DESIGN OF EXCAVATION SUPPORT SYSTEM FOR A COMMERCIAL PLAZA IN RAWALPINDI

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THESIS

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DESIGN OF EXCAVATION SUPPORT SYSTEM FOR A COMMERCIAL PLAZA IN RAWALPINDI

ABSTRACT

In Pakistan, commercial construction is leaning towards high rise buildings with multi-story basements which require substantial excavations. These type of excavations can lead to settlement and caving in of soil into the excavation. Which can lead to collapse of the surrounding buildings. Therefore, an efficient excavation support system is needed to prevent this from happening. The practices used in design of excavation supports are based on empirical methods and are highly conservative thus resulting in heavy costs, rendering the projects less feasible. Therefore, there is a need to use such methodologies which can meet the design requirements and reduce the cost at the same time. The aim of this thesis is to highlight the design of three different excavation support systems for a commercial plaza in Rawalpindi, Pakistan and to compare its costs to provide the most cost effective solution.

The assigned site is located behind Rania Mall, Saddar Rawalpindi and was surrounded by mega buildings on two sides and service roads on remaining other two. The excavation support systems undertaken for design included drilled shafts, piles with tie-backs and diaphragm wall with internal bracings. Field analysis was carried out to find the composition and shear strength parameters of the soil. The borehole data obtained as a result of extensive tests classified the soil as stiff silty clay. A 20ft excavation was to be carried out on site for the construction of two basements, each of 10ft height. The buildings located just over the edge of the excavation trench developed very heavy lateral stresses which was to be catered for to place the foundation of the plaza.

The design guide lines for each of the support system was followed differently, FHWA Circular No.4 for the design of secant piles with tiebacks, AASHTO manual on bridge design (2012) for drilled shafts and Deep Excavations by Chang Yu Ou for diaphragm wall with internal bracing.

This thesis includes a short comparison of all the three support systems based on design and cost. Extensive Cost analysis is carried out for each support system. Government issued rates of materials in Rawalpindi were used.

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CERTIFICATE OF ORIGINALITY

We hereby declare that this submission is our own work and that, to the best of my knowledge and belief, it contains no material previously published or written by another person, nor material which to a significant extent has been accepted for the award of any other degree at NUST or any other educational institution, except where due acknowledgment is made in this thesis. Any contribution made to the research by researchers and colleagues is explicitly acknowledged.

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DEDICATION

We dedicate our work to our beloved parents and teachers who enabled us to achieve education and meet our objectives with such dignity and respect.

DESIGN OF EXCAVATION SUPPORT SYSTEM FOR A COMMERCIAL PLAZA IN RAWALPINDI

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We are deeply obliged to acknowledge and thank those people who put their ever best contribution into our thesis. First of all thanks to Almighty Allah for blessing us with everything that he has provided us.

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CHAPTER 1

INTRODUCTION AND BACKGROUND

1.1 Introduction

In Pakistan the trend of adopting deep excavations is getting very common in urban areas where the limited space is utilized by construction of multistory plazas, underground road networks and basements for high rise buildings. Deep excavations are sustained by using different support systems when the excavation is close to adjacent infrastructure for example underground utilities, nearby buildings and roads that transport heavy traffic. Settlements in the adjacent ground may occur due to lateral movements of open faces as a result of excavations. Serious damage may be caused to nearby constructions which can result in collapse if the problem is not addressed in a timely manner. The indication of ground deformation may show up in the form of cracks in buildings or nearby ground surfaces due to lateral movements of soil mass. Therefore, the use of properly designed excavation support system is necessary to ensure the safety of excavated faces and adjacent structures. Another important aspect to consider is the health and safety of labor involved in the construction of retaining structures. The failure of excavation walls may result in life risk of people involved in the construction. An example of this is the collapse of 13-storey block of flats in Shanghai, due to piling adjacent to the building for making space for car parking, resulting in one death, several injured personnel and affecting the overall progress of the project.

Figure 1: Shanghai 13-storey block of flats collapsed on 28thJune 2009 due to piling under building for a car parking (The Wall Street Journal 2009)

The most common excavation support systems used word widely are diaphragm walls, ground anchors with soldier piles, secant piles walls and tangent piles walls. However, soil nails may also be used separately to resist lateral movements of soils to support the adjacent structures from falling into the excavation. In Pakistan the most commonly used excavation support system is ground anchors with soldier piles e.g Centaurus Mall Islamabad, Ufone Towers Blue Area Islamabad, Liberty Trade Center Lahore etc. However, an important concern regarding the use of these support systems is related to the soil condition and the built-up environment.

The overall design methods of the excavation support systems consider the effective settlements in soil and the structural stiffness of the supports to predict the performance thus providing the basis for the initial design.

1.2 Background

The design of excavation support systems depends upon the initial stresses, stiffness and strength parameters of subsoil and underground water conditions. It is also dependent upon the structural aspects of support system like type of retaining wall, its rigidity etc. and the construction method e.g. top-down, bottom-up construction, open-cut and excavation stages. All these factors contribute in prediction of total lateral ground movement and surface settlement. In Pakistan, the deep excavations are mostly supported by solider piles and ground anchors. The apparent earth pressures diagrams used for design of ground anchors are based on semi-empirical equations that have been developed from back analysis of field stresses by measuring strut loads. The design based on these correlations is therefore too conservative in some soil conditions and the stiffness of support system is such that the total predicted settlements are too small and hence design criteria can be relaxed to improve the overall economy of project while also ensuring the safety concerns as well.

1.3 Problem Statement

In Pakistan, commercial construction is leaning towards high rise buildings and thus the foundations needs to be embedded deep in the soil demanding for heavy excavations. The practices used in design of excavation supports are based on empirical methods and are highly conservative thus resulting in heavy costs, rendering the projects less feasible. Therefore, there is a need to use such methodologies which can meet the design requirements and reduce the cost at the same time. The aim of this thesis is to highlight the design of three different excavation support systems (mentioned later on) for a commercial plaza in Rawalpindi, Pakistan and to compare its costs to provide the most cost effective solution.

1.4 Content Summary

Chapter 1 of thesis provides a general introduction and brief of deep excavation and support systems and statement of the problem.

Chapter 2 presents technical background and existing literature on deep excavations and design methods. The chapter presents the methods proposed by different researchers to calculate the lateral earth pressures used in design of support systems.

Chapter 3 discusses the excavation practices in Pakistan. Data of 3 case studies have been presented in this chapter. The case studies presented are * Continued *

Chapter 4 discusses the methodology and approaches being followed for the design of excavation supports. It highlights the theories being incorporated for the calculation of lateral earth pressures and the factor of safety to be kept for long term sustenance of the support system.

Chapter 5 states the complete calculations for the design of each support system. Area needed, reinforcements required, center to center spacing, capacity of each support and related properties are being discussed thoroughly. Stepwise design method is explained in detail.

Chapter 6 discusses in details the economy of each support system. Each and every aspect is taken into consideration and a brief comparison between the discussed support systems is provided at the end.

Chapter 7 summarizes the general conclusions derived from the comparisons being carried out. Recommendations are also put forth at the end.

CHAPTER 2

AN OVERVIEW OF EXCAVATION SUPPORT SYSTEMS

2.1 Introduction

The behavior of deep excavations is a complex phenomenon that is influenced by many factors like structural properties, soil parameters and the insitu stress state. Many different methods are available in literature which can be used to predict the behavior of excavation and design for different support systems. The various aspects are; the use of theoretical and empirical based solutions, insitu and laboratory testing etc. This chapter discusses the basic concepts of excavations, existing methods, lateral support systems and finite element analysis.

2.2 Lateral Earth Pressures

To design an excavation support system determination of lateral earth pressures is the first and foremost step. Lateral stress depends upon the actual or governing conditions at site and behaviour of a typical soil present at site. To determine lateral earth pressures, we begin with the determination of lateral earth pressure coefficient. Rankine earth pressure theory and Terzaghi apparent earth pressure diagrams are the two prominent theories for measurement of lateral earth pressures.

2.2.1 Rankine Earth Pressure Theory

Rankine presented his theory of lateral earth pressures in 1857, after examining a number of soils and through testing. Rankine earth pressure theory is based on plastic equilibrium of soil and assumes that ; a) soil is cohesionless, b) there is no friction between wall and soil mass, c) the soil-wall interface is vertical, d) soil is isotropic and homogenous, e) failure surface of soil mass is planar, f) the friction resistance is uniform along the failure surface and g) the resultant force acts parallel to the inclination of backfill soil.

Total Horizontal stress is a function of vertical stress multiplied by the lateral earth pressure coefficient. It is shown in a figure below, the wall is represented by a line AB while pressures are shown by the corresponding arrows. As seen in figure 2.1 (b) a heavy soil mass is present behind the retaining wall due to which the wall from line AB moves laterally to A'B'. Due to the presence of active and passive zones across the retaining wall the pressure distribution in back and front of the wall changes. Due to heavy active zone the pressures at the front will increase and at the back it will decrease, the active pressure (pressure behind the wall) increases while passive pressure (pressure at the front wall) decreases. So, the calculations while designing a retaining wall are dealt in three states a) at rest condition b) at active state c) at passive state. The pressure distribution and the magnitude and direction of forces varies accordingly, this is illustrated as:

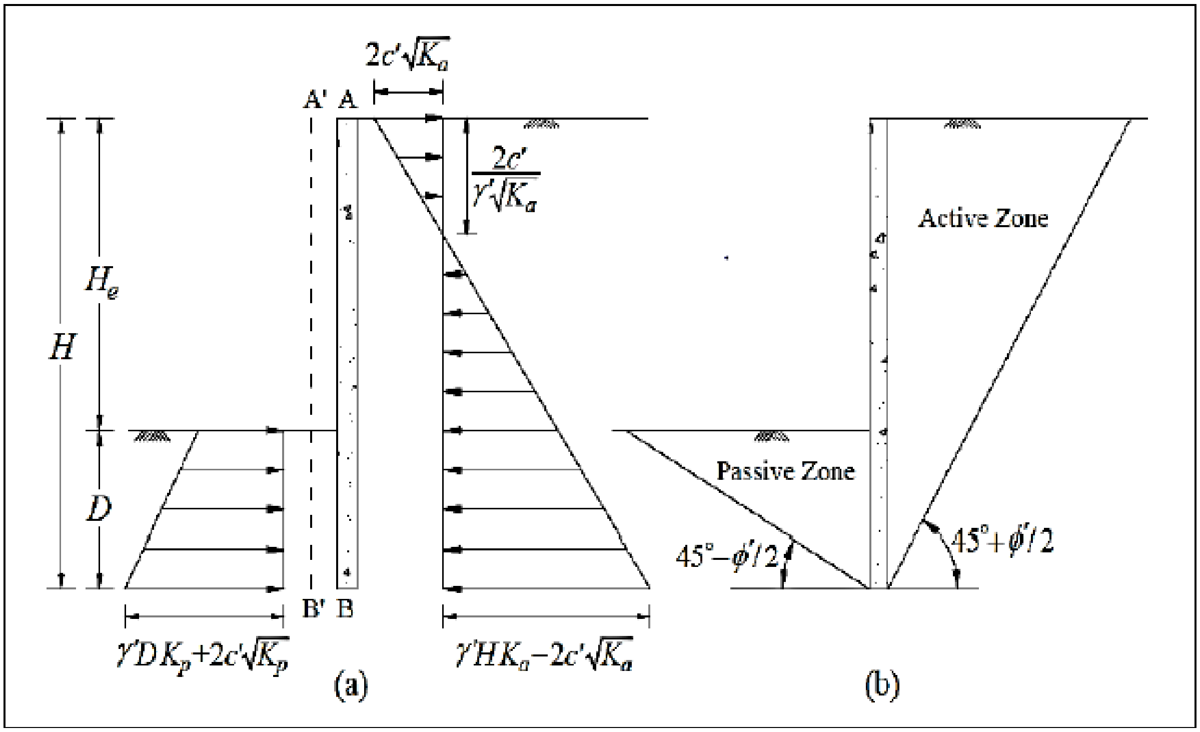


Figure 2: (a) Rankine's Earth Pressure Distributions; and (b) Passive and Active Zones

Rankine presented earth pressure equations for the active and passive cases of the soils having an intercept of cohesion c' and effective angle of internal friction. These are given as:

Active Case:

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

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

Passive Case:

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

short term analysis, undrained parameter is used and soil strength must be evaluated from CU or UU triaxial tests. So, $c' = s_u$ and $\varphi' = 0$ and $K_a = K_p = 1$ and Rankine earth pressure theory is given as:

Active Case:

$\sigma'_a = \sigma'_v K_a - 2s_u$

Passive Case:

$\sigma'_p = \sigma'_v K_p + 2s_u$

2.2.2 Peck's (1969) Apparent Earth Pressure Diagrams

Peck(1969) presented apparent earth pressure diagrams to calculate design strut loads for ground anchors. The distribution of stress depends upon the type on soil present at the site and corresponds to the type of soil. Peck presented earth pressure diagrams for sands and clays. Further in clays, there are soft clays and stiff clays, the behaviour of both these clays is different from each other, Peck also studied soft and stiff clays. The earth pressure diagrams for sands, soft to medium clays and stiff clays are presented below in figure 2.2.

Peck presented earth pressure diagrams on the basis of field measurements and their testing in laboratory. Peck studied excavations in Oslo, Chicago and Mexico and calculated the strut loads from back analysis. To develop these diagrams peck considered undrained conditions and total stress analysis for clays and for sands drained conditions were assumed.

Ou (2006) and Das (2007) presented the earth pressure diagram for soft to medium clays having stability number $N_b > 4$, and the apparent earth pressure to be the any of the large value:

$$\sigma = \gamma H_e (1 - 4ms_u / \gamma H_e) \quad \text{or} \quad \sigma = 0.3\gamma H_e$$

where m is the empirical coefficient related to stability number N_b. For $N_b \leq 4$, m=1 and for $N_b > 4$, m=0.40 and for reaching $\sigma=0.3\,\gamma H_e$, assume N_b=5.7, which is same as terzaghi's bearing capacity factor for clays.

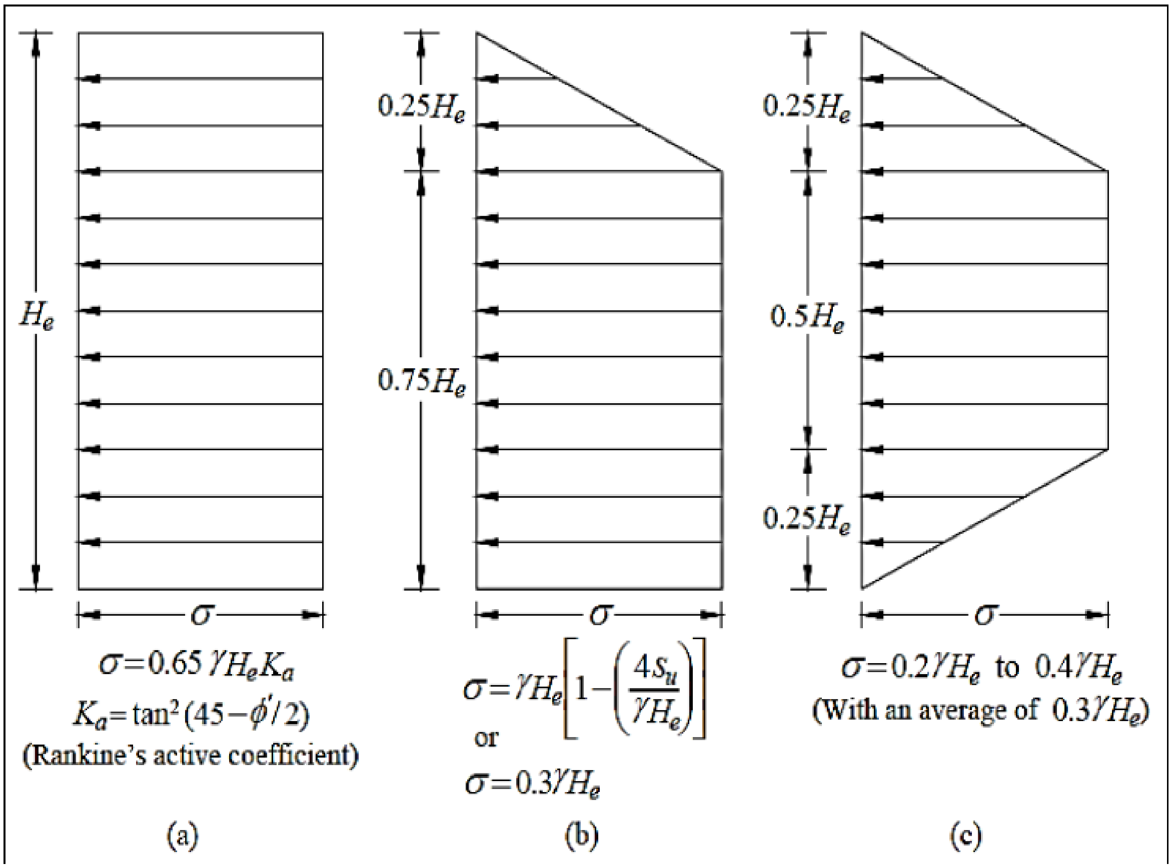


Figure 3: Peck's (1969) Apparent Pressure Envelopes: (a) Cuts in Sand; (b) Cuts in Soft Medium Clay; and (c) Cuts in Stiff Clay (After Peck, 1969)

For layered soil profile, use the properties of that soil layer which is dominant into the greater depth of excavation to design an appropriate support system. Peck (1943) gave a relationship to determine the average parameters to be used in pressure envelopes, which is as follows:

For two alternating layers of sand and clay as shown in the figure 2.3. $S_{u,avg}$and unit weight can be calculated as follows:

$S_{u,avg}= 1/2H_e[\gamma_s K_s H_s^2 \tan\varphi_s+2(H_e-H_s)n's_u]$

$\gamma_{avg}=1/H_e[\gamma_s H_s+(H_e-H_s)\gamma_c]$

where K_S is lateral earth pressure coefficient , n' is coefficient of progressive failure, H_e is excavation depth, $H_S$is sand layer thickness H_C is clay layer thickness, φ_S friction angle of sand layer S_u undrained shear strength of clay layer, γ_s= sand layer unit weight and γ_c clay layer unit weight.

Similarly, if clay layered strata is present $S_{u,avg}$ and γ_{avg} can be found outas:

$S_{u,avg}= 1/H_e(S_{u1}H_1 + S_{u2}H_2+...+S_{u,i}H_i+...+S_{u,n}H_n)$

$\gamma_{avg}=1/H_e(\gamma_1H_1+\gamma_2H_2+...+\gamma_iH_i+...+ \gamma_nH_n)$

where;

H_e= excavation depth

$S_{u,i}$= undrained shear strength of i_{th} layer

H_i= i_{th} layer thickness

γ_i= i_{th}layer unit weight

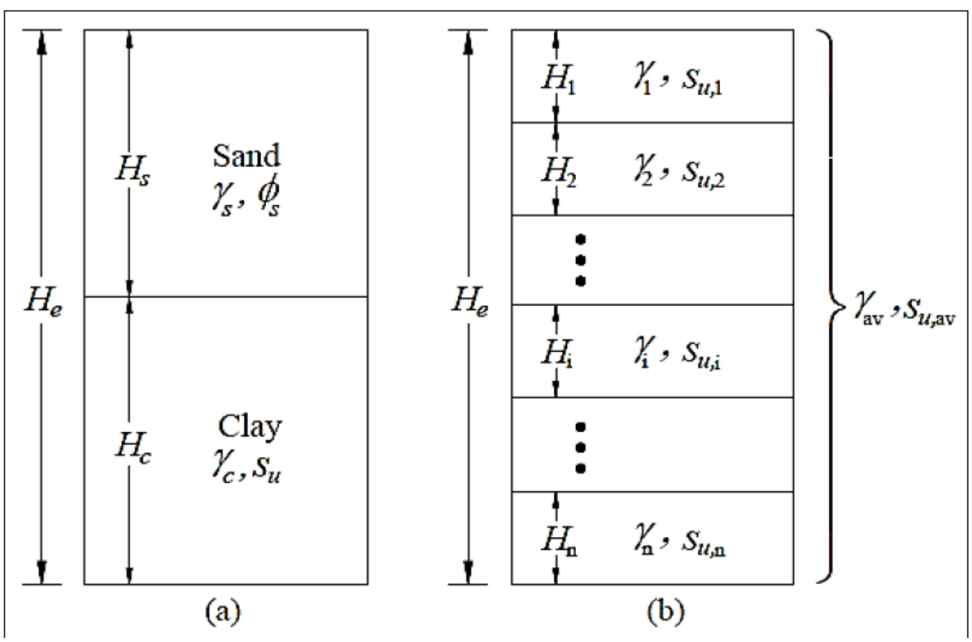


Figure 4: Layered Soil in Excavations: (a) Sand and Clay; and (b) Multilayered Clay (Adapted from Ou, 2006 and Das, 2007)

2.3 Stability Analysis (Basal Heave)

The basic purpose of an excavation support system is to support the excavations and this aspect is taken into consideration while designing an excavation support system. If FS approaches to 1, then large ground movements are to be expected and support system is the only remedy.

Basal Stability analyses can be carried out using Limit Equilibrium Methods or Non-Linear Finite element models. Limit Equilibrium is used for the initial phases of design due to their simpler nature as compared to non-linear finite element methods.

2.3.1 Terzaghi (1943) Method

Terzaghi's method is the simplest method to calculate the factor of safety for Basal Heave. Terzaghi assumed the circular sliding or failure surface of the excavations of infinite length. FS is given as

$FS_{(heave)}=S_u N_c/ \gamma_s H_e+q_s-(S_u H_e/B')$

where B' is limited to B/√2 or the thickness of the clay layer below the base of the excavation, smaller value is taken.

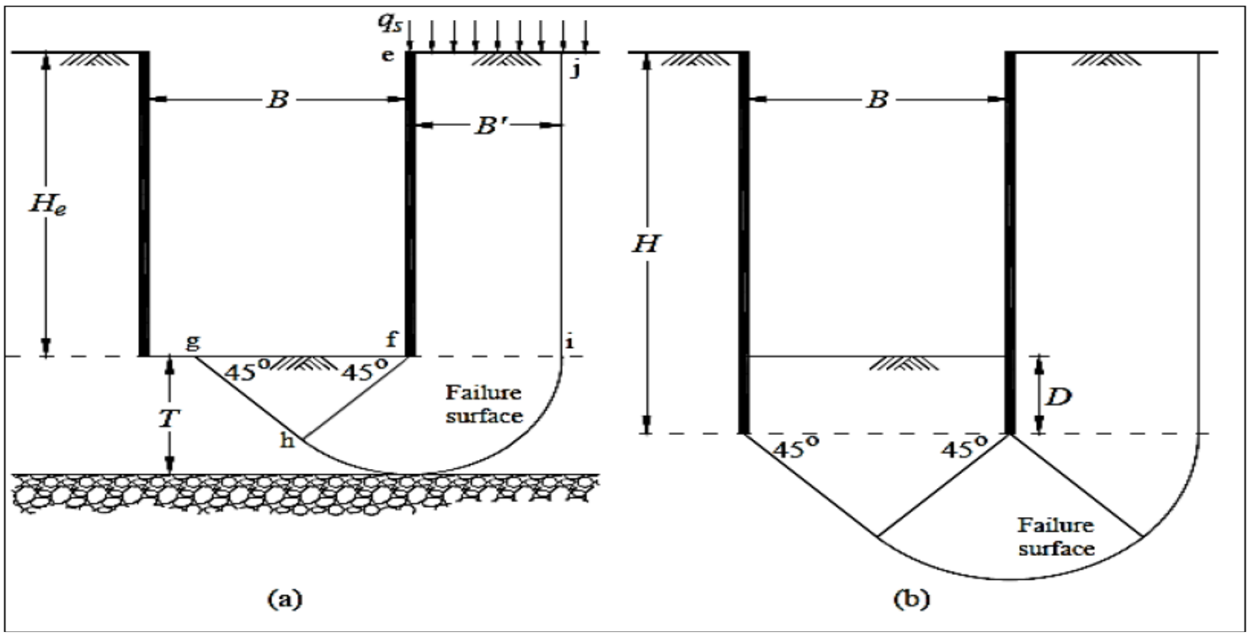


Figure 5: Factor of Safety against Bottom Heave Based on Terzaghi (1943a): (a) without wall embedment; and (b) with wall embedment (Adapted from Ukritchon et. al., 2003)

2.4 General Deflection Behavior of an Excavation Support System (Clough and O'Rouke)

2.4.1 Introduction

Twenty five years ago the concept of insitu walls involved a simple technology mainly including a sheet pile and soldier pile walls with crosslot bracing and earth berms. With a change in technology has come a growing interest in the movements of insitu wall systems, reflecting the increasing litigation over damages caused by excavations constructed within insitu wall systems, and applications in more critical situations.

2.4.2 Movements of Insitu Walls

The first practical approach for estimating the movements for insitu wall system was proposed by Peck. Data compiled was based on adjacent to temporary sheet pile and soldier pile walls. Below is the Peck's chart that gave the settlement and excavation depth relationship.

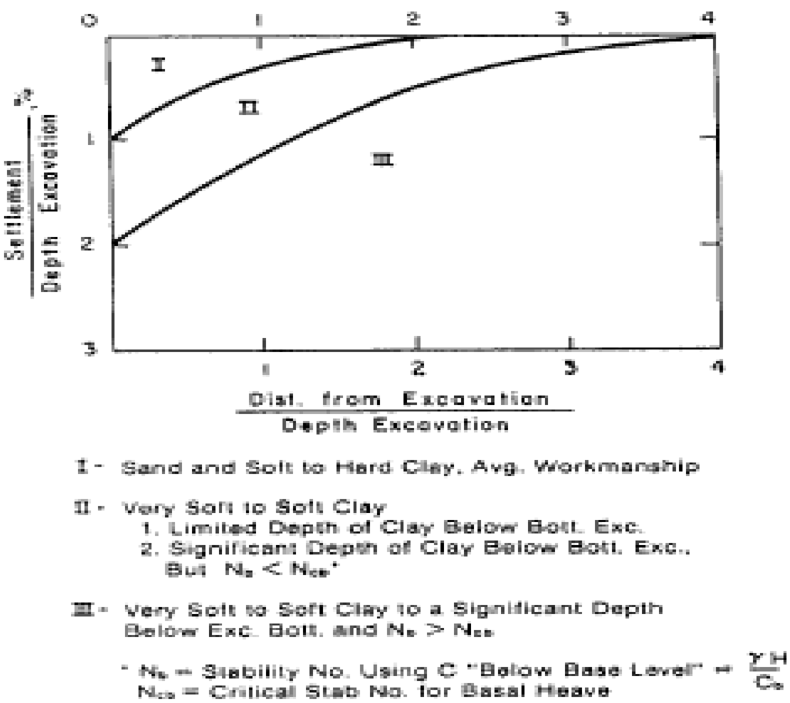


Figure 6: Peck's chart for the relationship between settlement and depth of excavation.

Three categories of behaviour were defined with the smallest movements indicated for sands, stiff clays and soft clays of small thickness (category I). The maximum movement movements in category I conditions near the wall were 1% of the excavation depth.

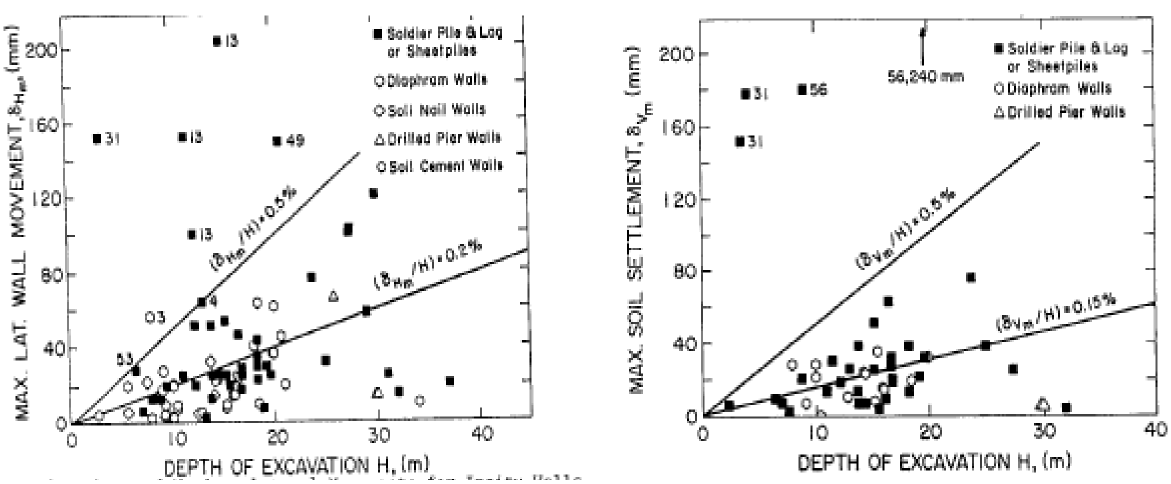
Peck's chart also included the data for the excavations in soft clays where basal stability was an issue and the thickness of the clay below excavation was large (categories II and III). Movements in these conditions exceeds those in category I because plastic yielding occurs below excavation.

2.4.3 Basic Movement Trends

Movements of insitu walls are function of soil and ground water conditions, changes in ground water level, depth and shape of excavations, type and stiffness of the wall and its supports, methods of construction of wall and surcharge loads. In this section of the paper the discussion concentrates on movements due to basic excavation and support process.

2.4.4 Maximum Movements-Stiff Clays, Residual Soils and Sands

Basal stability is not an issue in stiff clays, residual soils and sands. Peck's (1969) data shows that movements in these type of soils is restricted to 1%H. Later on studies showed that maximum horizontal movement is restricted to 0.5%H. To test these findings following figures were prepared.



(a) (b)

Figure 7: Peck's observation data for relation between excavation depth and (a) max. lateral movement of wall, (b) max. soil settlement.

Following conclusions can be drawn from the above figures:

1) The horizontal movements tend to average about 0.2% of H.

2) The vertical movements tend to average about 0.15% of H.

3) There is ample scatter in the data, with the horizontal movements showing more than the vertical movements.

4) There is no significant difference between trends of the maximum movements of different types of walls, and this includes even the soil nails and soil cement walls.

2.4.5 Maximum Movements- Soft and Medium Clays

Basal stability may be an issue inthese soils which result in movement patterns dominated by deflections beneath excavations. Peck has recognised this and accounted for stability number N_b, defined as $\gamma H/c_b$, where γ is the unit weight of the soil above excavation and $c_b$is the undrained shear strength of the clay beneath the excavation. When the magnitude of the stability number exceeds the bearing capacity factor for failure of the base of the excavation, then movements can become large.

The figure below shows that as FS falls below 1.5, the movements increase rapidly, it also illustrates the influence that wall stiffness and support spacing can have on wall movements. These factors are more important when FS is low.

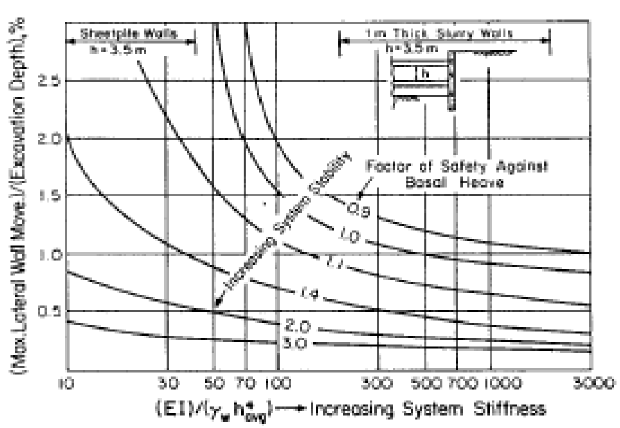


Figure 8: Maximum Lateral Wall Movements and Ground Surface Settlements forSupport Systems in Clay (After Clough et al., 1989).

2.4.6 General Patterns of Ground Movement

During initial stages of construction soil may be excavated before the installation of support. In some cases soil is excavated when the upper levels of support are not preloaded or lack sufficient stiffness to restrict inward movement. The wall deforms as a cantilever, and the adjacent soil settles such that vertical surface movements increase in inverse proportion to distance from the edge of excavation. When the excavations advances to deeper elevations upper wall movement is restrained by installation of support or stiffening of existing support members.

2.4.7 Displacements Adjacent to Excavation

Displacements adjacent to excavations is caused by activities such as dewatering and deep foundation construction within the excavation. So it is logical to treat these components separately to focus on the movements caused by excavations and support systems.

2.4.8 Excavations in Stiff to Very Hard Clays:

Response of stiff to very hard clays to excavations and support systems can be best understood by the diagram given below, obtained after testing and related case histories.

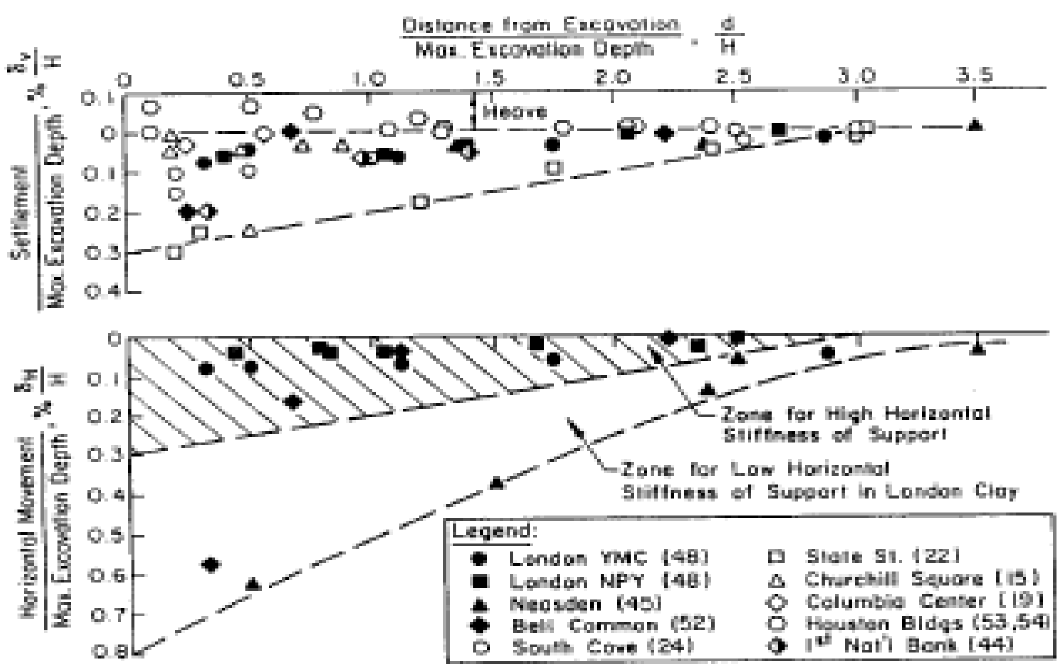


Figure 9: Summary of measured settlements and horizontal displacements in stiff to very hard clay (After Clough et al., 1989).

Vertical and horizontal movements observed during the installation of secant piles at Bell Common (52) which is screened in the above figure. The settlements are only a small percentage of excavation depth with maximum less than 0.3% but distributed over three times the excavation depth from the edge of the cut.

The majority of the horizontal displacements fall within the triangular boundary, this zone corresponds to excavations which have been braced with relatively stiff supports. The other zone contains measurements from Neasden (45) and Bell Common (52) highway excavations in London clay. At Bell Common the movements are influenced by a 3.5m cut in which temporary sheet piles were installed. At the Neasden underpass, block movement of London clay led to horizontal displacement in the zone of tie back anchor support.

2.4.9 Excavations in Soft to Medium Clays

The behaviour of soft to medium clays is best understood by the figure given below

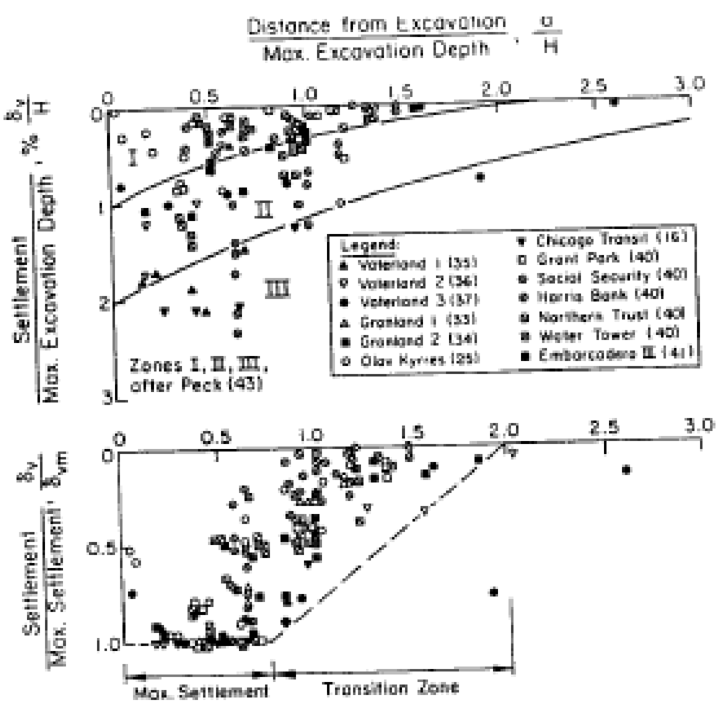


Figure 10: Summary of measured settlements and horizontal displacements in soft to medium clay (After Clough et al., 1989).

Settlements as a percentage of maximum excavation depth is plotted versus distance from the cut as a fraction or multiple of maximum excavation depth. When the settlements are plotted as a fraction of maximum settlements, a relatively well defined group of data is evident. The settlement distribution is bounded by a trapezoidal envelope in which two zones of movement can be identified. At $0 \leq d/H \leq 0.75$, there is a zone in which maximum settlement occurs. At $0.75 < d/H < 2.0$, there is a transition zone in which settlements decrease from maximum to negligible values.

2.4.10 Wall Installation Processes

In estimating the movements for an insitu wall project it is common to envision the wall in place, and consider what occurs beyond this point. However the placement of wall can generate movement.

Sheet piles are usually installed by driving, unless driving is particularly hard, it is usually done with vibratory hammer. The vibrations can cause problems for excavation project ranging from complaints from persons to the settlement of the ground in loose sand conditions. Using measured settlements from vibratory sheet pile driving in loose medium sands a plot of vertical strain was developed. To use this chart, the level of the vibrations caused by the driving needs to be selected based on the ground conditions.

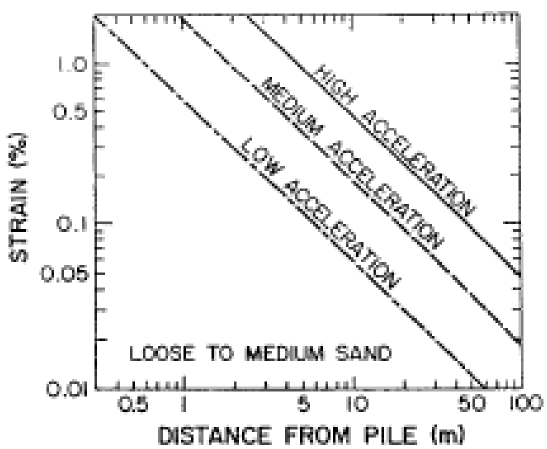


Figure 11: Vertical strain induced in loose to medium dense sand by vibratory sheetpile driving (After Clough et al., 1989).

The settlements concentrates near the sheet pile and decrease rapidly with the distance from the sheet pile. Importantly, significant settlements can occur near a sheet pile even though conventional criteria for structural damage due to vibrations show no damage occur.

2.4.11 Wall Settlement

Insitu wall designs are predicted on the assumption that the wall itself does not settle significantly. Settlement of an excavation support wall can de-stress a tieback system, and cause racking of a braced system. In excavations in soft clays with low FS values and a wall that does not bear on firm material, relative settlements can occur from one side of an excavation to other due to differences in soil conditions and surcharge loads.

In conditions where a soil layer is underlain by a rock, wall settlements can occur where the wall bearing is obtained at the top of the rock and the excavation extends below this point.

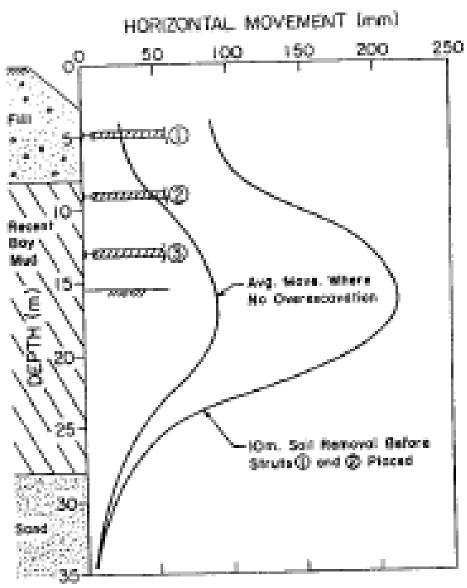


Figure 12: Effects of Over-Excavaion Below Support Levels in Soft Clays. (Adapted from Davidson 13)

2.4.12 Movements in the Anchorage Zone of an Anchored Wall

Highly OC clays are often characterized by the presence of high lateral stresses. The movements are greater with excavation for soils having high coefficient of lateral pressure, this effect could be higher in case of anchored wall where the movements can extend to include anchors themselves. Observation of London Clay showed movements progressed to several excavation depths, this occurred inspite of the long anchor with no load zone defined by a line sloped at 45° from the bottom of the excavation. In design, it belongs to the engineer to rely on local experiences and properly conducted consolidation tests to determine the importance of lateral stress relief factor.

2.5 Effect of Stiffness of Excavation Support System

2.5.1 Support System Design Considerations

In addition to the geotechnical and construction influences on insitu wall movements, the structural support system is equally an important factor. Thos element is particularly significant because it can be controlled by the designer. It is at once important to be prepared to utilize this element to improve wall behaviour while at the same time to be able to realize the limitations of the influence.

2.5.2 Wall Stiffness

Both theory and intuition lead to the conclusion that increasing the stiffness of the wall helps to counter movements. In conditions where clay is inherently stable, a stiff wall is much less effective in reducing movements than in conditions where there is a potential problem with basal stability.

In cohesionless soils there is no problem with the basal stability unless special conditions are encountered. In the presence of stable base increasing the wall stiffness theoretically does not significantly reduce the wall system movements. However there are subsidiary advantages for using a stiff cast in place concrete wall in cohesionless soils, first the construction of this type of wall is less subjected to risk than others such as sheet pile wall where the interlocks can be lost during driving. Second, as the wall is impervious so water movement is controlled. Finally, the concrete forms a strong bond with the soil during pouring and eliminates the hazard of voids.

2.5.3 Support Stiffness

In the interest of simplicity, the stiffness of a support system is defined in as few parameters as possible. Thus mainly wall stiffness and support spacing are included in system stiffness. However this does not mean that other factors do not have any influence on the support system behaviour. Clearly, the support themselves in the form of braces, tiebacks, soil nails, also will influence overall system stiffness to some degree. The theoretical Stiffness of the system can be defined in terms of AE/L, where A is the area of the steel, E is the modulus of steel and L is the unsupported length of the support. Because tiebacks are preloaded in tension, their theoretical stiffness is close to actual stiffness. Braces and Rakers on the other hand are compression members and their actual stiffness is affected by nature of the connections used to connect them to wall and the use of preloading.

The range of effect of support stiffness on predicted movement is on the order of plus or minus 20% relative to the values given in the charts. However, should an unusually stiff or flexible support be employed, the effect can be greater than 20%.

CHAPTER 3

EXCAVATION SUPPORT PRACTICES IN PAKISTAN

3.1 General

The construction of skyscrapers in Pakistan includes multi-storey parking spaces which requires deep excavation. Congestion in space and acceleration in land prices in urban areas of Pakistan is another important reason towards increasing underground construction. Major factors contributing towards poor construction practices in Pakistan are poor economy, lack of adequate expertise, poor practice in observing the construction bylaws etc. Here in this chapter a few case studies of excavations and their support systems in Pakistan has been summarized.

3.2 Excavation Case Studies

Summaries of few case studies are discussed below.

3.2.1 Jinnah Super Market Southern Parking Plaza, Sector F-7 Islamabad

Sector F-7 markaz is the center of commercial and shopping activities in Islamabad causing a serious need for adequate parking spaces. Keeping in view this need, Capital Development Authority (CDA) is constructing two parking plazas, one at southern side and other at northern side of Jinnah Super Market, F-7. These parking plazas are being constructed with areas of 4,200 and 3,307 square yards respectively. The southern site will have parking capacity of 213 cars, while the northern site will have parking capacity of 227 vehicles. The construction cost of the entire project is estimated to be Rs 239 Million. The parking plaza situated on southern side had been completed which consist of two levels of underground parking and floor level of upper slab will also be used for car parking while the other is under construction. This involved excavation to a depth of 20ft deep with respect to NSL. The excavation is surrounded by existing 2-3 storey shopping plazas at three sides of excavation while a service road runs on front side of excavation. The service road remains very busy due to extensive commercial activities in the sector.

The subsoil in this sector generally consist of medium stiff to stiff lean clay with thin layers of gravels at different depths. The vertical excavation cut was left unsupported because the excavation was exposed for very short period of time and the permanent retention system was in place within a time span of less than two months. The site excavation is shown in figure 3-1. It was observed that the no serious tension cracks were observed near end of excavation and also no visible distress was observed in adjacent buildings.

*Figure 13**:** Excavation for southern parking plaza Jinnah Super Market F-7, Islamabad*

3.2.2 Information Technology Tower Lahore

Information Technology (IT) Tower Lahore involved 64ft deep excavation and has been presented by Kibria et. al. (2010). IT Tower Lahore is located in Gulberg Lahore. It is one of the tallest buildings of Pakistan consisting of 28 floors including six basements, standing 200ft above the ground surface. The commercial activities at this building are related to Information Technology equipment and services. The IT Tower site is approximately trapezoidal in shape with an average length of 230ft and an average width of 160ft with excavation to a depth of 64ft. Two 46ft wide roads are running on northern and eastern side of the tower; whereas single/double storey residential buildings are located on other two sides.

The tower stands at a distance of at least 30ft or more from adjacent roads and buildings. Geotechnical site investigations at site were carried out by NESPAK which included a detailed excavation support system design to support deep excavation. Figures to Figure show the general layout of site and a view of adjacent buildings.

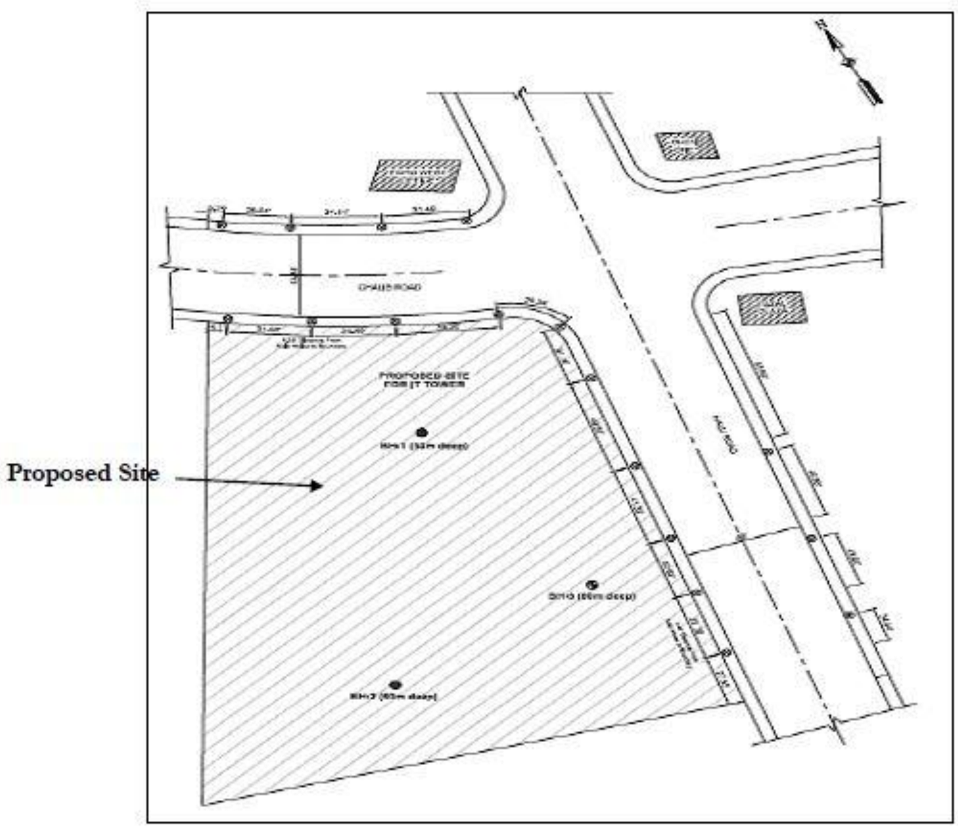


*Figure 14**:** Layout plan for IT Tower Lahore (Kibria et. al. 2010)*

Figure 15: An overview of adjacent buildings on rear end of building

Figure 16: An overview of adjacent buildings on right side of building

Detailed geotechnical site investigations were carried out by NESPAK Pakistan, which constituted drilling of three exploratory boreholes, insitu testing and laboratory testing. The geotechnical site investigations at site constituted drilling of 200ft deep exploratory boreholes and performing Standard penetration tests (SPTs) at every meter. The subsoil consists of two dominant soils layers till depth of investigations. The subsoil consists of approximately 45ft thick layer of silty clay in medium stiff to stiff insitu state underlain by silty fine sand till depth of investigations. The clay layer is categorized as medium plastic with liquid limit (LL) varying from 35% to 38%. Based on SPT blow counts, the insitu consistency of silty sand is categorized as dense to very dense state (Kibria et. al. 2010). The subsurface soil profile is shown in Figure 3-6 and the variation of SPT blow counts is shown in Figure 3-7. During site investigations, groundwater table was encountered at a depth of 90ft below NSL in all boreholes.

CHAPTER 4

DESIGN OF EXCAVATION SUPPORT SYSTEMS

4.1 Surcharge Calculation

In order to design the excavation support system, we need to calculate the surcharge that contribute to the lateral loading. Since we have buildings adjacent to the site, the structural loads need to be calculated based on the structural plans. The following data was used in the calculations:

- The dimensions of the building on both sides were 120 ft x 40 ft.
- The clear spacing of roofs was 10 ft.
- The center to center distance of columns was 10 ft.
- Therefore the no. of columns= 120 ft/10 ft= 10.
- Unit weight of RCC= 150 lbs/ft^3
- Mat foundation depth= 1 ft for 3 story and 1.5 ft for 5 story
- Beam dimensions: h=24in., b=18in.
- Column dimensions: h=24in., b=15in.
- Slab thickness= 6in.

Calculations:

- Load due to slab:

  (150) x (6/12) x (40 x 120)= 360,000 lbs= 360 kips.

- Load due to columns:

  (150) x (15/12 x 24/12) x (10) x (10)= 37,500 lbs

  For both sides (37500 x 2)= 75,000 lbs = 75 kips.

- Load due to beams: (along length)

  (150) x (18/12 x 24/12) x (120)= 54,000 lbs= 54 kips.

- Load due to beams: (along width)

(150) x (18/12 x 24/12) x (40)= 18,000 lbs= 18 kips.

For 10 beams= 18x10= 180 kips.

- Load due to tiles: (20 lbs/ft^2)

  Weight= 20 x(120x40)= 96,000 lbs=96 kips.

- Load of one floor= Wt. of slab + Wt. of beams + Wt. of columns + Wt. of tiles

  = 360+54+180+75+96=765 kips

- Load due to foundation:

  For 3 story building: 150 lb/ft^3 x 120 ft x 40 ft x 1 ft =720,000lbs = 720 kips

  For 3 story building: 150 lb/ft^3 x 120 ft x 40 ft x 1.5 ft =1080,000lbs= 1080 kips

- Load of 3 story building= 3 x (765) + 720 = 3015 kips
- Load of 5 story building= 5 x (765) + 1080 = 4905 kips
- Using a live load of 100 psf for commercial building (ACI) and using the LRFD method
- The total factored load for 3 story building = 910 psf.
- The total factored load for 5 story building = 1430 psf.

4.2 Design of Anchors

The design of anchors is an iterative process which require certain trial considerations. The trials must be reasonable. These include the no. of anchors, their depth, spacing and inclination. Terzaghi and Peck Earth Pressure Diagram is used for the lateral load calculations. For a 20 ft deep excavation, 2 anchors are used. The first at a depth of 7 ft and the second at a depth of 14 ft from the top of excavation.

The Tributary Area Method was used to determine the bending moments and the load on the anchors.

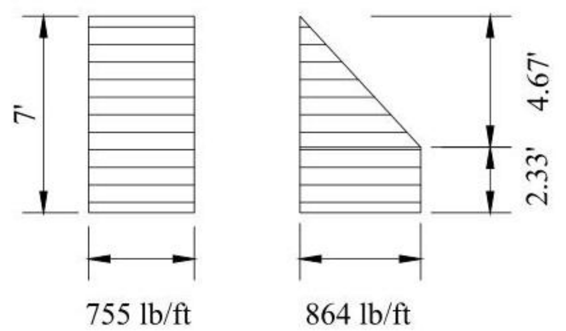


Figure 17: Surcharge and Soil profile for determination of maximum Moment on anchor.

M_1 = (1/2)(7x755) + (2.33/2)(864x2.33) + (1/2x4.67x864)(2.33+4.67/3)

M_1 = 28683.91 lb.ft

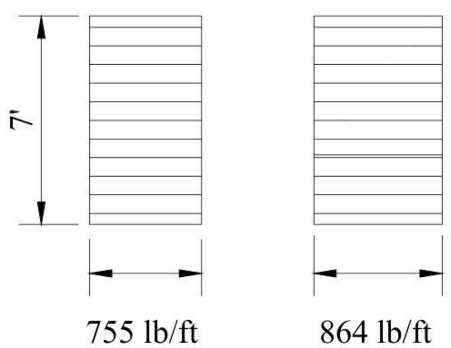


Figure 18: Surcharge and Soil profile for determination of maximum Moment on anchor

M_2= (7 x 755)(7/2) + (864 x 7)(7/2)

M_2 = 39665.5lb.ft = M_{max}

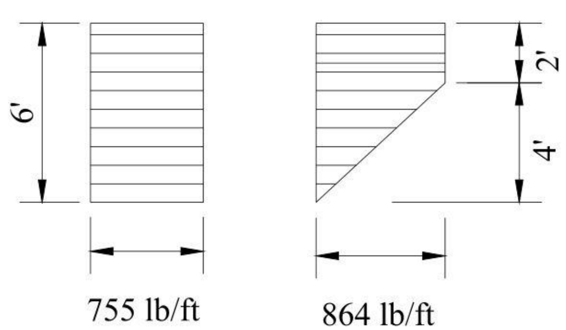


Figure 19: Surcharge and Soil profile for determination of maximum Moment on the anchor

M_3= (6 x 755)(6/2) + (2/3 x 4)(1/2 x 4 x 864)

M_3=19926 lb.ft.

4.2.1 Horizontal load on anchors (Area of the Diagrams)

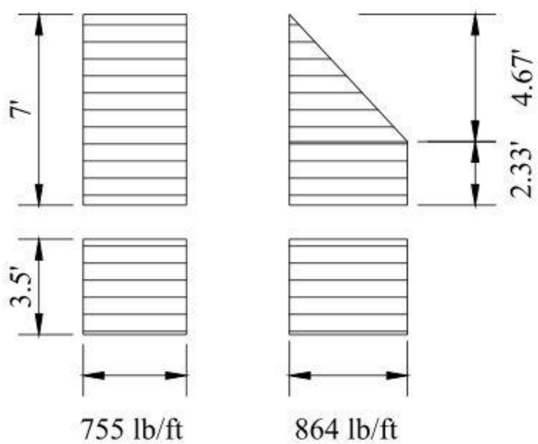


*Figure 20**:** Surcharge and Soil profile for determination of horizontal load on the first anchor*

T_{H1}= [(1/2 x 4.67 x 864) + (2.33 x 864)] +[7x755]+[3.5 x 864] +[3.5 x 755]

T_{H1}= 14982 lbs/ft

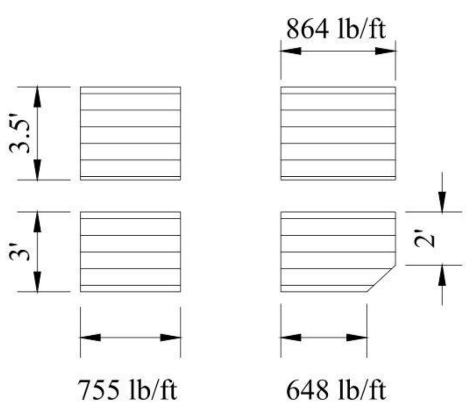


*Figure 21**:** Surcharge and Soil profile for determination of horizontal load on the second anchor*

T_{H2}=(3.5x755)+(3.5x864)+(3x755)+[(2x864)+(1/2x(648+864)x1)]

T_{H2}=10415.5 lbs/ft

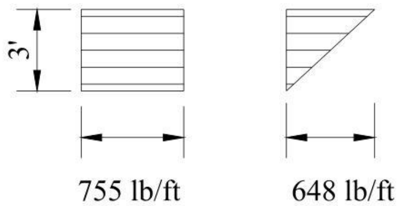


*Figure 22***:** *Surcharge and Soil profile for determination of horizontal load to be resisited by subgrade.*

R_S: Reaction force to be resisted by subgrade.

R_S= 3x755 + 1/2x3x648

R_S= 3237 lbs/ft

Assuming the inclination of anchor = 20°

Spacing = s = 4 ft.

4.2.2 Anchor Design Load

DL_1= (T_{H1})(s)/Cos 20° =14982x4/Cos 20°

DL_1=63774 lbs ≈ 63.77 kips

DL_2= (T_{H2})(s)/Cos 20° =104515.5x4/Cos 20°

DL_2=44336 lbs ≈ 44.34 kips

Max. anchor load = <u>63.77</u> kips

4.2.3 Unbonded Length

The unbonded length must be equal to the distance from the face of the excavation to the failure surface added to greater of 1.5m(5ft) or 0.2H. (H= height of excavation).

To determine the distance from the face of excavation to failure surface, SLIDE 6.0 software was used. For the upper anchor, it was= 9.44 ft and for the lower anchor it was 4.1 ft.

Unbonded length for the upper anchor= 9.44+5= 14.44 ≈ 15 ft.

Unbonded length for the lower anchor= 4.10+5= 9.10 ≈ 10 ft.

4.2.4 Bonded Length

The load transfer rate for hard clay was used to determine the bond length.

60 kN/m which is equal to 4111 lbs/ft. Ref. Table 1The max. bond length is 12m = 39 ft.

For a safety factor of 2, Max. design load= (4111 x 39)/2= 80164 lbs = 80.16 kips.

Since Max. load= 63.77 kips < 80.16 kips. No revision required.

Max. bond length= (63.77)(2)(1000)/4111= 31 ft.

Soil type	Relative density/Consistency (SPT range)[1]	Estimated ultimate transfer load (kN/m)
Sand and Gravel	Loose (4-10) Medium dense (11-30) Dense (31-50)	145 220 290
Sand	Loose (4-10) Medium dense (11-30) Dense (31-50)	100 145 190
Sand and Silt	Loose (4-10) Medium dense (11-30) Dense (31-50)	70 100 130
Silt-clay mixture with low plasticity or fine micaceous sand or silt mixtures	Stiff (10-20) Hard (21-40)	30 60

Note: (1) SPT values are corrected for overburden pressure.

4.2.5 Selection of Tendon

For the selection of tendon, the design load must be less than 0.6 of the Specified Minimum Tensile Strength (SMTS).

Using 270 ksi, 1/2 in. steel strands result in 0.6 SMTS= 105.5 kips > 63.77 kips. OK.

4.2.6 Soldier beam selection:

Required Section Modulus, S needs to be determined using the maximum bending moment.

$S = M_{max} / F_b$

F_b= allowable bending stress of steel. For permanent application, F_b= 0.55 of yielding stress.

For 50 ksi steel, F_b=(0.55)(50)= 27.5 ksi

M_{max}= 39665.5 x spacing = 39665.5x 4 = 158662 lb.ft≈ 159 kip.ft

S= (159x12)/27.5= 69.4 in^3

Using 2 MC 13x35, S= 72 in^3

4.2.7 Shaft Diameter

Let the open space between channels=6 in.

Flange width of MC 13x35=4.07 in., and Depth= 13 in.

Min. required dia. = $\sqrt{[(2x4.07+6)^2 + (13)^2]}$= 19.21 in.

Use 24 in. diameter shaft.

4.2.8 Timber lagging design

Required length= spacing of beams – space between channels

= 4 ft – (6/12)ft = 3.5 ft.

For stiff clays, recommended thickness = 3 in. Ref. FHWA-RD-75-130, 1976

4.2.9 Permanent facing design

M_{max}= 1/12(p_s + p_e) s^2

p_s= surcharge ordinate

p_e= Terzaghi Earth Pressure Diagram max. ordinate.

M_{max}= 1/12(755 + 864) $(4)^2$

M_{max}= 2158.67 lb.ft

$\frac{Mu}{\varphi bd^2}$= 2158.67(12)/(0.9)(12)(5^2)= 95.94 psi

For f_y=60 ksi, and f_c'= 3 ksi

ρ_{min}=0.0033 (for flexure) , ρ_{min}=0.0018 (for temp. & shrinkage)

Flexural Steel: $A_s=\rho bd$=(0.0033)(12)(5) =0.2 in^2/ft

Use #4 @ 10" C/C (A_s=0.24 in^2/ft)

Shrinkage Steel: $A_s=\rho bd$=(0.0018)(12)(5) =0.11 in^2/ft

Use #3 @ 9" C/C (A_s=0.15 in^2/ft)

4.2.10 Embedment Depth, D

Using wang-reese equations with the following parameters:

Depth of excavation = 20 ft,γ=108 pcf, φ=18°, q= 1430 psf, F.O.S = 2, D = <u>8</u>ft

4.2.11 Axial Capacity

- Vertical force of upper anchor = 14982 lbs/ Sin 20°= 5124 lbs
- Vertical force of lower anchor = 10415.5 lbs/ Sin 20°= 3562 lbs
- Weight of timber lagging:

  Thickness= 3.5 in, length= 3.5 ft, height= 20 ft,

  Unit weight= 50 pcf

  Wt = (3/12) x 3.5 x 20x50 = 875 lbs

- Weight of concrete in pile, after removing Wt. of timber:

  $[\pi(2)^2/4 \times 28 - (3/12)(3.5)(20)] \times 150$= 10570 lbs

- Total load= 5124+3562+875+10570= 20131 lbs.

Req. axial capacity= Q_a

Using F.O.S=2

$Q_a=(f_sA_s+qA)/2$

$q=c_uN_c$

$N_c=0.63(11.91- \ln(S_u))(\ln(S_u) - 4.39)= 7.06$

$q=(625)(7.06)=4412$

$A=(\pi)(3.5)^2/4=9.62\ ft^2$ (Assuming the base of pile is bell shaped of D = 3.5 ft)

$qA=4412 \times 9.62=42448$ lbs.

$f_s=\propto_u S_u$, $\propto_u=0.55$ for $S_u/P_{atm}< 1.5$

$f_s=184.52$

$A_s=(\pi d \times D)$ (d= pile dia., D= depth below excavation)

$A_s=(\pi \times 2)(8) = 50.27\ ft^2$

$f_sA_s=184.52 \times 50.27=9275$ lbs.

$Q_a=(9275+42448)/2=25860 > 20131$ OK.

4.2.12 Design Summary

4.2.12.1 Soldier Beams

Spacing = 4 ft.

Diameter = 2 ft

Embedment Depth = 8 ft

Section = 2 x MC 13x35, Grade 50.

4.2.12.2 Axial Capacity

Required = 20.1 kips.

Design Result = 25.8 kips

4.2.12.3 Anchors

Rows = 2.

Inclination= 20°

Size= 1 in.dia., steel grade = 150 ksi.

Depth= 7 ft (upper), 14 ft (lower)

Max. Required Capacity = 63.77 kips.

Design Result = 65 kips

Unbonded Length: 15 ft for upper anchor, 10 ft for lower anchor.

Bonded Length: 31 ft.

4.3 Design of Diaphragm Wall and Strut

For the design of strutted walls, the assumed support method is used. The retaining wall acts as a simply supported beam or a continuous beam and can therefore be solved easily by structural mechanics. For calculating the loads on the struts, the Terzaghi and Peck apparent earth pressure diagram and the Tributary Area method was used.

For two levels of struts, at a depth of 7 ft and 14 ft, from the top of the excavation, the loads on the struts are as follows.

T_{H1}=14982 lbs and T_{H2}=10415 lbs

4.3.1 Location of the assumed support

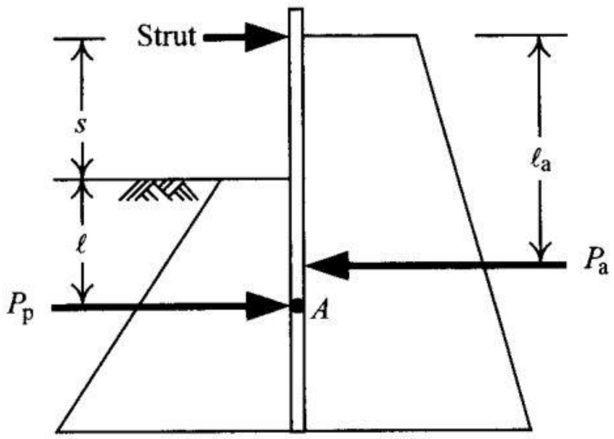


Figure 23: Schematic diagram for determination of the location of assumed support.

A is the location of the assumed support.

$\ell = P_a \ell_a / P_p - s$

Or it can be located using the stiffness of soil.

	Sandy soils	*Clayey soils*	*The locations of the assumed supports*
Dense soils	$N > 50$	$N > 15$	$\ell = 0$–0.5 m
Medium dense soils	$10 \leq N \leq 50$	$4 \leq N \leq 15$	$\ell = 1.0$–2.0 m
Soft soils	$N < 10$	$N < 4$	$\ell = 3.0$–4.0 m

N= standard Penetration number and ℓ the depth from the excavation surface.

N_{avg}= 12, clayey soil. Taking ℓ =1.5 m ≈ 5 ft.

4.3.2 Diaphragm wall design

With the loads on the struts and the location of the assumed support known, we can design the diaphragm wall. An initial thickness of 5% H_e(Excavation Height) can be assumed. For the design of reinforcement, the LRFD method is adopted. The nominal bending moment and shear force are as follows:

$M_n = L_F M_u / \varphi\lambda$

$V_n = L_F V_u / \varphi\lambda$

M_u and V_u are obtained from the Shear Force and the Bending Moment Diagrams.

φ= strength reduction factor. 0.75 for V_n and 0.90 for M_n.

λ= magnification factor = 0.6

4.3.2.1 Vertical main reinforcement

Initial thickness of wall = 5% H_e = 12"

$M_R=[\rho_{max}.f_y(1\text{-}0.59\ \rho_{max}.f_y/f_c'))]bd^2/\varphi$

Using cover = 1.5 in, d= 10.5 in, f_c'=4000 psi ,f_y=60,000 psi, $\rho_{max}= 0.75\rho_b$, β_1=0.85

ρ_b= 87000(0.85f_c'. β_1)/(87000+f_y)(f_y) = 0.022, ρ_{max} = 0.017

M_R = 106151.3 lb.ft

M_u= 25940 lb.ft

M_n= 76860 lb.ft < M_R Wall thickness of 12" is OK.

4.3.2.2 Tension Reinforcement

$\rho=1/m(1-\sqrt{1-(2mM/fybd^2})$

$m=f_y/0.85f_c'=17.65$

ρ = 0.13

As = ρbd = (0.13)(12)(10.5) = 1.66 in^2/ft

Use #9 @ 7" C/C (A_s = 1.71 in^2)

4.3.2.3 Horizontal Main Reinforcement

This will be the reinforcement for shrinkage and temperature effects.

$\rho_{min} = 0.0018$

$A_s = (0.0018)(12)(10.5) = 0.23\ in^2$

Use #4 @ 9” C/C(A_s= 0.26 in^2)

4.3.2.4 Shear Reinforcement

$V_u = 9315.6$ lbs

$\varphi V_c = 2\varphi\sqrt{f_c'}.b_w d$ ACI (11.3.1.1)

$\varphi V_c = 2(0.75)\sqrt{4000}\ .(12)(10.5) = 11953.4$ lbs.

$\varphi V_c/2 = 5976.7 < V_u$, Shear Reinforcement is needed.

4.3.2.4.1 spacing of stirrups

$s = (A_v)(f_y)/(0.75)(\sqrt{f_c'})(b_w) \leq A_v f_y/50b_w$

$s = 2(0.11)(60{,}000)/(\sqrt{4000})(12) \leq (2x0.11)(60{,}000)/50(12)$

s = 23 in $\leq$<u>22 in</u>

Or max. spacing = d/2 = 10.5/2 = 5.25 in.

Use #3 @ 5” C/C.

4.3.3 Strut System Design

4.3.3.1 Horizontal Struts

4.3.3.1.1 f_a calculation

$f_a = N/A$,f_a = axial compressive stress, N = axial load, A = Cross-sectional Area.

$N = N_1 + N_2$,N_1=load from excavation and N_2= load due to temp. (10-15 tons)

Take spacing = 10 ft. (Selected after trials) and Upper strut load = 14982 lb/ft (calculated earlier)

N_1 = (14982 x 10) = 149820 lbs.

N_2 = 10 tons = 20,000 lbs

For 14 x 132 W section, r_y = 3.76, A = 38.8 in^2

f_a= (14982 + 20,000)/38.8 = 4376.8 psi

4.3.3.1.2 f_b calculation

f_b = (M/S) (f_b = flexural stress)

M = wL^2/8

w = strut weight + live load ≈ 300 lbs/ft, S = Section Modulus.

M = (300)(10^2)/8 = 3750 lb.ft/ft

f_b = 3750x10/74.5 = 503.36 psi

4.3.3.1.3 allowable stress

KL/r_y = (1)(10x12)/3.76 = 31.91

C_c =$\sqrt{(2\pi^2 E/F_y)}$, E = 29000 ksi, F_y = 50 ksi (C_c = Critical Slenderness Ratio)

C_c= 107 > KL/r_y

$$\frac{KL}{r_y} < C_c \quad F_a = \frac{\left[1 - \frac{1}{2}((KL/r_y)/C_c)\right] F_y}{\frac{5}{3} + \frac{3}{8}[(KL/r_y)/C_c] - \frac{1}{8}[(KL/r_y)/C_c]^3} \cdot \lambda$$

$$\frac{KL}{r_y} > C_c \quad F_a = \frac{12}{23} \frac{\pi^2 E}{(KL/r_y)^2} \cdot \lambda$$

F_a = allowable compressive stress, λ = magnification factor = 0.6

F_a = (42.5/1.78)(0.6) = 14.32 ksi = 14320 psi

$F_b = (0.6)\ F_y.\ \lambda = (0.6)(50)(0.6) = 18000$ psi

F_b = allowable flexural stress

$f_a/F_a = 4376.8/\ 14320 = 0.3 > 0.15$

$$\frac{f_a}{F_a} \le 15\% \quad \frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0$$

$$\frac{f_a}{F_a} > 15\% \quad \frac{f_a}{F_a} + \frac{C_m f_b}{(1 - f_a'/F_e')F_b} \le 1.0$$

where

C_m = coefficient of modification = 0.85
$1/(1 - f_a/F_e')$ = amplification factor
F_e' = allowable Euler stress = $\lambda \cdot 12\pi^2 E/[23(KL/r_x)^2]$
KL/r_x = effective slenderness ratio on the flexural plane

$r_x = 6.28$, $KL/r_x = 19.11$, $F_e' = 10.67$ ksi,

$$\frac{f_a}{F_a} + \frac{C_m f_b}{(1 - f_a'/F_e')F_b} = 0.34 < 1.0.\ \text{OK}$$

Use W14x132 @ 10 ft C/C

4.3.3.4 Design of Wales

$M_{max} = pL^2/8$

$V_{max} = pL/2$

$p = p_s + p_e = 755 + 864 = 1619$ lbs/ft

$M_{max} = (1619)(10^2)/8 = 20237.5$ lbs.ft

$V_{max} = (1619)(10)/2 = 8095$ lbs.

$Z = M/\varphi F_y$

$\varphi = 0.9$, $F_y = 50$ ksi

$Z = 5.4\ in^3$

$V_n = 0.6F_yA_wCv$

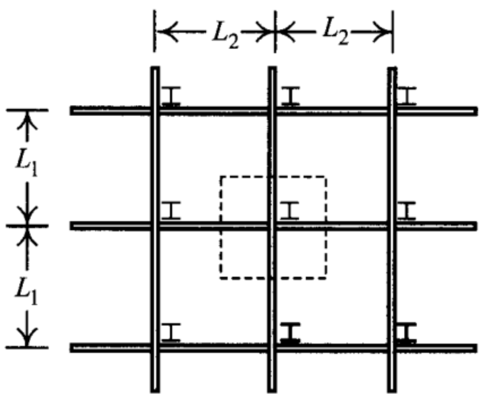
Use W4x13 (Z = 6.28 in^3)

$V_n = (0.6)(50{,}000)(4.16)(0.28)(1) = 34944 > 8095$ OK.

4.3.3.5 Center posts design

$P_1 = n(w\ x(L_1+L_2))$

n = no. of levels, w = Wt. of struts + live load $\approx$ 300 lbs/ft



L_1 = 10ft, L_2 = 15ft.

Figure 24: Schematic diagram for spacing of center posts.

$P_1 = 2(300\ (10 + 15\ )) = 15000$ lbs.

P_2 = Self Wt. of post = 520 lbs. (For HP 8x36)

P_3 = load induced due to settlement = $n[2(N)]\text{Sin}\ \Theta$

N=load on horizontal strut, n = no. of levels of struts, $\text{Sin}\ \Theta \approx 0.02$

$P_3 = 2[2(149820)](0.02)$

$P_3 = 11985.6$ lbs.

Total load = $P_1 + P_2 + P_3 \approx 27{,}500$ lbs

Capacity of Section **HP 8x36** = $F_y.A_g$ = (50)(10.6) = 530 kips. OK.

4.4 Drilled shaft design

For design of drilled shaft, AASHTO LRFD Bridge Design Manual was used. The SFD and BMD using Rankine Theory showed that the maximum bending moment was equal to 528 kip-ft and the maximum shear force was equal to 79.2 kips.

For max. steel area: $A_s/A_g \leq 8$ %

f_y = 60 ksi, f_c' = 4 ksi.

$A_s f_y / A_g f_c' \geq 0.135$

4.4.1 Longitudinal Reinforcement

Taking $\rho = 1.5\%$

$A_s = (1.5/100)(\pi)(2.5x12)^2/4 = 10.61$ in^2

Use 11#9 bars. (A_s = 11 in^2)

4.4.2 Shear Reinforcement

$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v$

$\beta = 2$, $b_v = D$, $d_v = 0.9\ (D/2 + D_r/\pi)$

D_r = Dia. of circle passing through center of longitudinal Reinforcement, D = Dia. of pile

D_r = 28.875 in, d_v = 21.77 in

V_c = 82.56 kips.

V_u = 79.2 kips.

Since $V_u > \varphi V_c/2$, Shear Reinforcement is required.

$s_{max} = 0.8d_v \leq 24$ in

$0.8(21.77) = 17.42$ in

Use s = 12 in.

$V_s = A_v f_y d_v / s = 2(0.11)(60)(21.77)/2 = 23.95$ kips.

$Vr = \varphi(V_c + V_s) = 0.9(82.56 + 23.95) = 95.86 > 79.2$ kips. OK.

4.4.3 Moment Capacity Check

For ρ = 1.5%, $M_n = 40D^3 = 40(2.5)^3 = 625$ kip-ft > 528 kip-ft. OK.

4.4.4 Embedment Depth

Using $K_a = 0.528$, $Kp = 1.89$, and taking moments about the final excavation level, we get the embedment depth required for the stability of pile.

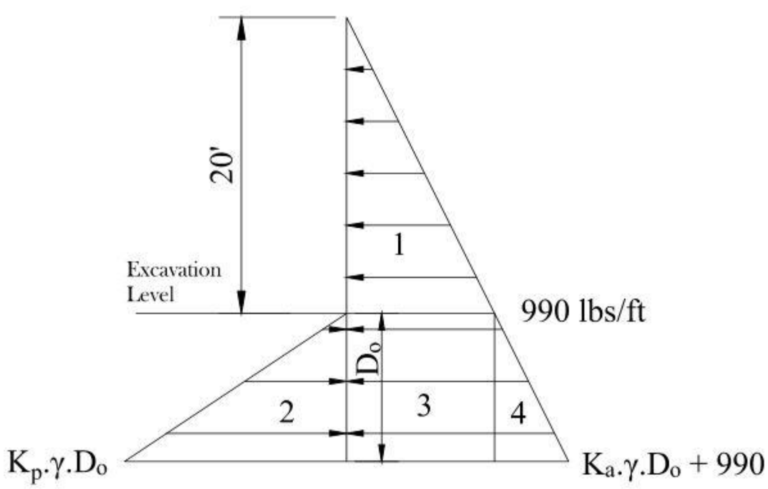


Figure 25: Schematic Diagram for determination of embedment depth, Do.

Solving for D_o, we get $D_o = 16$ ft.

Adding 25% for safety, we get the embedment depth, $D_o = 20$ ft. Similarly, and embedment depth of $D_o = 15$ ft was obtained for the drilled shaft on the 3 story building side.

CHAPTER 5

SOFTWARE MODELLING

5.1 DeepXcav

For the purpose of modelling the three support systems, DeepXcav software was used. It is a versatile and user friendly application capable of performing finite element analyses on deep excavations. The design of deep excavations can be a very complicated matter. The designer has to deal with many unknowns and factors that influence the behavior of the excavation. Performing detailed calculations for excavation systems can be a very time consuming process, especially when parameters have to be changed and iterations have to be performed. In addition, many current software programs do not offer an integrated platform of structural and geotechnical analyses required to design deep excavations. As a result, the designer is forced to use numerous software programs to analyze the excavation and the structural system separately. With the exception of finite element analyses, there are very few theoretical solutions for calculating lateral soil pressures from complex surface profiles. DeepXcav provides an integrated structural and geotechnical platform for designing deep excavations. The deformation calculated by the software can be used to check whether the designed support system is safe or not. The maximum allowed deformation according to Clough and O'Rouke (1990) is 0.5% of the height of excavation, which comes out to be 1.2 inches in our case.

5.2 Drilled Shafts Simulation

For the simulation of our designed drilled shafts in DeepXcav, staged construction was performed in steps of 7 feet, 14 feet and 20 feet. The surcharges were applied at the very start. The Deformations in the wall of excavation were calculated at each stage. The deformation in our model exceeded 1.2 inches, which indicated that our drilled shaft is not safe for our 20 feet excavation. Furthermore it reinforces NAVFAC's point, that drilled shafts must not be used for excavation greater than 15 feet.

The model is shown below:

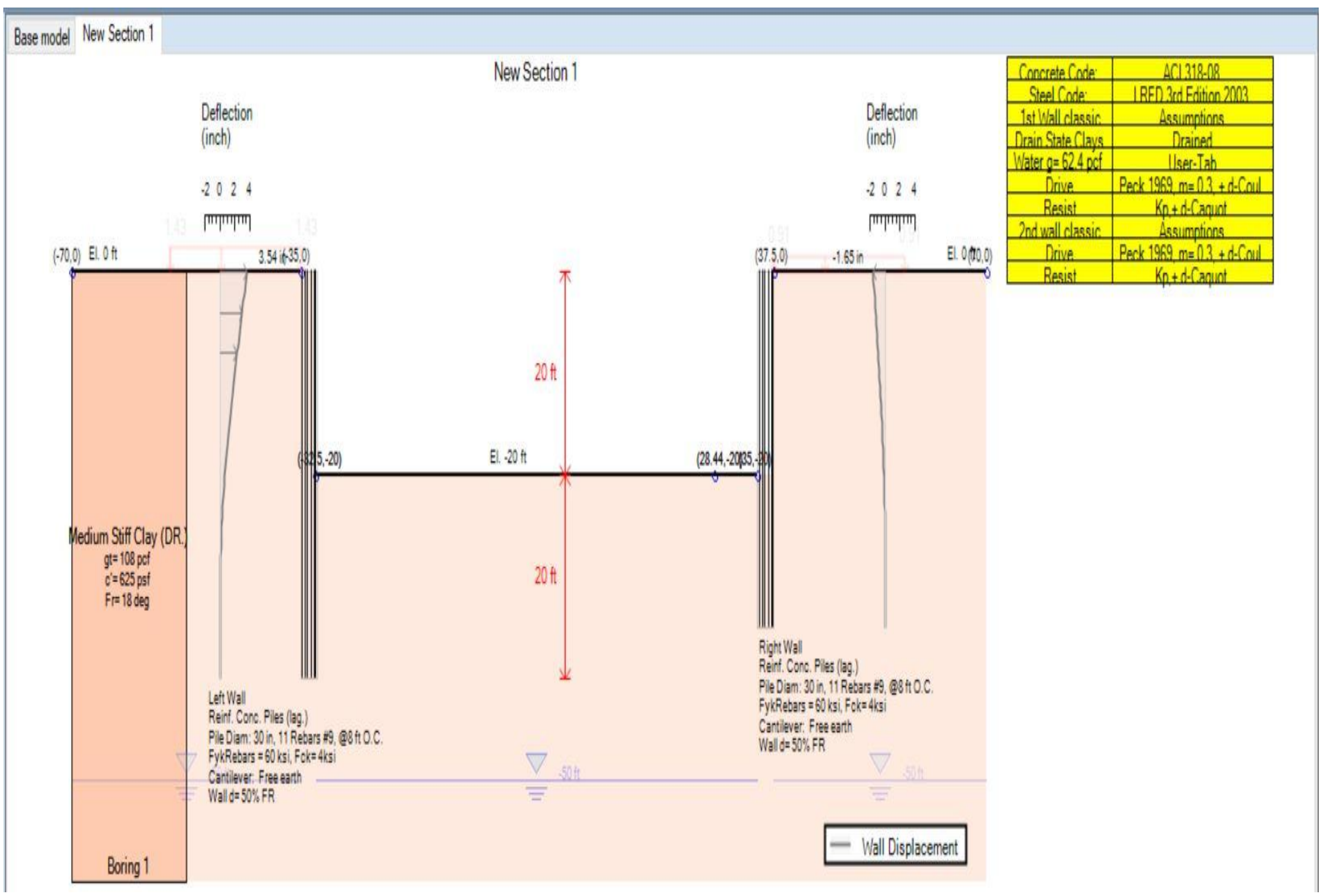


Figure 26: Software model for Drilled shaft.

(Maximum deformation on the 5 storey building side is 3.54 inches and on the 3 storey building side is 1.65 inches, both exceed 1.2 inches).

5.3 Soldier piles with Anchors:

The same staged construction procedure was carried out and deformations were calculated for each stage. It must be noted that the maximum deformation in this case was well below 1.2 inches.

The model is shown below:

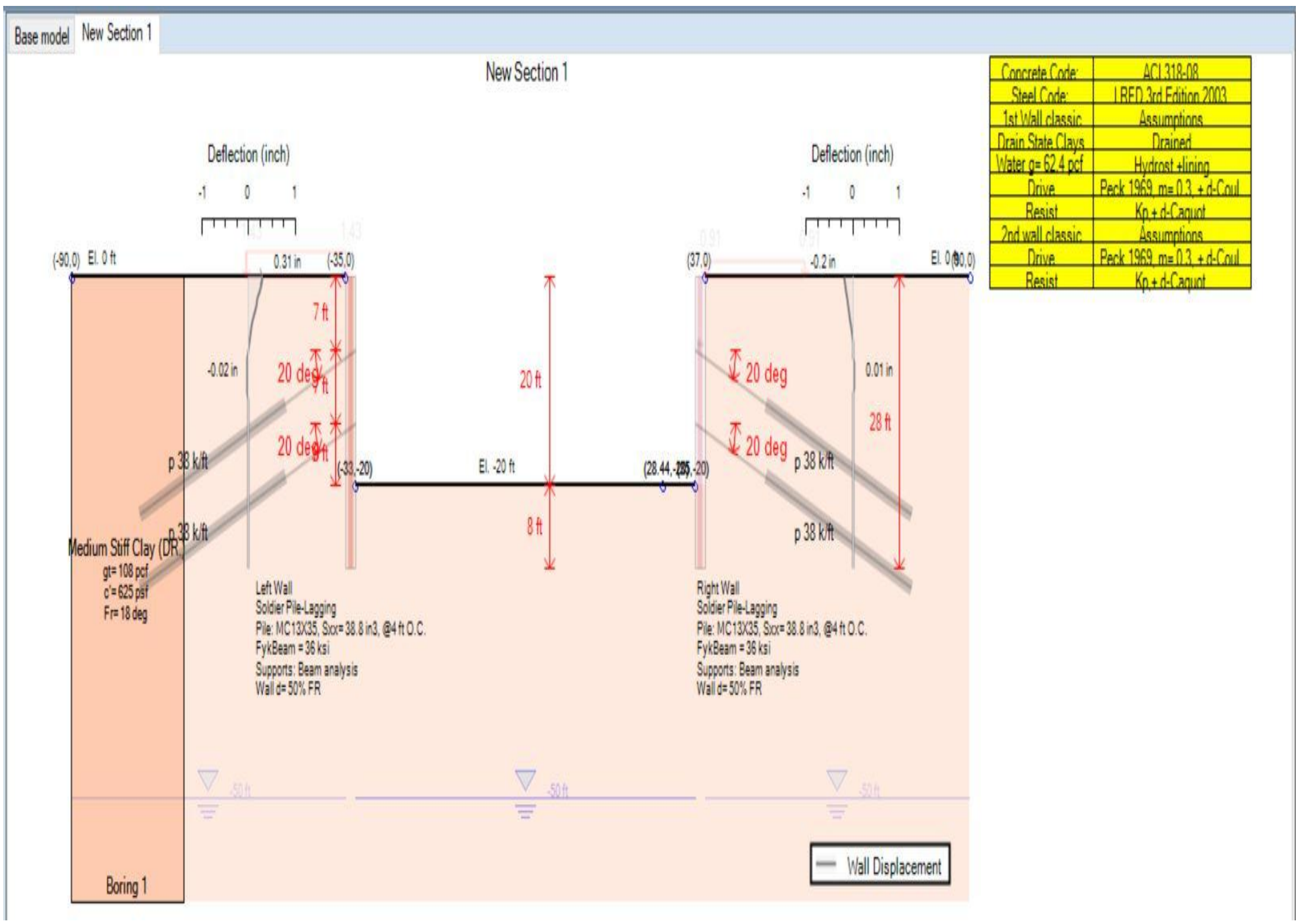


Figure 27: Software model for soldier pile with anchors.

(The max deformation in this case was 0.31 inches)

5.4 Diaphragm Wall with Struts:

A different staged construction procedure was carried out and results were obtained. First excavation this time was of 9 feet to allow strut installation at 7 feet. Second excavation at 16 feet for strut installation at 14 feet and finally 20 feet excavation was done.

The model is shown below:

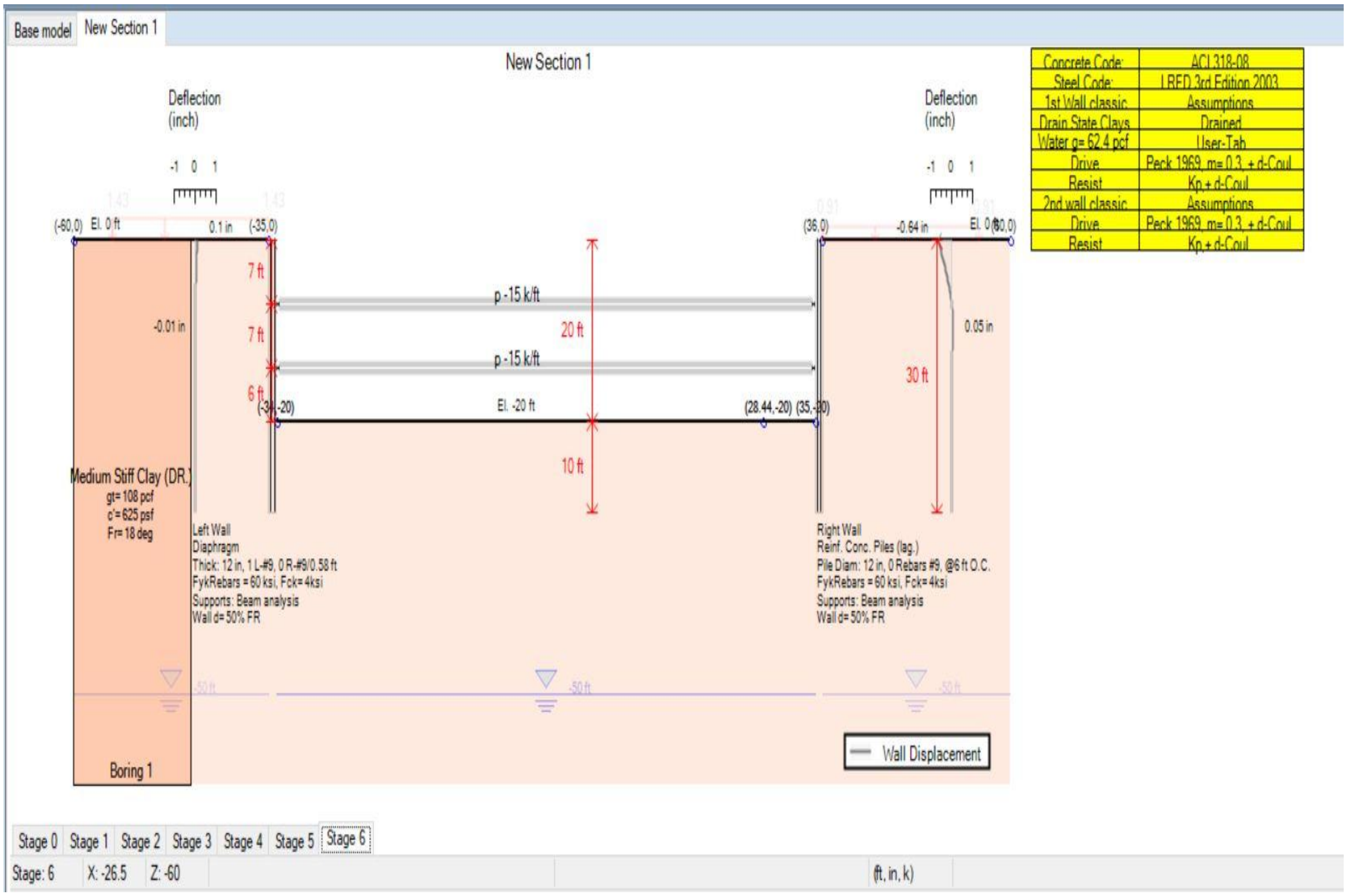


Figure 28: Software model for Diaphragm wall with struts.

(The least deformation was in this system i.e 0.1 inches maximum)

CHAPTER 6

COST ESTIMATION

6.1 Drilled Shafts

Market rate system (MRS) of Rawalpindi was used for estimating the quantities of material required and the cost that would be incurred for the construction of the support. The final cost calculated by adding cost of steel and concrete was approximately equal to 5.97 million Rs.

6.1.1 Concrete

The MRS gives the rate for a pile of a fixed diameter in Rupees per running foot. This is composite rate including the cost of labour and material. Rates were available for upto 2 ft dia., extrapolating the rate for 2.5 ft dia. pile, we got rate = 2700 Rs/ft. First we had to determine the no. of piles that were to used on either side by dividing the length by the spacing in between the piles, after that we multiplied that with the depth of the piles and the rate to get the total cost of concrete for piles.

No. of piles (one side) = 120/8 + 1 = 16.

Total length = (16 x 40) + (16 x 35) = 1200 ft.

Cost = 2700 x 1200 = 32,40,000 Rs.

6.1.2 Steel:

The composite rate of deformed bars = 59.45 Rs/ lb

Density of steel = 490 lbs/ft^3

Main reinforcement steel = 11#9 bars = 11 in^2 = 0.0764 ft^2

Total Depth of pile (5 storey side) = 40 ft

Volume of Steel in 1 pile = 0.0764 x 40 = 3.056 ft^3

Length of 1 stirrup = Dia. – 2(cover) = 24 in.

Spacing = 12 in.

No. of Stirrups per pile = (40x12)/12 +1 = 41

Total length = 41 x 2 = 82 ft

Cross sectional area of #3 bar = 0.11 in^2

Volume of steel = 82 x 0.11/144 = 0.063 ft^3/pile.

Total steel per pile = 3.056 + 0.063 = 3.12 ft^3

Weight of steel for 5 story building side = 490 x 3.12 = 1530 lbs.

Weight of steel for 3 story building side = 1530 x 35/40 = 1340 lbs.

Total steel weight = 1530 x 16 + 1340 x 16 = 45920 lbs.

Total steel cost = 45920 x 59.45 = 27,29,944 Rs.

6.2 Soldier Piles with Anchors

Mobilization and Demobilization of drilling and grouting equipment = 100,000 Rs.

Drilling per anchor = 35,000 Rs.

Preparation of tendon assembly = 1200 Rs.

Multistage grouting = 1500 Rs.

Stressing = 1000 Rs.

Plastic spacers = 500 Rs.

Admixture = 400 Rs.

PVC pipe = 1500 Rs.

Rubber tube, tape for end plug = 480 Rs.

Bearing plate, steel block and 5 wedges = 1680 Rs.

Channels = 9200 Rs.

Core Cutting = 2500 Rs.

41 ft of 3 strands of 270 ksi steel = 13500 Rs.

Total cost per anchor = 68,480 Rs.

For 2 levels = 68,480 x 2 = 1,36,920 Rs.

No. of piles on one side = (120/4) + 1= 31 Rs.

No. of piles on both sides = 31 x 2 = 62

Cost of anchors = 100,000 + (136920 x 62) = 85,89,040 Rs.

Cost of 2 pile dia. = 1890 Rs per foot

Total Length of piles = Depth x no. of piles = (28 x 62) = 1736 ft.

Cost of piles = (1736 x 1890) = 32,81,040 Rs.

Total Cost = 32,81,040 + 85,89,040 = 11,870,080 = 11.87 million.

6.3 Diaphragm wall with struts

The combined cost of diaphragm wall and struts was approximately equal to 30,840,000 Rs. Since the struts are dismantled and can be reused in other projects, therefore this value does not represent the cost in this project.

6.3.1 Diaphragm wall

The total cost of wall calculated was = 45,96,000. This included the cost of excavation, concrete and steel. The detail calculations have been shown below.

6.3.1.1 Concrete and Excavation

Excavation cost = 6.13 Rs/ft^3

Thickness = 1 ft, Depth = 30 ft, Length = 120 ft.

Volume for one side = 3600 ft^3

Volume for both sides = 7200 ft^3

Cost of excavation = 6.13 x 7200 = 44,000 Rs.

Cost of concrete = 229 Rs/ ft^3

Volume = 7200 ft^3

Cost = 229 x 7200 = 16,48,800 Rs.

6.3.1.2 Cost of Steel

Main Reinforcement = #9 @ 7” C/C

Horizontal Reinforcement = #4 @ 9” C/C

No. of main bars = 120 x 12/7 + 1 = 207

Total length = 30 x 207 = 6210 ft

Area of #9 bar = 1 in^2 = 0.00694 ft^2

Volume = 6210 x 0.00694 = 43.13 ft^3

No. of horizontal bars = 30x12/9 + 1 = 41 bars

Length = 120 x 41 = 4800 ft

Area of #4 bar = 0.2 in^2 = 0.00139 ft^2

Volume = 4800 x 0.00139 = 6.67 ft^3

Total Volume = 43.13 + 6.67 = 49.8 ft^3

Weight of Steel = 490 lbs/ft^3 x 49.8 ft^3 = 24,402 lbs.

Rate = 59.45 Rs/ lb.

Cost for one side = 59.45 x 24,402 = 14,51,675 Rs.

Cost for both sides = 29,03,350.

6.3.2 Cost of Struts

Fabrication cost of steel = 292.66 Rs/kg. Since struts can be reused in other projects, these do not reflect the cost that would be incurred for a single project. The total cost was equal to 2,64,44,000 Rs.

6.3.2.1 Horizontal struts

Weight of W14 x 132 = 132 lbs/ft

Length = 70 ft

No. of struts = 120/15 = 8, No. of levels = 2.

Total no. of struts = 8 x 2 = 16

Total Weight = 132 x 70 x 16 = 1,47,840 lbs = 67,200 kg

Cost = 1,96,66,752

6.3.2.2 Vertical Posts

HP 8x36 was used as a vertical post.

Total Depth = 25 ft, Spacing = 10 ft.

No. of posts in a row = 70/10 – 1 = 6

Rows = 8, Total no. of posts = 8 x 6 = 48

Weight = 36 x 25 x 48 = 43,200 lbs = 19636 kg

Cost = 292.67 x 19636 = 57,47,000 Rs.

6.3.2.3 Wales

W4x13 was used as a wale.

Length = 120 ft.

Levels = 2

Weight for both sides = 2 x 2 x 120 x 13 = 6240 lbs = 2836 kg.

Cost = 8,30,000

CHAPTER 7

CONCLUSION AND REFERENCES

7.1 Conclusion:

The main objective of this study was to determine which support system was most suitable in terms of economy and stability for our specific case. The strength parameters were C=625psf, φ=18 and γ=108 pcf while the soil was classified as stiff silty clay on the basis of SPT blows.

Three support systems were selected for the basis of this study namely, Drilled Shafts, Soldier Piles with Anchors and Diaphragm Wall with Struts. For the structural design of drilled shafts, LRFD 2012 AASHTO Bridge Design Manual was used while for the design of anchors and soldier piles FHWA Circular No.4 was taken as reference and finally diaphragm wall and internal struts were designed using guidelines from the book Deep Excavations by Chang-Yu Ou.

After the Design on paper, computations were run on the developed designs using DEEPXCAV software. For each support system, the maximum deformation was calculated by the software and it indicated whether the proposed support was safe or not by comparing with the allowed deformation. Drilled Shafts did not give favorable results as the deformation exceeded the allowable deflection. It also reinforced the instruction of NAVFAC that drilled shafts must not be used as support against lateral loading for excavations greater than 15 feet. Soldier piles and Anchors showed favorable results with a meager maximum deflection of 0.31 inches, it remained well within the allowable limits. Diaphragm wall and Struts proved to give the least deflection in our case and had superior structural stability with a negligible deflection of 0.1 inches.

According to Cost analysis drilled shafts benchmarked at 5.8 million rupees, the least in all three followed by soldier piles with anchors at 11.87 million rupees. Diaphragm walls and struts had the best stability factor but with a huge cost of 17.8 million rupees, although the struts could be reused for other projects and diaphragm wall becomes the part of the final structure, still it required a greater initial investment.

The recommended support system is Diaphragm wall and Struts as it most stable, although with a greater investment, the cost of diaphragm wall and struts not a true reflection for the cost in this project as the struts could be reused and diaphragm wall can be incorporated as a permanent part of the structure and acts as RCC retaining wall. For future studies to be conducted in this field, we would suggest using our study as reference, stabilize a similar excavation with similar soil properties using other means like Ground Freezing, Lime or Grout injection, Bacterial Injection, Secant or tangent walls etc.

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