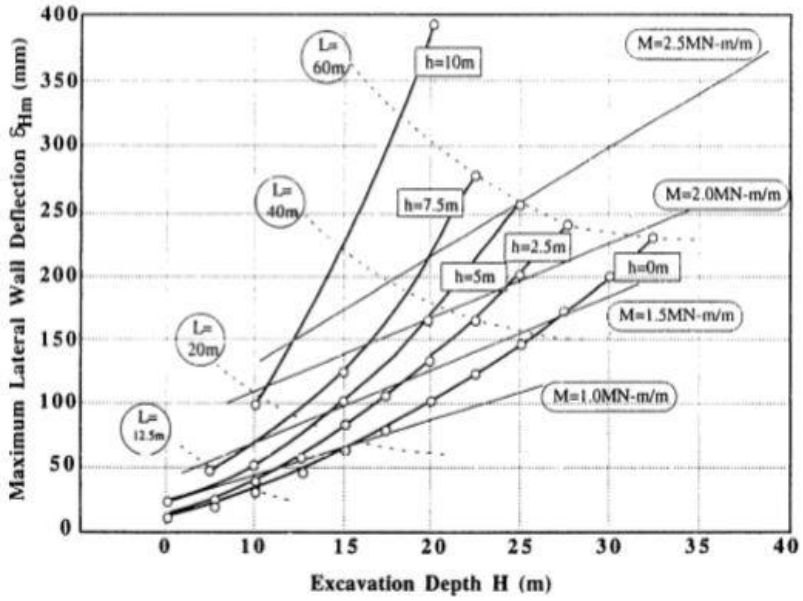

Figure 2-10 Numerical Predictions of Maximum Lateral Wall Deflection N.C Boston Blue Clay (Hashash 1992)

2.6.1.6 Clough et al.'s Method

Clough et al. (1989) proposed a semi-empirical procedure for estimating movement at excavations in clay in which the maximum lateral wall movement δhm is evaluated relative to factor of safety (FS) and system stiffness, which is defined as follows: System stiffness (η) = EI/ γ w h4 where EI is the flexure rigidity per unit width of the retaining wall, γw the unit weight of water and h the average support spacing.

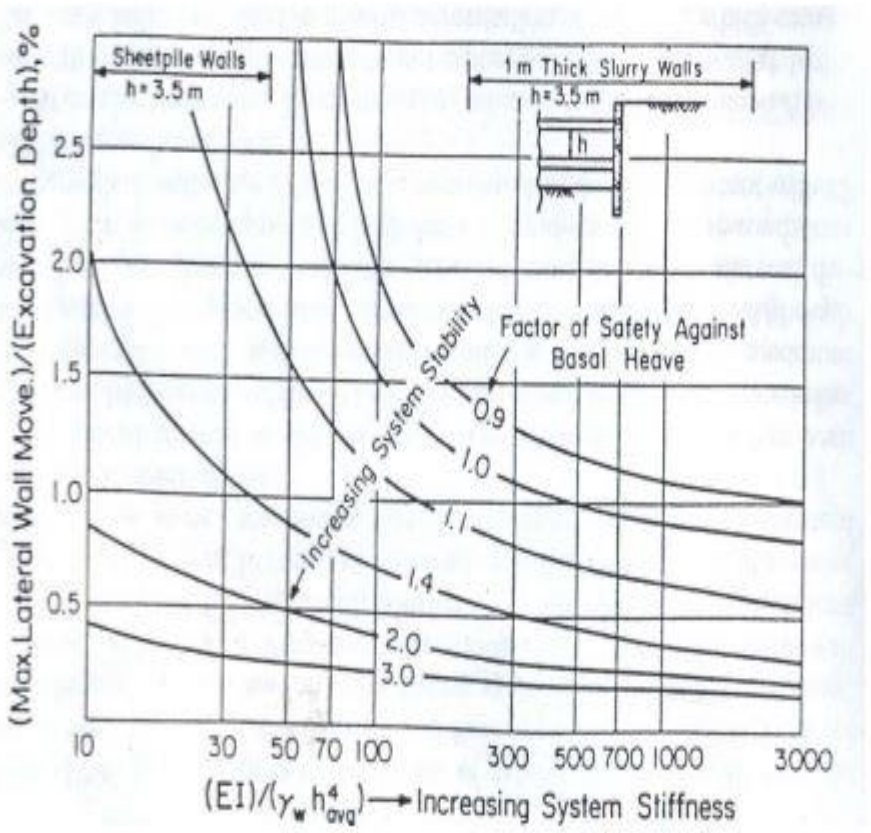

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Figure 2-11 Lateral Wall Movements as a Percentage of Excavation Depth Versus System Stiffness (Clough, et al. 1989).

2.6.1.7 Kung, Juang et al. (2007)

Juang et al. (2007) developed a semi-empirical model to determine the maximum wall deflection caused by a braced excavation in soft to medium clays, based on database of 33 case histories and results from a large number of finite element analyses. The developed model mainly consists of three components to estimate the maximum wall deflection and deformation ratio between maximum lateral wall deflections. Regression-based equations were used to analyses the relationship of the input variables which may affect the maximum wall deflection and the deformation ratio. Model bias was assessed, and precision of this model was regarded to be high enough for practical application. The proposed model was verified using case histories not used in the development of the model.

2.6.1.8 Clough, Smith et al. (1989)

Clough, et al. (1989) proposed a semi-empirical procedure to estimate wall movement induced by excavation in clay. The maximum lateral wall deflection is evaluated relative to factor of safety against basal heave by Terzaghi (1943) and system stiffness. The derived curves are based on average condition, good workmanship, and the assumption that cantilever deformation of the wall contributes only a small fraction of the total movement.

2.6.2 The Finite Element Approach (FEA)

The finite element method is one of the powerful numerical methods available to date to predict the ground movement and movement pattern in and around an excavation. In the last 3 to 4 decades several authors used the finite element method to predict or to back analyze the performance of an excavation. Since then there is a lot of progress in understanding the material behavior of soils, and the application of the finite element method to geotechnical problems. The finite element method (FEM) is often employed to model complex soil–structure interaction problems such as braced excavations. Many investigators (Finno & Harahap, 1991; Hashash & Whittle, 1996; Ng & Yan, 2000; Kung et al., 2007a; Kung et al., 2009) have verified the reliability of FEM applied to analyses of deep excavations even though the uncertainty of soil models and parameters adopted is concerned. Although the deflection of the braced wall can generally be predicted well using a routine FEM analysis. In engineering practice, two-dimensional (2D) finite element (FE) analysis is generally performed because of time and budget limitations the finite element code PLAXIS professional version was selected for the purpose of

analyzing the practical projects and the parametric studies. The PLAXIS program contains from simple linear elasticity to advanced elasto-plastic cap soil models. The details of each soil model can be found in PLAXIS user's manual (Brinkgreve (1995), Brinkgreve (1998), Brinkgreve (2002)).

The finite element method, FEM, consists of modeling the wall and the soil as made of small elements and assigning to the elements properties which control their behavior. Beam elements are usually chosen to represent the wall while brick elements are used for the soil. A typical input for the FEM is the mesh description including the geometry of the elements for the wall, the struts, the soil, the models for the wall material and water, boundary conditions, and the boundary loads. Typical programs include in PLAXIS.

2.6.2.1 Factors that affect the performance of finite element method (FEM)

Finite element analysis of deep excavations should exercise accurate modelling procedures for geotechnical and structural behavior, like; the geometry of the excavation, detailed retaining structures, correct construction sequence, and reliable material models and input parameters. The main issues involved with the numerical analysis of deep excavations include selecting appropriate constitutive models for soils and structures, the simulation of the construction procedure, and the modelling of the soil/structure interface (Zdravkovic, 2001).

2.6.2.2 Contact behavior between wall material and soil

The contact behavior between the retaining wall and the soil plays an important role in numerical simulations of retaining structures. Most standards recommend a wall friction value of zero for soft soils, which results in unrealistic deformations of the wall and the soil in numerical analysis. Terzaghi was the first to study the influence of wall friction on earth pressures in early 1930's. After him a number of studies had been conducted to study the frictional characteristics of interfaces between soils and structural elements (see for Clough (1971). The influence of interface element on retaining structures for numerical modelling has been studied by past researchers. (Day, 1994)Investigated the effect of interface properties on the behavior of a vertical retaining wall and the deformation of the ground through a series of finite element analyses. Beside the wall friction, the stiffness of the interface is equally important to simulate the contact behavior. Some FE-programs, for example PLAXIS, requires an input parameter $\mathbf{R}_{(inter)}$ (an interface factor, which is taken

as the ratio of the wall friction to friction of the soil) for the interface elements. The R ($_{inter)}$ between the soil and the continuous wall and slab was generally taken as 0.67 considering the relative movement. (R.B.J Bringreve, 2004). They used an elastic-perfectly plastic model without dilatancy effect in their finite element study of the soil reinforcement interaction. The interface elements take the material property from nearby soil cluster after reducing the values of all parameters by a factor $\mathbf{R}_{(inter),}$ even the shear modulus which is a key parameter as regard to the deformation of the soil should be reduced by factor $\mathbf{R}_{(inter)}$ (Freiseder, 1998)

2.6.2.3 Wall installation

Traditionally, wall installation has been ignored in design because of a lack of understanding of its effects, and also the tendency of designers to prefer conservative predictions. More of the times, the effects of installing a wall are ignore the finite element simulation and the wall is "wished into-place" with no change in the stress conditions in the ground or without any disturbance on surrounding soil (Ou, 1954).

O'Rourke and Clough (1990) summarized observed settlements that arose during installation of five diaphragm walls. They noted settlements as large as 0.12% of the depth of the trench. These effects can be evaluated by 2-dimensional modeling of the construction process of Taipei National Enterprise Centre excavation project in Taipei basin (Liao, 1998), in which both the wall installation and subsequent excavation process were included. They found an increase of up to 20% ground settlements and 15% wall movements generated from the WIM model than the WIP model. And others also mentioned that, it is less straightforward when modeling diaphragm wall installation effects in a plane strain analysis because the arching caused by the excavation of individual panels cannot be taken directly into account (Dang, 2014).

2.6.2.4 Modeling of wall element

When modelling embedded walls another issue is the choice of finite element type to represent the wall itself. The retaining wall is generally modelled using either shell elements or solid elements in finite element analysis, if the wall installation process is modelled using the Wished-In-Place (WIP) method. Shell elements have no geometric thickness and are easier to generate in the mesh. In addition, the internal forces and bending moments can be obtained directly and they are approximate in the sense that they reduce

one dimension of the structural member to zero. But for thicker walls this is not the case and there are differences based on considering the vertical shear stresses mobilized on the back of the wall between the analyses modelling the wall with solid and beam elements. As a result, it is found that the computed wall deflection and ground movement from shell element wall are larger than those from solid element wall (Zdravkovid, 2001).

2.6.2.5 Influence of domain analyzed.

When performing a numerical analysis, the choice of the depth and lateral extent of the finite element mesh is an important issue. The model dimension will be selected such that the boundaries are not affecting the model results. Because of soil strength and stiffness usually increase with depth, analyses are not so sensitive to the location of the bottom boundary as long as it is not unreasonably close to the bottom of the wall. The location of the vertical boundary is more problematic and will, in part, depend on the constitutive model employed to represent soil behavior. (David M.Potts, 2001) Investigates and found that for simple models, excessive settlement exhibited for shallower depth model and the influence of lateral extent reaches up to 8.5 times width of the excavation. For advanced model, increase in depth has no effect on the result of surface settlements and 4.5 times for lateral extents.

2.7 Soil Model

The essential features of soil behavior include: soil acts as a multi-phase material, soil response is non-linear and path-dependent, soil deformation include irrecoverable (plastic) strains, soils may dilate or compact, soil response is influenced by its load history, natural soils are anisotropic, and soils exhibit time-dependent behavior. Ideally a perfect soil model would be able to predict these soil behaviors under all type of loading condition. An increasing number of stress-strain relations have been formulated to model the behavior of soils. These models can be grouped into linear elastic, non-linear elastic (hyper elasticity, hypo elasticity), variable moduli, elasto-plastic, elastoviscous-plastic, cap models and hypo plasticity. Several references can be cited in connection with the formulation of constitutive soil models: (Brinkgreve, et al., 2002).

2.7.1 Mohr- Coulomb Model

The Mohr-Coulomb model coded in Plaxis version 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.

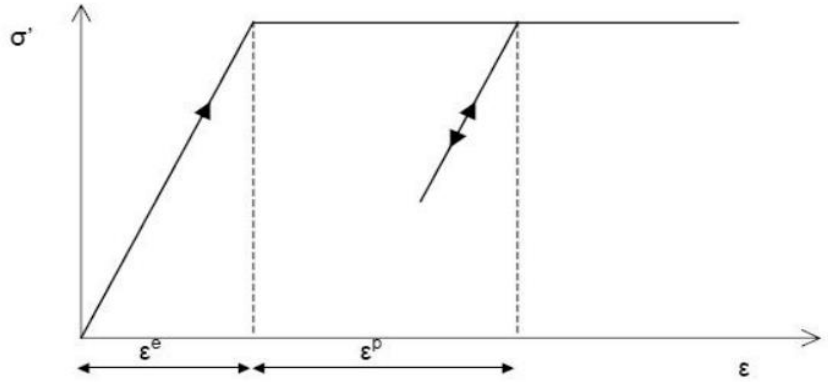

Figure 2-12 Basic Idea of an Elastic Perfectly Plastic Model.

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 inFigure2-13.

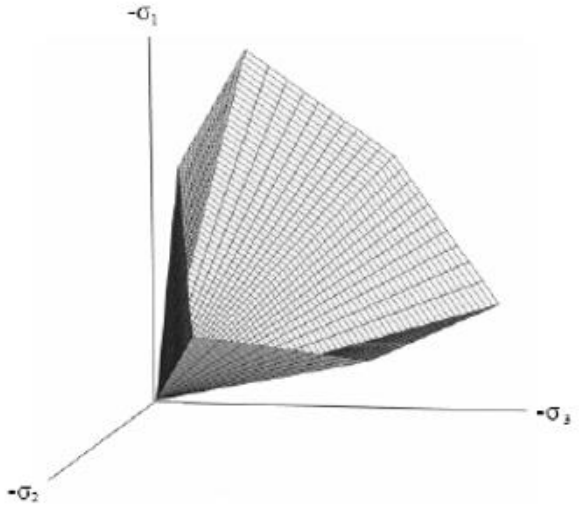

Figure 2-13 Mohr-Coulombs Yield Surface in Principal Stress Space (Plaxis v8.1, 2002).

$$f_{1a}=\frac{1}{2}(\sigma_2-\sigma_3)+\frac{1}{2}(\sigma_2-\sigma_3)\sin\varphi-C\cos\varphi \leq 0$$

$$f_{1b}=\frac{1}{2}(\sigma_3-\sigma_2)+\frac{1}{2}(\sigma_3+\sigma_2)\sin\varphi-C\cos\varphi \leq 0$$

$$f_{2a}=\frac{1}{2}(\sigma_3-\sigma_1)+\frac{1}{2}(\sigma_3+\sigma_1)\sin\varphi-C\cos\varphi \leq 0$$

$$f_{2b}=\frac{1}{2}(\sigma_1-\sigma_3)+\frac{1}{2}(\sigma_1+\sigma_3)\sin\varphi-C\cos\varphi \leq 0$$

$$f_{3a}=\frac{1}{2}(\sigma_1-\sigma_2)+\frac{1}{2}(\sigma_1+\sigma_2)\sin\varphi-C\cos\varphi \leq 0$$

$$f_{3b}=\frac{1}{2}(\sigma_2-\sigma_1)+\frac{1}{2}(\sigma_2+\sigma_1)\sin\varphi-C\cos\varphi \leq 0 \qquad (2\text{-}1)$$

2.7.2 Formulation of Mohr-Coulomb Model.

The Mohr-Coulomb yield condition is an extension of coulomb's friction law to generate states of stress. Actually, this condition ensures that coulomb's friction law is obeyed in any plane within a material element. The full Mohr –Coulomb yield condition consists of six yield functions when formulation in terms of principal stresses (see for instance smith & Griffith, 1982): The two plastic model parameters appearing in the yield function are the well-known friction angle ϕ and the cohesion C. In addition to Hooke's law and coulombs failure criteria, a dilatancy angle Ψ is used to model a realistic irreversible change in volume due to shearing (non-associated flow rule).

Table 2-1 Mohr Coulomb parameters with their standard units

Parameter		**Unit**
E	Young's Modulus	[kN/m^2]
ν	Poisson's ratio	[-]
ϕ	Friction angle	[°]
C	Cohesion	[kN/m^2]
Ψ	Dilatancy angle	[°]

2.8 Interface Elements

Retaining structures that used in excavation are stiff while the adjacent material which is soil is relatively soft. When the retaining structure deforms, relative displacement may be generated between the soil and the structure. Though interface elements can rationally simulate the relative displacement between soil and structures, since extra parameters, which are not easily obtained from conventional soil tests, will be introduced, and numerical instability during analysis often occurs, they have to be used carefully. The roughness of the interaction is modeled by choosing a suitable value for the strength (wall friction and adhesion) to the soil strength (friction and cohesion). (Clough, 1969). Studied the interaction between the wall and the back-fill material with the help of shear box test and showed that the stress-displacement behavior of the interface is similar to the stress-strain behavior of soils. In order to implement the interface behavior in the finite element analysis of retaining structure, (Clough D. , 1971)developed a non-linear, stress dependent stiffness, hyperbolic stress-strain constitutive relation to represent interface behavior similar to those developed by (Duncan, 1989)to model the stress-strain behavior of soils. Like the behavior of soils, interface behavior may also be represented by complex advanced models. However, Gens et al. (1989) underlined the use of less complex models. They used an elastic-perfectly plastic model without dilatancy effect in their finite element study of the soil reinforcement interaction. In PLAXIS the Mohr-Coulomb Model is used to represent the interface behavior, whatever model is applied to represent the soil behavior.

CHAPTER 3 METHODOLOGY

3.1 General Overview

This thesis work is conducted in order to develop a suitable simple model to estimate maximum wall deflection caused by braced excavation in clay soils. This study uses the data obtained from deep excavation works around the world.

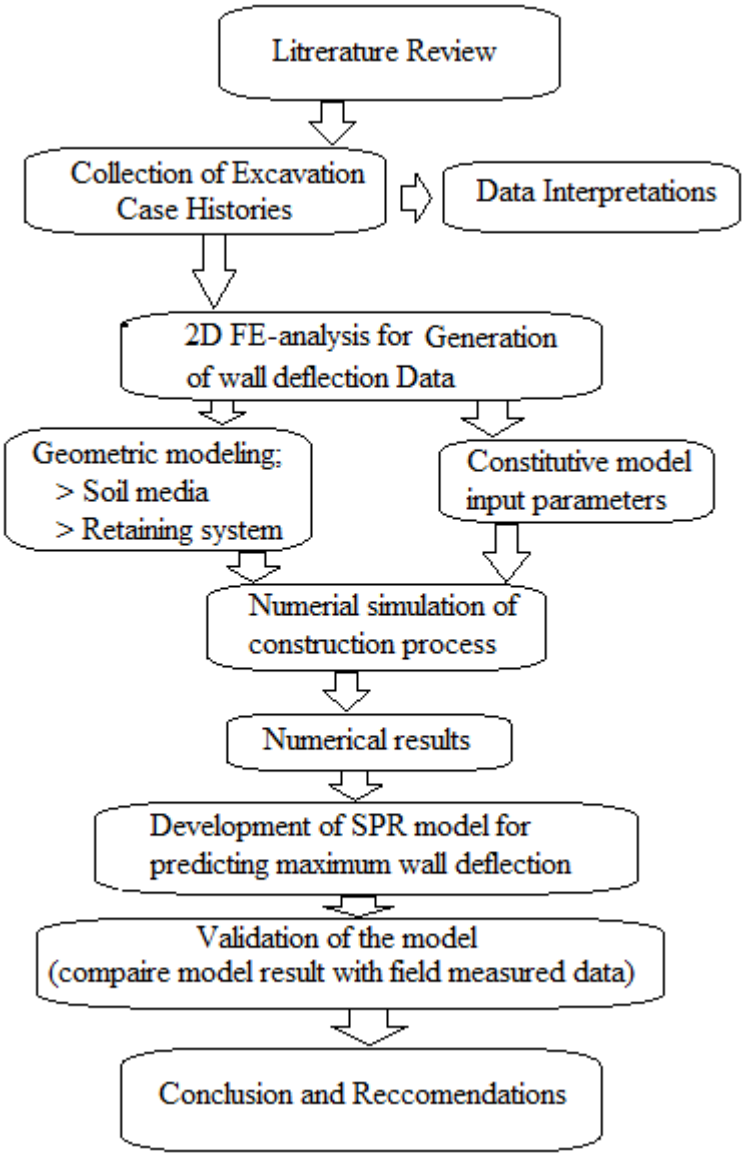

Figure 3-1. Methodology Flow Chart

3.2 Data Collection

Eight deep-excavation case histories are collected from different regions around the world uses in this research study. Prior to the compilation of deep-excavation case histories and generation of maximum wall deflection data from numerical experimentation, it is necessary to recognize the influential factors in terms of excavation-induced wall deflection. The case histories which are selected for analysis are: Central Jakarta in Indonesia, Post office Square Garage at Boston, and Far East enterprise in Taipei, Formosa, Tokyo, Farrer Park, AT & T corporate center, metro station and TNEC site, the complete information and numerical data are shown in Table 4-1. Excavation in such thick clay soils in urban areas is usually prone to movements of soils which could damage the nearby structures. The influential factors found are then included in the finite element method numerical experimentation for generating wall deflection data used for development of the intended models that is presented in Chapter five.

3.3 Components of Deep Excavation

3.3.1 Soil

. The performance of deep excavations is greatly influenced by the soil behavior. In finite element analysis, it is vitally important to use realistic soil models to obtain more reliable results. The selection of constitutive models largely depends on the software that is used. PLAXIS includes several basic models for soils (e.g. Mohr-Coulomb).

3.3.2 Diaphragm Wall

Diaphragm walls are also called slurry walls. Since first adopted in Italy in the 1950s they have been widely used around the world. With technological advances more and more news methods and construction equipment have been developed. The diaphragm wall modeled with plate element in Plaxis which consider axial stiffness (EA), flexural stiffness (EI), cross-sectional area of the wall (A), moment of inertia of the wall (I) and also consider thickness and weight of the wall. From these two parameters an equivalent plate thickness d_{eq} is calculated from the equation

$$d_{eq} = \sqrt{12\frac{EI}{EA}} \qquad (3\text{-}1)$$

3.4 Finite Element Method

The finite element method is one of the powerful numerical methods available to date to predict the ground movement pattern in and around an excavation. In the last 3 to 4 decades several authors used the finite element method to predict or to back analyze the performance of an excavation. Advanced finite element analysis of deep excavation should adopt accurate modeling procedure for geotechnical and structural problems e.g.: -

(I) The geometry of excavation
(II) Retaining structures
(III) Adequate construction sequence
(IV) Appropriate material model and input parameters

The interaction between the soil and the temporary retaining system can be analyzed by using Finite Element Method (FEM) to study the effects of the excavation to retaining system as well as to the surrounding ground and adjacent infrastructures. One of the powerful computer programs that use FEM is Plaxis 2D to analyses of soil structure interaction problems. Two-dimensional plane strain analysis is used to model the excavation. Finite element mesh is very fine to improve the accuracy of calculation near diaphragm wall because it is highly stress zone. The boundaries considering the symmetry of excavation and taking half for analysis Vertical model boundaries with their normal in the x-direction (i.e. parallel to the y-plane) are fixed in x-direction (Ux = 0) and free in y –direction.

The retaining wall was modelled as plate element with linear elastic properties and Mohr coulomb model is adopted to model the soil behavior.

A. Element

For the 2D simulations, the user may select either 6-node or 15-node triangular elements to model soil layers and other volume cluster.

a) 6-Node triangular elements

For the 6-node triangle the order of interpolation is two and the numerical integration involves three Gauss points. The type of element for structural elements and interfaces is automatically taken to be compatible with the soil element.

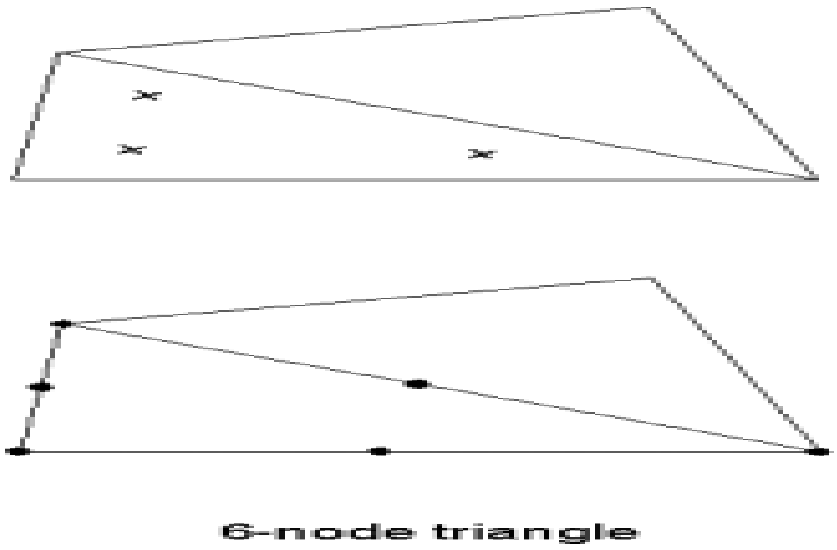

Figure 3-2 Position of Nodes and Stress Points in Soil Element

b) 15-Node triangular elements

The 15-node triangle is the default element. It provides a fourth order interpolation for displacement and the numerical integration involves twelve Gauss points (stress points).Fourth-order 15-node triangular elements, which are considered to be very accurate elements, were used to model. The use of 15-node triangles leads to relatively high memory consumption and quite slow calculation and operation performance. However, with the latest and powerful computer that available nowadays has made the calculation faster and the result can be obtained instantly.

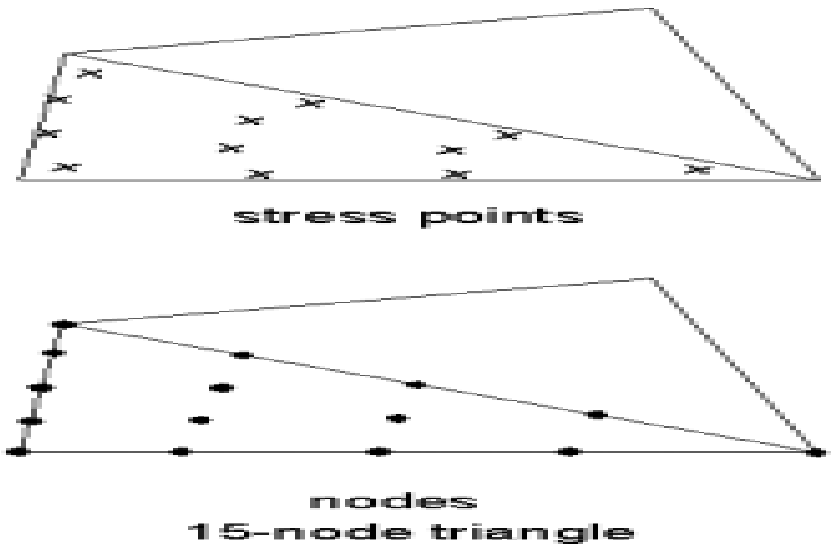

Figure 3-3 Position of Nodes and Stress Points in Soil Element

B. Model

Plane strain model is used for geometries with a (more or less) uniform cross section and corresponding stress state and loading scheme over a certain length perpendicular to the cross section (z-direction).Displacement and strain in z-direction are assumed to be zero. However, normal stresses in z-direction are fully taken into account. Plaxis 2D is finite element software that is only able to make two dimensional analyses and in plaxis 2D there are two finite element component which are Plane Strain.

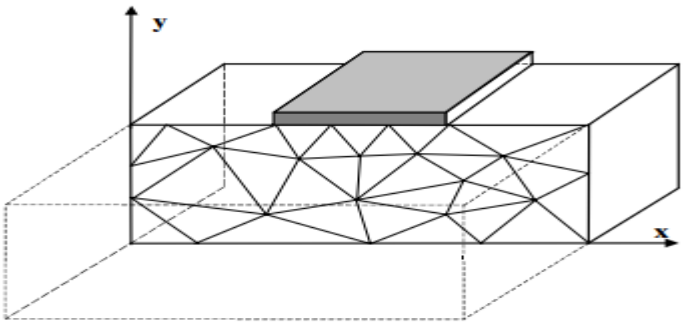

Figure 3-4 Plane Strain (Plaxis v8.1 2002).

C. Boundary Conditions

In Plaxis 2D prescribed displacement are special condition that can be imposed on geometry lines to control the displacement of these lines. A convenient option exists to use standard boundary conditions that apply in most cases.

D. Plates

Bending of retaining walls, tunnel linings, shells, and other slender structures can be modelled by using plate element. The behavior of these elements is defined using a flexural rigidity, normal stiffness and an ultimate bending moment. Plates with interfaces may be used to perform realistic analyses of geotechnical structures.

E. Geometry Model generation

The input of soil layers, structures, construction stages and boundary conditions are based on existing site data and drawing which allows to draw a detailed model of the geometry cross-section. From this geometry model, two-dimensional finite element meshes can be easily generated.

F. Plaxis 2D Soil Model

There are various types of advanced soil model provided in Plaxis2D. The conventional soil model such as Mohr Coulomb model, also other soil models such as Linear Elastic, Soft Soil, Soft Soil Creep, Modified Cam Clay, Critical State, Soil Hardening and Jointed Rock models. Soil structure interaction is modelled with interfaces. In deep excavation work, the designer must be able to choose the correct soil model so that overall analysis will be safe. For excavations in clays, the Mohr-Coulomb (MC) constitutive relationship is commonly used to model the clay stress-strain behavior, with no consideration of the soil small strain effect.

G. Plaxis 2D Calculation

Plaxis 2D considers both stress and deformation while distinguishing between plastic, consolidation, safety factor and dynamic calculations. A stage-construction feature provides a realistic simulation of construction and excavation process by activating and deactivating clusters of elements, for example changing water level.

H. Staged Construction

On this Plaxis 2D analyses feature enables a realistic simulation of construction and excavation process by activating and deactivating cluster elements, changing of water table, application of loads, etc. This procedure allows for a realistic assessment of stress and displacement as caused, for example by soil excavation during an underground construction.

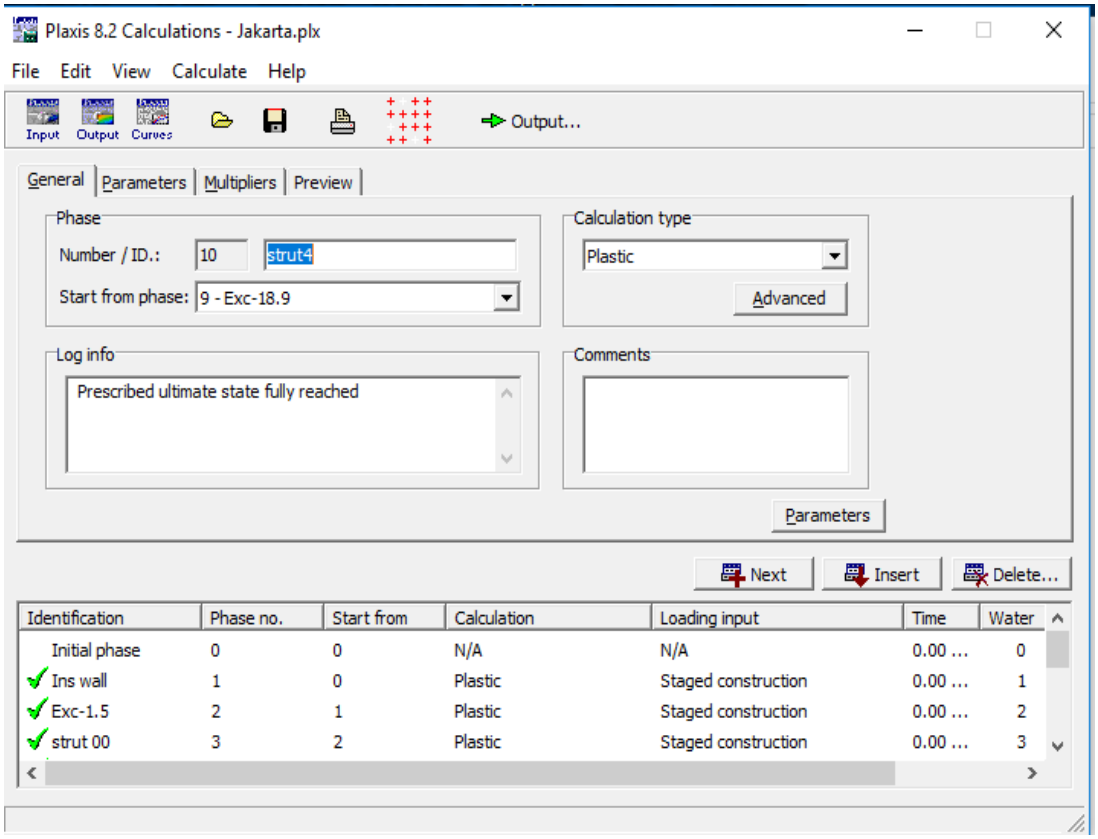

Figure 3-5 Calculation Window

I. Deformation

In Plaxis2D the result of deformed mesh, total or incremental displacements and total or incremental strains can be illustrated in graphical output. All displacement and strains graphs can be visualized as displacement arrows, contour lines or contour shadings as shown in Figure 3-6.

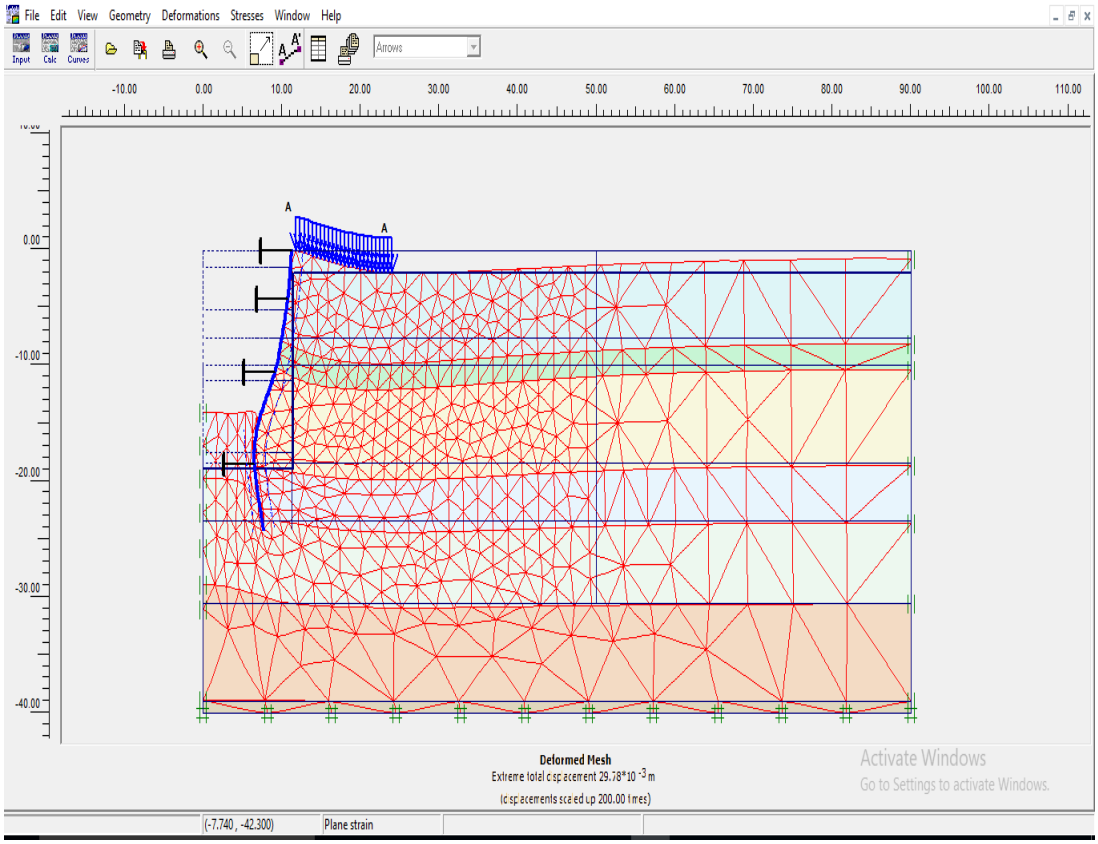

Figure 3-6 Deformation Mesh

H. Stresses

In Plaxis 2D the user can find the value of effective stresses, total stresses and excess pore water pressure. These stresses analyses can be visualized in contour shadings.

3.5 Least Square Regression

Microsoft Excel is a powerful mathematical tool for least square regression analysis, calculation, graphs, etc. After the parametric studies were conducted and generating of artificial data by PLAXIS 2D then by Add-ins –Data Analysis (Regression) Microsoft excel program were conducted to manipulate coefficients for each parameter to develop model.

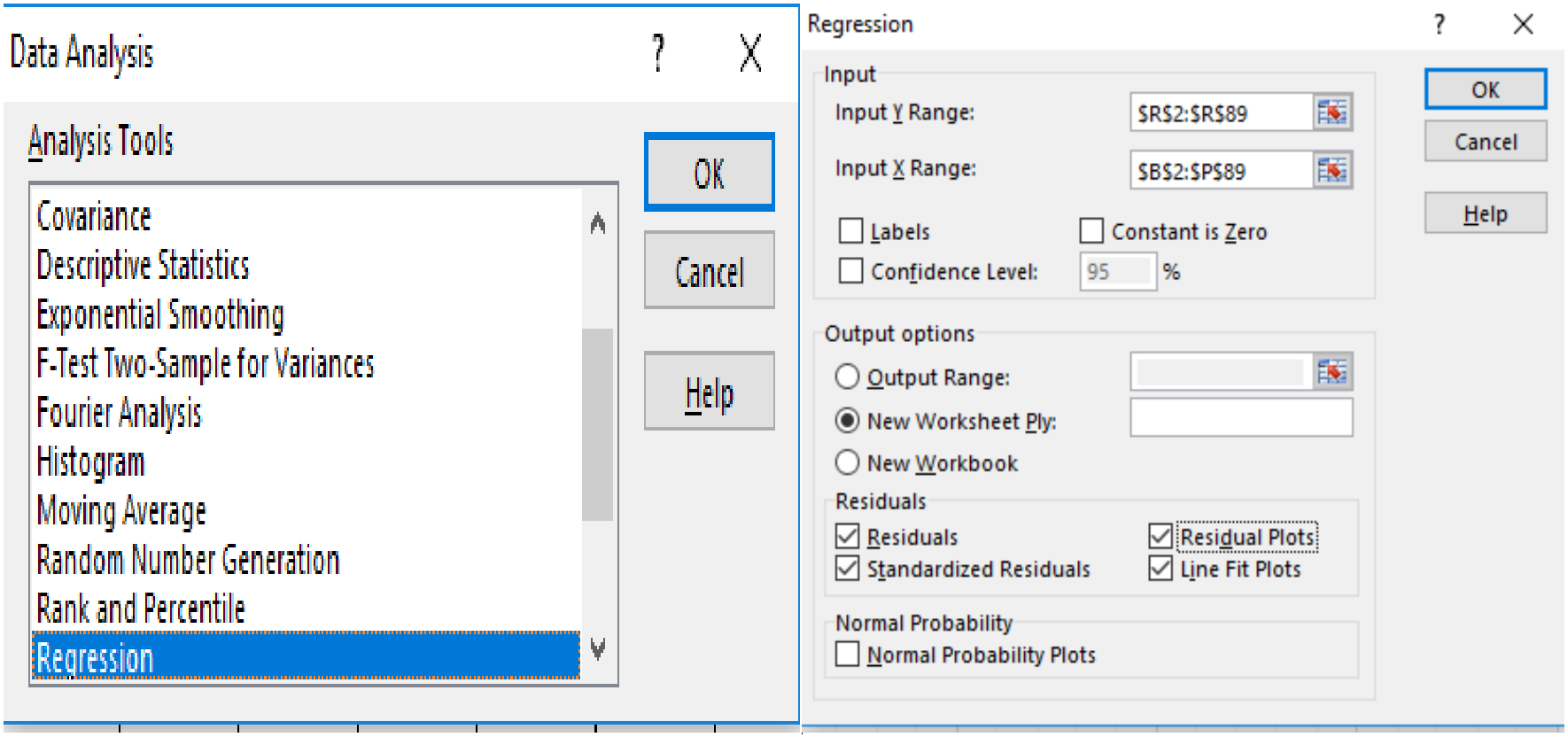

Figure 3-7 Regression Data Analysis Window

CHAPTER 4 ANALYSIS AND PARAMETRIC STUDIES OF EXCAVATION CASE HISTORIES AND GENERATION OF WALL DEFELECTION DATA.

4.1 Introduction

Over the past few decades, the number of well-documented deep excavation with field measurements and numerical data at each stage of excavation has increased significantly. However, experience from these cases has not been easily uses for any deep excavation projects. One of the tasks of this research study is to collect the well documented deep excavation cases and explore means of learning and extracting knowledge from them. It is noted that the total number of collected case histories had been less than ideal for the purpose of developing empirical models for estimating the wall deflection. Thus, in this study, secondary data were generated from numerical experimentation using finite element method solutions with the Mohr coulomb soil model. Eight deep-excavation case histories are collected in this research study. Prior to the compilation of deep-excavation case histories and generation of maximum wall deflection data from numerical experimentation, it is necessary to recognize the influential factors in terms of excavation-induced wall deflection.

4.2 Collection of Excavation Case Histories.

In this thesis study, eight case histories of braced excavation in clays from different country. Those are Central Jakarta in Indonesia, Post office Square Garage at Boston, Far East enterprise in Taipei, Formosa, Tokyo, Farrer Park, AT & T corporate center, Shanghai, metro station and TNEC site. This case histories include construction of building with basement, underground subway, underground garage, Mass transit and etc. And they are used for developing the intended model in Chapter five.

4.3 Parametric Studies

There are a number of parameters to be considered if one is interested in conducting a comprehensive parametric study. But as reported by earlier investigators (e.g. Goh, A.T.C., 1990), some of the variables have little or no effect in the performance of deep excavation supported by diaphragm wall. In this paper, variables that have practical importance and reported to have significant influence in the performance of deep excavations are considered

4.3.1 Finite Element Model

The finite element method is widely employed to model complex soil-structure interaction problems. For excavations in clays, the Mohr-Coulomb (MC) constitutive relationship is commonly used to model the clay stress-strain behavior.

In this thesis a total of 87 finite element simulation, performed with two-dimensional software package PLAXIS 2D. For plane strain analyses, the half mesh was used due to the symmetry in both geometry and material. Very fine mesh size was used for 2D analysis to improve the accuracy of FE calculations.

4.3.1.1 Model Generation

Numerical modeling is an effective way to investigate the soil-structure interaction mechanisms in deep excavations and has the ability to provide all the required information for design purposes. A typical numerical sample model and general dimensions for the study is shown in Figure 4-2.The model is simulated as an asymmetrical plane strain finite element employing 15-noded triangular elements and half of the geometry.

Two undrained functions, namely Undrained (A) and Undrained (B), were specified in PLAXIS: Undrained (A) requires effective stress parameters for both soil modulus and shear strength, whereas Undrained (B) uses the effective soil modulus and undrained soil shear strength. As discussed earlier, because a limited number of high-quality CU tests were conducted, a representative profile of effective shear strength parameters was not obtained. Therefore, the Undrained (B) function, using the effective soil modulus and undrained soil shear strength, was selected for analysis.

4.3.1.2 Boundary Condition

If considering the symmetry of excavation and taking half for analysis as shown figure 4-2 the symmetric boundary should be equipped. The boundary conditions in the finite element model are set automatically by PLAXIS 2D analysis as: -

(I) Vertical model boundaries with their normal in the x-direction (i.e. parallel to the y-plane) are fixed in x-direction (Ux = 0) and free in y –direction.
(II) The geometry model total fixities which is the combination (Ux =Uy =0).
(III) The ground surface of the model is free in all directions.
(IV) Pinned at the base to restrain any movements.

4.3.1.3 Mesh Generation

When the geometry of the model is fully defined and material properties are assigned to all clusters and structural object, the geometry has to be divided into finite elements in order to perform finite element calculation. The required input for the generator is a geometry model composed of points, lines, and clusters, of which the clusters (areas enclosed by lines) are automatically generated during the creation of the geometry model. PLAXIS allows for a fully automatic mesh generation of finite element meshes. The mesh generator is a special version of the triangle mesh generator developed by sepra. The generation of the mesh is based on a robust triangulation procedure, which results in unstructured meshes. The shape of the element used in the analysis can strongly affect the results obtained. The density of mesh is one of the main factors that can affect the result. In deep excavation work, the mesh in the area of stress concentration, of rapid strain changing, the crucial areas, and the object zones should be finer. The density of mesh for soil closest to the diaphragm wall should be finer than the soil further away from the diaphragm wall (Brinkgreve, 2002).

4.3.1.4 Sensitivity Analysis

Sensitivity Analysis is carried out for excavation, However Domain size to optimize the computational time and effort without affecting the accuracy of the results. A sensitivity analysis of model width has been conducted using different domain areas to examine influences on maximum wall deflection. Considering the width of the domain in the horizontal, x-direction was as 3He, 4He and 5He (He-Depth of excavation) from the center

of excavation. Varying the domain size in x-, directions is done until enlarging the size has a very negligible effect on the maximum wall deflection.

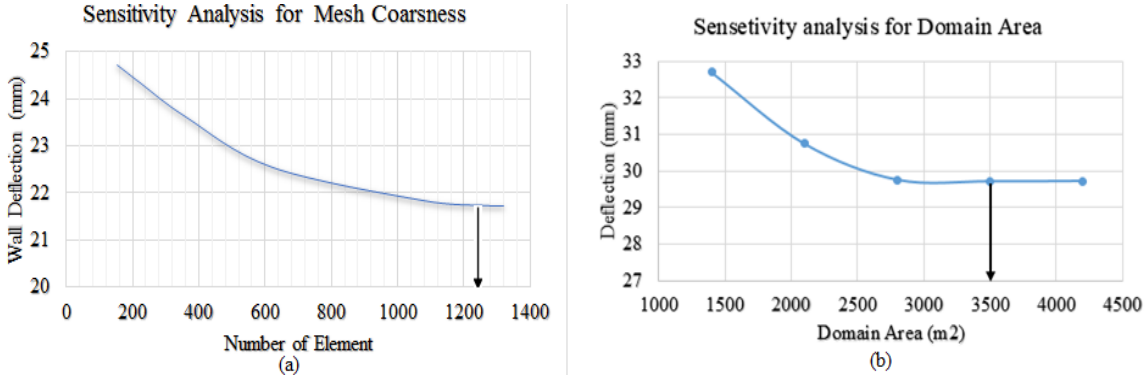

Figure 4-1Sensitivity analysis for (a) Mesh coarseness, (b) Domain area

As shown in figure 4.1 (a), sensitivity analysis on mesh coarseness was conducted for optimization of accuracy and computational time. PLAXIS 2D program allows for a fully automatic generation of finite element meshes using the 'robust triangulation scheme'. In PLAXIS 2D, five basic types of meshing were available namely 'very coarse', 'coarse', 'medium', 'fine', and 'very fine', each had a progressively refined mesh coarseness factor. The mesh should be sufficiently and optimally fine to obtain correct numerical results. A very coarse mesh may fail to capture the important responses of the domain. Moreover, very fine meshing should be avoided since it will take excessive time for calculations

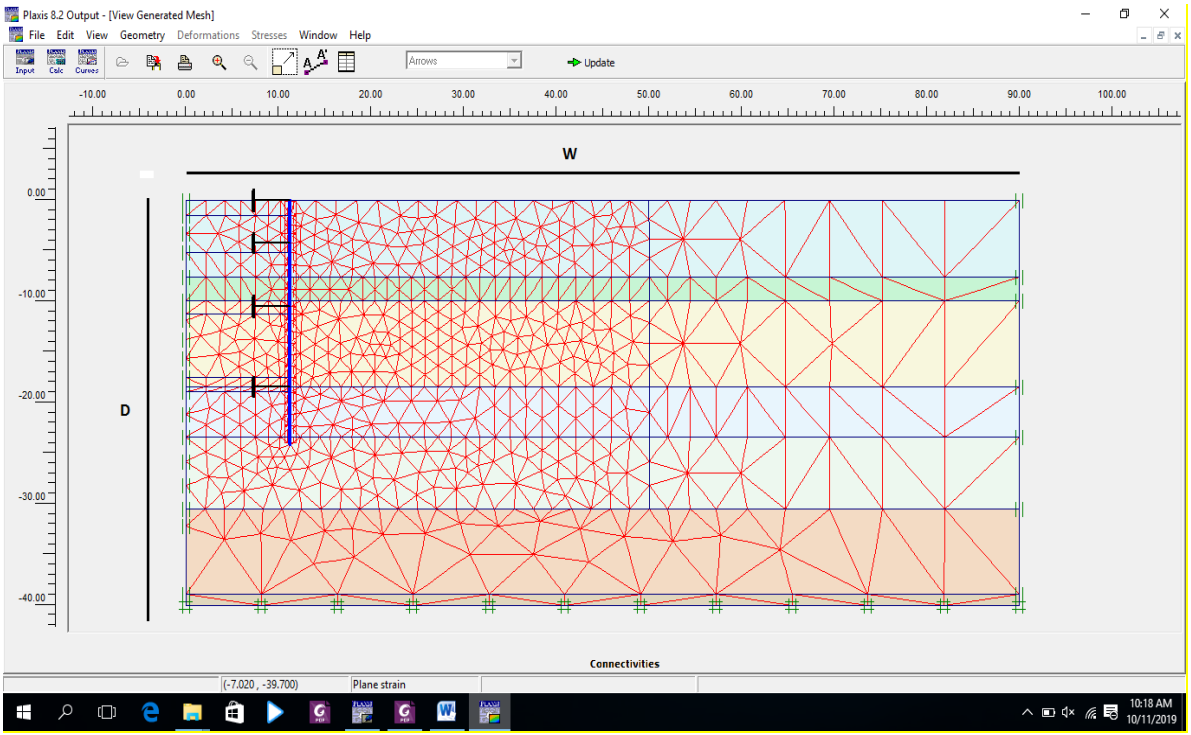

Figure 4-2 Schematic of Finite Element Model Input for Parametric Studies (1257 Elements)

4.4 Possible Factors Affecting Maximum Wall Deflection

Factors that might affect the maximum wall deflection of braced excavation are examined and analyzed by finite element software PLAXIS 2D and large number of data of wall deflection is generated. This generated data can used for development of intended simple model for the estimation of wall deflection caused by braced excavation in clays seen in next chapter. Those possible parameters induced unbalanced force on wall due to deep excavation the unbalanced force is synthetic result of many factors such as the excavation depth, excavation width, wall embedment depth, wall thickness and shear strength of soil. The combination of these parameters induces unbalanced force. However, some factors have greater influence on the maximum wall deflection while the others may have lesser influence.

For discussion three sites are selected, including are Central Jakarta, post office, and Far East enterprise site.

Case study 1: - Central Jakarta in Indonesia

Many studies have evaluated the effect of deep excavations on the behavior of walls. But limited studies have reported excavation cases in Jakarta, Indonesia. Furthermore, explorations of soil properties are limited, and few of the large-scale deep excavations in the city have been well documented. The length of the excavation was 430 m and the width excavation 22.6 m in the considered section. The construction was performed using a top-down method with five excavation stages, supported by four-level reinforced concrete slabs with various thicknesses. And the max excavation depth is 18.9m in the final excavation stage. The excavated pit was retained using a 1.0-m thick and 24.1-m deep concert diaphragm wall. The subsurface soil contained mostly clay, silty clay, and clayey silt. Notably, the soil is mainly classified as clay soil. In general, the stratigraphic profile consisted of soft to firm alluvial clay overlying stiff to hard and diluvial clay. The soil unit weight varied between 15 and 18kN/m3 (with average value of 16.9kN/m3). The site investigation work included 10 boreholes and standard penetration tests (SPTs) were performed at intervals of 1–2m.the undrained shear strength (Su) of the clay was obtained from triaxial UC tests and validated through several empirical methods (Su=6*N, where N is the SPT number) for estimating the Su of the clay; so, average value of 94.8 kPa was taken.The construction phase and sequences of excavation are listed below: -

Table 4-1: construction stages of Jakarta excavation site.

Phase	Construction Details.
1	Diaphragm wall installation
2	1st excavations to the depth of GL.−1.5 m
3	Deck slab installation at ground Level (slab thickness, t = 0.4 m)
4	2nd excavation to the depth of GL. −5.08 m
5	Top slab construction at GL. −4.18 m (slab thickness, t = 0.8 m)
6	3rd excavation to the depth of GL. −11.18 m
7	Middle slab construction at GL. −10.48 m (slab thickness, t = 0.4 m)
8	4th excavation to the depth of GL. −17.53 m
9	5th excavation to the depth of GL. −18.93 m
10	Bottom slab construction at GL. −18.13 m (slab thickness, t = 1.0 m)

Wall deflection profile and effect of input parameters on maximum wall deflection are summarized below.

Effect of Excavation Depth:

It is evident that the wall deflection and bending moment increase with the excavation depth. All analysis results and parametric studies confirm this fact. The depth of excavation (H_e) is one of the key factors influencing the performance of deep excavation; this section was concentrated on this factor. The maximum wall deflection as a fraction of the excavation depth He is typically in the range of (0.107–0.133%). Wall deflection profile and effect of excavation depth on maximum lateral deformation profile is described graphically in figure 4-3 as obtained from the FE analysis.

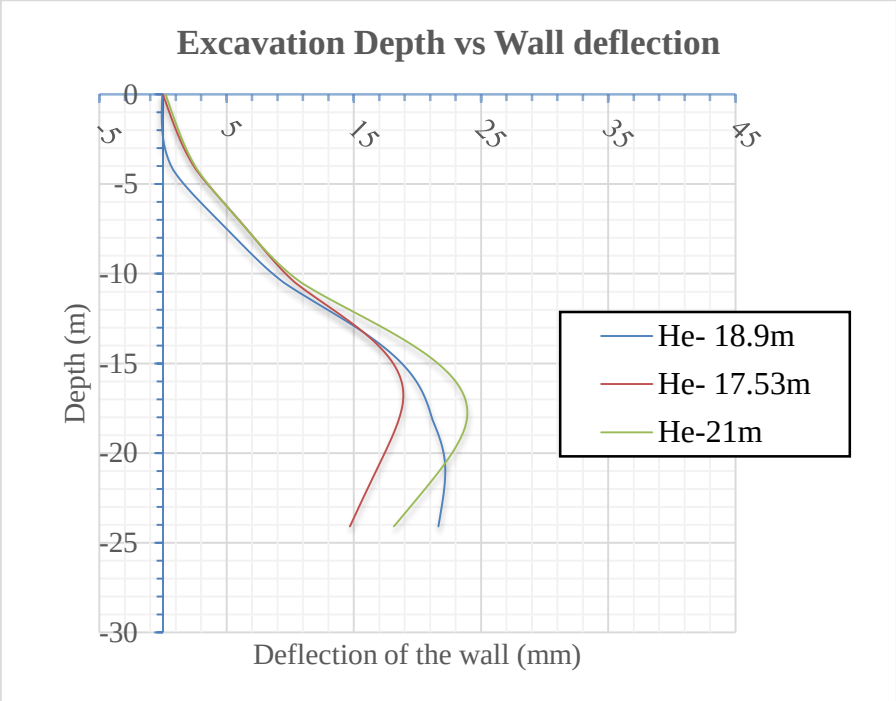

Figure 4-3 Wall deflection Profile Different Excavation Depth (He).

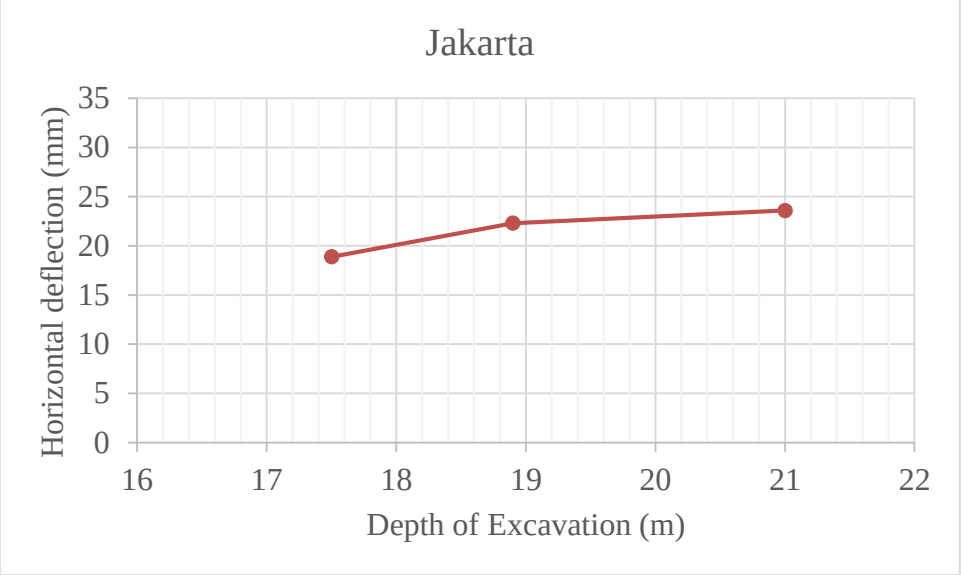

Figure 4-4 Effect of Excavation Depth (He) on Maximum Wall Deflection

The output of this analysis is presented as shown in Figure 4-3 and Figure 4-4, the depth of excavation for analysis He (17.53, 18.9 and 21) all parameters are the same as the original site the only varied data was the depth of excavation, due to these the effect of change in depth of excavation on the maximum horizontal displacement of the diaphragm wall would be changed. For the change of excavation depth (He) from (17.5-21) m the result showed that the computed maximum deformation is (Δ_{max}) increased typically from (27-52. 3) %.

Effect of Excavation Width:

Another geometrical key factor for lateral wall deflection is excavation width. Mana (1978) investigated the effect of width of excavation and showed that the wider the excavation, the larger are the magnitude of ground movements and the size of the yield zones as expected. He also indicated that the predicted zone of yielding is larger than those assumed by Terzaghi (1943), especially on the active side of the wall. In this part of analysis when varying excavation width, the maximum wall deflection also varied, in the analysis, all parameters are kept the same as the original except the width of excavation, which is varied (17.6, 22.6 and 27.6) m.

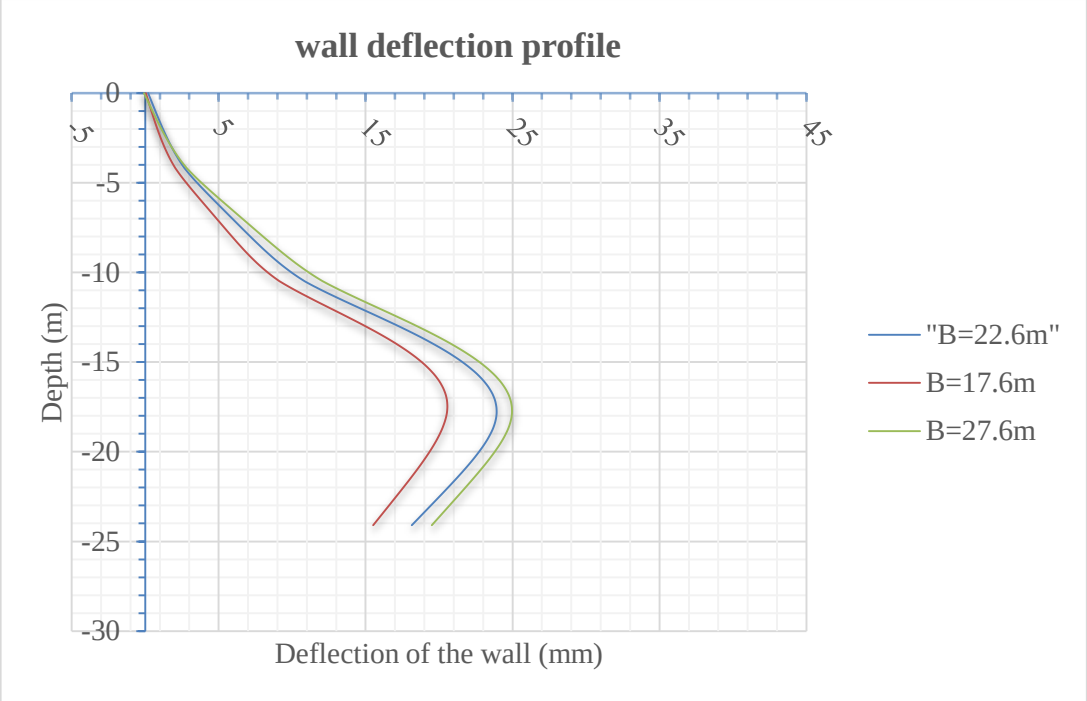

Figure 4-5 Wall Deflection Profile for Different Excavation Width (B) from 2D Analyses

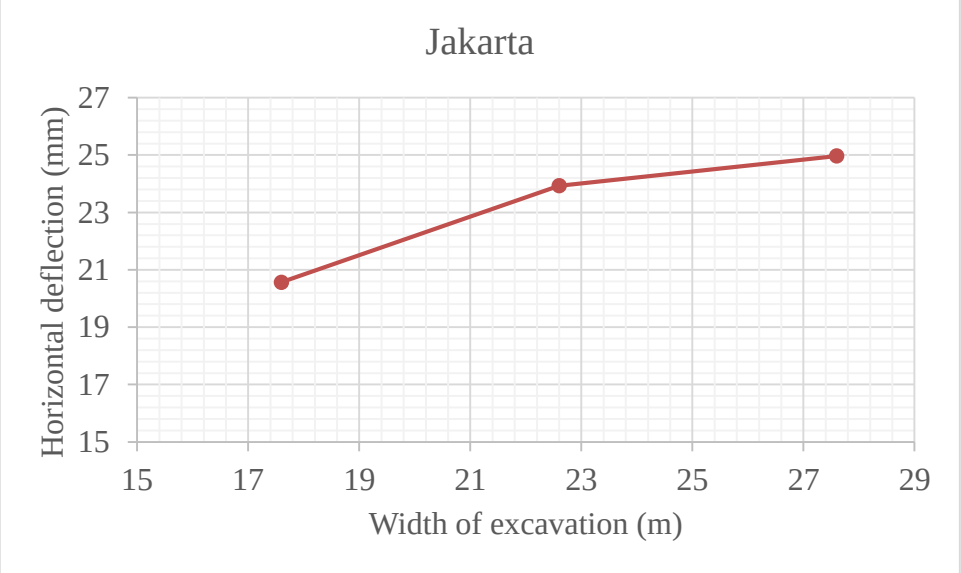

Figure 4-6 Effect of Excavation Width (B) on Maximum Wall Deflection

The output of the analysis as shown in Figure 4-5 and Figure 4-6. A radical change on the deformation of diaphragm wall of deep excavation is observed on Jakarta site, as

excavation width increase from (17.6, 22.6 and 27.6m) the maximum wall deflection increases from 20.8% to 67.2%.

Effect of shear strength of soil:

Shear strength of the soil have the greatest influence on the performance of deep excavation especially in weak soils. In this part of the analysis all parameters are kept the same as in above. The only varied parameter is the shear strength of the soil, which is varied from (19,108,144,84,90) -(24,114,150,90,96) -(30,120,156,96,102) kPa.

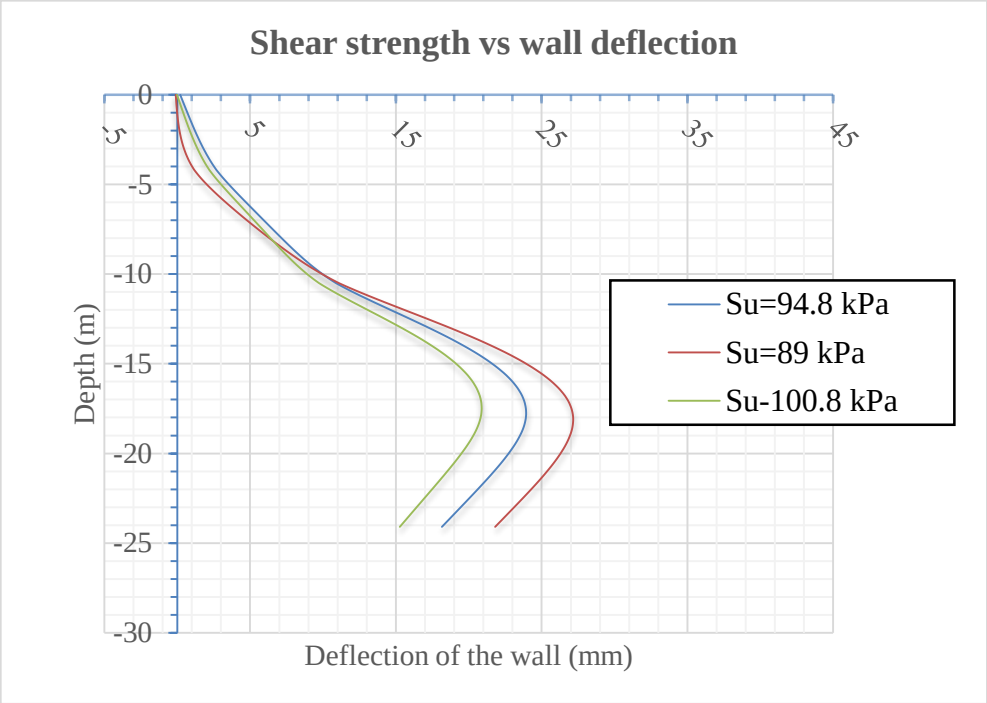

Figure 4-7 Wall Deflection Profile for Different Shear Strength (Su).

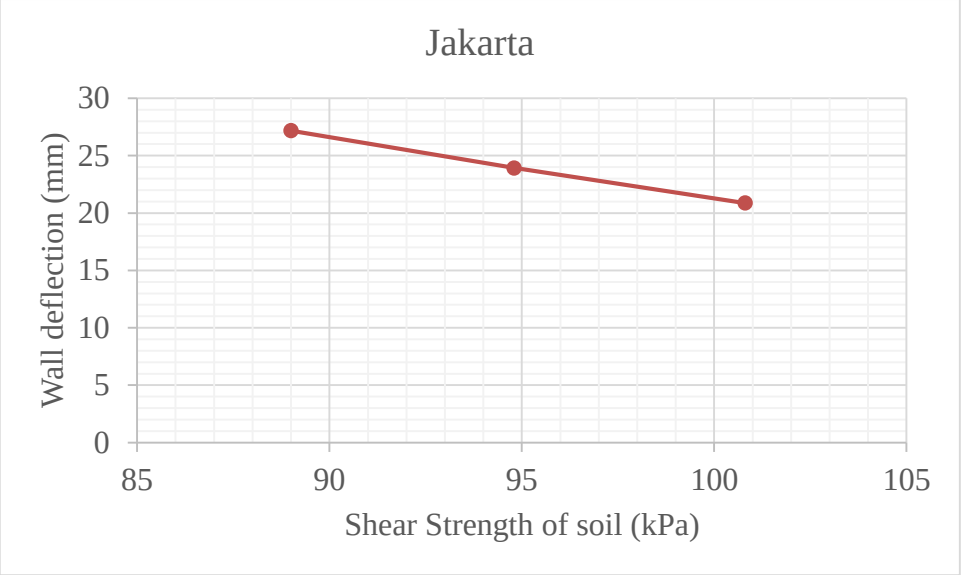

Figure 4-8 Effect of Shear Strength (Su) on Wall Deflection.

The output of the analysis summarized in figure 4_7 and figure 4_8 shows that; when shear strength goes to stiffer the maximum wall deflection decreased from (50-55%). Many researchers mentioned that in the case of excavation with flexible wall, or with stiffer soils, or partially penetrating wall, the soil strength might have greater influence.

Effect of wall thickness:

Stiffer lateral support system tends to reduce the deflection of the wall at support levels and increase strut loads. In this part of the analysis, this thesis concentrates on how the change in stiffness of the diaphragm wall affects the performance of deep excavation. The output from this analysis is presented in the Figure 4-9 and Figure 4-10, which also give the range of values for the thickness of the diaphragm wall used in the parametric study. Since the young's modulus of the diaphragm wall is kept unchanged for all the analyses, a change in thickness of the diaphragm wall represents a change in both the bending stiffness and the axial stiffness of the diaphragm wall. For all the analyses conducted in this section of the parametric study, the input parameters for wall and soil are kept the same. The wall thickness is varied for parametric study (i.e. 0.8, 1, and 1.2) m were considered.

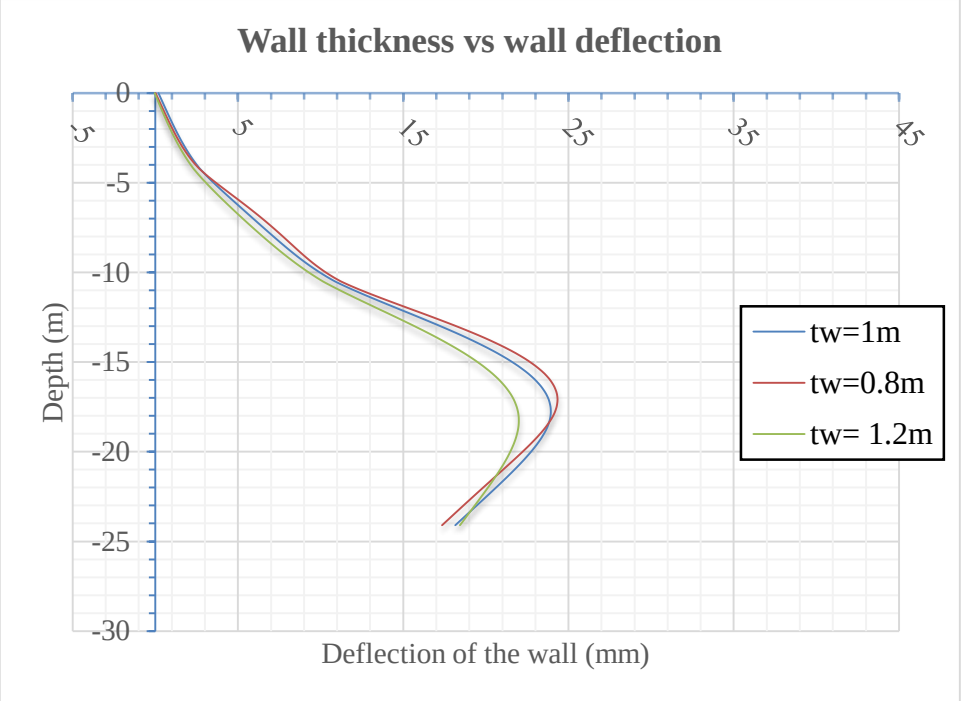

Figure 4-9 Wall deflection profile for different wall thickness (tw) from 2D analyses

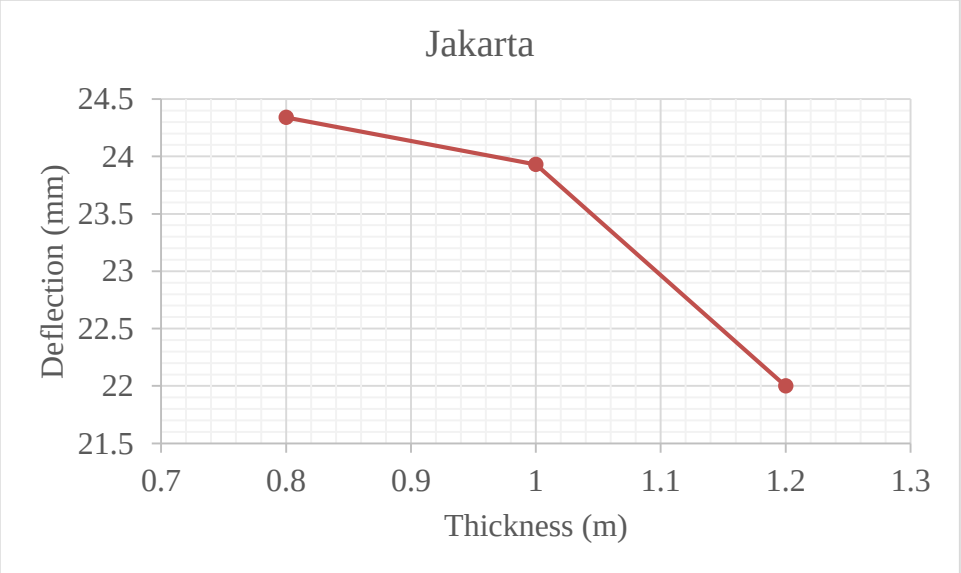

Figure 4-10 Effect of Wall Thickness (Tw) on Maximum Wall Deflection.

The maximum lateral wall displacement (U) decreased with increasing wall thickness. It is also seen that the maximum variations in the values of maximum lateral wall

displacement were (4.7-10) %. And the trend of wall deflection was reversed in the case of maximum lateral wall displacement.

Case study 2: - Post Office Garage in Boston

The observations during construction in the Post Office Square Garage (POSG) excavation case history were performed by Whittle et al. this case is in the heart of the financial district of Boston. Existing buildings up to 40 stories tall are located adjacent to the site. The diaphragm wall (31 m deep and 0.8 m thick) extending down into the bedrock was used as the permanent lateral earth pressure support. The POSG case was performed by the Top-down construction method. The maximum excavation depth was 18.5m. The detailed construction sequences, including the excavation depth and the depth where the floor slab was constructed.

Table 4-2: construction stages of post office garage excavation site.

Phase	Construction Details
Phase 1	Wall installation
Phase 2	First excavation up to 2.5 m.
Phase 3	Install strut @ 1m.
Phase 4	Second excavation up to 5.5m.
Phase 5	Install strut @ 2.5.
Phase 6	Third excavation up to 7.3m.
Phase 7	Install strut @ 6.2m.
Phase 8	Fourth excavation up to 10m.
Phase 9	Install strut @8.5m.
Phase 10	Fifth excavation up to 13m.
Phase 11	Install strut @11.5m
Phase 12	Sixth excavation up to 15.5m
Phase 13	Install strut @ 14m.
Phase 14	7th excavation up to 18.5m
Phase 15	Install strut @ 16.2m.

Wall deflection profile and effect of each parameter on wall deflection summarized below:

Effect of Excavation Depth:

As the depth of excavation is increase the deformation of deep excavation increase, these sections will summarize the maximum wall deflection to a fraction of the excavation depth He, typically in the range of (0.39–0.64) % of He.

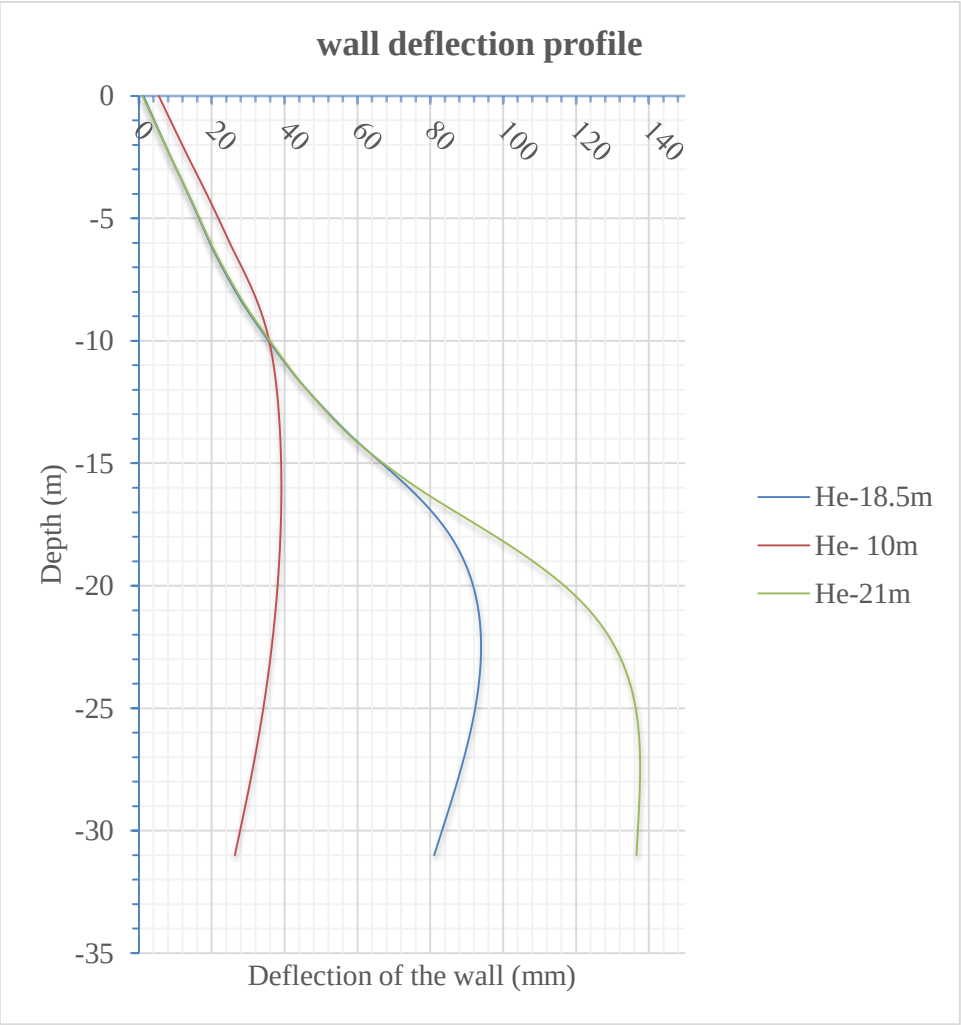

Figure 4-11 Wall Deflection Profile for Different Excavation Depth (He) from 2D Analyses

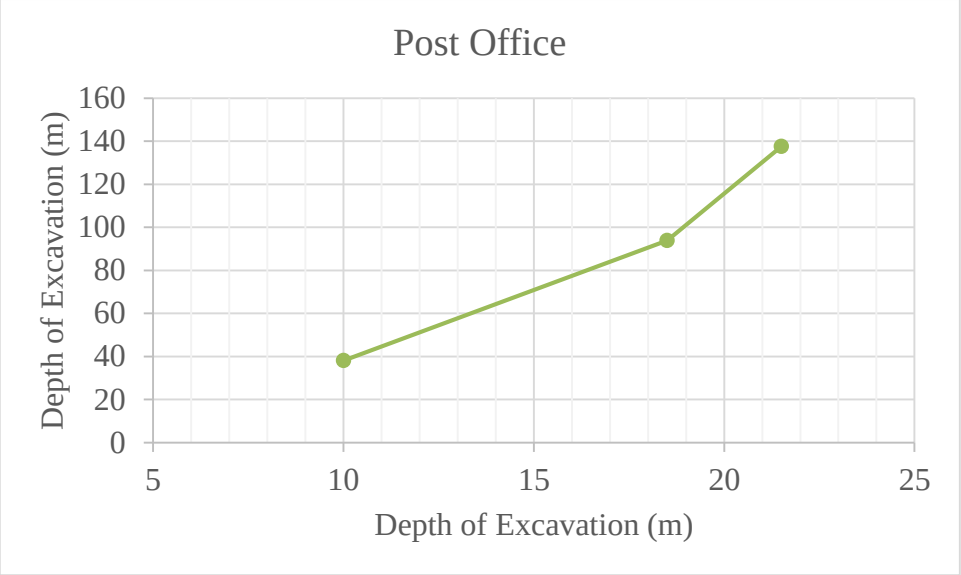

Figure 4-12 Effect of Excavation Depth (He) on Maximum Wall Deflection

The output of this analysis is presented from Figure 4_11 and Figure 4_12 are summarized below. The depth of excavation for analysis He (10,18.5 and 21.5) all parameters are the same as the original site but the only varied data was the depth of excavation, due to these the effect of change in depth of excavation on the maximum horizontal displacement of the diaphragm wall would be changed. For the change of excavation depth (He) from 10-21.5 m the result showed that the computed maximum deformation is (Δ_{max}) increased typically in range of (15-20) %.

Effect of Excavation Width:

In this part of analysis when varying excavation width, the maximum wall deflection also varied, for analysis all parameters are same as original except width of excavation, width of excavation varied for analysis B (28, 33 and 38) m.

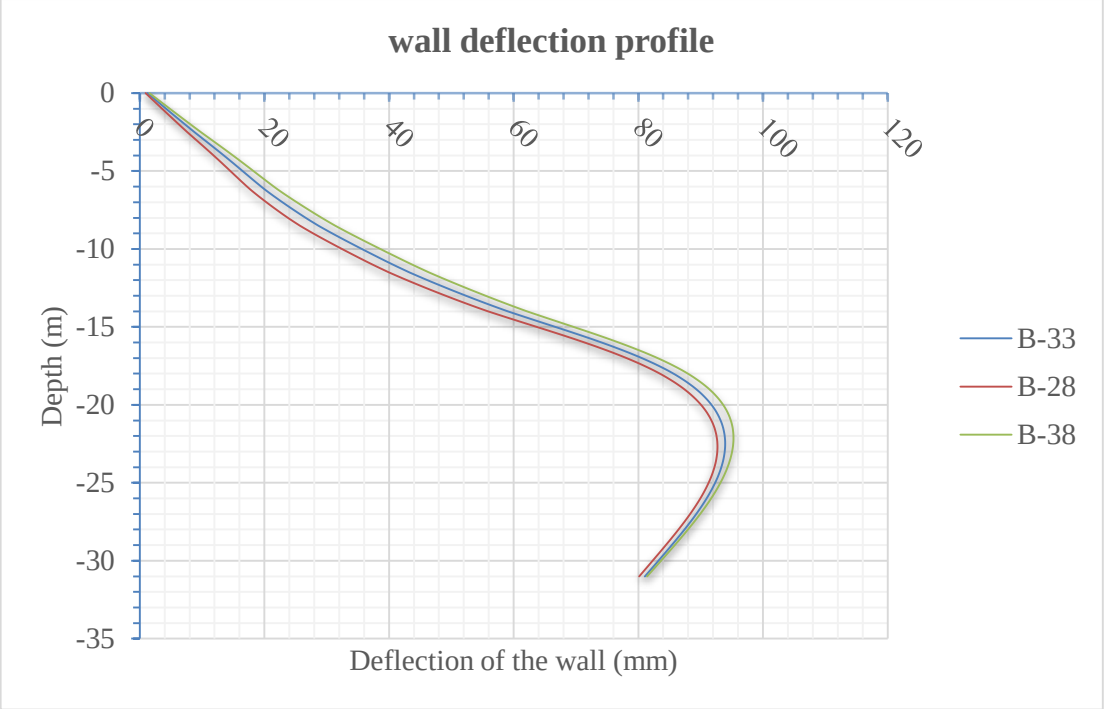

Figure 4-13 Wall Deflection Profile for Different Excavation Width (B) from 2D Analyses

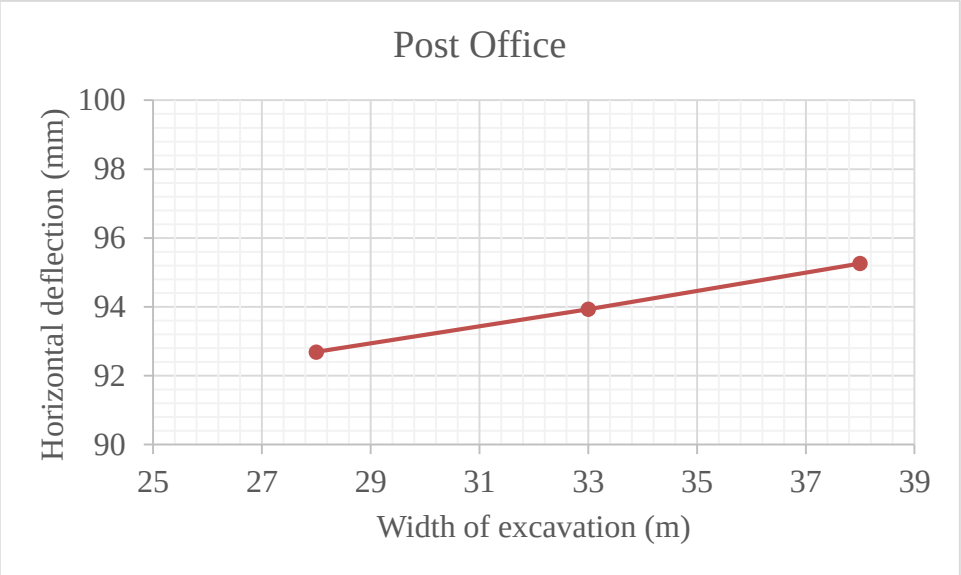

Figure 4-14 Effect of Excavation Width (B) on Wall Deflection.

The output of analysis is summarized below in figure 4_13 and figure 4_14. The change on the deformation of diaphragm wall of deep excavation is observed on post office site, as excavation width increase from (28-38) m the maximum wall deflection increases from 24.8% to 26.6%.

Effect of shear strength of soil:

Shear strength of the soil has the greatest influence on the performance of deep excavation in especially in soft clay soil.in these part of the analysis, all parameters keep same as previous the only varied parameter shear strength of soil, the shear strength of soil varied from (64,70 and 76) kPa.

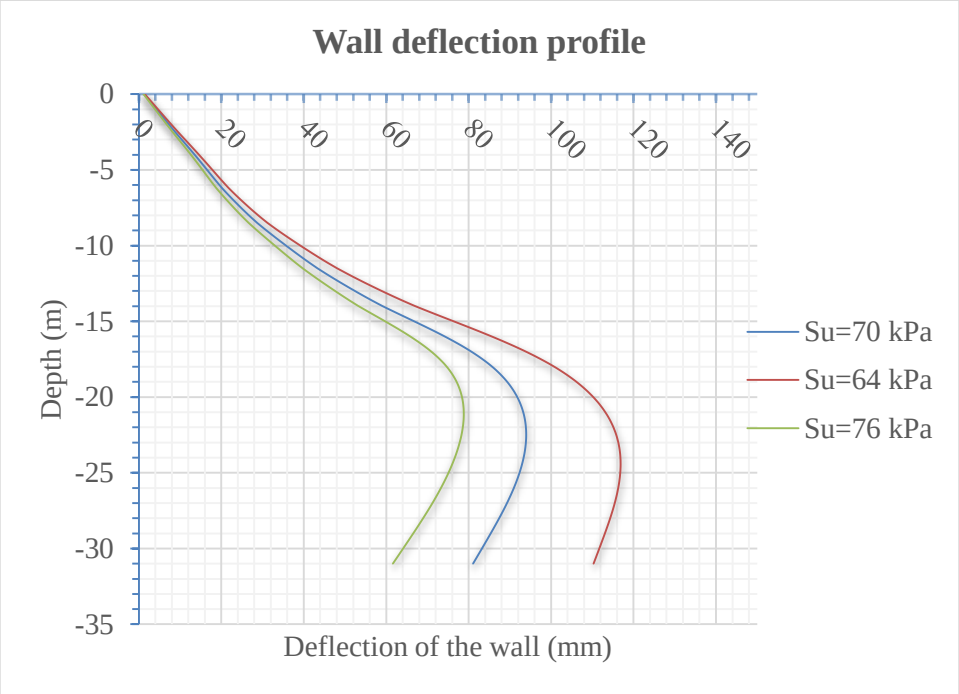

Figure 4-15 Wall Deflection Profile for Different Shear Strength (Su) from 2D Analyses

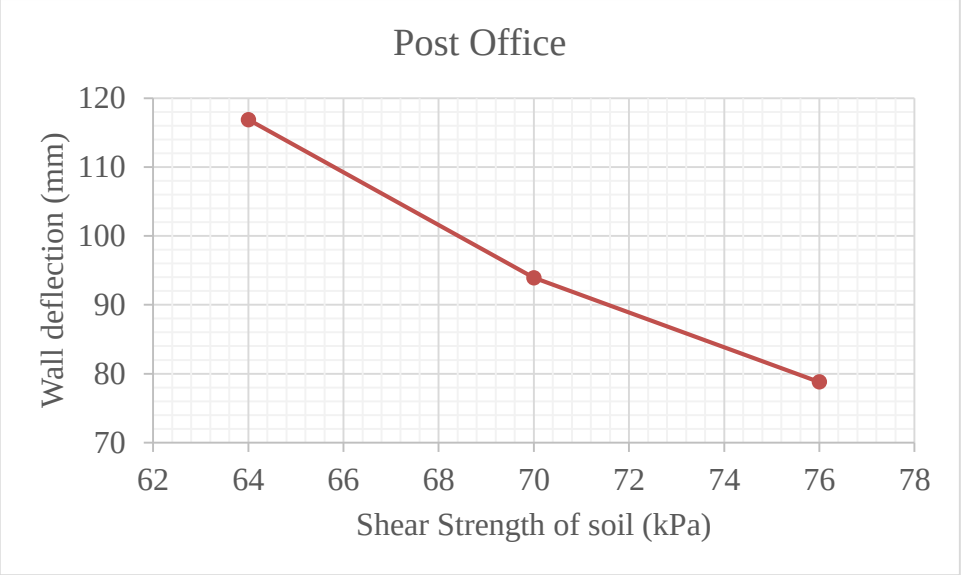

Figure 4-16 Effect of Shear Strength (Su) on Maximum Wall Deflection

The output due to shear strength variation would change the max lateral wall deflection the output summarized in figure 4-15 and figure 4-16 when shear strength increased (65-70-75) kPa the max deformation decreased from (39.65-26.16%). They admitted, however, that in the case of excavation with the flexible wall, or with stiffer soils, or partially penetrating wall, the soil strength might have greater influence on the maximum wall deflection.

Effect of wall thickness:

Stiffer lateral support system tends to reduce the deflection of the wall at support levels and increase strut loads. In this part of the analysis, the paper concentrates on how the change in stiffness of the diaphragm wall affects the performance of deep excavation. The output from this analysis is presented in figure 4-17 and figure 4-18, the range of values for the thickness of the diaphragm wall used in the parametric study. Since the young's modulus of the diaphragm wall was kept unchanged for all the analyses, a change in thickness of the diaphragm wall represents a change in both the bending stiffness and the axial stiffness of the diaphragm wall. For all the analyses conducted in this section of the parametric study, the following quantities or features are kept the same.

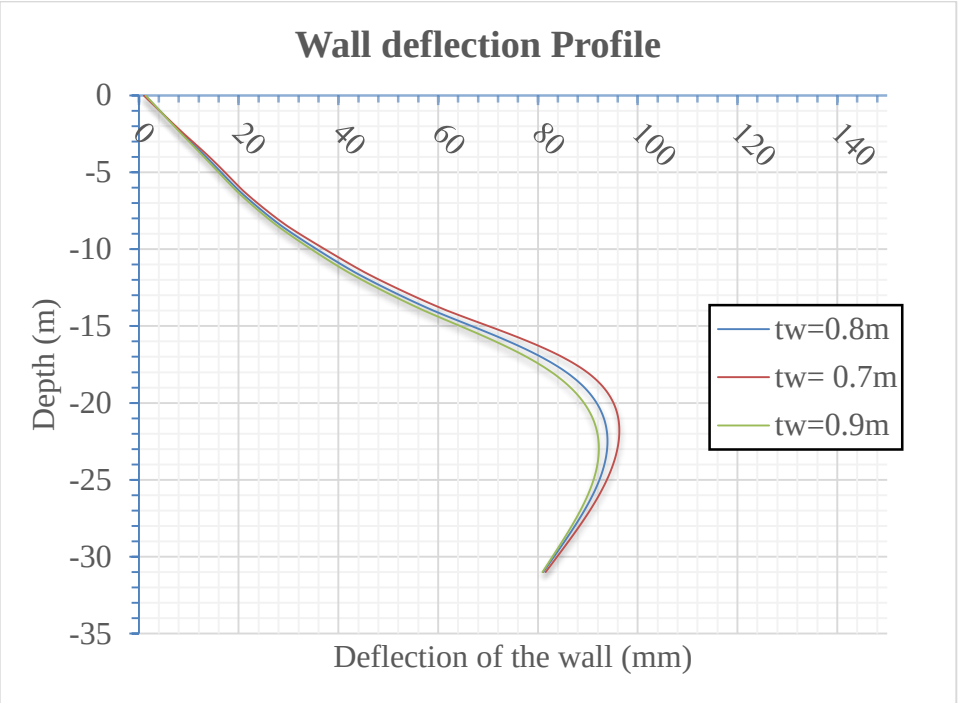

Figure 4-17 Wall deflection Profile for Different Wall Thickness (Tw) from 2D Analyses



Figure 4-18 Effect of Wall Thickness (Tw) on Maximum Wall Deflection

There are three different wall thicknesses for analysis (i.e. 0.7, 0.8, and 0.9) m were considered to study. The maximum lateral wall displacement (u) decreased with increasing

wall thickness. It is also seen that the maximum variations in the values of maximum lateral wall displacement were 5.74- 4.14%. The maximum bending moment in the wall increased as the thickness of the wall increased, whereas the trend was reversed in the case of maximum lateral wall displacement.

Case study 3: - Far East enterprise Center project in Taipei.

The Far East enterprise center project is found in a very congested city in Taiwan with an Excavation width of B=70m and Excavation Depth He=19.7m with five struts and a diaphragm wall is used as to support the excavation. The case was performed by the Top-down construction method. The detailed construction sequences, including the excavation depth and the depth where the floor slab was constructed. Wall deflection profile and effect of each parameter on wall deflection summarized below:

Effect of Excavation Depth:

As the depth of excavation is increase the deformation of deep excavation increase, these sections will summarize the maximum wall deflection to a fraction of the excavation depth He, typically in the range of 0.12–0.64% of He. When the excavation depth increases from 15.4 to 24.6 m the deformation increased from 12%-15%.all parameters are the same as original analysis the only excavation depth varied for analysis.

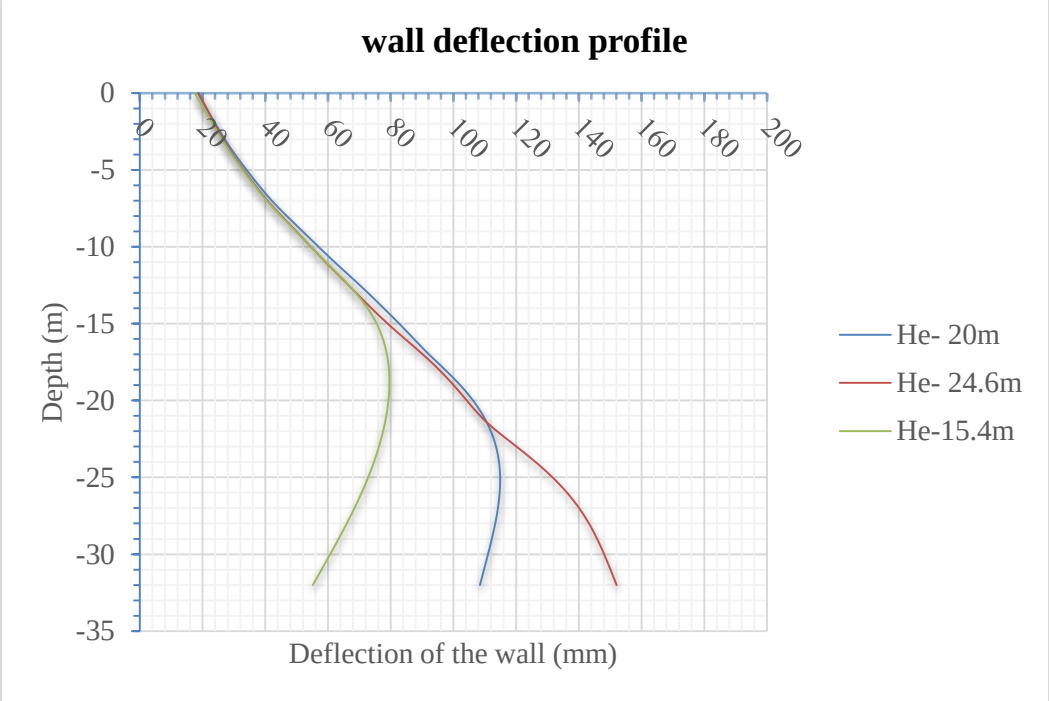

Figure 4-19 Wall Deflection Profile for Excavation Depth (He) Variation from 2D Analyses

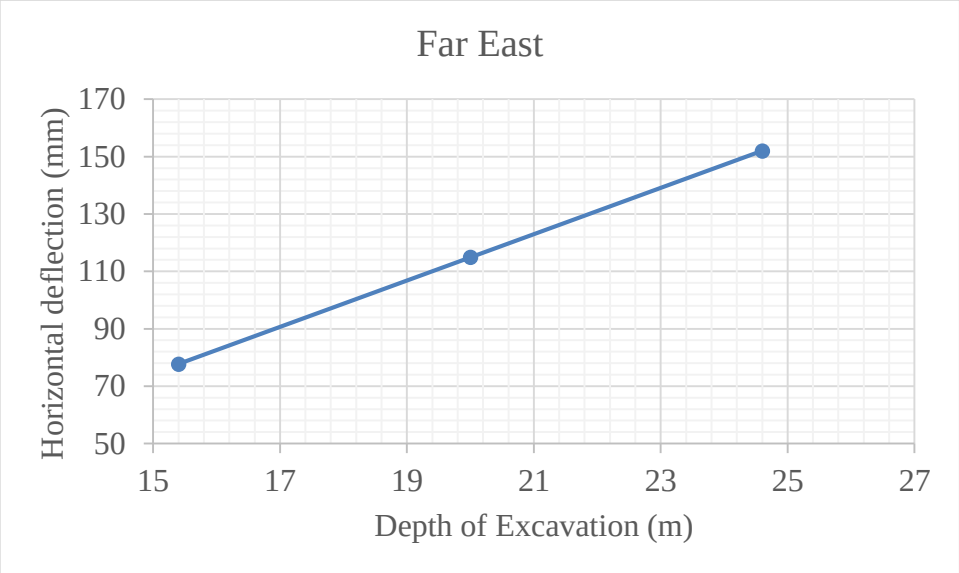

Figure 4-20 Effect of Depth of Excavation (He) on Maximum Wall Deflection.

Effect of Excavation width variation:

Among the geometry of excavation, the width would influence lateral wall deformation significantly. For Far East analysis when excavation width varied and all parameters keep the as original deformation also variety when excavation width changed from (65, 70, 75) m, deformation changed from 37.8% to 39.6%.

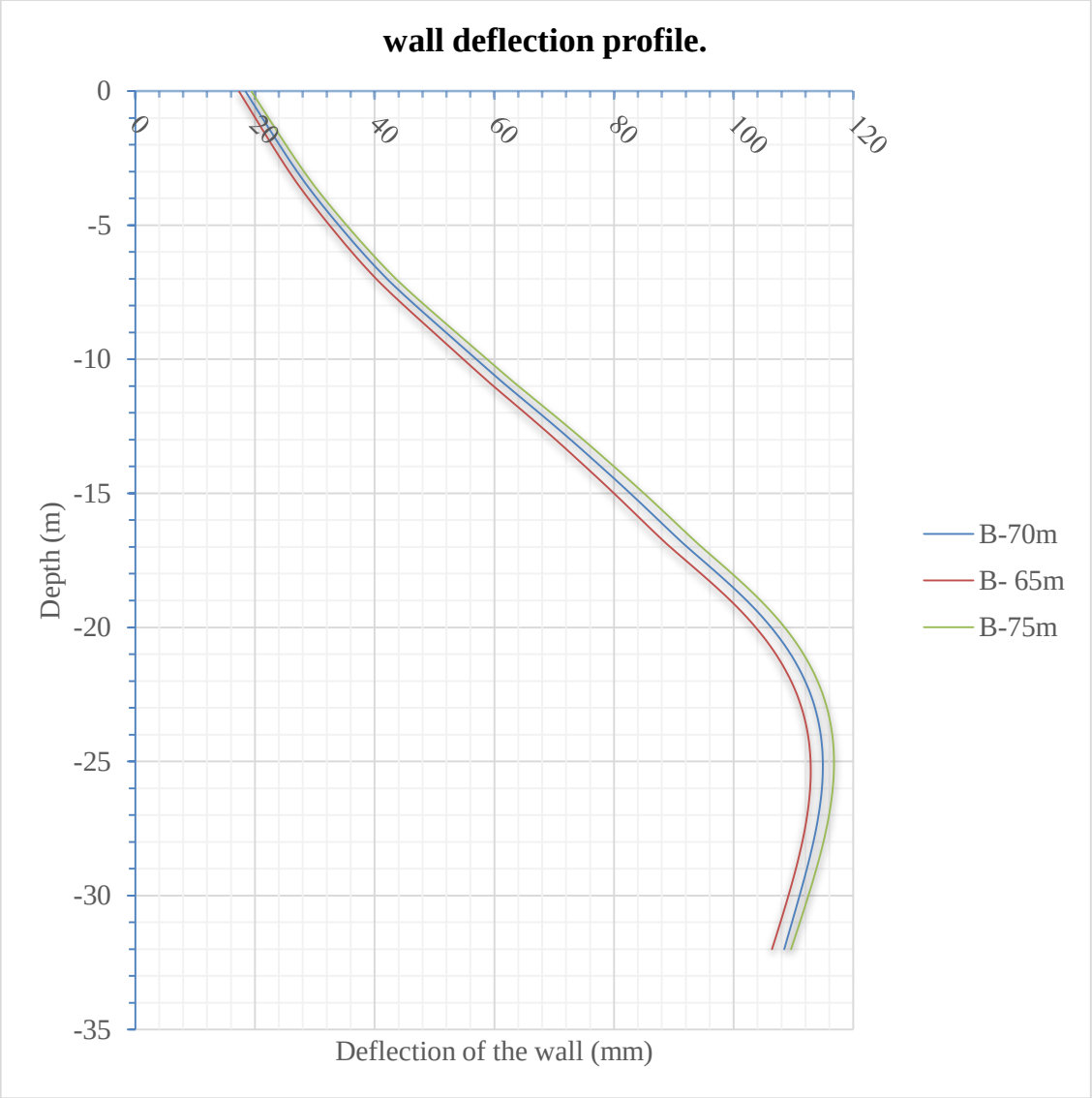

Figure 4-21 Wall Deflection Profile for Different Excavation Width (B) from 2D Analyses

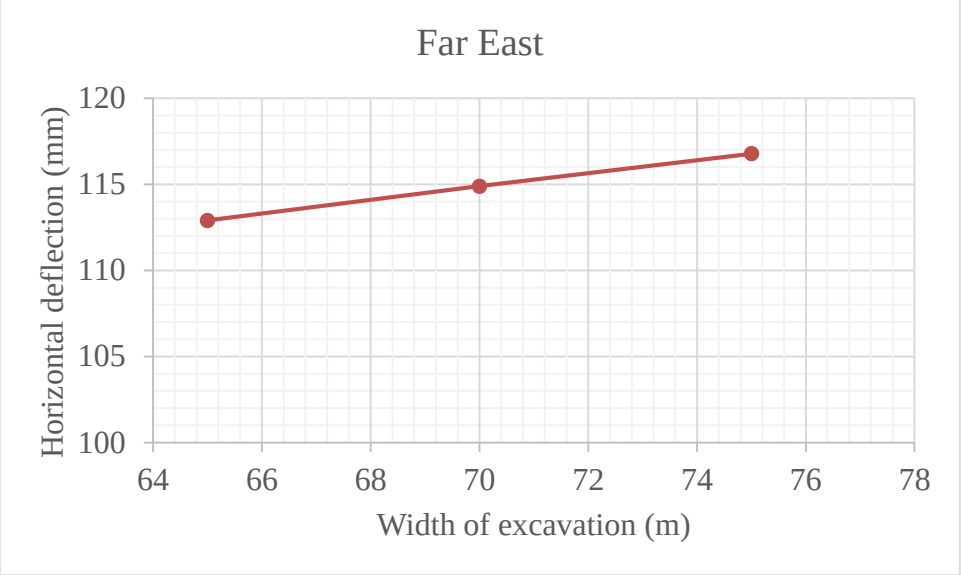

Figure 4-22 Effect of Excavation Width (B) on Maximum Wall Deflection

Effect of soil shear strength:

Shear strength variation has an influence on lateral deformation of wall and performance of deep excavation, when shear strength of soil increases the deformation of the wall decreases b/c when shear strength of high i.e. the soil goes to stiff w/c have the ability to stand by itself. In these sites when shear strength value for study lateral deformation, (70.5, 76.5, and 81.5) KPa, all other parameters are the same as original analysis. Then, lateral deformation decreased from (43.7%-24.1%). These indicate that the shear strength of soil has a significant effect on lateral wall deformation on a braced excavation in clay.

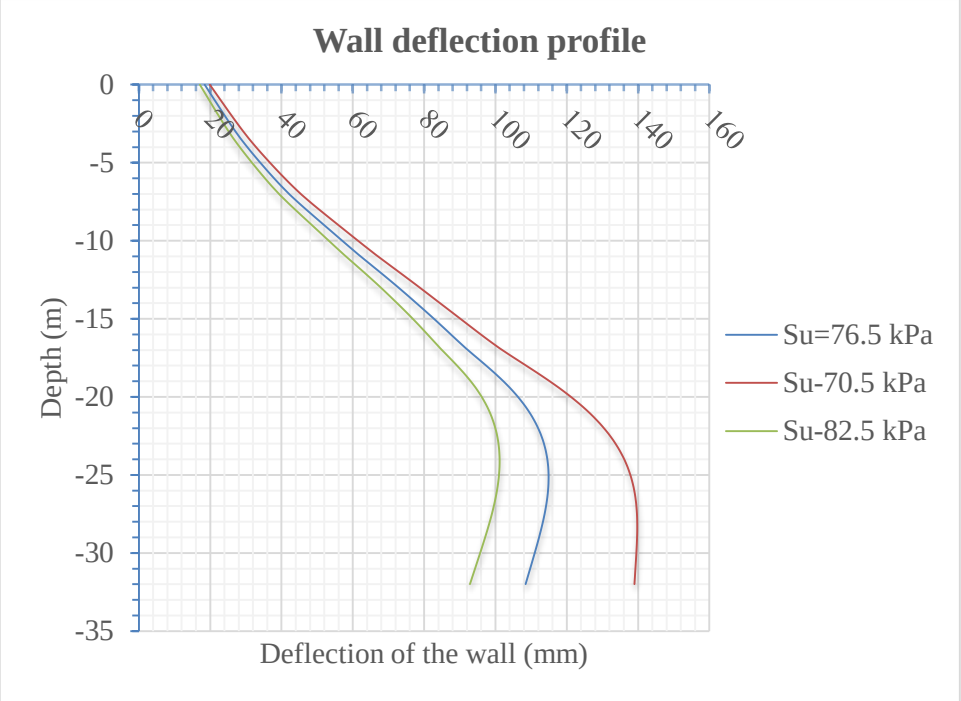

Figure 4-23 Wall deflection Profile for Different Shear Strength (Su) from 2D Analyses

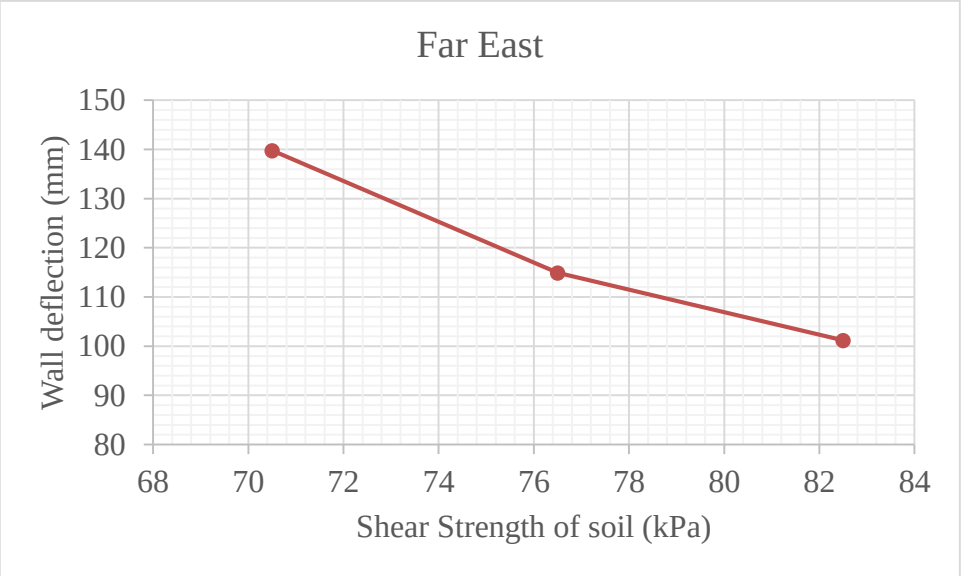

Figure 4-24 Effect of Shear Strength (Su) on Maximum Wall Deflection

Effect of wall thickness variation:

Theoretically, the deformation of the wall will decrease with the increase of the stiffness of retaining structure. In this study, three different wall stiffness values were used to analyze these change system stiffness's. All other parameters are the same as the original analysis. The wall thickness are (0.6, 0.7, and 0.8) m and the lateral deformation from these analyses typically in the range of (5.18%-4.4%).

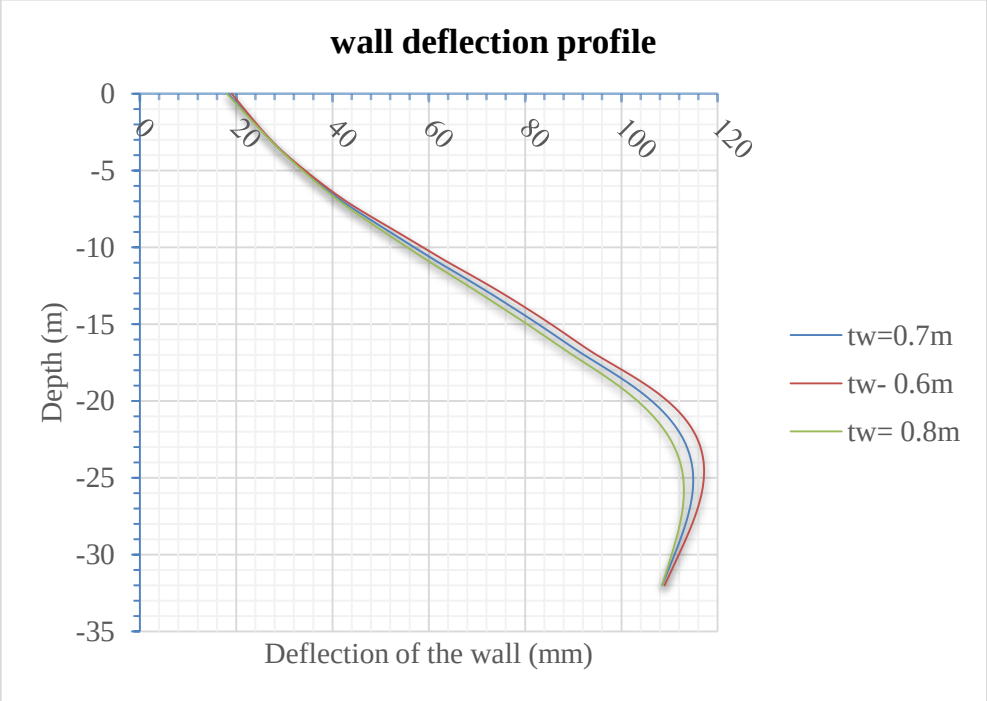

Figure 4-25 Wall Deflection Profile for Different Wall Thickness (Tw) from 2D Analyses

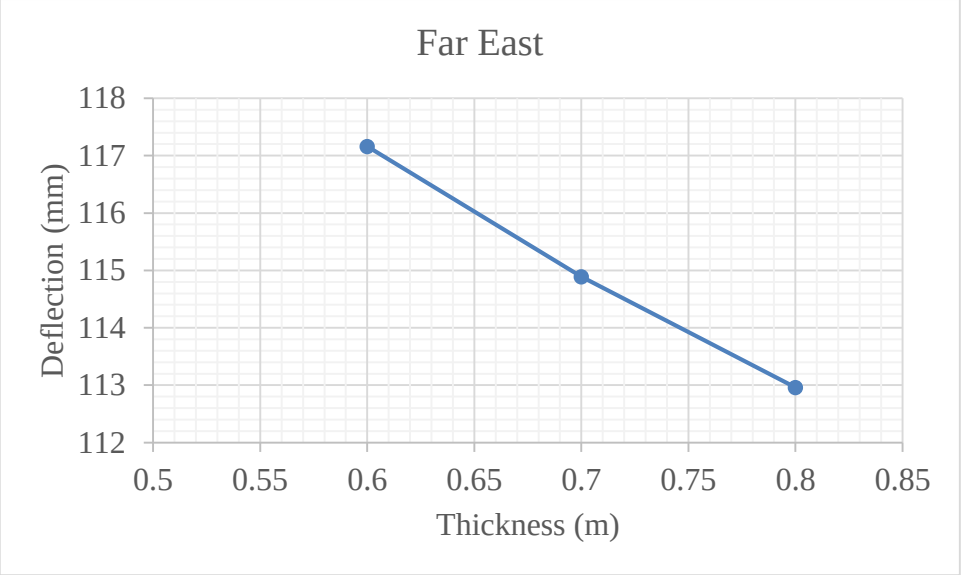

Figure 4-26 Effect of Wall Thickness (Tw) on Maximum Wall Deflection

4.5 Data generation for Wall Deflection

The possible factors discussed above are believed to influence the maximum wall deflection of the wall in clays, the previously described input parameter of the wall and soil for the FEM numerical analysis is varied to generating secondary wall deflection for the development of the intended model. For the analysis the strut member is modeled as anchor which is installed after each excavation stage and installation effect not considered.

Table 4.2-4.4 show a summary of input parameters incorporated in FEM analyses and generated wall deflection data.

The numerical simulations were performed using various soil moduli estimated using PMT method for Jakarta case study and empirical correlations with the soil shear strength, as proposed by Teparaksa (2007) for the other case studies.

$$E_{50} = 500Su \text{ (kPa)}$$

The drained soil modulus was converted to the effective soil modulus E'_{50} using,

$$E'_{50} = E_{50}\frac{2(1+v')}{3}$$

Where E'_{50} and E_{50} are the drained and Undrained soil moduli at the 50% stress level and v' is the effective Poisson's ratio, assumed to be 0.3 for clay and can be further simplified into $E'_{50} = 0.867E_{50}$.

Summary of Excavation Case Histories and generated wall deflection data

Table 4-3 Deflection of Wall Due to Excavation Depth Variation.

Case	Wall type	t (m)	He(m)	B(m)	Su(kPa)	E'(kPa)	E(GPa)	EI(kN.m2)	ϒ(kN/m3)	Δ(mm)
			17.53							18.88
			18.9							22.3
Jakarta	Diaphragm	1	21	22.6	94.8	76078.4	15.19	1265783	16.9	23.59
			10							39.14
			18.5							93.93
post office	Diaphragm	0.8	21.5	70	76.5	33162.75	10	447000	19	137.6
			15.4							77.74
			20							114.89
far east	Diaphragm	0.7	24.6	33	70	30345	24.8	709000	19	152.04
			6							39.13
			8.3							79.06
AT & T	Diaphragm	0.76	10	25	21.5	9320.25	26	951080	19	132.5
			12							76.94
			19.7							94.2
Tokyo	Diaphragm	0.9	25	30	50	21675	27.6	1675320	18.9	120
			13.2							58.9
			18.5							64.89
Formosa	Diaphragm	0.8	21	35	29.6	14986.4	27.6	1177600	18.84	71.76
			11.8							81.89
			19.7							106.61
TNEC	Diaphragm	0.9	24	40	50	21675	27.6	1675320	18.6	122.6

Table 4-4 Deflection of Wall Due to Wall thickness Variation

Case	Wall type	He(m)	T	B(m)	Su(kPa)	E'(kPa)	E(GPa)	EI(kN.m^2)	ϒ(kN/m3)	Δ(mm)
			0.8							24.34
			1							23.93
Jakarta	Diaphragm	18.9	1.2	22.6	94.8	76078.4	15.19	1265783	16.9	22
			0.7							96.34
			0.8							93.93
post office	Diaphragm	18.5	0.9	70	76.5	33162.75	10	447000	19	92.19
			0.6							117.16
			0.7							114.89
far east	Diaphragm	20	0.9	33	70	30345	24.8	709000	19	112.96
			0.6							92.87
			0.76							79.06
AT & T	Diaphragm	8.3	1	25	21.5	9320.25	26	951080	19	62.87
			0.8							112.8
Tokyo	Diaphragm	19.7	0.9	30	50	21675	27.6	1675320	18.9	94.2

			1							84.1
			0.7							68.45
			0.8							64.89
Formosa	Diaphragm	18.5	0.9	35	29.6	14986.4	27.6	1177600	18.84	59.87
			0.8							124.3
			0.9							106.61
TNEC	Diaphragm	19.7	1	40	50	21675	27.6	1675320	18.6	91.5

Table 4-5 Deflection of wall due to excavation width variation

Cases	Wall type	t (m)	B(m)	He(m)	Su(kPa)	E'(kPa)	E(GPa)	EI(kN.m^2)	ϒ(kN/m3)	Δ(mm)
			17.6							20.57
			22.6							23.93
Jakarta	Diaphragm	1	27.6	18.9	94.8	76078.4	15.19	1265783	16.9	24.97
			28							92.69
			33							93.93
post office	Diaphragm	0.8	38	18.5	76.5	33162.75	10	447000	19	95.26
			65							112.91
			70							114.89
far east	Diaphragm	0.7	75	20	70	30345	24.8	709000	19	116.78
			20							76.7
			25							79.06
AT & T	Diaphragm	0.76	30	8.3	21.5	9320.25	26	951080	19	81.5

			25							85.6
			30							94.2
Tokyo	Diaphragm	0.9	35	19.7	50	21675	27.6	1675320	18.9	110.4
			30							61.45
			35							64.89
Formosa	Diaphragm	0.8	40	18.5	29.6	14986.4	27.6	1177600	18.84	66.87
			35							92.56
			40							100.78
TNEC	Diaphragm	0.9	45	19.7	50	21675	27.6	1675320	18.6	123.67

Table 4-6 Deflection of wall shear strength variation

Case	Wall type	t (m)	Su(m)	B(m)	He(kPa)	E'(kPa)	E(GPa)	EI(kN.m^2)	ϒ(kN/m3)	Δ(mm)
			89							27.17
			94.8							23.93
Jakarta	Diaphragm	1	100.8	22.6	18.9	76078.4	15.19	1265783	16.9	20.88
			64							116.86
			70							93.93
post office	Diaphragm	0.8	76	70	18.5	33162.75	10	447000	19	78.8
			70.5							139.78
			76.5							114.89
far east	Diaphragm	0.7	82.5	33	20	30345	24.8	709000	19	101.17
			15.5							84.96
			21.5							79.06
AT & T	Diaphragm	0.76	27.5	25	8.3	9320.25	26	951080	19	30.33
Tokyo	Diaphragm	0.9	36	30	19.7	21675	27.6	1675320	18.9	120.3

			50							94.2
			58							78.6
			28.5							73.67
			29.6							64.89
Formosa	Diaphragm	0.8	40.57	35	18.5	14986.4	27.6	1177600	18.84	56.56
			44							112.3
			50							106.61
TNEC	Diaphragm	0.9	56	40	19.7	21675	27.6	1675320	18.6	96.4

CHAPTER 5 SIMPLE POLYNOMIAL REGRESSION MODEL

This chapter focuses on the development of a simplified polynomial regression model for estimation of maximum wall deflection on braced excavation in clays as a function of five input parameters: depth of excavation (He), width of excavation (B), thickness of the wall (t), shear strength of soil (Su) and wall embedment depth (D).

5.1 Regression Analysis

5.1.1 Linear Regression

The regression is modeling the linear relationship between a dependent (response) variable and an independent variables (predictors, repressors). The linear regression can be presented in a scatter plot with the dependent and independent variable on the y- and x-axis, respectively. The case of simple linear regression considers a single repressor variable or predictor variable *x* and a dependent or response variable *Y*. Suppose that the true relationship between *Y* and *x* is a straight line and that the observation *Y* at each level of *x* is a random variable. Then, the expected value of *Y* for each value of *x* is:

$$E(Y \mid x) = \beta_0 + \beta_1 x \tag{5-1}$$

Where the intercept β_0 and the slope β_1 are unknown regression coefficients. We assume that each observation, *Y*, can be described by the model

$$Y = \boldsymbol{\beta_0} + \boldsymbol{\beta_1 x_i} + u \tag{5-2}$$

Where *u* is the random error term. This model is called a simple linear regression model because it has only one independent variable or repressor. Sometimes a model like this arises from a theoretical relationship. The magnitude and direction of that relationship are given by the slope parameter (β_1), and the status of the dependent variable when the independent variable is absent is given by the intercept parameter (β_0). An error term (u) captures the amount of variation not predicted by the slope and intercept terms. The correlation coefficient (R^2) shows how well the values fit the data.

To gain more insight into this model, suppose that we can fix the value of *x* and observe the value of the random variable *Y*. Now if *x* is fixed, the random component *u* on the right-hand side of the model in Equation 5-2 determines the properties of *Y*. Suppose that the mean and variance of *u* are 0 and R^2, respectively. Then,

$$E(Y|x) = E(\boldsymbol{\beta_0}+\boldsymbol{\beta_1}x+u) = \boldsymbol{\beta_0}+\boldsymbol{\beta_1}x+E(u) = \boldsymbol{\beta_0}+\boldsymbol{\beta_1}x \quad (5\text{-}3)$$

In most real-world problems, the values of the intercept and slope (β_0,β_1) and the error variance R^2 will not be known and must be estimated from sample data. Then this fitted regression equation or model is typically used in prediction of future observations of *Y*, or for estimating the mean response at a particular level of *x*., the variance of *Y* given *x* is:

$$V(Y|x) = V(\boldsymbol{\beta_0}+\boldsymbol{\beta_1}x+u) = V(\boldsymbol{\beta_0}+\boldsymbol{\beta_1}x) + V(u) = 0 + R^2 = R^2 \quad (5\text{-}4)$$

Thus, the true regression model $\mu_{Y|x} = \beta_0 + \beta_1 x$ is a line of mean values; that is, the height of the regression line at any value of *x* is just the expected value of *Y* for that *x*. The slope, β_1 , can be interpreted as the change in the mean of *Y* for a unit change in *x*. Furthermore, the variability of *Y* at a particular value of *x* is determined by the error variance R^2. This implies that there is a distribution of *Y*-values at each *x* and that the variance of this distribution is the same at each *x*.

In the statistical analysis, scatter plots may be presented with regression lines fit the data and confidence or prediction intervals to verify or reject a hypothesis. The confidence interval contains the parameter of interest with a prespecified probability within an interval. The sample statistics are used as estimators of population parameters and confidence intervals are applied to these estimates. The intervals are determined at a confidence level, such as 95%. This means that if the same population is sampled on several occasions, the confidence interval would cover the true population parameter in approximately 95% of the cases. The confidence interval is defined as:

$$\left(\mu - z\frac{\boldsymbol{\sigma}}{\sqrt{\boldsymbol{N}}}\,,\ \mu + z\frac{\boldsymbol{\sigma}}{\sqrt{\boldsymbol{N}}}\right) \quad (5\text{-}5)$$

Where *z* is corresponding to the confidence level, and e.g. *z*=1.96 for a 95% confidence interval.

While the confidence interval predicts the distribution of estimates of the true population mean, the prediction interval predicts the distribution outcome of individual points in a future sample. The prediction interval is constructed using the observed sample mean and standard deviation (Råde and Westergren, 2004). It is defined as:

$$(\mu - z\boldsymbol{\sigma}\ ,\ \mu - z\boldsymbol{\sigma}\ ) \quad (5\text{-}6)$$

Where *z* is corresponding to the prediction interval range, and e.g. *z*=1.15 for a 75% prediction interval. The prediction interval considers a future sample as a statistic and computes its distribution.

In regression analysis, the coefficient of determination, R^2 *Linear*, is a measurement of how well independent variables explains the variance in the dependent variable. The value of R^2 varies from 0 to 1, where 1 means that all variance in the dependent variable is described from the independent variables, and 0 means that there is no correlation between the variance and the independent variables.

5.1.2 Multiple Regression

Multiple regression refers to regression applications in which there is more than one independent variable. Multiple regression includes a technique called polynomial regression. In polynomial regression, we regress a dependent variable on the powers of the independent variables. The multiple regression analysis is a useful method for generating mathematical models where there are several (more than two) variables involved. A polynomial regression model is consisting of successive power terms. Each model will include the highest order term plus all lower-order terms. We can view polynomial regression as a particular case of multiple linear regression. Polynomial models are an effective and flexible curve fitting technique. The most widely used method of regression analysis is ordinary least squares analysis. This method works by creating a "best fit" line through all of the available data points and parameter estimates are chosen to minimize the error sum of squares.

In order to provide a systematic estimate of the line, statisticians have devised procedures to obtain an estimate of the line that fits the points better than other possible lines. The procedure most commonly used is the least-squares criterion, and the regression line that results from this is called the least-squares regression line.

5.2 Determination of Simple Polynomial Model

Because of the complexity of modeling this highly nonlinear problem involving a large number of variables, the serviceability failure domain was constructed artificially through repeated point-by-point numerical analyses. However, on this section is to develop a simple Regression model, which assists for predicting wall deflection caused by a braced excavation in soft clays. The desired models are developed upon the 87 generated data of

selected case histories which are generated in Chapter 4, and verified using collected case histories and the published results in the literatures as a guide.

The case histories in the first step are used to investigate the relationship between each of the five variables and the maximum wall deflection, and then to provide data for the polynomial regression analysis. Before performing regression analysis, each of the five input variables is transformed so that approximately a relationship trend between the maximum wall deflection and each of these variables is achieved. The form of the transformation function is established through Excel analyses with an objective of maintaining a uniform and simple functional form across all input variables

Based on the results, a Polynomial Regression (PR) model has been developed for estimating the maximum wall deflection d_{hm} as a function of five input parameters: width of excavation (B), height of excavation (He), undrained shear strength (su), embedment depth (D) and wall thickness (t). Data generated for wall deflection by changing input parameters of the original case histories were considered in developing the regression model. To develop the regression model, EXCEL program was used with 95 % confidence level. The input parameters and EXCEL output model coefficients are described in detail in Appendix B.

A total of 87 generated data are then used to establish the regression equation for estimating the maximum wall deflection (δ_{hm}). The developed regression equation is expressed as:

$$\delta_{hm}(mm) = \alpha_o + \alpha_1 D^1 + \alpha_2 D^2 + \alpha_3 D^3 + \alpha_4 H^1 + \alpha_5 H^2 + \alpha_6 H^3 + \alpha_7 B^1 + \alpha_8 B^2 + \alpha_9 B^3 + \alpha_{10} s_u^1 + \alpha_{11} s_u^2 + \alpha_{12} s_u^3 + \alpha_{13} t^1 + \alpha_{14} t^2 + \alpha_{15} t^3 \quad (5\text{-}7)$$

The input variables (He, B, Su, t, and D) are the transformed variables. The coefficients for each variable are obtained through error minimization using the artificial data generated from FEM analyses using Excel program as shown in Table 5-1.

The appropriateness of these regression is reflected in the coefficient of determination (R^2) that was 0.97.

Table 5-1 Coefficients of a Simple Regression Model

Coefficients	Value	Coefficient.	Value	Coefficient.	Value	Coefficient.	Value
α_0	50.703	α_4	29.642	α_8	-0.100	α_{12}	-0.001
α_1	-15.829	α_5	-1.719	α_9	0.001	α_{13}	59.519
α_2	0.694	α_6	0.037	α_{10}	-7.235	α_{14}	-197.161
α_3	-0.006	α_7	4.196	α_{11}	0.177	α_{15}	92.038

5.2.1 Assessment of the Proposed PR Model

The maximum wall deflection computed by the polynomial regression model compared to FEM is generally satisfactory as shown in the table 5-2.

Table 5-2 Comparisons of Wall Deflection by Polynomial Regression Model with FEM.

FEM Δ(mm)	**Equation Δ(mm)**	**Difference Δ(mm)**
18.88	21.66433	2.784328
23.93	26.33307	2.403071
27.94	36.25881	8.318808
93.93	91.91677	-2.01323
114.89	114.1857	-0.70435
152.04	149.5468	-2.49319
39.13	49.90487	10.77487
79.06	74.53665	-4.52335
76.94	64.03049	-12.9095
94.2	88.90142	-5.29858
120	130.3656	10.36565
58.9	59.46086	0.560858
64.89	75.06855	10.17855
71.76	86.46941	14.70941
81.89	80.05292	-1.83708
106.61	105.774	-0.83601
24.93	26.33307	1.403071
22	18.48976	-3.51024
96.34	99.9846	3.644599
94.93	91.91677	-2.98323
92.19	84.32356	-7.86644
117.16	122.1759	5.015858

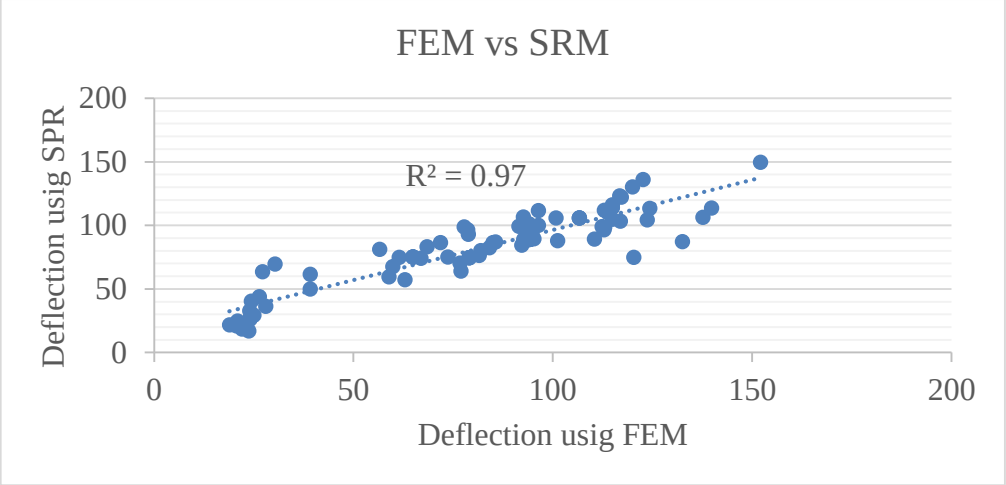

Figure 5-1 Predicted Maximum Wall Deflection dδh Versus dhm by FEM

Figure 5-2 shows the predicted wall deflections (with error bars depicting the upper and lower bounds) versus the measured wall deflection for the cases. The plot indicates that the PR model is able to predict reasonable well (within a factor of 2) the excavation induced wall deflections for the case histories considered.

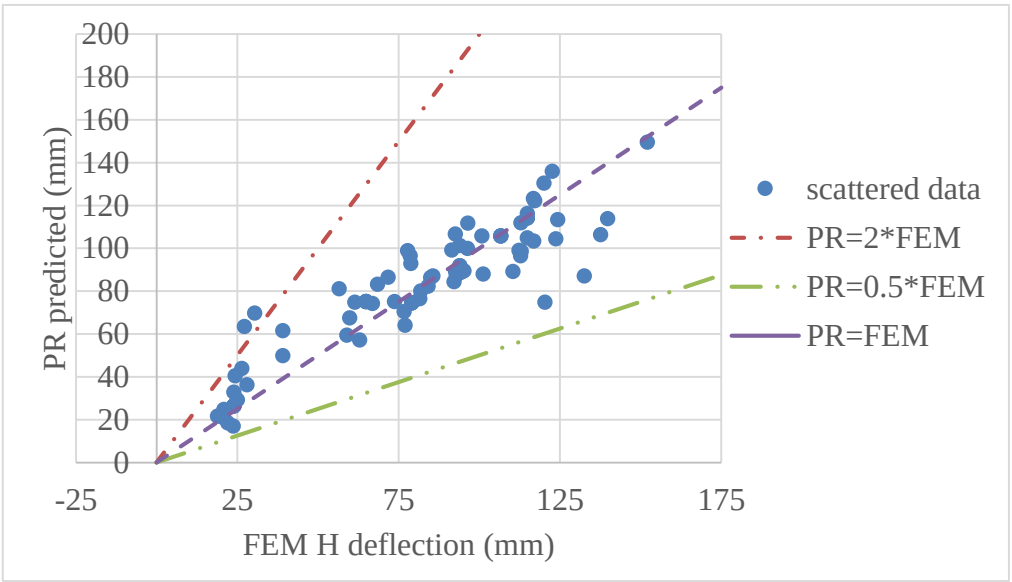

Figure 5-2 Measured δh, FM Versus Predicted δh, PR

5.2.2. Validation of the Proposed Model

To validate the proposed PR model, a total of 10 well-documented excavation case histories from various countries listed in Table 5-3 were analyzed which are not used in development of intended model and some are case studies of inclinometer data. A comparison was done of the predicted and observed maximum wall deflection excavation case histories.

The predicted (by simple polynomial regression model) wall deflections versus the observed (measured) wall deflection for the cases as shown in Table 5-3. The result shows that the wall deflection estimated by the proposed model is closer to observed wall deflection. The performance of the developed model in predicting the maximum wall deflection is considered agreeable.

Table 5-3 Data of Five Case Histories Used for Validating the Proposed Model.

Cases	H_e(m)	B(m)	Su (kPa)	t(m)	D(m)	Observed Δ (mm)	Predicted Δ (mm)	Reference
Sukhumkit station	21	23	92.33	1	6.9	38	40.09	Bangkok MRT Sukhumkit station
IFCT building	11.3	40	82.9	0.8	6.9	60	69.85	Deformation of diaphragm wall deep excavation Bangkok subsoil IFCT building.
Lavender	15.7	24	31.5	1	12.3	32	50.23	ctcgoh@ntu.edu.sg
Roose wood Hotel	24.2	60	107	1	3.8	43.34	43.8	(http://creativecommons.org/licenses/by-nc-nd/4.0/).
One market plaza in sanifrancisico	11	11	25	0.4	21	101.6	82.57	Clough and Denby (1997)
Post office Garage	20.2	61	91	0.9	5.4	53.61	62.2	Whittle et al, 1993
Formosa	18.5	33	29.6	0.8	12.5	62	75.13	Case Study
Hasin-Yi	20.05	68	88.3	1	20.9	58.38	61.61	Louis Ge, 2017
TNEC	19.7	41	68.2	0.9	15.3	106.4	92.97	Case Study
Far east enterprise	20	70	76.5	0.7	12	124	106.82	Case Study

5.2.3 Model Bias of the Proposed Model

The proposed model consists of five component models, for estimation of the maximum wall deflection. The entire model is referred as the simple polynomial regression model (SPR).

For practical application, it should be of interest to estimate model uncertainty or model bias of the developed SRM. The model bias if often expressed as a bias factor (BF):

$$\text{BF}= \frac{observed\ value}{estimated\ value}$$

The bias factor BF is often assumed to follow normal or lognormal distribution (Phoon and Kulhawy 2005; Juang et al. 2006). The closer the mean of BF is to 1.0, the more accurate a model is, and the smaller the variation of BF is, the more precise a model is.

Table 5-4 Model Bias Factor (from Table 5-3)

Cases	Observed Δ (mm)	Predicted Δ (mm)	Bias factor (BF)=Observed/Predicted
1	38	40.09	0.86
2	60	69.85	0.83
3	32	50.23	0.94
4	43.34	43.8	1.14
5	101.6	82.57	1.16
6	53.61	62.2	0.94
7	62	75.13	0.86
8	58.38	61.61	0.63
9	106.4	92.97	0.98
10	124	106.82	1.20

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This thesis presents a polynomial regression model for estimating the maximum wall deflection for braced excavation on clay soil which takes 2D based problems. And the analysis of major parameters that contribute to the wall deflection. The excavation geometries, wall embedment depth, soil strength and, and the wall stiffness were varied to study the wall deflection behavior. Based on the results, a simple polynomial regression (PR) model was developed for estimating the maximum wall deflection, PR as a function of five input parameters: excavation width (B), excavation depth (He), wall embedment depth D, soil shear strength (Su), and wall thickness (t).

The results of FEM analyses of the well-documented case histories and observed maximum wall deflection by PR model agreed well.

The predicted maximum wall deflection by PR model is well agreed with observed maximum wall deflection from different case histories which are not used in development of model.

The following conclusions are drawn from the results of the study on the compilation of excavation case histories, generation of maximum wall deflection data and on development of simplified models for wall deflection.

- In this thesis study, eight (8) case histories of braced excavations in clays are obtained from Taipei, Singapore, Oslo, Tokyo, and Chicago. However, the total number of collected case histories is less than ideal for the purpose of developing simple models for estimating the wall deflection. Thus, in this study, secondary data were generated from numerical experimentation using FEM solutions with the Mohr-Coulomb soil model.
- The proposed simple polynomial regression model is able to predict reasonably well the maximum wall deflection, caused by braced excavations in clays. It has improved upon the important contributions of earlier investigators.
- The model bias of the simple polynomial regression model is assessed at the component level as well as the entire model as a whole. The simple polynomial

regression model is judged to be accurate (with all mean bias factors approximately equal to 1.0).

- The proposed simple polynomial regression model assumes normal workmanship and no basal failure in the braced excavation. Possible uncertainty caused by the over excavation prior to support installation, are not explicitly addressed in this paper.
- Computed values agreed well with results observed value from different case records.

5.2 Recommendation for Further Studies

All the analysis performed in this study was assumed that there is no effect of wall installation. Thus, to simulate an actual condition at the site and to get a more accurate result in terms of ground movement the wall installation effect needs to be considered.

Besides that, all the analyses in this study also were performed by using Plaxis2D. For further study, because arching effect of wall installation can be simulate with the use Plaxis3D. Thus, the applicability of this approach is limited to the use of the SPR model in case of the soil types like sand.

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APPENDIX A

Diaphragm Wall and Struts Input Parameters.

Sample calculation of Diaphragm Wall and Struts Input Parameters

I). **Diaphragm wall**

- Compressive strength of concrete (f_c) = 28MPa
- Young's Modulus (E)=$4700\sqrt{f\,c}$ (MPa) = $4700\sqrt{28}$=$24.8*10^6$kPa
- Thickness of diaphragm wall (D) = 0.7m
- Axial Stiffness (EA) = $0.9*24.8*10^6$ =$17.36*10^6$ kN/m
- Flexural Stiffness (EI) = $24.8*10^6$kPa*(bh^3/12)
  = $24.8*10^6$kPa*($1*(0.9)^3$/12)
  =$0.708*10^6$ kN m^2/m

Weight of plate element to simulate wall in soil using PLAXIS software:

a) W_{real}= $\gamma_{concrete}*d_{real}$
   W_{model}= $\gamma_{soil}*d_{real}+W_{plate}$

   Then, W_{real} = W_{model}

   W_{plate}= $(\gamma_{concrete}-\gamma_{soil})*d_{real}$

b) Wall with excavated soil
   W_{model}= $\gamma_{soil}*\frac{1}{2}d_{real}+W_{plate}$

   Then, W_{real} = W_{model}

   W_{plate}= $(\gamma_{concrete}-\frac{1}{2}\gamma_{soil})*d_{real}$

   By considering wall with in soil Mass

   W_{plate}= $(\gamma_{concrete}-\gamma_{soil})*d_{real}$

   = (25-19.5)*0.9m = 4.95kN/m/m

II). **Struts**

- Cross-sectional area, A = 0.6m^2
- $E_{concert}$ = 24.8GPa
- EA= 24.8*10^6 kPa*0.6m^2 = 14.88*10^6 kN

APPENDIX B

Excel regression program output.

Table B-6-1: Excel regression program output.

SUMMARY OUTPUT	
Regression Statistics	
Multiple R	0.903482
R Square	0.81628
Adjusted R Square	0.777466
Standard Error	16.15245
Observations	87

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 99.0%*	*Upper 99.0%*
Intercept	**50.70269**	391.7582	0.129423	0.897389	-730.4409934	831.8464	-986.228	1087.633
D	**-15.8288**	6.847262	-2.3117	0.0237	-29.48184171	-2.17574	-33.9526	2.294979
D2	**0.693747**	0.500842	1.38516	0.17034	-0.30490438	1.692398	-0.63191	2.019408
D3	**-0.00561**	0.011367	-0.49358	0.623127	-0.028276417	0.017055	-0.0357	0.024477
He	**29.6423**	16.86518	1.757604	0.083125	-3.985899853	63.27051	-14.9975	74.28212
He2	**-1.71943**	1.03291	-1.66464	0.100391	-3.778990156	0.340137	-4.4534	1.014544
He3	**0.036557**	0.020342	1.797126	0.076568	-0.004003613	0.077117	-0.01729	0.090399
B	**4.196018**	6.257366	0.670572	0.504668	-8.280816229	16.67285	-12.3664	20.75841
B2	**-0.10018**	0.150456	-0.66582	0.507681	-0.400179164	0.199824	-0.49842	0.29806
B3	**0.000744**	0.001122	0.663515	0.50915	-0.001492149	0.00298	-0.00222	0.003713
Su	**-7.23517**	2.720313	-2.65968	0.009661	-12.65931924	-1.81102	-14.4355	-0.03487
Su2	**0.177478**	0.048329	3.672312	0.000462	0.081113391	0.273843	0.049559	0.305398
Su3	**-0.00125**	0.000271	-4.63556	1.58E-05	-0.001793838	-0.00071	-0.00197	-0.00054
t	**59.51909**	1308.408	0.04549	0.963845	-2549.372186	2668.41	-3403.66	3522.696
t2	**-197.161**	1489.219	-0.13239	0.895048	-3166.579707	2772.257	-4138.92	3744.598
t3	**92.03826**	555.8167	0.165591	0.868949	-1016.228795	1200.305	-1379.13	1563.209

ANOVA					
	Df	*SS*	*MS*	*F*	*Significance F*
Regression	15	82303.22	5486.881	21.03047	2.55173E-20
Residual	71	18524.01	260.9015		
Total	86	100827.2			

Table B-6-2: Input data's to Excel

No.	D	D2	D3	He	He2	He3	B	B2	B3	Su	Su2	Su3	t	t2	t3	parameter to vary
1.0	5.2	27.0	140.6	17.5	307.3	5387.0	22.6	510.8	11543.2	94.8	8987.0	851971.4	1.0	1.0	1.0	
2.0	5.2	27.0	140.6	18.9	357.2	6751.3	22.6	510.8	11543.2	94.8	8987.0	851971.4	1.0	1.0	1.0	
3.0	5.2	27.0	140.6	21.0	441.0	9261.0	22.6	510.8	11543.2	94.8	8987.0	851971.4	1.0	1.0	1.0	
4.0	12.5	156.3	1953.1	10.0	100.0	1000.0	70.0	4900.0	343000.0	76.5	5852.3	447697.1	0.8	0.6	0.5	
5.0	12.5	156.3	1953.1	18.5	342.3	6331.6	70.0	4900.0	343000.0	76.5	5852.3	447697.1	0.8	0.6	0.5	
7.0	12.0	144.0	1728.0	15.4	237.2	3652.3	33.0	1089.0	35937.0	70.0	4900.0	343000.0	0.7	0.5	0.3	He
8.0	12.0	144.0	1728.0	20.0	400.0	8000.0	33.0	1089.0	35937.0	70.0	4900.0	343000.0	0.7	0.5	0.3	
9.0	12.0	144.0	1728.0	24.6	605.2	14886	33.0	1089.0	35937.0	70.0	4900.0	343000.0	0.7	0.5	0.3	
10.0	5.2	27.0	140.6	6.0	36.0	216.0	25.0	625.0	15625.0	21.5	462.3	9938.4	0.8	0.6	0.4	
11.0	5.2	27.0	140.6	8.3	68.9	571.8	25.0	625.0	15625.0	21.5	462.3	9938.4	0.8	0.6	0.4	
13.0	15.3	234.1	3581.6	12.0	144.0	1728.0	30.0	900.0	27000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	

14.0	15.3	234.1	3581.6	19.7	388.1	7645.4	30.0	900.0	27000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
15.0	15.3	234.1	3581.6	25.0	625.0	15625.0	30.0	900.0	27000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
16.0	12.5	156.3	1953.1	13.2	174.2	2300.0	35.0	1225.0	42875.0	29.6	876.2	25934.3	0.8	0.6	0.5	
17.0	12.5	156.3	1953.1	18.5	342.3	6331.6	35.0	1225.0	42875.0	29.6	876.2	25934.3	0.8	0.6	0.5	
18.0	12.5	156.3	1953.1	21.0	441.0	9261.0	35.0	1225.0	42875.0	29.6	876.2	25934.3	0.8	0.6	0.5	
19.0	20.3	412.1	8365.4	11.8	139.2	1643.0	40.0	1600.0	64000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
20.0	20.3	412.1	8365.4	19.7	388.1	7645.4	40.0	1600.0	64000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
21.0	20.3	412.1	8365.4	24.0	576.0	13824.	40.0	1600.0	64000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
22.0	5.2	27.0	140.6	18.9	357.2	6751.3	22.6	510.8	11543.2	94.8	8987.0	851971.4	0.8	0.6	0.5	t(m)
23.0	5.2	27.0	140.6	18.9	357.2	6751.3	22.6	510.8	11543.2	94.8	8987.0	851971.4	1.0	1.0	1.0	
24.0	5.2	27.0	140.6	18.9	357.2	6751.3	22.6	510.8	11543.2	94.8	8987.0	851971.4	1.2	1.4	1.7	
25.0	12.5	156.3	1953.1	18.5	342.3	6331.6	70.0	4900.0	343000.0	76.5	5852.3	447697.1	0.7	0.5	0.3	
26.0	12.5	156.3	1953.1	18.5	342.3	6331.6	70.0	4900.0	343000.0	76.5	5852.3	447697.1	0.8	0.6	0.5	
27.0	12.5	156.3	1953.1	18.5	342.3	6331.6	70.0	4900.0	343000.0	76.5	5852.3	447697.1	0.9	0.8	0.7	
28.0	12.0	144.0	1728.0	20.0	400.0	8000.0	33.0	1089.0	35937.0	70.0	4900.0	343000.0	0.6	0.4	0.2	
29.0	12.0	144.0	1728.0	20.0	400.0	8000.0	33.0	1089.0	35937.0	70.0	4900.0	343000.0	0.7	0.5	0.3	
31.0	5.2	27.0	140.6	8.3	68.9	571.8	25.0	625.0	15625.0	21.5	462.3	9938.4	0.6	0.4	0.2	
32.0	5.2	27.0	140.6	8.3	68.9	571.8	25.0	625.0	15625.0	21.5	462.3	9938.4	0.8	0.6	0.4	
33.0	5.2	27.0	140.6	8.3	68.9	571.8	25.0	625.0	15625.0	21.5	462.3	9938.4	1.0	1.0	1.0	
35.0	15.3	234.1	3581.6	19.7	388.1	7645.4	30.0	900.0	27000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
36.0	15.3	234.1	3581.6	19.7	388.1	7645.4	30.0	900.0	27000.0	50.0	2500.0	125000.0	1.0	1.0	1.0	
37.0	12.5	156.3	1953.1	18.5	342.3	6331.6	35.0	1225.0	42875.0	29.6	876.2	25934.3	0.7	0.5	0.3	
38.0	12.5	156.3	1953.1	18.5	342.3	6331.6	35.0	1225.0	42875.0	29.6	876.2	25934.3	0.8	0.6	0.5	
39.0	12.5	156.3	1953.1	18.5	342.3	6331.6	35.0	1225.0	42875.0	29.6	876.2	25934.3	0.9	0.8	0.7	

40.0	20.3	412.1	8365.4	19.7	388.1	7645.4	40.0	1600.0	64000.0	50.0	2500.0	125000.0	0.8	0.6	0.5	
41.0	20.3	412.1	8365.4	19.7	388.1	7645.4	40.0	1600.0	64000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
42.0	20.3	412.1	8365.4	19.7	388.1	7645.4	40.0	1600.0	64000.0	50.0	2500.0	125000.0	1.0	1.0	1.0	
43.0	5.2	27.0	140.6	18.9	357.2	6751.3	17.6	309.8	5451.8	94.8	8987.0	851971.4	1.0	1.0	1.0	
44.0	5.2	27.0	140.6	18.9	357.2	6751.3	22.6	510.8	11543.2	94.8	8987.0	851971.4	1.0	1.0	1.0	
45.0	5.2	27.0	140.6	18.9	357.2	6751.3	27.6	761.8	21024.6	94.8	8987.0	851971.4	1.0	1.0	1.0	
46.0	12.5	156.3	1953.1	18.5	342.3	6331.6	28.0	784.0	21952.0	76.5	5852.3	447697.1	0.8	0.6	0.5	
47.0	12.5	156.3	1953.1	18.5	342.3	6331.6	33.0	1089.0	35937.0	76.5	5852.3	447697.1	0.8	0.6	0.5	
48.0	12.5	156.3	1953.1	18.5	342.3	6331.6	38.0	1444.0	54872.0	76.5	5852.3	447697.1	0.8	0.6	0.5	
49.0	12.0	144.0	1728.0	20.0	400.0	8000.0	65.0	4225.0	274625.0	70.0	4900.0	343000.0	0.7	0.5	0.3	
50.0	12.0	144.0	1728.0	20.0	400.0	8000.0	70.0	4900.0	343000.0	70.0	4900.0	343000.0	0.7	0.5	0.3	
51.0	12.0	144.0	1728.0	20.0	400.0	8000.0	75.0	5625.0	421875.0	70.0	4900.0	343000.0	0.7	0.5	0.3	
52.0	5.2	27.0	140.6	8.3	68.9	571.8	20.0	400.0	8000.0	21.5	462.3	9938.4	0.8	0.6	0.4	
53.0	5.2	27.0	140.6	8.3	68.9	571.8	25.0	625.0	15625.0	21.5	462.3	9938.4	0.8	0.6	0.4	
54.0	5.2	27.0	140.6	8.3	68.9	571.8	30.0	900.0	27000.0	21.5	462.3	9938.4	0.8	0.6	0.4	
55.0	15.3	234.1	3581.6	19.7	388.1	7645.4	25.0	625.0	15625.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
56.0	15.3	234.1	3581.6	19.7	388.1	7645.4	30.0	900.0	27000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
58.0	12.5	156.3	1953.1	18.5	342.3	6331.6	30.0	900.0	27000.0	29.6	876.2	25934.3	0.8	0.6	0.5	
59.0	12.5	156.3	1953.1	18.5	342.3	6331.6	35.0	1225.0	42875.0	29.6	876.2	25934.3	0.8	0.6	0.5	
60.0	12.5	156.3	1953.1	18.5	342.3	6331.6	40.0	1600.0	64000.0	29.6	876.2	25934.3	0.8	0.6	0.5	
61.0	20.3	412.1	8365.4	19.7	388.1	7645.4	35.0	1225.0	42875.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
62.0	20.3	412.1	8365.4	19.7	388.1	7645.4	40.0	1600.0	64000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
63.0	20.3	412.1	8365.4	19.7	388.1	7645.4	45.0	2025.0	91125.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
64.0	5.2	27.0	140.6	18.9	357.2	6751.3	22.6	510.8	11543.2	89.0	7921.0	704969.0	1.0	1.0	1.0	Su

B(m)

65.0	5.2	27.0	140.6	18.9	357.2	6751.3	22.6	510.8	11543.2	94.8	8987.0	851971.4	1.0	1.0	1.0	
66.0	5.2	27.0	140.6	18.9	357.2	6751.3	22.6	510.8	11543.2	100.8	10160.6	1024192.5	1.0	1.0	1.0	
67.0	12.5	156.3	1953.1	18.5	342.3	6331.6	70.0	4900.0	343000.0	64.0	4096.0	262144.0	0.8	0.6	0.5	
68.0	12.5	156.3	1953.1	18.5	342.3	6331.6	70.0	4900.0	343000.0	70.0	4900.0	343000.0	0.8	0.6	0.5	
69.0	12.5	156.3	1953.1	18.5	342.3	6331.6	70.0	4900.0	343000.0	76.0	5776.0	438976.0	0.8	0.6	0.5	
71.0	12.0	144.0	1728.0	20.0	400.0	8000.0	33.0	1089.0	35937.0	76.5	5852.3	447697.1	0.7	0.5	0.3	
72.0	12.0	144.0	1728.0	20.0	400.0	8000.0	33.0	1089.0	35937.0	82.5	6806.3	561515.6	0.7	0.5	0.3	
73.0	5.2	27.0	140.6	8.3	68.9	571.8	25.0	625.0	15625.0	15.5	240.3	3723.9	0.8	0.6	0.4	
74.0	5.2	27.0	140.6	8.3	68.9	571.8	25.0	625.0	15625.0	21.5	462.3	9938.4	0.8	0.6	0.4	
75.0	5.2	27.0	140.6	8.3	68.9	571.8	25.0	625.0	15625.0	27.5	756.3	20796.9	0.8	0.6	0.4	
76.0	15.3	234.1	3581.6	19.7	388.1	7645.4	30.0	900.0	27000.0	36.0	1296.0	46656.0	0.9	0.8	0.7	
77.0	15.3	234.1	3581.6	19.7	388.1	7645.4	30.0	900.0	27000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
78.0	15.3	234.1	3581.6	19.7	388.1	7645.4	30.0	900.0	27000.0	58.0	3364.0	195112.0	0.9	0.8	0.7	
79.0	12.5	156.3	1953.1	18.5	342.3	6331.6	35.0	1225.0	42875.0	28.5	812.3	23149.1	0.8	0.6	0.5	
80.0	12.5	156.3	1953.1	18.5	342.3	6331.6	35.0	1225.0	42875.0	29.6	876.2	25934.3	0.8	0.6	0.5	
81.0	12.5	156.3	1953.1	18.5	342.3	6331.6	35.0	1225.0	42875.0	40.6	1645.9	66775.2	0.8	0.6	0.5	
82.0	20.3	412.1	8365.4	19.7	388.1	7645.4	40.0	1600.0	64000.0	44.0	1936.0	85184.0	0.9	0.8	0.7	
83.0	20.3	412.1	8365.4	19.7	388.1	7645.4	40.0	1600.0	64000.0	50.0	2500.0	125000.0	0.9	0.8	0.7	
84.0	20.3	412.1	8365.4	19.7	388.1	7645.4	40.0	1600.0	64000.0	56.0	3136.0	175616.0	0.9	0.8	0.7	

85.0	20.0	400.0	8000.0	18.9	357.2	6751.3	22.6	510.8	11543.2	89.0	7921.0	704969.0	1.0	1.0	1.0	D
86.0	24.0	576.0	13824.0	18.9	357.2	6751.3	22.6	510.8	11543.2	94.8	8987.0	851971.4	1.0	1.0	1.0	
87.0	28.0	784.0	21952.0	18.9	357.2	6751.3	22.6	510.8	11543.2	100.8	10160.6	1024192.5	1.0	1.0	1.0	