Design of Support System for deep Stratified Excavation

Final Year Project UG-2018

BACHELOR OF

CIVIL ENGINEERING

NUST Institute of Civil Engineering

School of Civil and Environmental Engineering

National University of Science and Technology Islamabad, Pakistan

This is to certify that the final year project title

Design of Support System for deep Stratified Excavation

Submitted by

KHUZAIFA JAVED (Group Leader)	252371
FURHANA AKBAR	248367
SHAHZAIB	241709
MUHAMMAD ABDULLAH SULEMANI	241715

has been accepted towards the requirements

for the undergraduate degree

in

CIVIL ENGINEERING

Dr.Syed Muhammad Syed Jamil

NUST Institute of Civil Engineering

School of Civil and Environmental Engineering

National University of Sciences and Technology, Islamabad, Pakistan

TABLE OF CONTENTS

Chapter 1 10

INTRODUCTION 10

1.1 BACKGROUND 10

1.2 PROBLEM STATEMENT 111

1.3 AIMS & OBJECTIVES 11

1.4 ORGANIZATION OF THE THESIS ....…11

Chapter 2 13

LITERATURE REVIEW 13

2.1 GENERAL...13

2.2 LATERAL EARTH PRESSURES 13

2.2.1 RANKINE EARTH PRESSURE THEROY 14

2.2.2 PECK’S (1969) APPARENT EARTH PRESSURE DIAGRAMS 14

2.3 GENERAL DEFLECTION BEHAVIOR OF AN EXCAVATION SUPPORT 16

2.3.1 PECK(1969) 14

2.3.2 CLOUGH AND O’ROURKE (1990)..14

2.3.3 S.J BOONE (2003) 14

2.3.4 BRYSON (2012) 14

2.3.5 HAHASH (1996)...14

2.3.6 KUNG (2007) 20

Chapter 3 21

METHODOLOGY 21

3.1 INTRODUCTION 21

3.2 CALIBRATION 21

3.3 FINITE ELEMENT MODELLING WITH PLAXIS 2D 21

3.3.1 INPUT 22

3.3.1.1 GEOMETRY 22

3.3.1.2 MATERIAL PARAMETERS USED IN PLAXIS......................................22

3.3.1.3 MESH 22

3.3.1.4 CALCULATIONS...22

3.3.2 OUTPUT 22

3.4 VISUAL BASIC APPLICATION...24

3.5. CALIBRATION OF PARAMETERS FOR MC MODEL AND THE MODULI USED IN REFERENCED EQUATIONS:...

Chapter 4 25

OBSERVATIONS...25

4.1 STIFF CLAYS: 25
4.2. MEDIUM CLAYS 27
4.3. SOFT CLAYS: 30
CHAPTER 5 33
DISCUSSIONS AND CONCLUSION 33
5.1. SOIL MODELS 33
5.2. VARIATIONS OF RESULTS 33
5.3. ON THE WANG & REESE EQUATIONS IN CIRCULAR NO. 4 33
5.4 CONCLUSION 34
REFERENCES 36

TABLE OF FIGURES

Figure 2. 1 (a) “Rankine’s Earth Pressure Distributions; and (b) Passive and Active Zones” 14

Figure 2. 2 Peck’s (1969) “Apparent Pressure Envelopes: (a) Cuts in Sand; (b) Cuts in Soft toMedium Clay; and (c) Cuts in Stiff Clay (After Peck, 1969) 14

Figure 2. 3 Layered Soil in Excavations: (a) Sand and Clay; and (b) Multilayered Clay 16

Figure 2. 4 Peck (1969) Settlement Profiles 17

Figure 2. 5 Wall Movement in Braced Excavations 17

Figure 2. 6 Relationship between horizontal and vertical excavation induced movements 18

Figure 2. 7 Estimation of Maximum Lateral Wall Deflections from Numerical Experiments ... 19

Figure 2. 8 Performance of the proposed KJHH model for predicting maximum wall deflection 20

DEDICATION

We would like to dedicate our works to our parents, our teachers our institution NUST and all our friends. We executed our work with the impressive assurance and determination and applied best of ourselves to the errand at hand.

DECLARATION

It is hereby reverently and truthfully declared that all the work alluded to this thesis is composed by us and it has not been submitted by any institution, in whole or in part in any previous application for a degree. Any references to the work done by any other person or University have been appropriately cited.

ACKNOWLEDGEMENTS

In the name of Allah, the most Beneficent, the most Merciful as well as peace and blessings be upon Prophet Muhammad, His servant and final messenger.

We are thankful to Allah almighty for bestowing us an opportunity to be here in a prestigious institute and intellectual strength with continuous guidance to work up to the mark.

This work would not have been possible without support of generous faculty of NUST Institute of Civil Engineering. We are especially indebted to Dr. Muhammad Jamil, who provided is with his unending guidance and motivation as supervisor of the project.

Most importantly, we are grateful to our families for unconditional, unequivocal, and loving support.

We are thankful to our department and school for providing us with a leaning environment which helped us in achieving we dreamt of.

ABSTRACT

Deep excavations are becoming common in urban environments, be it for the construction of multi-story buildings or tunnels or underground subway networks etc. These excavations result in lateral movements of retaining structure and consequently lead to vertical settlement in the retained soil mass which in turn, leads to settlement in the foundation of any neighboring structure. With the ever-growing demand for structures mentioned previously in developing nations as well where a lack of adequate field data presents a challenge in estimating these settlements it is imperative that a method be devised or adequate modifications to existing equations/ methods be made, such that suitable yet conservative estimates of loads, deflections and settlements can be made to aid in the design of an adequate support structure that can mitigate the threat of any potential loss or damage to life or property. The design despite being conservative should also be such that it does not compromise the economy of the project to an extent where it is deemed unfeasible.

.

Chapter 1

INTRODUCTION

While carrying out excavation for the construction of Structural foundation or any other utility, provision of lateral support to the excavated open face is mandatory otherwise lateral movement of the soil might result is vertical settlement or tension cracking of soil adjacent to the excavated part, which eventually can damage adjacent structures or any other facility near excavation. These settlements can we observed in influence area which is as far as four times the depth of excavation in length from the point of excavation and these settlements are more severe in an area called Primary influence zone which is about two times the depth of the excavation in length from the point of excavation. If these lateral movements are not mitigated timely, collapse of adjacent structures or other facilities becomes imminent. Therefore, it’s very necessary to use properly designed excavation support system to ensure safety of excavated face and adjacent property. Safety of the workers during the excavation is also a very important aspect as there has been number of incidents which resulted in causality of the workers due to the collapse.

Lateral support systems that are usually provided in Pakistan to restrict lateral movements include Tangent pile walls, secant pile walls, diaphragm walls, Soldiers piles coupled with anchor systems etc. Selection and installation of these support systems depends upon the location of the project, type of the soil, construction techniques being implemented, economic considerations, contractors’ expertise etc. These support systems can either be temporary or permanent in nature and sometimes temporary support system is followed by construction of permanent support system. This research has primarily focused upon anchored support systems.

1.1 BACKGROUND

Anchored walls provide a sufficient lateral support to soil movements. But these anchor support systems need to be designed efficiently for maximum effectiveness and ensuring economy of the project at the same time. Anchor systems are designed based on loads according to Circular 4 without factoring in the deflection and deformation of the wall.

Design based on load is employed for homogenous, stratified and Ci-phi soils. Design of anchors begin with determination Earth pressures. Determination of Earth pressures differ in each type of soil. For homogenous strata, apparent earth pressure diagrams proposed by Terzaghi, and Peck are used to determine acting earth pressures. For stratified soil, Peck (1943) and Terzaghi, Peck & Mesri (1996) have proposed equation for converting stratified soil strata into equivalent homogenous strata. Earth pressures in Ci-phi soils could be determined using apparent earth pressure diagrams proposed by Alavinezhad (2020).

Once the earth pressures acting over soil due to excavation are determined, loads are calculated that will act upon the supporting wall. Based on analysis of these loads, number of anchors, bonded length, unbonded length, number of strands in anchor and pile design is evaluated.

Design of anchor support systems based on deflection and deformation of installed wall yields better results as its more helpful in determination of vertical and horizontal spacing in-between the anchors and the pile diameter.

1.2 PROBLEM STATEMENT

Determination of deflections and deformations in supporting anchored wall could be done using empirical equations suggested in multiple research papers. Aim of our research is to identify the most suitable equation from the number of given equations to be applied for conditions in Pakistan and modify the identified equations to give acceptably conservative but economical results.

1.3 AIMS & OBJECTIVES

Empirical equations suggested for determination of deflections and deformations in previous research consist of multiple parameters such as soil Moduli. For modification in existing equations to be suitable for use in Pakistani soil conditions, these equation parameters should be determined. These parameters could either be determined using field measurements or by using co-relations suggested in literature. Using field measurements for finding these parameters is a difficult task in Pakistan due to non-availability of sophisticated equipment and skilled manpower. We opted for using co-relations to determine these parameters for empirical equations. The problem with using co-relation is that application of these co-relations is confined in broad range of values, due to which too much conservative results were given by the empirical equations. One of our aims was to shorten these ranges without compromising on conservativeness and economy of the results.

1.4 ORGANIZATION OF THE THESIS

This thesis has been organized in five (5) chapters. A brief description of every chapter is given below:

First chapter is the introductory chapter that familiarizes the reader with the basic knowledge of design excavation support systems and their importance and states the aim and objectives of this thesis.

Second chapter provides detailed literature review of design of support system for deep stratified excavation.

Second chapter is about the methodology adopted in this project for achieving the objectives. It focuses on the selection of methods used and the modelling techniques.

Forth chapter describes the findings of the study, descriptive and mathematical analysis performed on the generated results.

Fifth chapter illustrated the major outcomes of the project, recommendations and suggestions in accordance with the obtained data.

Chapter 2

LITERATURE REVIEW

2.1 GENERAL

The behavior of deep excavations is a complex phenomenon that is influenced by many factors like soil parameters, structural properties and the insitu stress state. Many methods are available in literature that can be used to predict behavior of excavation and design for different support systems. The various aspects are; the use of theoretical and empirical based correlations and methods, insitu and laboratory testing, use of elaborate field measurements during construction stages and the use of more sophisticated numerical based solutions.

2.2 Lateral Earth Pressures

The determination of lateral earth pressures is the first and important step towards deign of excavation support system. Different theories have been proposed by different researchers to calculate lateral earth pressures that involves the determination of lateral earth pressure coefficient. Few theories have been discussed briefly.

2.2.1 Rankine Earth Pressure Theory

Rankine (1857) developed his classical theory to calculate lateral earth pressures in 1857, called the Rankine Earth Pressure Theory.

For a soil exhibiting both effective cohesion intercept, **c'**, and effective angle of internal friction, the Rankine earth pressures are given by:

Active case:

$$\boldsymbol{\sigma'_a = \sigma'_v K_a - 2c' \sqrt{K_a}}$$

where: $K_a = \tan^2(45° - \varphi/2)$

Passive case:

$$\boldsymbol{\sigma'_p = \sigma'_v K_p + 2c' \sqrt{K_p}}$$

where: $K_p = \tan^2(45° + \varphi/2)$

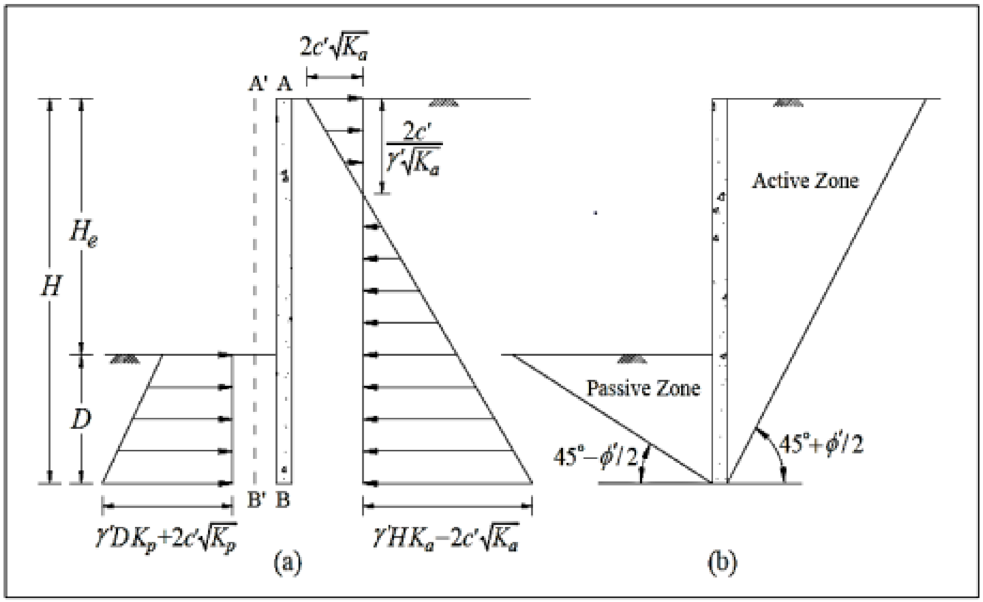


figure 2. 1 (a) “Rankine’s Earth Pressure Distributions; and (b) Passive and Active Zones”

2.2.2. Peck’s (1969) Apparent Earth Pressure Diagrams

The earth pressures diagrams for sands, soft to medium clays and stiff clays are presented in Figure 2.2. Soft to medium stiff conditions are applicable when the stability number Nb = $\gamma H_e/s_u$ > 6; and when Nb ≤ 4 the stiff clay conditions apply.

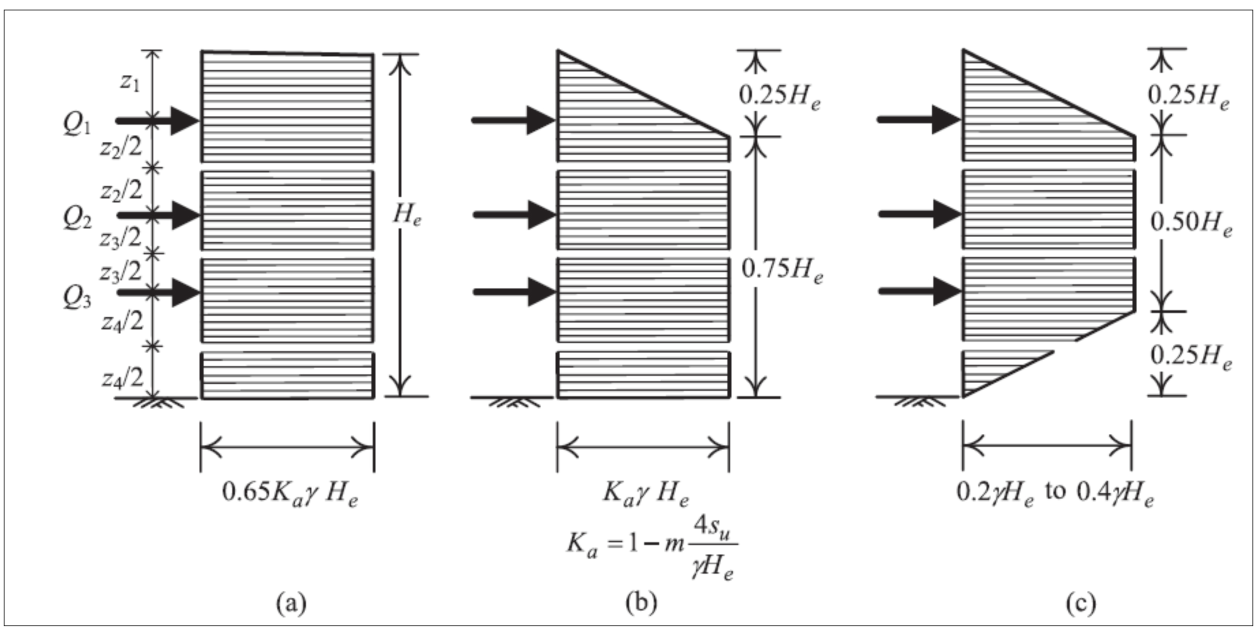


Figure 2. 2 Peck’s (1969) “Apparent Pressure Envelopes: (a) Cuts in Sand; (b) Cuts in Soft to Medium Clay; and (c) Cuts in Stiff Clay (After Peck, 1969)

In case of layered soil profile, it is important to know which soil parameters are to be used in design of support system. In that case either to use the properties of soil layer that is dominant within the depth of excavation or to determine equivalent undrained shear strength, $s_{u,eq}$ and unit weight, γeq parameters. Peck (1943) gave the following equations to determine the average parameters to be used in pressure envelops. For two alternating layers of sand and clay as shown in Figure $s_{u,eq}$ and unit weight, γ_{eq} can be calculated as

$$s_{u,eq} = \frac{1}{H_e}\left[(K_s \gamma_s H_s \tan\delta) H_s / 2 + H_c s_u\right]$$
$$= \frac{1}{2H_e}\left[\gamma_s K_s {H_s}^2 \tan\delta + 2H_c s_u\right]$$

$$\gamma_{eq} = \frac{1}{H_e}\left[\gamma_s H_s + \gamma_c H_c\right]$$

γ_c and γ_s = unit weight of clay and sand respectively
H_s and H_c = thickness of sand clay respectively
δ = angle of friction of sand
Ks = coefficient of lateral earth pressure. For simplicity, Ks can be assumed to be equal to Rankine's Ka
H_e = excavation depth,
Similarly, for layered clay strata average undrained shear strength $s_{u,eq}$ and average unit weight, γeq can be calculated as;

$$s_{u,eq} = \frac{1}{H_e}\left(s_{u1} H_1 + s_{u2} H_2\right)$$

$$\gamma_{eq} = \frac{1}{H_e}\left[\gamma_1 H_1 + \gamma_2 H_2\right]$$

$S_{u.1,2}$ = undrained shear strength of first and second layer respectively
$H_{1,2}$ = height of first and second layer respectively
$\gamma_{1,2}$ = unit weight of first and second layer respectively

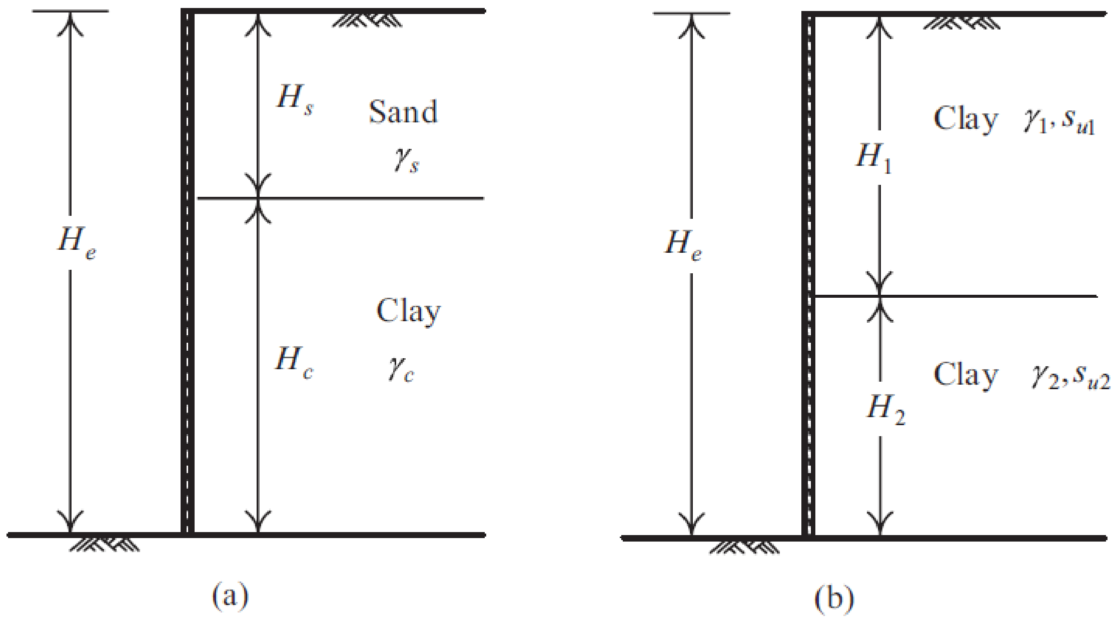


Figure 2. 3 Layered Soil in Excavations: (a) Sand and Clay; and (b) Multilayered Clay

2.3 General Deflection behavior of an Excavation Support

Lateral wall deflections and adjacent ground surface settlements are important parameters to predict the performance of an excavation support system. The magnitude of lateral deformations are a function of excavation support system stiffness, excavation geometry, the insitu soil and groundwater conditions, stability against basal heave and the construction procedures and workmanship. Excavation activities generally comprises of following three stages: (a) installation of solider piles or retaining wall, (b) excavation of soil mass and installation of lateral support elements at different levels, (c) structural backfill. However, the removal of temporary support system after the construction of permanent retaining wall is optional.

2.3.1. Peck (1969)

Peck developed the plots for maximum surface settlement δv (max) for braced excavations (mostly sheet pile and soldier pile walls) using data from existing case histories. He classified the soil based on undrained shear strength in three zones. Zone I for sands and hard clays and Zone III for very soft to soft clay with a low margin of safety against excavation basal heave. According to his findings the maximum ratio of δv (max) normalized by depth of excavation (He) was 1.0% for Zone I and > 2.0% for Zone III soils.

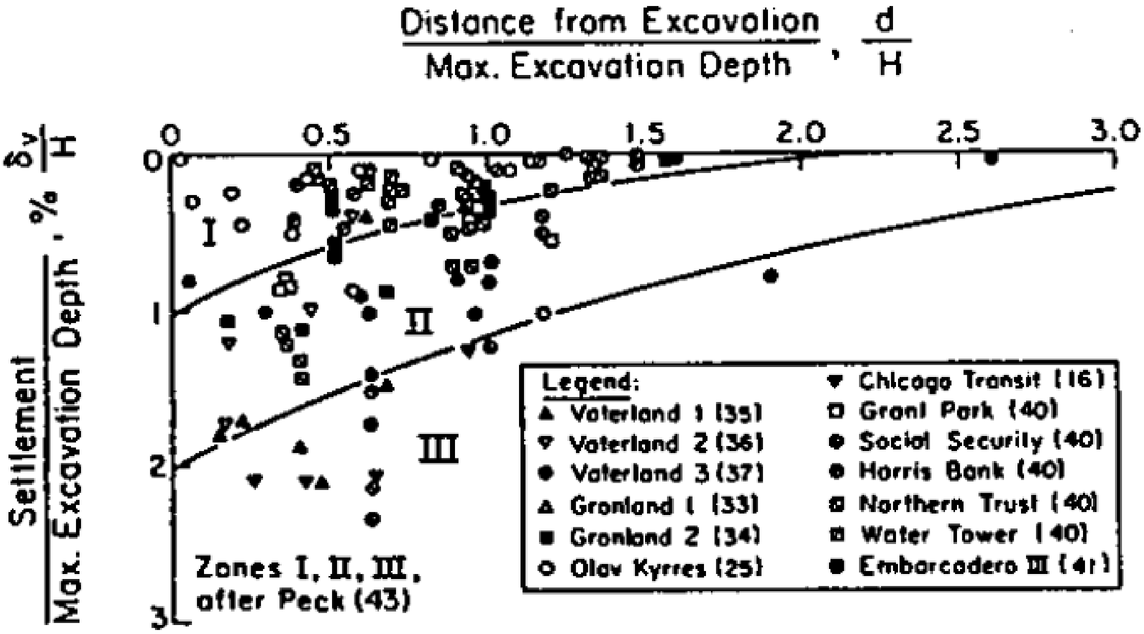


Figure 2. 4 Peck (1969) Settlement Profiles

2.3.2. Clough and O'Rourke (1990)

Clough and O'Rourke (1990) studied the behavior of lateral deformations and associated ground settlement. The general deflection behavior of lateral support systems during different stages of construction are presented in Figure 2.5.

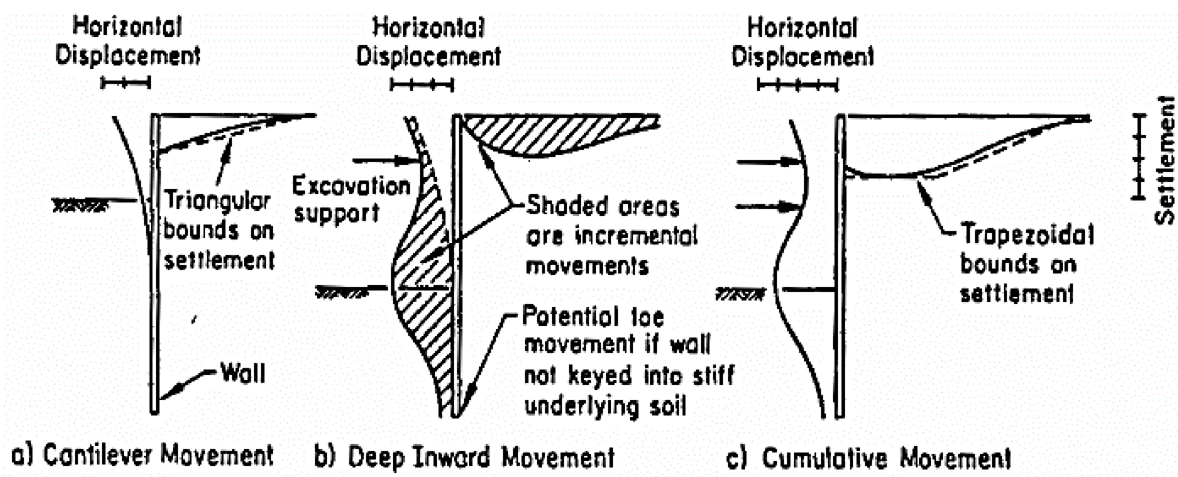


Figure 2. 5 Wall Movement in Braced Excavations

2.3.3. S.J Boone (2003)

Boone's method of determining displacements provides a logical and stepwise method to consider different important factors that control the movement of excavation support system.

Characteristics	Conditions	Equations
maximum unfactored lateral support installation and δ_{hmax} construction Stage, αcs preloading, αPL excavation width, αB strut stiffness, αS soil modulus, αM max. lateral displacement, δ_{hmax}	Support installation and removal Supports removed tiebacks remaining stressed percent of preload maintained	$\delta^*_{hmax} = (8.5Sr^{-0.5} + 0.4)FS-1.7$ αCS = 1 αCS = 1 − (Eur/pa) 3000/S 0.3 r + (Eur/pa) αPL = e −{PL/(60+4Sr)} αB = 0.75 + H/(4B) αS = 0.3(e^-Sr /1000 + e^-Sr /200) + 0.7 αM = 6.67E−2/3 $\delta_{hmax} = \delta^*_{hmax*}$αMαSαPLαDαBαCS

Where αCS αPL αB αS αM are factors for Construction Stage,PreLoad, Breadth (of Excavation), Soil Modulus and δ^*_{hmax} is the unfactored deflection

2.3.4. Bryson (2012)

Bryson presented the development of a semi empirical method for designing excavation support systems, which helps the selection of the excavation support system stiffness in such a way that it limits excavation-related deformations. In methodology proposed by Bryson, a new parameter was developed called the relative stiffness ratio. This new parameter relates the

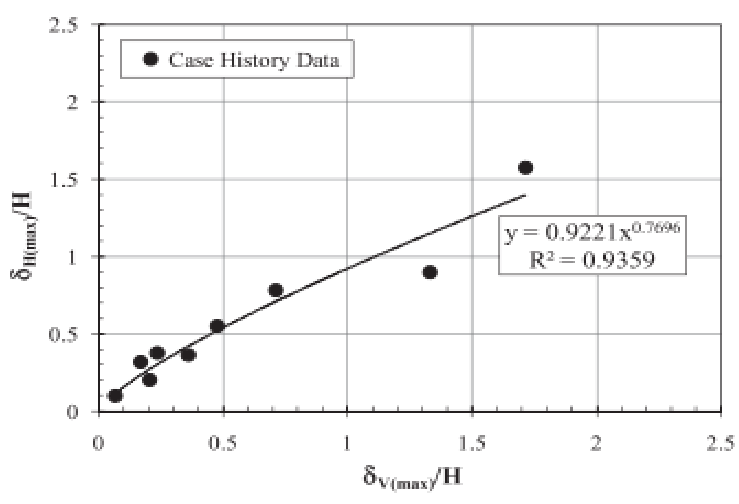


Figure 2. 6 Relationship between horizontal and vertical excavation induced movements

strength and stiffness of the soil with the stiffness of the excavation support system and was developed from a dimensional analysis of the parameters that appear to contribute to the overall stiffness of an excavation support system. The performance of the proposed methodology was evaluated using several excavation case histories reported worldwide and was compared with the performance of the Clough et al. (1989) method.

2.3.5. Hashash (1996)

Hashash presented results of numerical experiments, based on nonlinear finite-element analyses, which facilitated new insights to explain the ground movements caused by deep excavations, where the diaphragm wall is embedded in a very deep layer of soft clay. The principal structural parameters considered in the experiments are the diaphragm wall length and support spacing, while the excavation width and wall thickness are kept constant. For a given stress history profile, prototype design charts show the computed maximum wall deflections as functions of the excavation depth, H, and support spacing, h. Excavation stability can be assessed from contours of (H/L), corresponding to predicted failure mechanisms in the soil, and $M_{max,}$ the maximum bending moments in the wall. The maximum lateral wall deflections, surface settlements, and centerline heave can be interpolated from a simple four-parameter empirical equation.

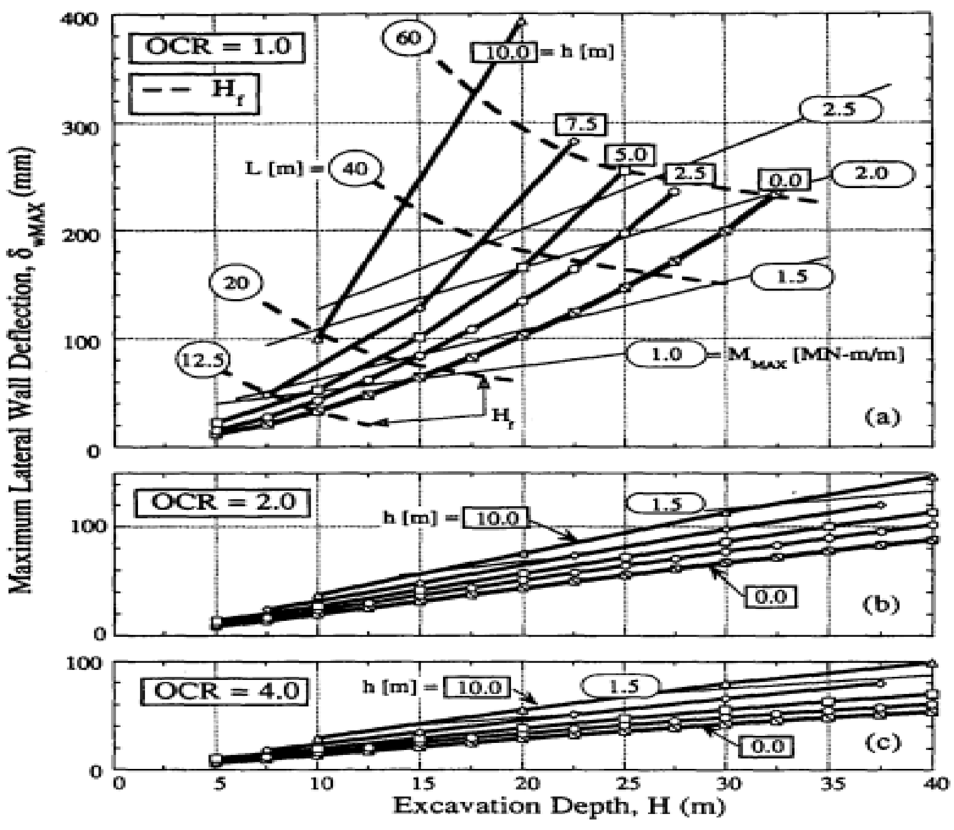


Figure 2. 7 Estimation of Maximum Lateral Wall Deflections from Numerical Experiments

2.3.6. Kung (2007)

Kung developed a semi empirical model by which the maximum wall deflection and the ground-surface settlement caused by a braced excavation in soft to medium clays can be fairly accurately determined. A database of 33 case histories and the results of a large number of well-calibrated FEM analyses was used to develop KJHH model. The model uses factors that are considered essential for predicting the maximum wall deflection. The factors include height of excavation, stiffness of wall, excavation width, depth to hard strata, stiffness of soil, and shear strength of soil.

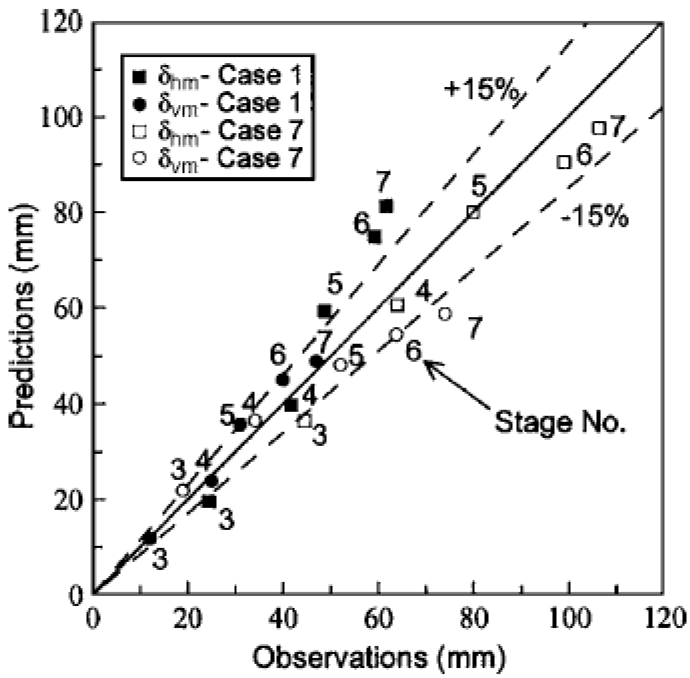


Figure 2. 8 Performance of the proposed KJHH model for predicting maximum wall deflection

Chapter 3

METHODOLOGY

3.1. Introduction

There is no standardized method available to design support system in stratified deep excavation. Aim is to narrow down the parameters along with deflection equations proposed by different authors as discussed in Chapter 2 as to which empirical equation should be used in a specific soil type giving most accurate result. Plaxis has been used to model soil sample to observe its behavior after excavation. It is a finite element software which is mainly used to analyze stabilities and deformations of soils. It gives opportunity to users to model the soil with different soil models like Mohr-Coulomb model, hardening soil model (HS). Finite element analysis adds to the picture by giving us results based on stress strain behavior and considering infinitesimal element in comparison to empirical equations formed on the basic of regression analysis formed by trial and error.

3.2 Calibration

Initially, Plaxis was calibrated by modelling a case from Excavation Support Practices by Jamal Ali. The case of excavated soil given in the thesis had all parameters given with stated known stress strain behavior and the deflections and settlements of that soil on field were given. The purpose of calibration is to make sure the output given by Plaxis is in accordance with known data results obtained from field and with the empirical equations. The results obtained are coherent with the empirical equations. As a result of this positive outcome, we modelled 103 cases of sand and clays.

3.3. Finite Element Modelling with Plaxis 2D

Plaxis 2D is mainly composed of two programs which are input and output. Input program consists of 5 subsections. These are soil, structures, mesh, flow conditions and staged construction. Model geometry, soil and material parameters and load properties can be created; in addition, soil and material parameters can be assigned from the structure subsection of the input program. From the material sets toolbar; material model, drainage type, soil parameters and groundwater conditions can be characterized. Before creating a mesh, all geometry must be modelled and properties of the materials must be assigned, otherwise, computational errors may be encountered. After creation of the geometry, generation of mesh process starts in order to perform finite element calculations. In the mesh subsection, the geometry is separated into finite elements with a desired level of fineness. Water levels can be added from the flow conditions subsection. After this part, one may start calculations. According to the purpose of the study, the model can be divided into calculation steps in stage construction part. Stage construction starts with the initial phase.

After calculations are completed, analysis results can be obtained from the output program. The main outputs of this program are the displacements and the stresses. Moreover, if the finite element program contains structural elements and forces applied to these elements, additional outputs related to these structural elements like deformations, axial shear forces, bending moments, etc. may be obtained.

3.3.1 Input

In this part of the thesis, detailed information will be given on how the geometry of the model is created, what parameters of soil and material are taken and how mesh sizes and calculation stages are created.

3.3.1.1 Geometry

Creating geometry starts with determining the size of the model. Plaxis 2018 fixes the boundaries of the model in all directions automatically. Boundaries of the model are selected large enough so that soil zone of interest will not be affected from these fixities. On the other hand, enlarging the soil boundaries also increases computation time. Therefore, an optimal boundary size should be determined. Height of excavation was taken as 12m for Stiff and Medium Clays 10m. The extent of the soil model was taken as7 times the height of excavation as referenced in Khoiri & Ou (2013). Geometry of the deep excavation problem is modelled in the structure subsection of the input program by using points, lines, plates, node to node anchors, and geogrids. Bored pile is modelled with plate element in Plaxis 2D analysis. Also, the interfaces are included to the plate element to simulate the soil structure interaction. Free length was modelled as anchor while bond length of the anchors was modelled as geogrid.

3.3.1.2 Material Parameters used in Plaxis

The choice of material properties in finite element analysis is important in order to reflect the system precisely. In this study for clays, Mohr Coulomb model was used because it is a simple and well-known linear elastic perfectly plastic model, which can be used as a first approximation of soil behavior. The linear elastic part of the Mohr-Coulomb model is based on Hooke's law of isotropic elasticity. The perfectly plastic part is based on the Mohr-Coulomb failure criterion, formulated in a non-associated plasticity framework. For sands the Hardening Soil model was used which is an advanced model for simulating the behavior of different types of soil. The reasoning for these decisions is explained in section 6.

The cases for Stiff and Medium Clays were modelled assuming and excavation depth of 12m with three rows of anchors and an embedment length of 3 m. The cases for Soft Clays were modelled with an excavation depth of 10m with 3 rows of anchors with embedment length of 2m and 5m. The cases for sands were modelled with an excavation depth of 10m with two rows of anchors and an embedment length of 2m.

Parameters for sand modelling:

$E_{ref} = E_{oed} = 1200*N$

$E_{ur} = 3*E_{ref}$

Parameters for stiff clay modelling:

$E_{ref}= 300\text{-}350*s_u$

Parameters for medium clay modelling:

$E_{ref}= 300\text{-}350*s_u$

φ = 1-1.5 degrees (for cases with Ns>4.5 and depth of clay strata exceeding 0.7B)

Parameters for soft clay modelling:

$E_{ref}= 500*s_u$

φ = 1.5-2.5 degrees

3.3.1.3 Mesh

The geometry of the excavation is in rectangular shape. Therefore, the plane strain model is used for the analysis with considering uniform cross section of the excavation. The plane strain models assume uniform cross section and no deformation in z direction during analysis. Also, 15 node triangular elements, which is the default element, are selected for modelling of soil layers.

Mesh size is important in terms of accuracy and calculation time. When the size of the mesh decreases, complexity of the model and the accuracy of the analysis increases. However, mesh refinement non-linearly increase the calculation time. In Plaxis, default mesh size is “Medium”; however, for the analysis “Coarse” mesh was selected.

3.3.1.4 Calculations

The full height of the excavation cannot be performed in a single stage; therefore, stage construction is needed to reflect the real situation. Analysis starts with initial phase from the activation of pile and its interfaces. The calculation is continued to excavation performed up to a level below the first -row anchor level. This stage is followed by the activation of the first-row anchor and application of prestress. Other construction stages continue with the subsequent excavation phase and anchor activation phase, these come one after another. After final anchorage level is activated, stage construction is finalized with the excavation up to the bottom excavation level.

3.3.2 Output

Calculation results in the form of plots and tables can be obtained for the full model or selected structures from the output program.

3.4 Visual Basic Application

Microsoft Excel is used to create a sheet incorporating all empirical equations under study, applied to determine the loads as well as the deflections and settlements of adjacent supported soil after excavation. The complex formulas were incorporated in Microsoft excel with the help of Visual basic Application (VBA). It is a computer language which including many, has the benefit of coding that helps you formulate formulas in Excel that are not present by default like avg, sum etc. Results obtained from Plaxis were also incorporated along with that values obtained from the use of empirical equations.

3.5 Calibrations of parameters for MC model and the moduli used in referencing equations

In the absence of adequate field data especially triaxial test results it was important to determine what value for Soil Modulus was to be used for both the FEM analysis and the value of initial modulus for Kung (2007) and unloading reloading modulus for Boone (2003) at the same time ensuring that these values were consistent with existing observations and ranges. For Medium and Stiff Clays Eref was taken as 500*su (Bowles 1995) for modelling in Plaxis; for Soft Clays Eref=250*su (Humza & Aye (2012) and Johansson and Sandeman (2014)). The value for initial modulus in the cases of Stiff and Medium Clays was taken as 3*Eu, where Eu is the undrained modulus (Bowle's (1996)) and Eu=500*s_u, for cases involving Soft Clays Ei = Eu. The reason for taking such a high initial modulus for Stiff and Medium clays is because according to what is mentioned in Bowle's (1996) "A number of investigators [Leonards (1968), Soderman et al. (1968), Makhlouf and Stewart (1965), Larew and Leonards (1962)] have suggested that a better initial tangent modulus for settlement analyses might be obtained by cycling the deviator stress $\Delta\sigma_1$ to about half the estimated failure stress several times [Leonards (1968) suggests at least five cycles] and then compressing the sample to failure in the CU triaxial test. The initial tangent modulus (may be called Er) by this method may be three to five times larger than Es obtained on the first cycle".The value of Eur was taken as 300*su and 350*su for Stiff Clays and Medium Clays respectively and finally for cases involving Soft Clays it was determined that Eur=250*su yielded results that were most consistent with the values obtained from FEM analysis. These parameters were by no means arbitrary, and the deflections yielded were verified with existing and widely accepted relations proposed by Clough & Rourke (1990).

Chapter 4

OBSERVATIONS

4.1.1. Stiff Clays:

For Stiff Clays (Ns<4) , where Ns represents the stability number, the average value for the predicted lateral deformation based on the equations proposed by Hashash (1996), Kung (2007), Bryson (2012) and Boone (2003) are within a range of 2-3 mm for values of Ns<3 and upwards of 5-10mm for Ns>3 when compared to the results obtained from Finite Element Method (FEM) analysis. However, the results of each individual equation show variance of a much greater degree. Boone (2003) and Bryson (2012) tend to under predict the lateral deformation by as much as 20%-30% and 15-25% respectively, while Kung (2007) overestimates it by 15-20%. Similarly, Hashash (1996) can over predict the deformation by a scale of up to 1.5 times the actual value when the value of Ns<3 but can offer realistic estimates with a small margin of error of 5% for values of Ns>3.2.

Case No	Kung (2007) (mm)	Boone (2003) (mm)	Hashash (1996) (mm)	Bryson (2012) (mm)	Average (mm)	Plaxis (mm)
1 (Ns=2.52)	45	25	62	32	41	40
2 (Ns=2.6)	48	28	62	33	43	42
3 (Ns=2.78)	55	33	62	35	46	47
4 (Ns=3.26)	74	50	62	48	59	62
5 (Ns=3.6)	86	66	62	49	66	75

Table 4.1 Summary of horizontal deflections (Stiff Clays)

When dealing with vertical displacements in stiff clays, the values estimated by Hashash (1996) were ignored as it overestimated the vertical settlements by an unreasonable amount i.e., up to 2times the actual value, however, the average value for the results by Kung (2007), Bryson (2012) and Boone (2003) predicted the vertical settlement within a small margin of 3-5mm when compared with the results obtained from Plaxis; although for Ns>=3, Kung (2007) tends to overestimate the settlement by a margin of greater than 15%.

Case No	Kung (2007) (mm)	Boone (2003) (mm)	Hashash (1996) (mm)	Bryson (2012) (mm)	Average (mm)	Plaxis (mm)
1 (Ns=2.52)	12	16	-	15	14	13
2 (Ns=2.6)	13	18.5	-	15	15.5	14
3 (Ns=2.78)	15	23	-	16	18	16
4 (Ns=3.26)	21	35	-	30	25	20
5 (Ns=3.6)	26	43	39	22	32.5	26

Table 4.2 Summary of vertical settlements in Stiff Clays

4.1.2. Medium Clays:

With regards to Medium Clays (4<Ns<5.14) two significant observations were made:

The first observation was for when the value of Ns<4.5; for this case the average value from the specified equations would yield results that were 10-15% smaller than the values given by FEM analysis mainly because Boone (2003) and Hashash (1996) underestimated the deflections by 10% and Bryson by about 20%.

The second observation was the generation of a failure surface in the retained soil mass that resulted from the ground heaving when Ns exceeds 4.5, this resulted in unrealistic and excessive deformations that would be greater than 10% of the excavation depth when the highest estimates and observations have failed to yield values greater than 2-3% of the excavation depth.

Case No	Kung (2007) (mm)	Boone (2003) (mm)	Hashash (1996) (mm)	Bryson (2012) (mm)	Average (mm)	Plaxis (mm)
6 (Ns=4.11)	103.5	93	89	58	86	100
7 (Ns=4.32)	110	105	89	60	90	114
8 (Ns=4.8)	123	117	89	65	94	324

Table 4.3 Summary of horizontal deflections in Medium Clays

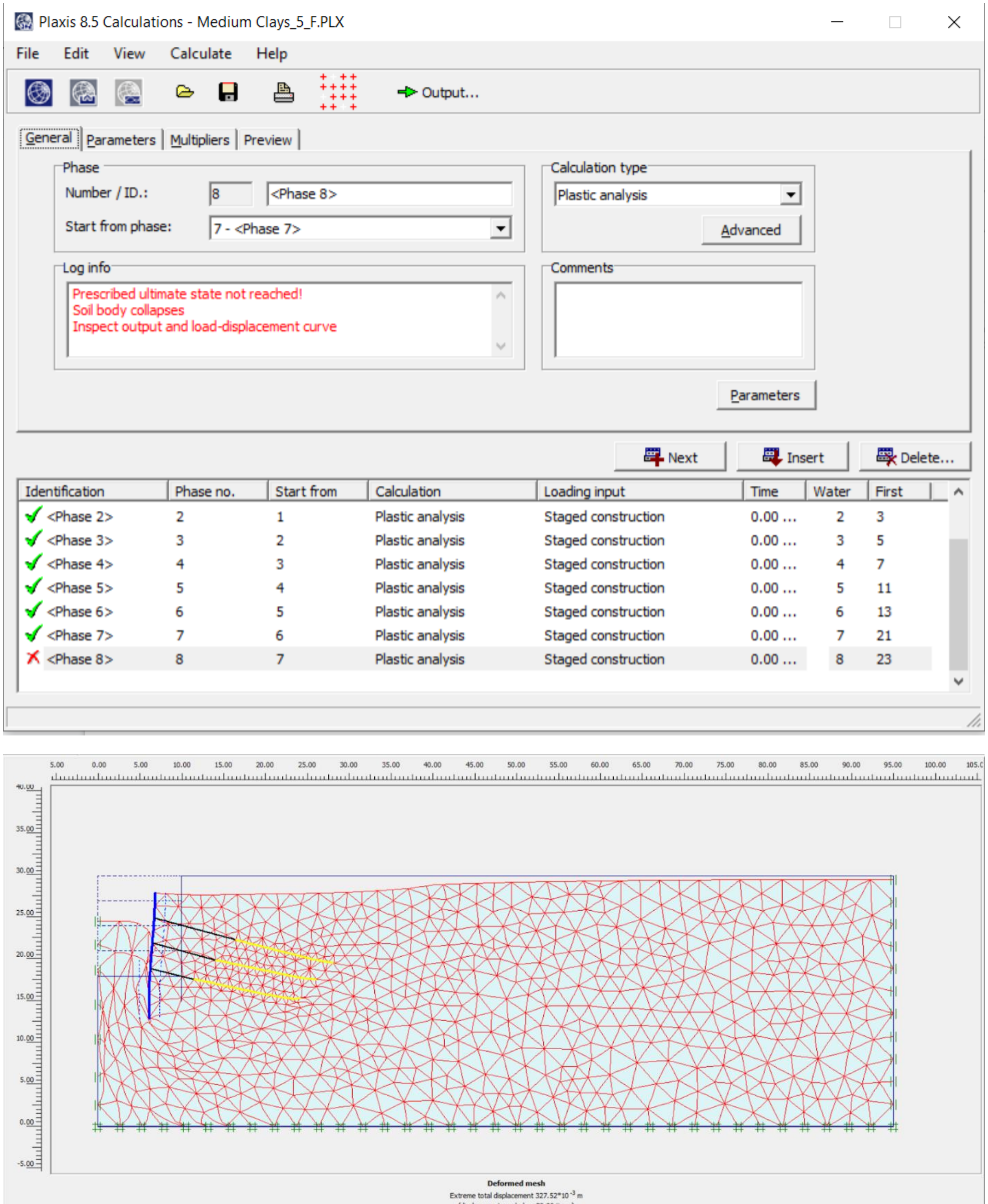


Figure 4. 1 FEM of medium Clays

To mitigate the problem, it was necessary to assign a value of angle of friction that would be able to simulate undrained conditions without it making any significant contribution to the decrease in lateral ground movement; based on trial-and-error procedure a value of 2.5 degrees was determined to be suitable. It is important to note here that provided the value of Ns<5.14 and there is no competent strata located at a depth of smaller than 0.7B below the base of excavation, heave will not occur so angle of internal friction can be taken as 0.

Case No	Kung (2007) (mm)	Boone (2003) (mm)	Hashash (1996) (mm)	Bryson (2012) (mm)	Average (mm)	Plaxis (mm)
8 (Ns=4.8)	123	117	89	65	98	102
9 (Ns=4.93)	126	128	89	65	102	105
10 (Ns=5.11)	131	141	89	67	107	109
11 (Ns=5.05)	129	137	89	67	105.5	107

Table 4.4 Summary of horizontal deflections in Medium Clays (with the incorporation of phi)

For these cases Hashash (1996) shows a value that is 5-10% smaller than the value of FEM analysis while both Bryson (2012) and Boone (2003) offer a more conservative estimate which is 15-30% larger than the value obtained from FEM analysis. At this point i.e., 4.5< Ns<5.14 for Hashash (1996) a small modification is suggested discussed later in conclusions.

For vertical settlements in Medium Clays the average value for maximum settlement when compared to results from Plaxis shows a minute difference of 5-10%. Though Kung (2007) can overestimate maximum settlement by as much as 20-30%, while Bryson (2012) underpredicts it by roughly the same margin. Since angle of internal friction for medium clays with 4<Ns<4.5 was not incorporated these would generate slightly larger settlement profiles, but for the remaining cases Boone (2003) and Hashash (1996), without the modification, can be used to reasonably predict maximum settlement.

Case No	Kung (2007) (mm)	Boone (2003) (mm)	Hashash (1996) (mm)	Bryson (2012) (mm)	Average (mm)	Plaxis (mm)
6 (Ns=4.11)	52	33.5	39	26	38	46
7 (Ns=4.32)	55	37	39	27	39.5	56
8 (Ns=4.8)	60	45	39	31.5	44	40
9 (Ns=4.93)	61	46	39	32	44.5	40
10 (Ns=5.11)	64	49	39	33	46	44
11 (Ns=5.05)	62	48	39	32	45.5	42

Table 4.5 Summary of vertical settlements in Medium Clays

4.1.3. Soft Clays:

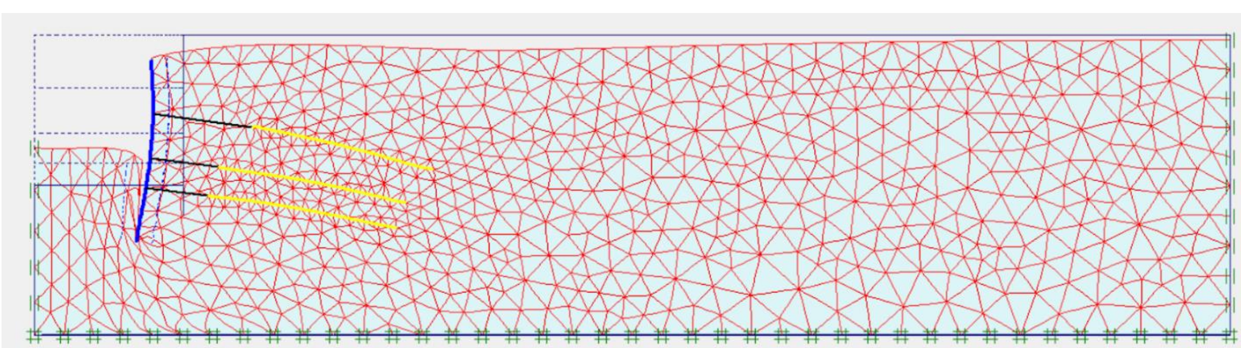
In the discussion of the above cases, it is worth mentioning that when the equation proposed Hashash (1996) was used, instead of using the excavation height, as originally intended by the author, the total wall height was used to get results more consistent with those of FEM analysis and those of Boone (2003) and Kung (2007). Admittedly this entails that a larger penetration depth yields a larger value for deflection which is the exact opposite of what takes place. The only reason this is not of significant concern is that, at max the penetration depth is 20-30% of the excavation depth and no significant decrease in wall deflections takes place with increasing penetration depth when dealing with medium to soft clays (discussed later) and the suggested modification, as results have shown, predicts deflections for medium clays within a 10% margin of error when compared with results obtained from FEM analysis.

Finally, for Soft Clays the values for prediction of ground movement by Hashash (1996) and Bryson (2012) were neglected as comparison of data showed that only Boone (2003) and Kung (2007) offered any reasonable estimate for the value of lateral deformation when compared with results from FEM Analysis. The difference between the average value of Boone (2003) and Kung (2007) and the results from Plaxis was within an acceptable range of 5-10%, however, for cases where Ns> 5.5 the average value was observed to be 10% less than the Plaxis value.

Case No	Kung (2007) (mm)	Boone (2003) (mm)	Hashash (1996) (mm)	Bryson (2012) (mm)	Average (mm)	Plaxis (mm)
12-A (Ns=5.14)	119	119	59	50	87	110
13-A (Ns=5.29)	122	127	-	-	124.5	117
14-A (Ns=5.45)	124	138	-	-	131	143
15-A (Ns=5.6)	127	149	-	-	138	145

Table 4.6 Summary of horizontal deflections in Soft Clays

In case of soft clays, it was also observed that the settlement profile generated showed heave in the Primary Influence Zone and significant settlement was observed in the Secondary Influence Zone. This problem was also reported by Johansson and Sandeman (2014) while modelling a case for sot clays but, when the results were compared with actual field measurements, no heave was reported in the soil mass. This can be attributed to lack of sufficient passive resistance developing in the top most anchor.



Unfortunately, due to the phenomena of heave occurring, at least according to what Plaxis shows, the only sensible course of action would be to take the safest approach. Therefore, in order to avoid any unforeseen complications from developing, the most conservative estimate based upon the results of the equations by Kung (2007), Boone (2003) and Hashash (1996) must be used.

Another thing worth considering is that for soft clays there were cases where the desired FOS for passive resistance would be achieved at a penetration depth exceeding the maximum recommended value of 0.2*He. To see just how much of an effect the penetration depth has on wall deflection the initial 6 cases modelled with Hp= 0.2* He were re-modelled with penetration depth increased to 50% of excavation depth

Case No	Plaxis (mm)	Plaxis (mm)	Case No
12-A (Ns=5.14)	110	102	12-B (Ns=5.14)
13-A (Ns=5.29)	117	108	13-B (Ns=5.29)
14-A (Ns=5.45)	143	115	14-B (Ns=5.45)
15-A (Ns=5.6)	145	117	15-B (Ns=5.6)
16-A (Ns=5.8)	158	129	16-B (Ns=5.8)
17-A (Ns=6)	175	160	17-B (Ns=6)

Table 4.7 Left: Deflections with Hp=.2*He, Right: Deflections with Hp=.5*He

Chapter 5

DISCUSSIONS AND CONCLUSION

5.1. Soil Models

On the HS model it is important to mention that adequate data is available for estimating the values of Eref, Eoed and Eur when modelling sands that gives a reliable estimate of ground movement (Hsiung (2014), Khoiri & Ou (2013)). On the contrary we were unable to determine or find such a relationship for reasonably estimating these parameters for clays; the relationships or parameters that were used would either not meet a certain criterion in Plaxis or would yield questionable results and hence it was determined suitable to use the MC model which, apart from settlement profiles in soft clays and sands, would show satisfactory results.

5.2. Variations of results

Based upon the previous observations, the reason for Bryson (2012) greatly underestimating the deflections compared to the results of finite element analysis and those determined using the relations proposed by the other authors is because, the relationship derived by Bryson (2012) was a result of observations made purely by finite element analysis while the remaining equations are either purely empirical or semi-empirical. Significance in this piece of information is that finite element analysis doesn't consider the human factor especially in the deflections observed in medium and soft clays, as indicated in Clough & Rourke (1990) publication. The reason being that when carrying out finite element analysis, an ideal scenario is considered while there is a significant time margin between the conduction of the excavation and installation of the anchors. In addition, the experience, and the skill level of the labor in carrying out the task also a determinant of deflections. This and the time lag can lead to additional wall movements. Hence, why Bryson (2012) is not recommended in predicting the deflections in medium and soft clays.

5.3. On the Wang & Reese equations in Circular No. 4

Another observation that was made, is that the formulas mentioned in FHWA circular no.4, was that the formulas used to calculate flow resistance developed for the embedded length for both sands and clays are inconsistent with the cited material.

For sands, the equation used to calculate flow resistance was as follows:

$P_{pu} = K_A b\gamma d\ tan^8\beta + K_o \gamma d\ tan^4\beta$ (Equation B-5)

The first portion of the equation gives a value in kN/m, while the second portion gives a value in kN/m^2. The adjustment made was that the second portion of the equation be multiplied by the diameter of the pile. This was further verified by comparing the results with the examples in appendix b example 1 and the spreadsheet in appendix c.

For clays, the equation used to calculate flow resistance was as follows:

$P_{pu} = 2S_u + \gamma d$ (Equation B-12)

There are two things to note, the first being that the equation gives us the stress instead of the force per unit length. The second being that when compared to the results in the second spreadsheet in appendix c, the results indicate that the flow resistance in clays is independent of the embedded length. However when consulting the reference document Wang-Reese (1986), the equation for flow resistance is $P_{pu} = 11S_u b$. The results obtained from this equation are the same as the spreadsheet in appendix c. Also, these results support a similar equation proposed by -, that is $P_{pu} = 9S_u b$ and these equations support the observations made by – and the FEM analysis. That increasing the embedded length has very little effect on the increase in passive resistance and hence wall movement.

5.4 Conclusion

The following conclusions can be drawn so far based off of the observations made:

For Stiff Clays with Ns<3, average of Bryson (2014) and Kung (2007) results in a more realistic estimate of wall movements

For Stiff Clays with Ns>3, average value of Kung (2007) and Hashash (1996) is recommended

For vertical settlements in stiff clays any of the equations excluding that of Hashash (1996) can be used

For wall deflection in Medium Clays avg. of Boone (2003), Kung (2007) and Hashash (1996) with modification; this modification entails that instead of using the excavation depth, a more realistic estimate can be obtained using the entire wall length

For settlements in medium clays either the average of all 4 equations should be used for a more conservative estimate or Boone (2003) and Hashash (1996) should be considered for a more precise estimate

In the case of clays especially when considering heave and based on the results obtained from FEM analysis the bearing capacity factor should be taken as 5.14 as opposed to the value of 5.7 as suggested by Terzaghi (1996) when dealing with deep deposits of clays. This is only for cases where the soil mass is assumed to be fully saturated, and an undrained condition exists.

For soft clays avg. of Boone (2003) and Kung (2007) is highly recommended as the margin of error is within a small envelope of 5-10%, though for cases having a stability number higher than 5.5 shows a slight underestimation by a margin of 5-10% therefore an appropriate FOS should be applied

The above point should only be taken into consideration when analyzing how much of an impact the excavation can have on any adjacent infrastructure and how important that structure is. It is also important to consider the “human factor” as poor workmanship can yield excessive deformations (Clough & Rourke (1990))

Keeping in view the arguments presented above it is only reasonable to have a FOS of 1.1-1.2 times the average value obtained by Boone (2003) and Kung (2007)

For deep deposits of soft clays Hashash (1996), with modification mentioned above, and Kung (2007) should be prioritized for a more conservative estimate due to lack of a proper comparison with FEM results.

In addition, when dealing with excessive wall movements one should also consider the economy of increasing either the system stiffness or the penetration depth; based on the observations made the former step is highly encouraged as it offers greater flexibility when it come to the factors that can be altered

More reasonable to use settlement profiles proposed by Kung (2007), Bryson (2014) and Ou (1996) for Clays

For Sands results were more consistent with profile presented by Clough & Rourke (1996)

References

Alavinezhad and Shahir (2020). “Determination of apparent earth pressure diagram for anchored walls in c–w soil with surcharge”.

Boone, S.J. 2003. “Design of Deep Excavations in Urban Environments. Ph.D. Thesis. Toronto: University of Toronto”.

Bowles, J. E. 1995. Foundation analysis and design, 5th Ed., McGraw-Hill, New York.

Bryson L. and Zapata-Medina D. (2012). “Method for Estimating System Stiffness for e excavation Support Walls”.

Clough, G. W., and O’Rourke, T. D. (1990). “Construction induced movements of in situ walls.”

Cosenza, Galasso, Maddaloni (2010). “A simplified method for flexural capacity assessment of circular RC cross-sections”.

Han (1964). “Principles and practice of ground improvement”.

Hashash, Y. M. A., and Whittle, A. J. (1996). “Ground movement prediction for deep excavations in soft clay”.

Hsiung, B. C. and Dao, S.D. (2014). “Evaluation of Constitutive Soil Models for Predicting Movements Caused by a Deep Excavation in Sands”.

Humza & Aye (2012). “Numerical modeling of diaphragm wall behavior in Bangkok soil using hardening soil model”.

Jamal Ali (2013). “Excavation Support Practices in Pakistan”.

Johansson and Sandeman (2014). “MODELLING OF A DEEP EXCAVATION IN SOFT CLAY”.

Khoiri & Ou (2013). “Evaluation of deformation parameter for deep excavation in sand through case histories”.

Kung, Juang, Hsiao and Hashash (2007). “Simplified Model for Wall Deflection and Ground Surface Settlement Caused by Braced Excavation in Clays”.

Ou, Hsieh and Chiou (1993). “Characteristics of ground surface settlement during excavation”.

Ou and Hsieh (1998). “Shape of ground surface settlement profiles caused by excavation”.

Ou a, Pio-Go Hsieh (2011). “A simplified method for predicting ground settlement profiles induced by excavation in soft clay”.

Peck, R.B. (1969). "Deep Excavations and Tunneling in Soft Ground, State of Art Report.”

Sabatini, D.G. Pass, R.C. Bachus (1999). “GEOTECHNICAL ENGINEERING CIRCULAR NO. 4”.

Terzaghi, Peck and Mesri (1996). “Soil Mechanics in Engineering Practice 3rd edition”.

Wang & Reese (1986). “STUDY OF DESIGN METHOD FOR VERTICAL DRILLED SHAFT RETAINING WALLS”.