DEEP EXCAVATIONS IN BUILT UP AREAS AND PILING TECHNIQUES

FINAL YEAR PROJECT

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SYNOPSIS

For deep excavations, a retention system is mandatory not only for the safety of surrounding structures, but also for safe and proper execution of construction work within the parameter. This document serves as a comprehensive evaluation and comparison of design procedures available for retention systems comprising of piles and anchors. The design procedure consists of two components to be worked out separately, the piles, and the anchorage system. The approaches of FHWA and Canadian manual have been used as a major reference to evaluate the design procedures, and hence to draw a comparison between the methods available. Detailed procedure is discussed for design method of anchors, and different methods for piles, including ; Brom's, Wange & Reese's, Brinch Hansen's, Evans & Duncan's and Reese & Matlock's method. To demonstrate the design, a case study has been discussed. The comparison has been made on the basis of results obtained by the application different methods on the case study.

In addition to the design considerations and procedures, general detail has also been discussed to highlight the necessity of retention systems and the key theory of excavation procedures and types of pile.

Chapter 1: INTRODUCTION

1.1. PURPOSE

The purpose of this project is to study the various methods of designing piles and anchors, and using this understanding to solve the problems in the case studies assigned to us. To study the stresses arising due to existing structures around the excavation, like buildings, roads etc, and implementing this knowledge to design adequate and reasonable piles and anchors.

1.2. BACKGROUND

During deep excavations, especially in built up areas, a large number of problems can be encountered. Therefore to eliminate the difficulties to carry out required excavations, the methods for retaining the soil from falling in have been extensively studied to achieve a reasonable solution.

The first use of anchors was for temporary support of excavation systems. The use of permanent ground anchors did not become common until the late 1970s and today, represent a common technique for earth retention and slope stabilization.

In design and construction conditions, anchored systems offer several economic and technical benefits.

These include:

- Unobstructed workspace for excavations
- Ability to withstand relatively large horizontal wall pressures without requiring a significant increase in wall cross section;
- Elimination of the need to provide temporary excavation support since an anchored wall can be incorporated into the permanent structure;
- Elimination of need for select backfill;
- Elimination of need for deep foundation support;
- Reduced construction time;

1.3. Scope

The purpose of this project is study various case studies and to come up with reasonable and logical solutions to the problems that may be encountered during the process. It includes the study of the stresses, due to the loads of the structures around the excavation, and stress distribution and the design of piles and anchors based on that stress distribution.

- Understanding of challenges expected during deep excavations.
- Causes of the expected problems.
- Solutions for safe and successful excavations in built up area.
- Learn about the stress distribution for strutted excavation in various types of soils.
- Design of micro piles.
- Design of anchors
- Recommendation of suitable approaches, economically and practically.

1.4. Area of application

The purpose is to keep the sides of the excavation stable. Essentially, excavations deeper than 1.5 m need stabilizing methods. The main area of application of this project is sub structures. The following are categorized in this category:

- Buildings requiring deep excavation.
- Basement parkings.
- Underpasses.
- Where excavation may cause damage to existing surrounding structure

CHAPTER 2: DEEP EXCAVATIONS

2.1. INTRODUCTION

An excavation is considered to be deep excavation if the depth is more than 6m (20 ft). Deep excavation is usually advanced by using earthmoving material. Vertical excavation of such great depths cannot stand without any support system during the construction period as stresses and pressure may lead to the collapse of the excavation. In this topic, the various techniques of deep excavation and providing the support systems, along with the causes of failures in the deep excavation shall be discussed.

The support system for the excavation is a critical aspect during the construction period of the project as the damages could be fatal in case of a potential failure.

2.2. DEEP EXCAVATION TECHNIQUES

Deep excavation is a procedure of various activities such as excavation, installation and construction of sheet pile and retaining walls etc. Different techniques have been developed to achieve this purpose. Various factors like size of excavation, ground conditions, economy etc. Based on these factors, the best method may be chosen.

However, there are two main methods of excavating deep soil, which further have more types.
All of these methods have been discussed below:

2.2.1. OPEN EXCAVATION USING SLOPING OR BENCHING

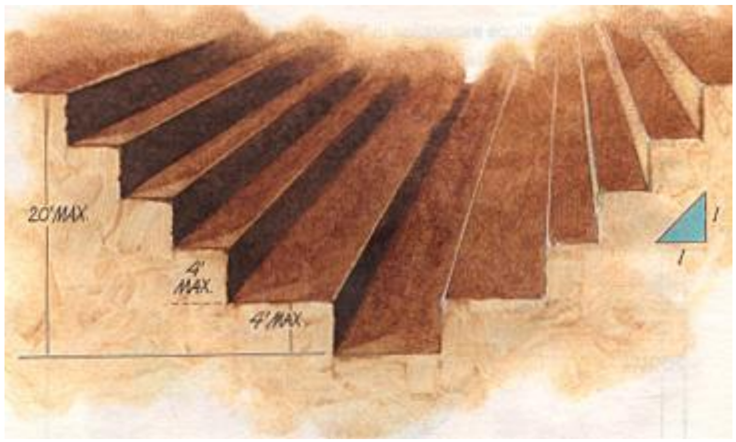
Excavation is done by earthmoving equipment and heavy machinery, with the sides being sloped or benched to provide support to the excavation and to prevent the collapse. The main advantage of this method is that unlimited excavation depths can be achieved by using it along with shoring.

However, the limitations of this method are:

- It requires large stockpiles and separate disposal facilities;
- It may affect surface features;

- And more importantly, it requires large areas to be worked upon because of the sloping.

Due to this reason, it is out of the scope of our study because we are provided with limited area where excavation has to be done.



2.2.2. EXCAVATION WITH BRACED SIDE WALLS

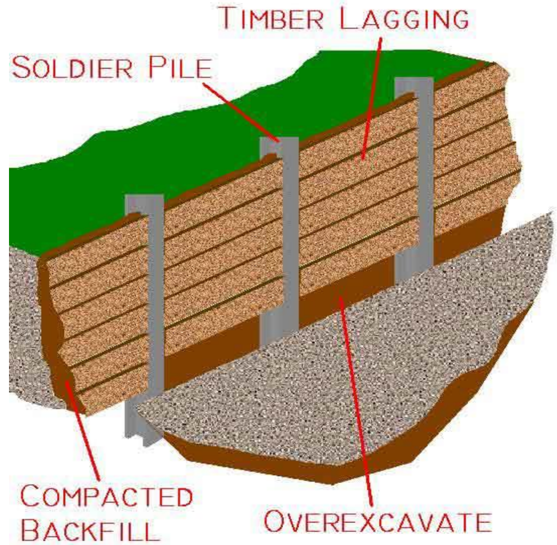
Braced excavation techniques are the most widely employed worldwide. Conventional equipment is employed in this method.

This method has been further classified into the following different types.

a) SHEET PILING

The maximum depth in this method that can be attained is 50 feet. Walls of the excavation are supported to prevent damages. These walls are constructed by inserting prefabricated sections into the ground, which provides structural resistance.

The limitations of this method are that the boulders and cobbles present in the soil may prevent the insertion of the wall. Moreover, wall support may be impractical at greater depths.



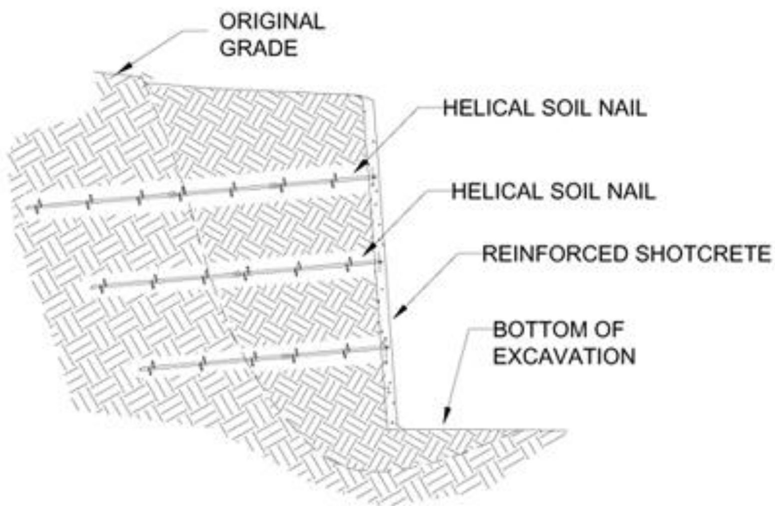
b) SOLDIER PILE AND LAGGING WALLS

The steel H-piles are inserted into the drilled holes in the soil at regular intervals by driving or placing them. Timber or steel lagging is placed between the piles to support the ground, while advancing the excavation. Conventional equipment is used. The maximum depth is about 100 feet and the H-piles are supported by anchors.

Loose material can make lagging difficult to insert in between the piles. Boulders and cobbles present in the soil make vertical control difficult.

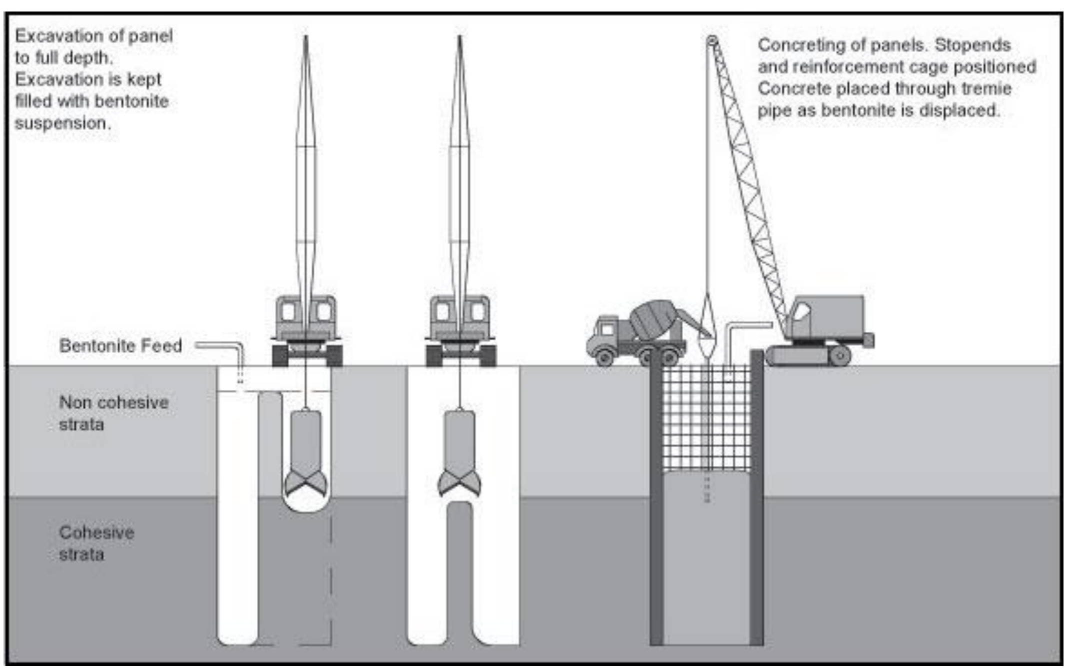
c) SOIL NAIL WALLS

Steel reinforcing bars are inserted into the shallow cuts made into the face of the soil at regular intervals. Wire mesh is applied on the face to support it and the depth is limited to about 35-40 feet. However, the limitations being that it should be applied in cohesive soil and in minimal water flow conditions.



d) DIAPHRAGM WALLS

Reinforced concrete wall is constructed in panels that are supported by bentonite slurry. The maximum depth that can be achieved is 200 feet, with the walls being supported by anchors. The limitations of this technique are that it employs highly specialized equipment and wide corridor (75 – 100 feet) is required along wall alignment.



2.3. Causes of failure in deep excavations

Following are the general causes of failure in the methods of braced excavations:

- Inadequate site investigation resulting in optimistic design approach.
- Lack of coordination of the designer and the constructor.
- Poor workmanship in site temporary works.
- Change in the loading due to natural conditions or phenomena.
- Lack of flexibility in the design in case of changed loading and lack of attention given to the consequences in case of changes in rock or soil conditions.
- Influence of deflections of the soil on the support system.
- Temporary plant loads overload the support structure.
- Special techniques such as diaphragm walling require special attention and inadequate attention may result in consequences

CHAPTER 3: PILES

3.1. GENERAL

Piles are a type of deep foundations generally recommended due to some common reasons such as; very large design loads, a poor soil at shallow depth, or site constraints.

A pile foundation usually consists of cylindrical structural member embedded to required depths. Piles are capped with a structural base known as pile cap which transfers the load from the super structure to the pile or a group of piles.

Piles can be designed to cater both vertical loads from the super structure and/or the lateral load in certain conditions. Where deep excavations are carried out, piling is often used to take the lateral loading from the pressure exerted by the surcharge acting near the site.

3.1.2. TYPES OF PILES

a) DRILLED PILES

Piles embedded by drilling the ground to the required depth are classified under drilled piles. They can be further classified as;

- **Driven Piles**

  Driven piles are usually pre-casted/pre-engineered before being driven or hammered into the ground. The piles can be of steel or precast concrete.

- **Bored Piles**

  A hole as per requirement of design is bored into the ground and the pile is then formed usually of reinforced concrete.

b) AUGER-CAST PILE

Auger-cast piles are cast-in-place, using a hollow stem auger or drill. The auger is drilled into the soil and then slowly extracted, removing the drilled soil as concrete or grout is pumped through the hollow stem.

In addition to these two main types, there are some ‘specialty’ piles. These piles are designed for different purposes according to requirement.

c) MICRO-PILES

Small sized pile foundations with a diameter of 60 mm to 200 mm (3 inches to 10 inches). Micro-piles are usually made out of high strength steel.

d) SHEET PILES

Sheet piling is a form of driven piling using thin interlocking sheets of steel to obtain a continuous barrier in the ground.

e) SOLDIER PILES

Concrete or steel piles spaced close together and covered with a horizontal timber lagging to provide a continuous wall.

3.1.3. CLASSIFICATION OF PILE TYPES

Classification by:	**Piles subgroup**
Installation	Driven, bored, auger-cast.
Displacement	High/Low displacement, no displacement.
Function	End bearing, Skin bearing, Lateral resistance, combination.
Capacity	High, medium or low.
Shape	Round, square, H-section.
Environment	Land, marine.
Inclination	Vertical, battered.
Length	Long, short.

3.1.4. APPLICATIONS OF PILES

Pile systems are used in many different structures and conditions. However the main uses can be narrowed to the following:

1. Deep Foundations
2. Retention in deep excavations
3. Dams and cofferdams

3.2. BASIC PRINCIPLES OF PILE DESIGN

3.2.1. GENERAL DESIGN CONSIDERATIONS

Introduction

For analysis of foundation pile, several assumptions are held affecting the accuracy of the results. The calculated results should always be checked using safety factor by the design engineer to ensure that the values are reasonable assumptions.

Generally, the proposed structure should be evaluated on the basis of the factors that affect the losses of lives and property. Therefore it is, to reduce the frequency of losses and reduce costs, and hence we should apply designer appropriate safety factors for the design. So the factors of safety depend upon proper functioning of the structure, assurance of the foundation parameters, sufficient analysis tools and construction controls. For the analysis and design of piles the designer must be aware of all the factors.

Failure

Failure to the structure and organization of the actual collapse or functional failure can be the result of excessive deviation, seismic load and premature deterioration because of environmental factors. Therefore, we must be aware of the design and not only of the safety factor against collapse, but also from the effects of settlement and vibration functionality.

Factor of safety

Uncertain parameters and design loads require a higher safety factor. Hydraulic structures mostly affected by these parameters, so designers should have a high level of assurance in the properties of the soil, and stack parameters analyzed. Therefore, high factor of safety should not be considered instead of minimizing the uncertainties. And structures that are less important, it is permissible to use a high factor of safety to make it economical.

Pile analysis and design

The kind of tests to be performed for foundation design is determined by the economy of structure and its significance. Following tests are further performed for analytical purposes and for the determination of type and degree of foundation exploration programs e.g. the pile test program, the settlement and seepage analyses and the analytical models for the pile and structure. Following criteria should be fulfilled for designing critical structure and foundation i.e. soil type, soil profile and its strength etc. For construction of large structures, the pile load test should be performed for design of its piles.

While designing the analytical model, the pile and structure should be designed while considering the structural significance in mind. The structural model should consider actual stiffness of the structure for the determination of correct load factors and design parameters.

3.2.2. EARTH PRESSURE EVALUATION FOR LATERALLY LOADED PILES

Introduction

Laterally applied load on any structure by the soil, usually in horizontal direction is called lateral earth pressure. So laterally loaded piles are the one on which such pressure is applied. The purpose of analyzing earth pressure on the piles is to determine the stiffness of a single pile against lateral load.

States of earth pressure

There are three states of earth pressure which are

a) Earth Pressure at Rest

b) Active Earth Pressure

c) Passive Earth Pressure

a) Earth pressure at rest

It is the pressure applied laterally by soil when it is in rest or stationary condition. In this condition the wall in front of the soil mass is rigid and does not move by the pressure exerted on the wall.

For rest condition the expression is,

$$\frac{\sigma h}{\sigma v} = \text{Constant} = K_o \qquad \text{(Eq. 3.1)}$$

Where K_O is called the coefficient of earth pressure at rest condition and given by

$$K_o = 1\text{-}\sin\Phi \qquad \text{(Eq. 3.2)}$$

b) Active earth pressure

Active earth pressure is the pressure exerted on wall by soil mass that causes the wall to move away from the soil mass. It is represented by **K_A** and determined by,

$$K_A = \frac{1-\sin\Phi}{1+\sin\Phi} \qquad \text{(Eq. 3.3)}$$

c) Passive earth pressure

In this state the wall moves towards the soil mass. The expression for the coefficient of passive earth pressure is given by,

$$K_P = \frac{1+\sin\Phi}{1-\sin\Phi} \qquad \text{(Eq. 3.4)}$$

Calculation of total lateral earth pressure

Initially we have to determine whether earth pressure is active or passive. If the pressure is active, then the soil pressure is determined by using equation,

$$\sigma_h' = K_a \sigma_v' - 2c\sqrt{K_a} \quad \text{(Eq. 3.5)}$$

The effective soil pressure is minimum in this case because soil is in active state which is lesser than passive state and rest state.

However, if the pressure is in passive state then effective stress is calculated by,

$$\sigma_h' = K_p \sigma_v' + 2c\sqrt{K_p} \quad \text{(Eq. 3.6)}$$

This equation gives the maximum stress value. Here **σ_h'** is effective horizontal stress , **σ_v'** is the effective vertical stress and **c** is the cohesion of soil

3.3. DESIGN OF PILES

3.3.1. BROM'S METHOD

The Brom's method is a relatively easy procedure to determine the lateral loads and pile deflections at ground surface. As this method ignores axial load on the pile, it is only suitable for smaller projects. This method calculates ultimate soil resistance against the lateral load and can be used to determine fixed or free headed pile condition in both purely cohesive and non-cohesive soils. For other soils, including mixed cohesive or non-cohesive this method is unsuitable.

Procedure

1) Calculation of the horizontal sub-grade reaction
For cohesive soils, the horizontal sub-grade reaction, K_h is given by:

$$\mathbf{K_h = \frac{n_1 n_2 q_u 80}{b}} \qquad \text{(Eq. 3.5)}$$

Where; q_u = Unconfined compressive strength (kPa)
b = diameter of pile (m)
n_1 and n_2 = Empirical coefficients and their values are found using the table 3.1;

VALUES OF COEFFICIENTS n_1 AND n_2 FOR COHESIVE SOILS	
Unconfined Compressive Strength, q_u, (kPa)	n_1
Less than 48 kPa	0.32
48 to 191 kPa	0.36
More than 191 kPa	0.40
Pile Material	n_2
Steel	1.00
Concrete	1.15
Wood	1.30

Figure 3. 1

For cohesion-less soils, Kh is noted from the table 3.2;

VALUES OF K_h FOR COHESIONLESS SOILS		
	K_h, (kN/m^3)	
Soil Density	Above Ground Water	Below Ground Water
Loose	1900	1086
Medium	8143	5429
Dense	17644	10857

Figure 3. 2

The value of Kh is adjusted for cyclic loading and creep in cohesion-less and cohesive soils respectively. We use a factor of 1/3 in both cases.

2) Determine if the pile is long or short.

   a) Calculate the dimensionless length factors :

Dimensionless length factor for cohesive soils is given by; β x L, where:

$$\boldsymbol{\beta} = \sqrt[4]{\frac{K_h b}{4EI}} \qquad \text{(Eq. 3.5.a)}$$

Dimensionless length factor for non-cohesive soils is given by; η x L, where:

$$\boldsymbol{\eta} = \sqrt[5]{\frac{K_h}{EI}} \qquad \text{(Eq. 3.5.b)}$$

L is the embedded length of the pile and EI is the material elastic stiffness.

b) For cohesive soil :

β x L > 2.25 = Long pile
βxL < 2.25 = Short pile

For non-cohesive soil:

ηxL > 4 = Long pile
ηxL < 2 = Short pile
2 <ηxL < 4 = Intermediate pile

3) Determine soil parameter

a) Rankine passive pressure (for non-cohesive soils)
b) Unit weight of the soil
c) Cohesion (for cohesive soils), ½ of unconfined compressive strength.

4) Calculate dimensionless factor by: L/b and use the respective graph of dimensionless factor against dimensionless load factor for free or fixed headed and long or short piles.

5) Find the ultimate load Q_u from the dimensionless load factor calculated from graph.

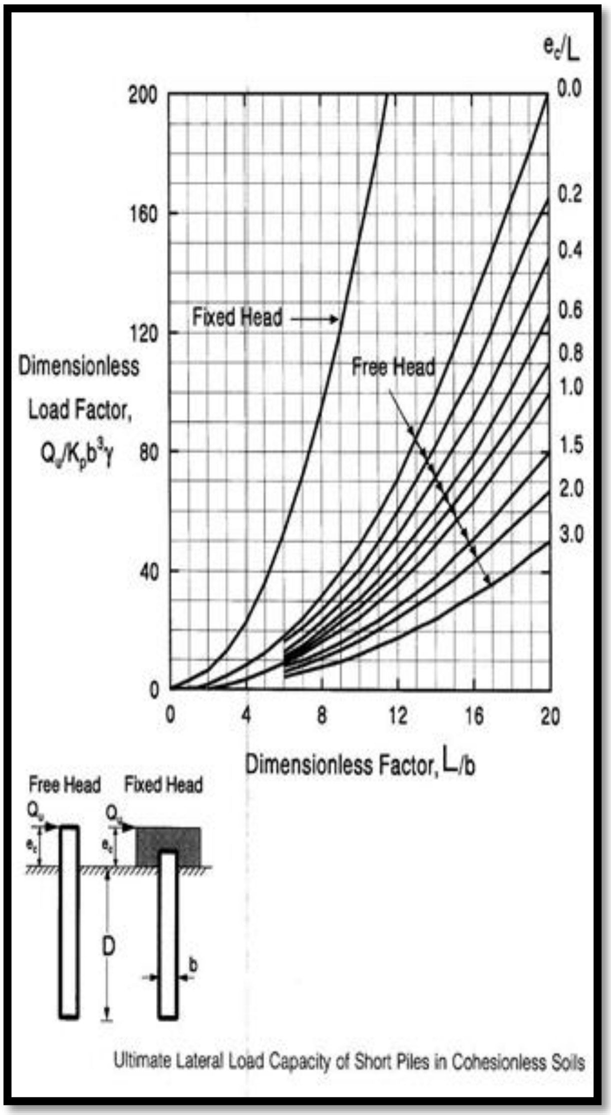


Figure 3. 3

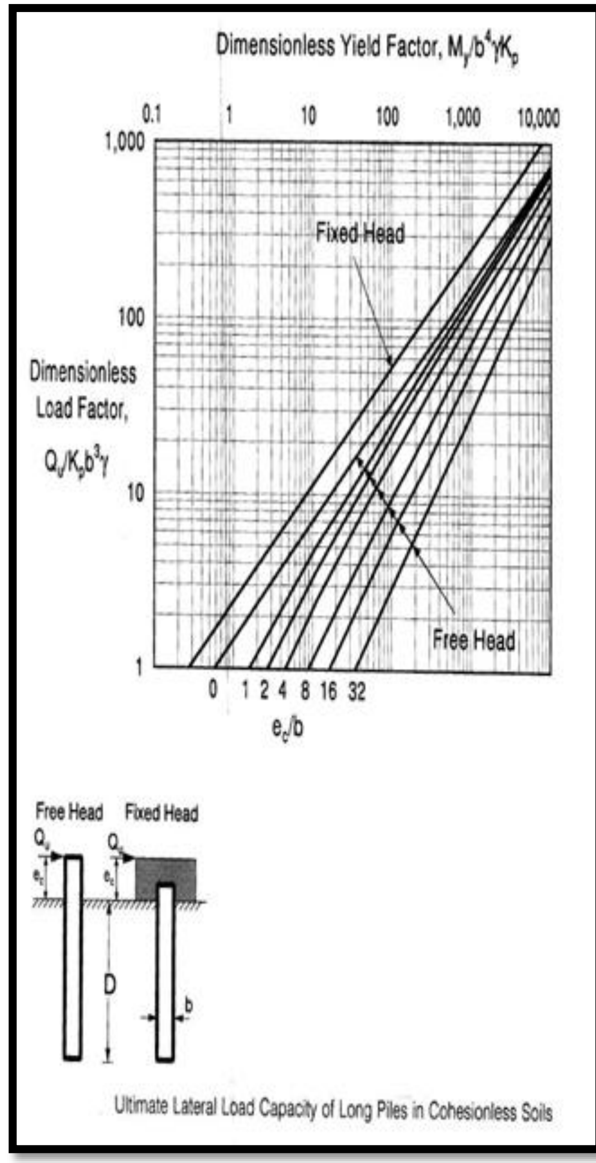


Figure 3. 4

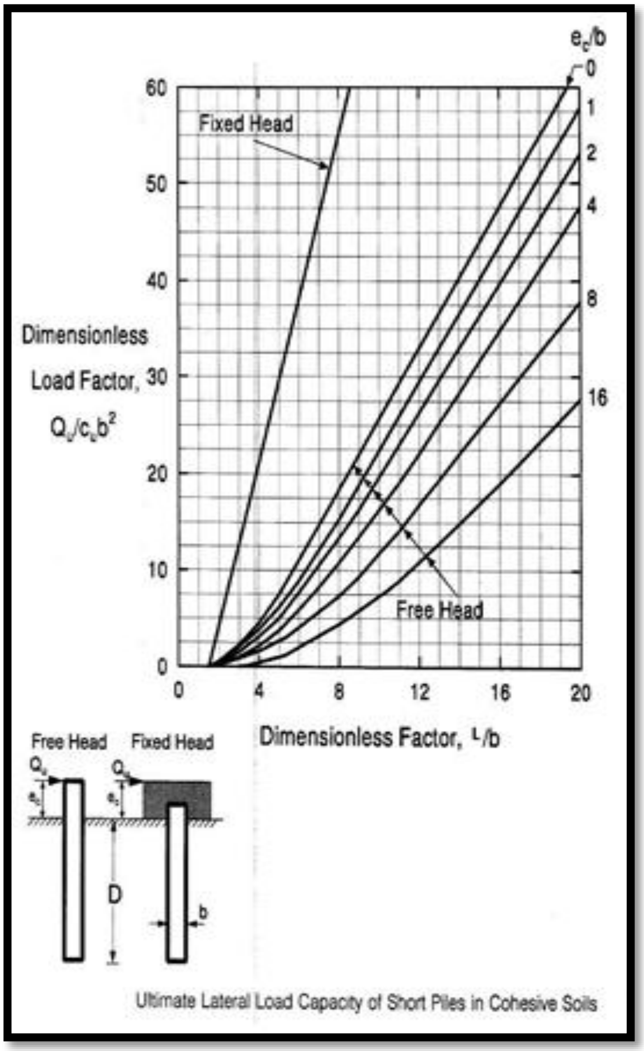


Figure 3. 5

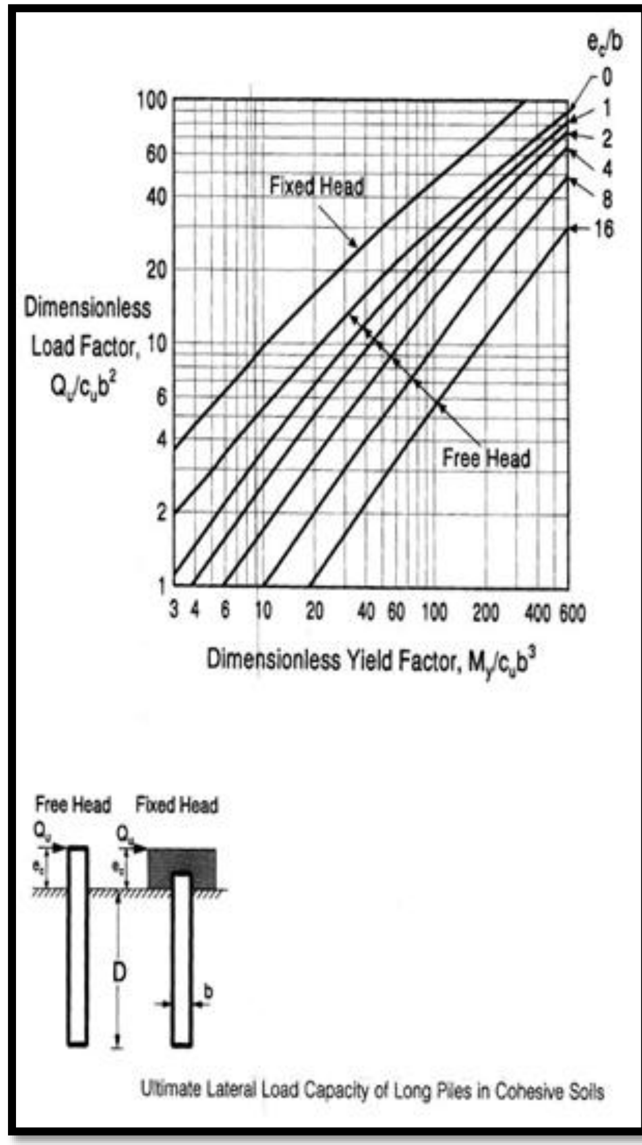


Figure 3.6

3.3.2. WANGE AND REESE METHOD

The Wange and Reese method considers three potential failure mechanisms;

1. Wedge failure for a single pile,
2. Overlapping or intersecting wedge failure for grouped piles,
3. Plastic flow of soil around the pile(s).

During the design, the minimum passive resistance by these three mechanisms is taken to be the ultimate passive resistance.

Procedure

1) Determine the passive force

   a) For non-cohesive soils, the passive force is given by:

$$F_p = \left(\frac{K_o.d.tan\Phi.sin\beta}{3.\tan(\beta-\Phi).cos\alpha} + \frac{tan\beta}{\tan(\beta-\Phi)}\left(\frac{b}{2}+\frac{d}{3}.tan\beta.\tan\alpha\right) + \frac{K_o.d.tan\beta}{3}(tan\Phi.sin\beta - \tan\alpha\right)$$

(Eq. 3.6)

   b) For cohesive soils:

$$\mathrm{F_p} = \mathrm{S_u db(\tan\theta + (1+K)\cot\theta) + \frac{1}{2}\gamma b D^2 + S_u D^2 \sec\theta} \quad \text{(Eq. 3.7)}$$

2) Wedge failure mechanism for a single pile:

   a) In non-cohesive soils, the ultimate soil resistance is given by differentiating the passive force:

$$P_{pu} = \gamma d\left(\frac{K_o.d.tan\Phi.sin\beta}{\tan(\beta-\Phi).cos\alpha} + \frac{tan\beta}{\tan(\beta-\Phi)}(b + d.tan\beta.\tan\alpha) + K_o.d.tan\beta(tan\Phi.sin\beta - \tan\alpha\right)$$

(Eq. 3.8)

b) For cohesive soils, the passive force is differentiated with the angle between the pile and the inclined plane of the wedge assumed to be 45^{o}, and no reduction factor is consideration for average un-drained shear strength of the soil to give:

$$\mathbf{P_{pu} = 2S_u b + \gamma bd + 2.83S_u d} \qquad \text{(Eq. 3.9)}$$

3) Wedge failure mechanism or intersecting piles (Pile groups) :

a) For non-cohesive soils, the intersection depth of the failure wedges is given by :

$$\mathbf{d_i = d - \frac{S_c}{2\tan\alpha\tan\beta}} \qquad \text{(Eq. 3.10)}$$

If this depth of the pile group is greater than the wedge intersection depth, the adjacent piles are not affected i.e wedges do not intersect. However, in case where the depth of piles lie within the wedge intersection depth, the ultimate passive resistance is given by:

$$P_{pu} = \gamma d\left(\frac{K_o.d.\tan\Phi.\sin\beta}{\tan(\beta-\Phi)}\left(\frac{1}{\cos\alpha}-1\right)+\frac{d\tan\beta\tan\alpha}{\tan(\beta-\Phi)}-K_o d\frac{\sin^2\beta}{\cos\beta}\tan\Phi(\tan\alpha+1)\right)$$

(Eq. 3.11)

b) In cohesive soils, critical spacing is determined. This is the spacing between adjacent piles, at which the behaviour changes from single to group piles :

$$\mathbf{S_{cr} = \frac{2.83S_u d}{d\gamma + 6S_u}} \qquad \text{(Eq. 3.12)}$$

If the critical spacing is smaller or equal to the actual pile spacing, group behavior governs and the ultimate passive resistance is given by:

$$\mathbf{P_{pu} = 2S_u(b + s_c) + \gamma d(b + S_c) + S_u s_c} \qquad \text{(Eq. 3.13)}$$

However if the critical spacing becomes zero (continuous wall):

$$\mathbf{P_{pu} = 11S_u b} \qquad \text{(Eq. 3.14)}$$

4) Plastic flow of soil around the pile(s):

Soil presence between the piles aid in resisting the applied lateral load. This is due to the plastic flowing action of soil in the spacing between the piles.

a) For non-cohesive soils :

$$\mathbf{P_{pu} = K_A b\gamma d.\tan^8\beta + K_o\gamma d.\tan\Phi.\tan^4\beta} \qquad \text{(Eq. 3.15)}$$

However this plastic flow resistance cannot exceed :

$$\mathbf{P_{pu} = K_p\ d\gamma(b + s_c)} \qquad \text{(Eq. 3.16)}$$

Hence:

$$\mathbf{K_p\ d\gamma(b + s_c) > K_A b\gamma d.\tan^8\beta + K_o\gamma d.\tan\Phi.\tan^4\beta}$$

b) For cohesive soils :

$$\mathbf{P_{pu} = 2S_u + d\gamma} \qquad \text{(Eq. 3.17)}$$

However this plastic flow resistance cannot exceed;

$$\mathbf{P_{pu} = (2S_u + d\gamma)(b + s_c)} \qquad \text{(Eq. 3.18)}$$

Hence:

$$(2S_u + d\gamma)(b + s_c) > 2S_u + d\gamma$$

Description of various symbols used in this method is given below in table 3.1;

Symbol	Description	Symbol	Description
Φ	Drained friction angle of soil	β	$45 + \varphi/2$
γ	total unit weight	α	φ for dense sands, $\varphi/3$ to $\varphi/2$ for loose sands
b	pile diameter or width	d	depth of bottom of pile
K_o	at rest earth pressure coefficient	S_c	clear spacing between adjacent piles
S_u	average un-drained shear strength	d_i	depth of intersection
D	toe depth		

Table 3. 1

3.3.3. REESE AND METLOCK METHOD

This method primarily includes finding the elastic deflections of the pile which should satisfy the following:

- The predicted non-linear soil deformation relations.
- The elastic bending properties of the pile
- The stiffness of the upper structure – pile connection.

A set of load-deflection curves are used for this method to evaluate the deflection for a given loading.

Procedure:

1) Choose a trial depth of the pile below the ground level. This depth is denoted as ‘T’.

2) Select different depths that are to be checked. These depths are denoted as ‘X’
3) Determine the ratio of selected depths to the trial depth:

$$\mathbf{Z = \frac{X}{T}} \qquad \text{(Eq. 3.19)}$$

4) Calculate the deflections ‘y’ by :

$$\mathbf{y = A_y \frac{P_t T^3}{EI} + B_y \frac{M_t T^2}{EI}} \qquad \text{(Eq. 3.20)}$$

Where;
P_t is applied lateral load.
A_y and B_y are functions of ‘z’ and are taken from following table,

5) For the calculated deflections, load ‘P’ is noted from load-deflection curves.

Z	A_y	B_Y
0.0	2.435	1.623
0.1	2.273	1.453
0.2	2.112	1.293
0.3	1.952	1.143
0.4	1.796	1.003
0.5	1.644	0.873
0.6	1.495	0.752
0.7	1.353	0.642
0.8	1.216	0.540
0.9	1.086	0.448
1.0	0.962	0.364
1.2	0.738	0.223
1.4	0.544	0.112
1.6	0.381	0.029
1.8	0.247	-0.030
2.0	0.142	-0.070
3.0	-0.075	-0.089
4.0	-0.050	-0.028
5.0	-0.009	0.000

Table 3. 2

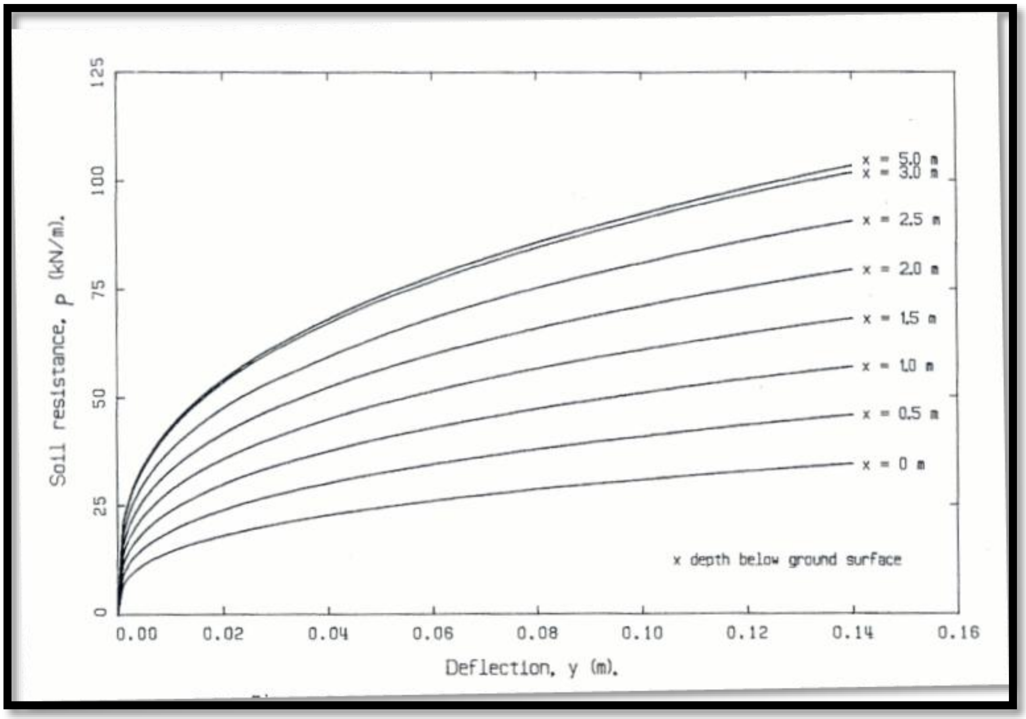
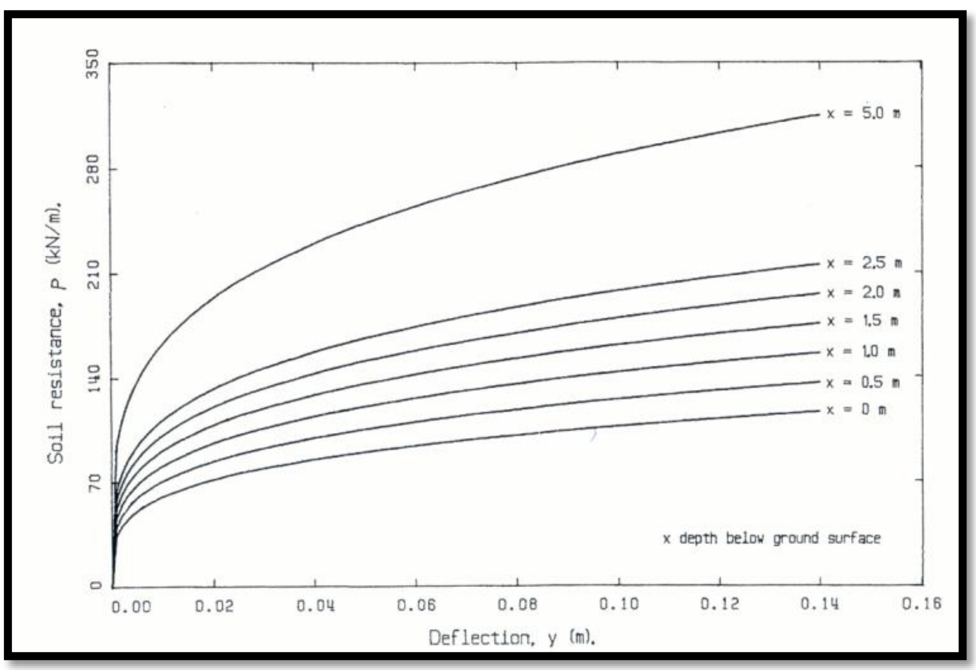


Figure 3. 6

Deflection curve for normally consolidated clay

Figure 3. 7



Deflection Curve for Over-consolidated Clay

6) Now, calculate the secant modulus of soil by:

$$\mathbf{E_s = \frac{P}{Y}} \tag{Eq. 3.21}$$

7) A graph of E_s vs X is obtained. The gradient of the graph is determined and is denoted 'k'.

8) $T_{obtained}$ is calculated by :

$$\mathbf{T_{obt} = \frac{EI}{k}} \tag{Eq. 3.22}$$

9) Now Trial depth T and $T_{obtained}$ are compared. If they are significantly different, the procedure is repeated for a different value of trial depth.

3.3.4. BRINCH HANSEN'S METHOD

Procedure

1) Determine the Resultant Earth Pressure:

The earth pressure is calculated at different depths it is sum of

- Pressure caused by the vertical effective over burden and
- That caused by the cohesion of the material

$$P_z = P_o K_q + cK_c \tag{Eq. 3.23}$$

Where;
K_q = resultant earth pressure coefficient caused by vertical effective over burden pressure
K_c = resultant earth pressure coefficient caused by cohesion
C = Cohesion
p_o = effective vertical over burden pressure (KN/m^2)

a) Value of K_q at any arbitrary depth can be found by :

$$K_q = \frac{K_q^o + K_q^\infty \alpha_q \frac{Z}{D}}{1 + \alpha_q \frac{Z}{D}} \tag{Eq. 3.24}$$

b) Values of K_q^o and K_q^∞ are obtained by using graph and for α_q:

$$\alpha_q = \frac{K_q^o}{K_q^\infty - K_q^o} \times \frac{K_o \sin \Phi}{\sin(\frac{\pi}{4} + \frac{\Phi}{2})} \tag{Eq. 3.25}$$

For value of K_q^o and K_q^{∞} consult the figure 3.9

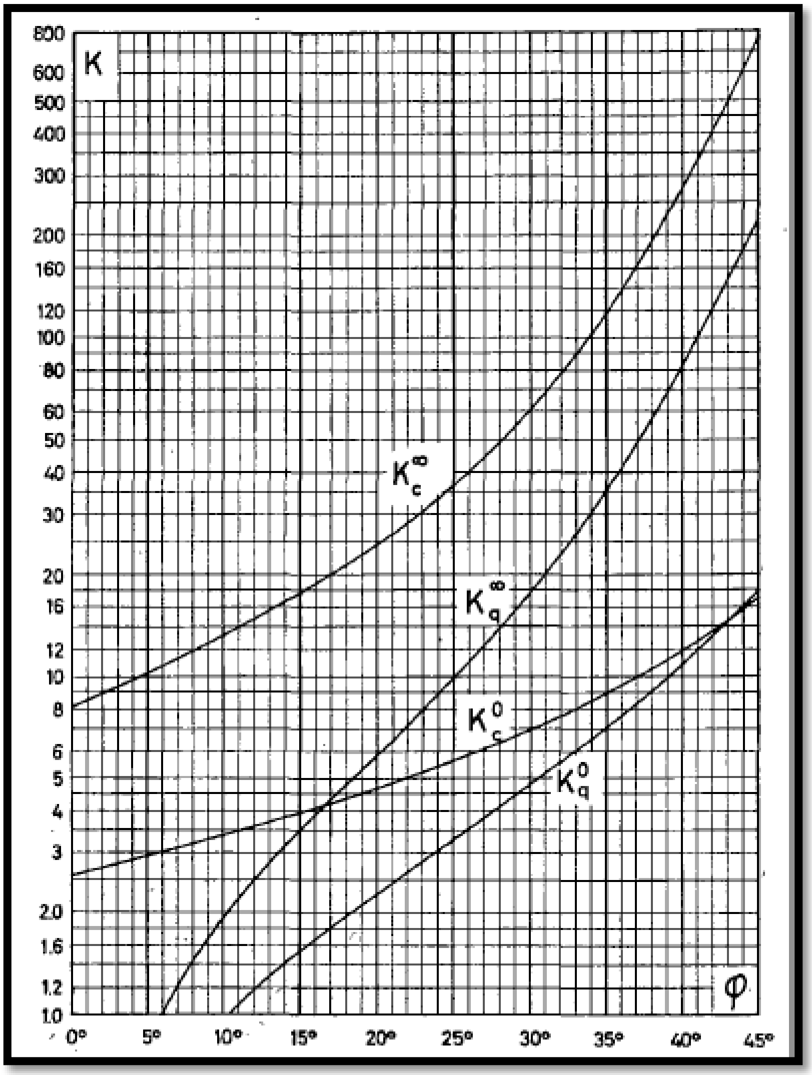


Figure 3. 8

c) Now for value of K_c :

$$\mathbf{K_c} = \frac{K_c^o + K_c^{\infty}\alpha_c\frac{Z}{D}}{1+\alpha_c\frac{Z}{D}} \qquad \text{(Eq. 3.26)}$$

d) Values of K_c^o and K_c^∞ are obtained by using graph and for α_c

$$\boldsymbol{\alpha_c} = \frac{2K_c^o}{K_c^\infty - K_c^o}\ \sin\left(\frac{\pi}{4} + \frac{\Phi}{2}\right) \qquad \text{(Eq. 3.27)}$$

2) Now assume point of rotation at different depths and take moment about the point of application of load P_{ult}.As suggested by Brom's the resistance caused by top 2m is neglected. The sign of moment values above and below the assumed point of rotation will be opposite to each other.Select the point of rotation with value closer to zero.

3) After selecting point of rotation, calculate the value of P_{ult} in KN/m by taking moment about point of rotation.

4) Now calculate total load in KN by multiplying P_{ult} with diameter of pile.

5) Now for

$$\mathbf{P_{allowable}} = \frac{P_{ult}}{F.O.S} \qquad \text{(Eq. 3.28)}$$

While Pile efficiency is calculated by

$$\textbf{Pile Efficiency} = \mathbf{P_{allowable}}\ \mathbf{x}\ \boldsymbol{\eta} \qquad \text{(Eq. 3.29)}$$

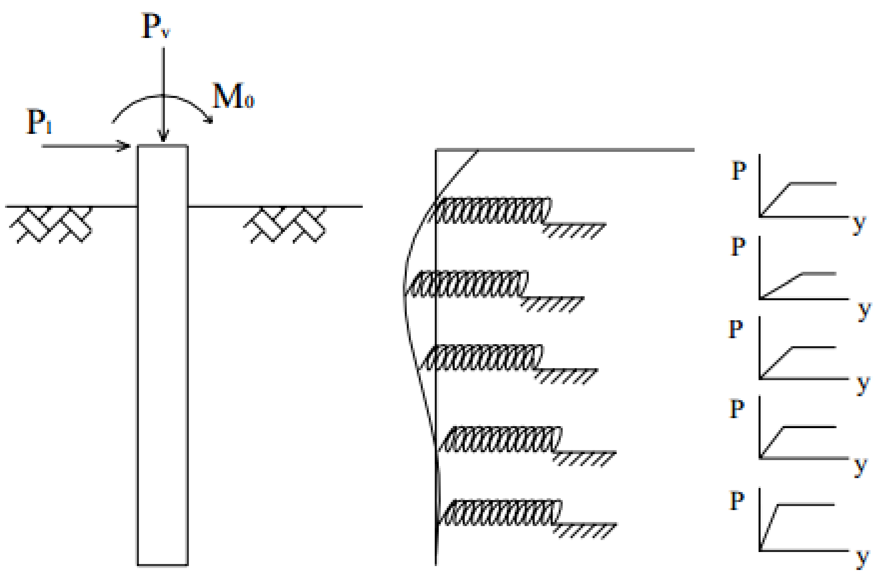
3.3.5. P-Y CURVES

The p-y method is a method of analysing the ability of deep foundations to resist loads applied in the lateral direction.Based on the sub grade reaction approach, the soil pressure, p (kN/m^2) is correlated to the lateral deformation as follows (Matlock, 1970):

$$\mathbf{p} = \mathbf{k_{ho}y}$$

Where, k_{ho} is the coefficient of sub grade reaction, y is the deflection of the spring, and p is the force applied to the spring.

In the sub grade reaction approach for analysis of laterally loaded piles and shafts, the soil is replaced by a series of springs attached to an element of foundation. P-y curves are defined at various depths as a function of soil type and geometry.



The P-y curves are different for different soil types.

Procedure

P-y curves from measured data can be evaluated using principles of statics. Two sets of equations are used to establish the governing differential equation based on geometry and structural element.

The constitutive equation for the pile is defined as:

$$\mathbf{M = EI\varphi = EI\left(\frac{d^2y}{dz^2}\right)} \qquad \text{(Eq. 3.30)}$$

Where, M = bending moment at depth, z;

E = modulus of elasticity of the pile;

I = moment of inertia of the pile around the centroidal axis of the pile section;

φ = pile curvature;

y = pile lateral displacement; and,

z = depth

Based on assumption that the pile is embedded in a linear elastic medium, a number of methods have been developed to predict the lateral pile head stiffness. The behavior of the pile in elastic medium on the differential equation for the beam column on a foundation, given by Hetenyi (1946):

$$\mathbf{(d^2M/dx^2) + P\,(d^2y/dx^2) - p = 0} \qquad \text{(Eq. 3.31)}$$

P = Axial Load on the pile.

The unit soil resistance 'p' varies with the depth of the pile, and can be expressed in the P-y curves. Combining the above two equations, it can be expressed as:

$$\mathbf{(EI)_p\,(d^4y/dx^4)\; +\; P(d^2y/dx^2) - E_s y\; =\; 0} \qquad \text{(Eq. 3.32)}$$

Here,

$P = E_s y$;

E_s = soil stiffness;

$(EI)_p$ = pile stiffness

3.3.6. EVANS AND DUNCAN METHOD

Assume pile parameters

- Width or Diameter 'D'
- Young's Modulus 'E_P'
- Moment of Inertia 'I_p'

1) Calculate ratio of moment of inertia of the shaft to the moment of the inertia of a solid, unreinforced cross section 'R_i'

$$R_i = \frac{I_p}{I_{solid}} \quad \text{(Eq. 3.33)}$$

2) Soil properties:
   - For Clays----------- average un-drained shear strength 'S_U'
   - For Sands----------- average angle of internal friction $'\Phi'$,γ'

3) Calculation of characteristic load P_c

   - For Clays

$$P_c = 7.34D^2(E_pR_i)\left(\frac{S_u}{EpRi}\right)^{0.683} \quad \text{(Eq. 3.34)}$$

   - For Sands

$$P_c = 7.34D^2(E_pR_i)\left(\frac{\gamma' D\phi K_p}{EpRi}\right)^{0.57} \quad \text{(Eq. 3.35)}$$

4) Determine the load ratio $\frac{P_s}{P_c}$

5) Calculate $\frac{Y_s}{D}$ using figures

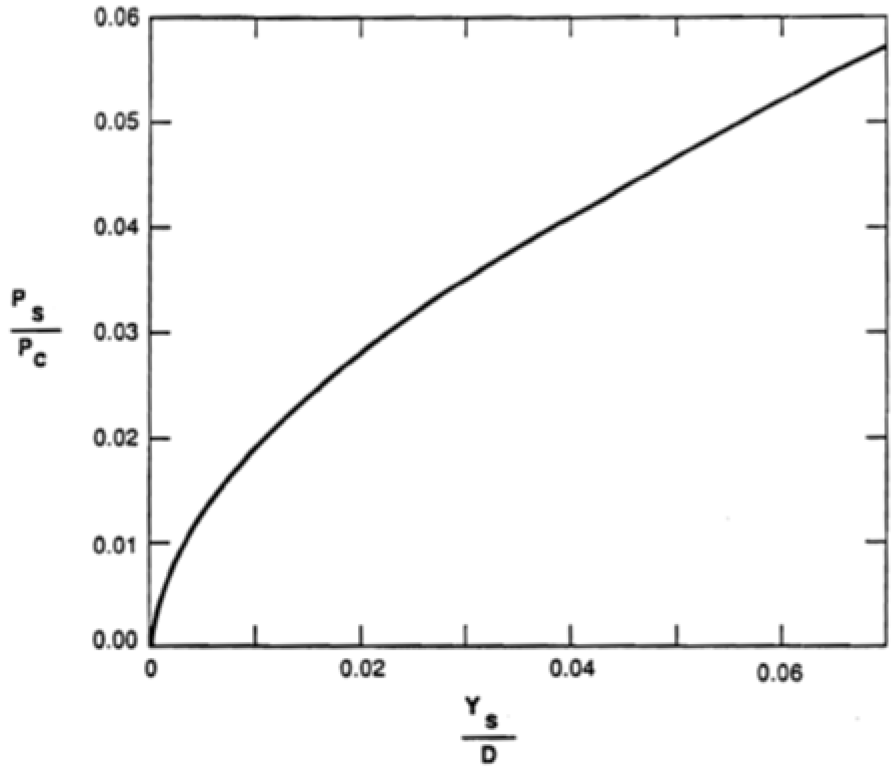


Figure 3. 9

Lateral load vs deflection for fix-headed piles in clay

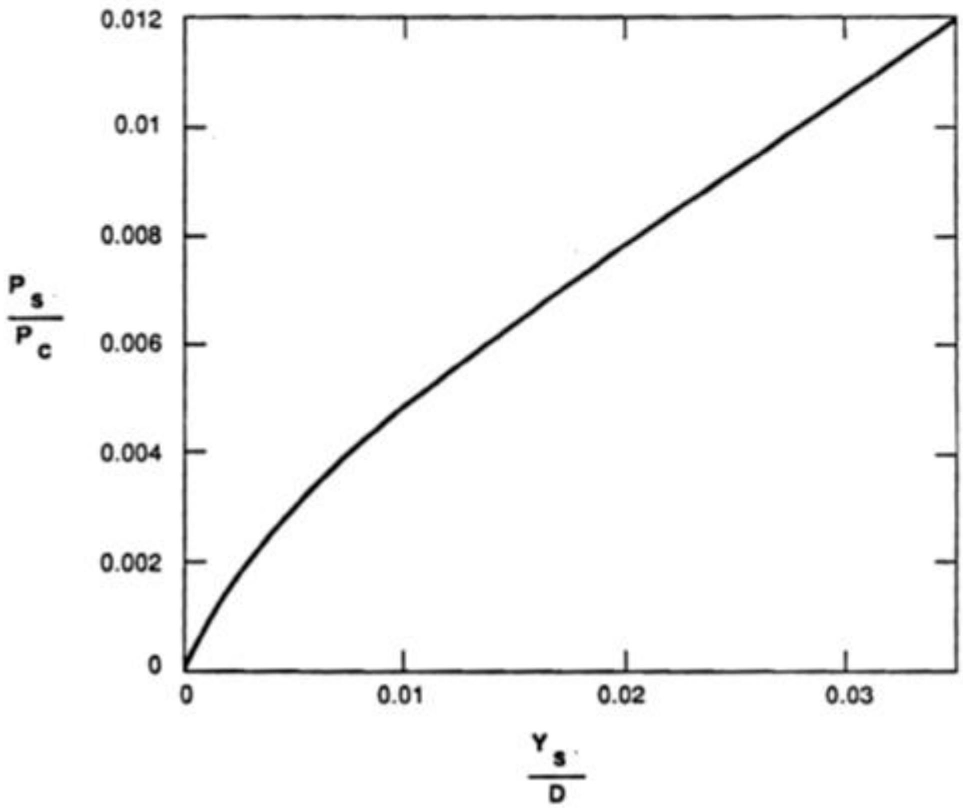


Figure 3. 10

Lateral load vs deflection for fix-headed piles in sands

6) Find lateral deflection ' Y_s' using:

$$Y_s = D\left(\frac{Y_s}{D}\right) \qquad \text{(Eq. 3.36)}$$

7) If the pile is free headed then moment is also required in the analysis. This characteristic moment is given by:

- For Clays

$$M_c = 3.86D^3(E_pR_i)\left(\frac{S_u}{E_pR_i}\right)^{0.46} \qquad \text{(Eq. 3.37)}$$

- For Sands

$$M_c = 1.33D^3(E_pR_i)\left(\frac{\gamma' D\phi K_p}{E_pR_i}\right)^{0.4} \qquad \text{(Eq. 3.38)}$$

8) Determine the moment ratio $\frac{M_s}{M_c}$

9) Calculate $\frac{Y_{sm}}{D}$ using figures 3.12 and 3.13

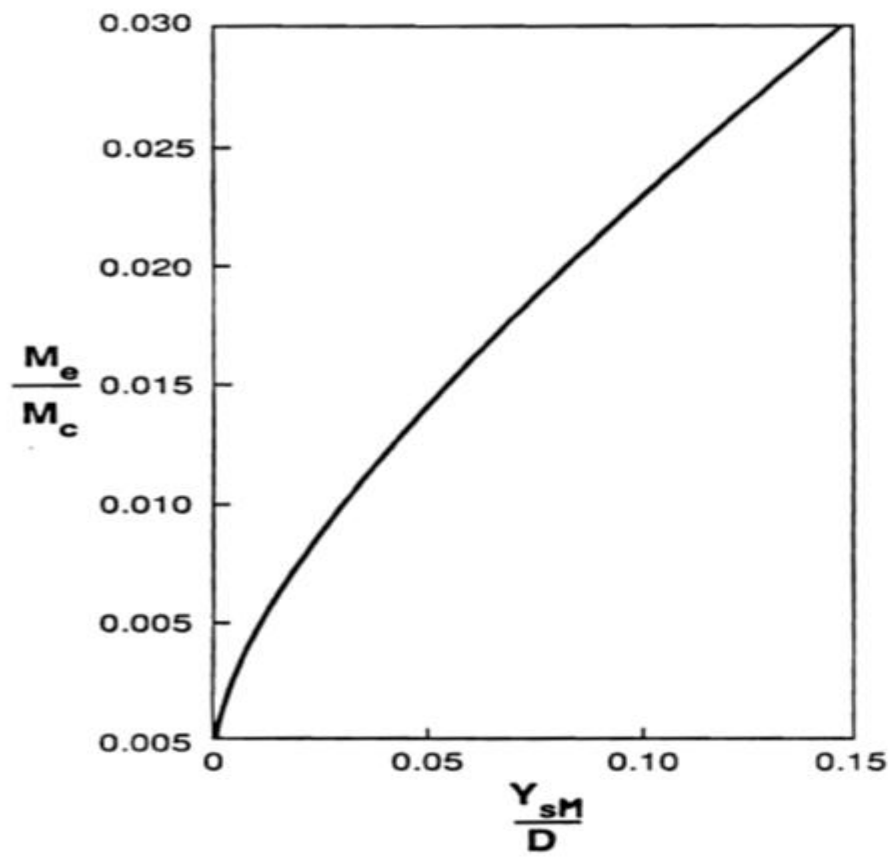


Figure 3. 11

Moment vs Deflection for Free-head piles in clay

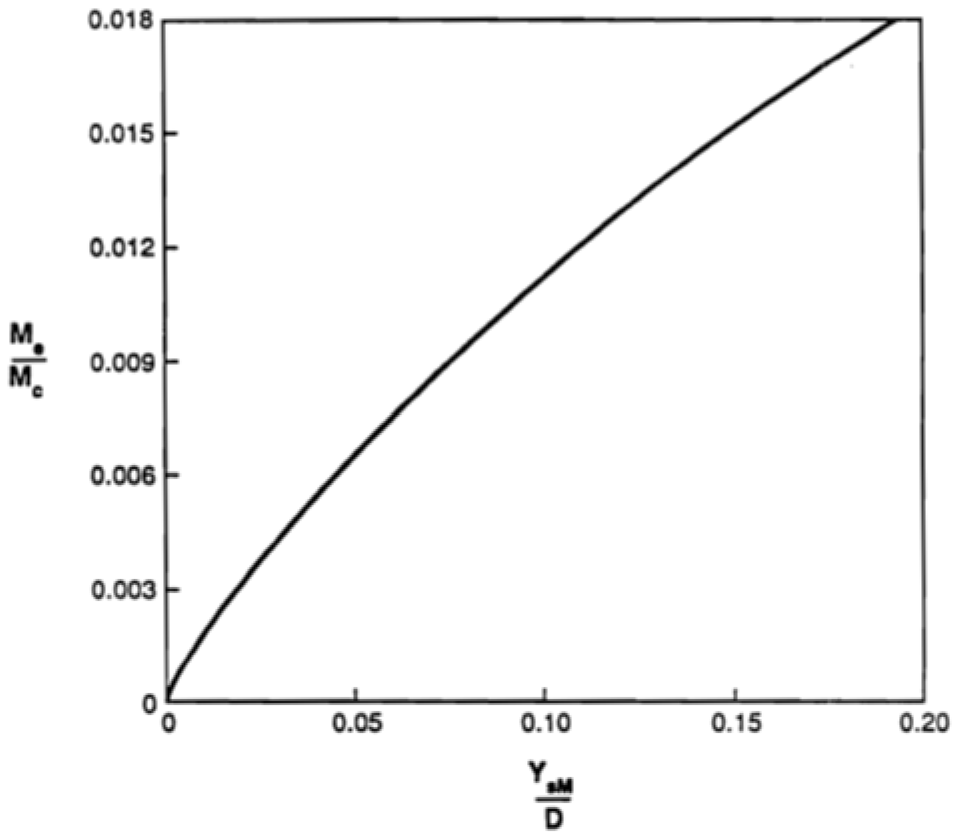


Figure 3. 12

Moment vs Deflection for Free-head piles in sands

10) Find deflection '$\mathbf{Y_{sm}}$' using:

$$Y_{sm} = D\left(\frac{Y_{sm}}{D}\right) \qquad \text{(Eq. 3.39)}$$

CHAPTER 4: ANCHORS

4.1. GENERAL

Anchors are structural elements that are installed to transmit the tensile loads applied in them to competent soil mass.

4.1.1. COMPONENTS OF GROUND ANCHORS

A ground anchor can be divided into following components

a) Anchorage
b) Un-Bonded Length
c) Bond Length
d) Sheath
e) Anchor Grout

a) ANCHORAGE

Anchorage is a combined system of anchor head, bearing plate and trumpet that is capable of transmitting pre-stressing force from pre-stressing steel (bar or strand) to ground surface or supported structure. So anchorage includes

- Anchor head
- Bearing plate
- Trumpet

b) UN-BONDED LENGTH

The portion of pre-stressing that elongates freely with in elastic limit and transfer the resistive force to the structure.

c) BOND LENGTH

The length of pre-stressing steel bonded with the grout. This length transmits the load to the ground. Bond length is always located behind the critical failure surface.

d) SHEATH

Sheath is a smooth or corrugated pipe or tube which provides corrosion protection to the pre-stressing steel in un-bonded length.

e) ANCHOR GROUT

Grout is a mixture that is based on Portland cement. Grout transfers the load from the tendon to the ground. Tendon is the portion of complete ground anchor that consists of pre-stressing steel and sheathing.

4.1.2. TYPES OF GROUND ANCHORS

TYPE (A)

STRAIGHT SHAFT GRAVITY-GROUTED GROUND ANCHORS

These anchors are installed in rocks and very stiff to hard cohesive soil deposits. Rotary drilling and hollow-stem auger are the methods used for installing these types of anchors. Gravity displacement (tremie) methods are used to grout these anchors in a straight shaft borehole. Depending on stability borehole can cased or uncased.

TYPE (B)

STRAIGHT SHAFT PRESSURE-GROUTED GROUND ANCHORS

These are useful for coarse granular soils and weak fissured rocks. This type of anchors is also used for fine grained cohesion-less soils. Grout is injected in bond zone under pressure greater than 0.35 MPa. Grout is injected till the time when entire bonded length is grouted. Pull out resistance is more than gravity grouted anchors.

TYPE (C)

POST-GROUTED GROUND ANCHORS

In this type of anchors the body of gravity grouted anchors is increased by delayed multiple injections of grout. Delay time is one or two days. Post-grouting is done using a sealed grout tube installed with the tendon. This tube has the check valves in the bond zone. These check valves are used to inject grout whenever it is needed.

TYPE (D)

UNDER-REAMED ANCHOR:

This type of anchors is used in firm to hard cohesive soil deposits. These anchors consist of gravity grouted boreholes including a series of under-reams. The resistance is provided through side shear as well as by end bearing. Cleaning and forming of under-reams is done carefully.

4.1.3. APPLICATION OF ANCHORS

1. HIGHWAY RETAINING WALLS

Anchored walls are used for construction of grade separated depressed roadways, roadways widening and roadway realignment. The gravity excavation wall is expensive than a permanent anchored wall because of the reasons that temporary excavation support, deep foundation and backfill is required for gravity excavation walls. Anchored walls may also be used for construction of bridge abutments.

2. SLOPE AND LANDSLIDE STABILIZATION

To stabilize landslide and slopes ground anchors are used in combination with horizontal beams, concrete blocks and walls. Ground anchors when used to stabilize the soil mass above the slip surface provide large force to make it stable. This force can be considerably larger than that provided by gravity walls. Beams or blocks are selected keeping in mind cost and their maintenance.

3. TIE-DOWN STRUCTURES

Permanent ground anchors may be used to provide resistance to vertical uplift forces. Vertical uplift forces may be generated by hydrostatic and overturning forces. Using anchors to resist uplift forces reduces the volume of concrete slab, excavation and dewatering. Disadvantages include constructing a water tight connections variation of stress in slab.

4.2. BASIC PRICIPLES OF ANCHOR DESIGN

4.2.1. GENERAL DESIGN CONSIDERATION

Anchors are installed to transmit the loads applied on them to competent soil mass. Anchors usually transmit the forces caused by soil, surcharge, and water to the soil mass which is at appropriate distance from the potential failure zone. This potential failure zone is usually adjacent to the excavation in equilibrium.

The depth up to which the anchors must be installed is based on the deepest potential failure zone so that an acceptable factor of safety is achieved.

4.2.2. FAILURE MECHANISMS OF ANCHORED SYSTEMS

Failure mechanisms are generally due to excessive static loading of an anchor. Excessive loading can be related to surcharge, construction of adjacent structures, tension placed in anchor, excavation sequence or combination of these factors. Various modes of failures involved in failure mechanism of ground anchors are as follow,

- Failure of steel tendon
- Failure of ground mass
- Failure of ground grout bond
- Failure of grout tendon bond

4.2.3. SELECTION OF SOIL PARAMETERS FOR DESIGN

Determination of soil parameters is required to find the shear strength of soil which in turn is necessary to find the earth pressure acting on a wall, stability of anchored system (external), and axial and lateral capacity of embeded portion. Different types of shear strength is required for different types of soils and we will discuss them one by one here.

Granular Soils (Drained Shear Strength)

For granular soils the friction angle φ for drained effective stress is used to find the drained shear strength. This friction angle is determined by using SPT and CPT.

Normally Consolidated Clay (Un-Drained Shear Strength)

Un-drained shear strength is found by using CPT (in-situ) and different laboratory methods. The preferable method for laboratory testing is consolidated un-drained tri-axial testing with pore pressure measurements. Un-drained shear strength is not considered as the fundamental property of soil. Therefore it is affected by method of testing, rate of loading, initial stress state, boundary conditions etc. as a result of this the calculated un-drained shear strength may be different depending upon the method of testing. The designer should choose the appropriate method to find value of un-drained shear strength which is closer to the actual.

Over Consolidated Clays (Un-Drained Shear Strength)

In clay soils due to the mobilization of frictional shearing resistance the soil mass attempts to expand. Over-consolidated clays are usually fissured and due to this reason at the level of discontinuities these types of soils allow relatively rapid local drainage. Therefore, in over-consolidated clays it is almost impossible to define the duration during which the enhanced un-drained shear strength can be assumed to apply. So in these types of soils it is recommended to perform design analysis in terms of drained, effective stress parameters.

Over Consolidated Clays (Drained Shear Strength)

First of all designer should decide which level of strength (peak, fully softened or residual) will be used for anchored system. Fully softened strength is determined by using tri-axial compression test with pore-pressure measurements. Moreover fully softened strength is conservative drained shear strength for analysis of anchored walls.

Values of residual strength are used when failure surface already exist within the clay. For such conditions it is assumed that strength has reduced to a residual value due to sufficiently large deformations.

4.2.4. EARTH PRESSURES

Following three types of earth pressures are considered for the design of anchored system,

1. Earth pressure at rest
2. Active Earth Pressure
3. Passive Earth Pressure

Details of these types have already been mentioned in chapter 3 (section 3.2.2).

Earth pressures are developed due to the retained soil masses, surcharge loads, and ground motions caused by earthquake etc.

Some simple assumptions are made about active and passive earth pressure based upon theoretical analysis that transforms complex processes into simple techniques. These assumptions are based on following factors,

- Soil stiffness and strength properties
- Wall flexibility
- Soil interface friction
- Mode of wall movement
- Horizontal pre-stress in the ground

4.3. DESIGN OF ANCHORED SYSTEM

In this chapter our concentration is based on the design of permanently anchored soldier beam and lagging walls. It is suggested that the engineer, however, should ensure that the specific components and combinations of components used for the anchored system are consistent with all performance requirements.

4.3.1. EVALUATION OF EARTH PRESSURES

In this section our discussion is based on the evaluation of earth pressure for wall design. The factors like magnitude and distribution of lateral wall deformations govern distribution of earth pressure that develops on an anchored wall. Moreover soil shear strength, wall stiffness, anchor inclination, vertical spacing of the anchors, and anchor lock-off loads directly influence the wall deformation pattern and the earth pressure acting on these types of soils.

The use of apparent earth pressure, sliding wedge type, and the limit equilibrium calculations are the various methods that can be used for evaluating earth pressure for anchored walls.
For design of these systems, theoretical active earth pressure diagrams using either Rankine or Coulomb analysis method can be used.

Dwelling deeper into Terzaghi and peck's diagram, it is explained that it is rectangular or trapezoidal in shape and based on following factors,

- It is assumed that the excavation is greater than 6m deep and relatively wide. Moreover wall movements are assumed to be large enough so that full value of soil shear strength may be mobilized.
- For clays ground water is assumed blow the base of excavation and for clays its position is not of much concern.
- Homogenous soil mass is assumed and behaviour of soil during shearing is assumed to be drained for sands and un-drained for clays, i.e. only short term loadings are considered.
- These loading diagrams are applicable only to the exposed portion of the wall and not to the wall embedded below the bottom of the excavation.

The pressure envelopes are different for sands, stiff-hard fissured clays and soft to medium clays. A different diagram for each of these types is recommended and has been explained in a lot of detail.

For sands, the earth pressure diagram is rectangular and for the given value of K_A the maximum earth pressure ordinate is

$$\mathbf{p = 0.65\, K_A \gamma H} \qquad \text{(Eq. 4.0)}$$

Where K_A is the active earth pressure coefficient and is given by

$$\mathbf{K_A = \tan^2(45 - \varphi/2)} \qquad \text{(Eq. 4.1)}$$

There are two conditions in reference to Stiff to Hard Fissured Clays those are temporary conditions and permanent conditions. In temporary conditions, the most important factors that affect or influence the Earth pressures are degree of fissuring or jointing in the clay and the potential reduction in strength with time. The strength may not necessarily be the shear strength of the intact clay. However, in permanent conditions, earth pressures associated with long-term drained conditions for excavations in stiff to hard fissured clays may be greater than those computed based on envelopes for temporary conditions. The range of maximum earth pressure is from $0.2\gamma H$ to $0.4\gamma H$.

Terzaghi and Peck diagram has been used to evaluate apparent earth pressures for design of temporary walls in soft to medium clays. For these type of soils it is required that a competent layer for forming the anchor bond zone should be within a reasonable depth below the excavation.

4.3.2. DESIGN OF ANCHORS

In this section we will discuss the general procedure adopted to design ground anchors using the criteria presented by Terzaghi and Peck. Here we will discuss the method to calculate the apparent earth pressure, earth pressure due to surcharge load, total horizontal load acting on each anchor, bonded and un-bonded length and anchor capacity.

Before installing ground anchors they are tested for loads, during testing the anchors are loaded about 133 percent of actual load they are going to carry after installation.Most of the calculations are carried out using the tributary area method.

Procedure

The step wise procedure for anchors design is mentioned below,

1. Location of critical potential failure surface

The purpose of finding the location of critical potential failure surface is important so that the transfer of load to "no-load" zone can be avoided. For that purpose anchors must be installed sufficiently behind the critical potential failure surface. No-load zone is also called the un-bonded length.

For cohesion-less soils the critical potential failure surface extends up from the corner of excavation and sloped at an angle of $45^o+\varphi/2$ from horizontal. The expression for finding the horizontal force is given by

$$\mathbf{0.65\{\tan^2(45-\varphi/2)\}\gamma H^2} \qquad \text{(Eq. 4.2)}$$

2. Calculation of earth pressure

The earth pressure is calculated due to the apparent earth pressure and depends upon unit weight and angle of friction of soil and height of wall and earth pressure caused by the surcharge or overburden.

The expression for apparent earth pressure is given by

$$\mathbf{P_e = \frac{0.65\{\tan^2(45-\varphi/2)\}\gamma H^2}{H-\frac{H1}{3}-\frac{H3}{3}}} \qquad \text{(Eq. 4.3)}$$

While earth pressure using surcharge load can be calculated using the following expression

$$\mathbf{P_s = K_A q_s} \qquad \text{(Eq. 4.4)}$$

Where $K_A$is the coefficient of earth pressure and q_s is the surcharge load acting on the wall. In case when the surcharge load is due to traffic lanes and these traffic lanes are located within half the wall height behind the wall then AASHTO (1996) recommends that a surcharge pressure equal to 0.6m of soil above the wall be included in the calculation of lateral earth pressure against the wall.

3. Horizontal anchor loads

Tributary area method is used to calculate the horizontal load acting on each anchor, in most of cases the general expression for all the anchors is same expect the top one as it is located at a different distant with respect to others.

The expression for the horizontal load on the top anchor T_{H1}

is given by

$$\mathbf{T_{H1} = (\tfrac{2}{3}H_1 + \tfrac{H2}{2})P_e + (H_1 + \tfrac{H2}{2})P_S} \qquad \text{(Eq. 4.5)}$$

While the horizontal load acting on lower anchors can be calculated using the general expression given as,

$$\mathbf{T_{Hn} = (\tfrac{Hn}{2} + \tfrac{23}{48}H_{n+1})P_e + (\tfrac{Hn}{2} + \tfrac{Hn+1}{2})P_S} \qquad \text{(Eq. 4.6)}$$

4. Wall bending moment

Wall bending moments are calculated between top of excavation and upper anchor level (M_1) , between the upper anchor and the second one (M_2) and so on. Maximum value is selected among all the values and designated as M_{max},

Expression for calculating M_1 is given as,

$$\mathbf{M_1} = \frac{13}{54}\mathbf{H_1}^2\mathbf{P_e} + \mathbf{P_s}\ \mathbf{H_1}\frac{\mathbf{H1}}{2} \qquad \text{(Eq. 4.7)}$$

While values of bending moments for remaining anchors can be calculated using the general expression

$$\mathbf{M_n} = \frac{1}{10}(\mathbf{H_n})^2(\mathbf{P_e} + \mathbf{P_s}) \qquad \text{(Eq. 4.8)}$$

Choose the maximum value of bending moment.

5. Reaction force

Tributary area method is used to calculate the reaction force resisted by the sub-grade. Reaction force is assumed to act at the base of excavation.

$$\mathbf{R} = \left(\frac{3}{16}\boldsymbol{H_{n+1}}\right)\mathbf{P_e} + \left(\frac{\boldsymbol{H_{n+1}}}{2}\right)\mathbf{P_S} \qquad \text{(Eq. 4.9)}$$

6. Anchor design load

To calculate the load for which anchors are designed following expressions are used. We will select the maximum value of design load.

For upper most anchor the expression is

$$\mathbf{DL_1} = \frac{\mathrm{T}\boldsymbol{H_1 XS}}{\boldsymbol{COS\theta}} \qquad \text{(Eq. 4.10)}$$

While for the rest of anchors we use general expression,

$$\mathbf{DL_n} = \frac{\mathrm{T}\boldsymbol{H_n XS}}{\boldsymbol{COS\theta}} \qquad \text{(Eq. 4.11)}$$

S = centre to centre spacing of soldier beams

θ= inclination angle.

7. Design of un-bonded length

Values of Un-bonded length are different for bar anchors and strands anchors. For Bar Anchors the minimum un-bonded length is greater of either 3m or distance from the wall to a location of 2m beyond the critical failure surface.

While for strands Anchors the minimum un-bonded length is greater of either 4.5m or the distance from the wall to a location of 2m beyond the critical failure surface.

Following figure 4.1 can be used for better understanding and calculation of un-bonded length,

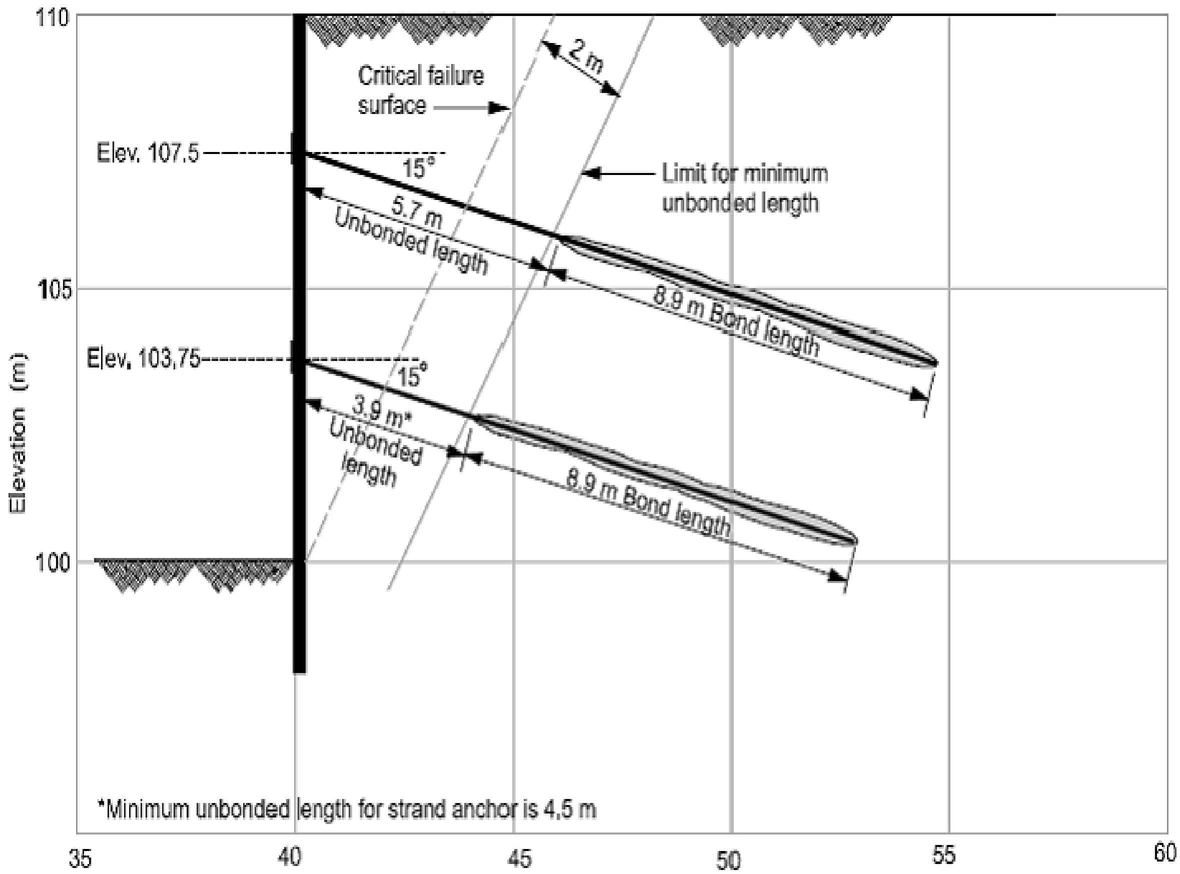


Figure 4. 1

8. Anchor capacity

Maximum load that anchor can carry is calculated using

$$\text{Allowable anchor capacity} = \frac{\{\boldsymbol{loadtransferrate}\}\boldsymbol{XBondLength}\ (\boldsymbol{assumed})}{\{\boldsymbol{F.O.S.}\}} \qquad \text{(Eq. 4.12)}$$

Anchor Capacity should be greater than the maximum design load.

A F.O.S. of 2 is assumed and any appropriate value of bond length is selected. Load transfer rate for different types of soils is selected using the following table 4.1;

Soil Type	Relative Density/consistency (SPT Range)	Estimated Ultimate Transfer Load (KN/m)	Estimated Ultimate Transfer Load (Kip/ft)
Sand and Gravel	Loose (4-10) Medium Dense (11-30) Dense (31-50)	145 220 290	10 15 20
Sand	Loose (4-10) Medium Dense (11-30) Dense (31-50)	100 145 190	7 10 13
Sand and Silt	Loose (4-10) Medium Dense (11-30) Dense (31-50)	70 100 130	5 7 9
Silt-Clay mixture with low plasticity or fine micaceous sand or silt mixture	Stiff (10-20) Hard (21-40)	30 60	2 4

Table 4. 1

9. Maximum bond length

The expression for maximum bond length is given as

$$\text{Maximum bond length} = \frac{[\boldsymbol{max.designload}][\boldsymbol{F.O.S.}]}{\boldsymbol{LoadTransferRate}} \qquad \text{(Eq. 4.13)}$$

CHAPTER 5: **CASE STUDIES**

5.1. INTRODUCTION

In this chapter we will discuss the design procedure and solution of different case study sites. These sites includes

1. Design Excavation Support system for Haly Towers DHA, Lahore.
2. Design of Ground Anchor System for Federal Courts, Lahore.
3. Design of Laterally Loaded Pile at Jhika Gali, Murree.

5.1.1. DESIGN EXCAVATION SUPPORT SYSTEM FOR HALY TOWERS DHA, LAHORE

DHA shopping Mall is under construction at 103-R, DHA Phase II, Lahore. In order to carry out excavation down to 52 ft. required for the construction of basements, an excavation retaining system comprising periphery cast-in-situ piles and ground anchors has been proposed.

. The given site has surcharge load due to traffic lanes on three sides and structural surcharge load on one side. So for 52 ft. deep excavation depth two separate anchor systems are proposed. Each system includes two rows of anchors.




HITACHI
CONDRILL


Design of Ground Anchors

Two types of ground anchor system (different both for building and traffic lanes) have been proposed and we will discuss them separately. Top row of ground anchors is installed at a depth of 3.05 m (10 ft) from NSL while the second row of ground anchors is at a depth of 9.45 m (31 ft) from NSL

Anchor System with Traffic Surcharge

Design results for ground anchors is given in following tables, as mentioned earlier top anchor is installed at a depth of 3.05 m (10 ft) from NSL while distance between top and bottom anchor is 6.40 m (21 ft). Surcharge load due to traffic lanes is 11.48 KN/m^2 (0.24 ksf).

Soil Properties			
Unit weight of soil	18.83		KN/m3
Friction angle	30.00	0.52	
Spacing between piles	1.22		m
Inclination angle of anchors	15.00	0.26	

For anchors,		
Excavation depth	15.85	m
Number of anchors	2.00	
Depth of first anchor from ground level	3.05	m
Spacing of anchors	6.40	m
Surcharge load from adjacent structures	11.48	KN/m2
Surcharge load from traffic	0.00	

The value of earth pressure due to retained soil mass P_e and due to traffic surcharge load P_s is calculated using equations (4.3) and (4.4) and results are shown below

Apparent Earth Pressure		
P_e	80.70	KN/m2
Earth Pressure due to Surcharge		
P_s	3.83	KN/m2

Consequently, the anchor loads, moments and the final capacity is calculated to find the bonded length. To find their values equations (4.5 to 4.11) have been used.

Anchor No.	Horizontal Anchor Load (KN/m)	Moment between Anchors (KN-m)	Design Anchor Load (KN)
1.00	446.27	198.54	563.65
2.00	530.24	346.24	669.71
		Max Design Load (KN)	669.71

Reaction Force (KN/m)	109.09
Load transfer rate (KN/m)	100.00
F.O.S	2.00
Estimated bond length (m)	13.39
Assume bond length (m)	15.00
Allowable anchor capacity for assumed bond length	750.00
Final bond length (m)	15.00
Unbonded length (m)	5.00
Total length (m)	20.00

Design Summary

Anchor Design load Capacity = 750 KN

Unbonded Length = 5 m

Bonded Length = 15 m

Angle of Anchor Installation = 15°

Spacing between Adjacent Piles = 1.22 m (4 ft)

Anchor System with Structural Surcharge

Total surcharge load due to adjacent building is 71.78 KN/m^2 (1.5 ksf). Design procedure is given in the following tables

Soil Properties			
Unit weight of soil	18.83		KN/m3
Friction angle	30.00	0.52	
Spacing between piles	1.22		m
Inclination angle of anchors	15.00	0.26	

For anchors,		
Excavation depth	15.85	m
Number of anchors	2.00	
Depth of first anchor from ground level	3.05	m
Spacing of anchors	6.40	m
Surcharge load from adjacent structures	71.78	KN/m2
Surcharge load from traffic	0.00	

The value of earth pressures due to retained soil mass $\mathbf{P_e}$ and due to building surcharge load $\mathbf{P_s}$ is calculated using equations (4.3) and (4.4). Results are shown below

Apparent Earth Pressure		
P_e	80.70	KN/m2

Earth Pressure due to Surcharge		
P_s	23.93	KN/m2

Consequently, the anchor loads, moments and the final capacity is calculated to find the bonded length. (use equations 4.5 to 4.11).

Anchor No.	Horizontal Anchor Load (KN/m)	Moment between Anchors (KN-m)	Design Anchor Load (KN)
1.00	571.89	292.03	722.32
2.00	658.88	428.57	832.19
		Max Design Load (KN)	832.19

Reaction Force (KN/m)	173.41
Load transfer rate (KN/m)	100.00
F.O.S	2.00
Estimated bond length (m)	16.64
Assume bond length (m)	18.00

Allowable anchor capacity for assumed bond length (KN)	900.00

Final bond length (m)	18.00
Unbonded length (m)	5.00
Total length (m)	23.00

Design Summary

Anchor Design load Capacity = 900 KN

Unbonded Length = 5 m

Bonded Length = 18 m

Angle of Anchor Installation = 15^{o}

Spacing between Adjacent Piles = 1.22 m (4 ft)

Design of Piles

Piles of given sites are designed using various methods of pile designing and we will discuss their results one by one.

Brinch Hansen's Method

Design of Piles installed along the road side

Total pile length (L) is 26.85 m. Pile and soil properties are given in following tables

Soil Properties		
Cohesion (C)	0	KPa
Friction Angle (ɸ)	30	Deg/Rad
Unit Weight (ɣ)	18.83	KN/m^3
Surcharge (P)	4.67	KN/m

Pile Properties		
Diameter (D)	0.61	m
Depth (Z)	10	m

Where Depth (Z) is the embedment depth of the pile and and P is surcharge pressure due to traffic lanes.

Now using the figure (3.8) find the values of coefficient of cohesion and coefficient of surcharge at the surface and at bottom of pile against angle of friction.

Enter Values From Graph	
Coeff. Cohesion at surface (Kc')	7
Coeff. Cohesion at depth (Kc")	60
Coeff. Surcharge at surface (Kq')	4.8
Coeff. Surcharge at depth (Kq")	18

Consequently calculate the values of α_q and α_c using equation (3.27) and (3.25). The results obtained are illustrated below.

Sin (45+Φ/2)	0.87
Sin (Φ)	0.50
Alpha c	0.23
Alpha q	0.10

Following Calculations are carried out to find the resultant earth pressure which is sum of earth pressure caused by vertical effective overburden and that caused by cohesion of material. (see equation (3.23)) . While K_q and K_c are the coefficients of cohesion and surcharge respectively at any depth Z and their values are calculated using equations (3.24) and (3.26).

Depth (Z)	Z/D	Kc	C x Kc	Kq	Po	Po x Kq	Pz	For Moment
0.00	0.00	7.00	0.00	4.80	4.67	22.42	22.42	
1.00	1.64	21.46	0.00	6.74	23.50	158.34	158.34	90.38
2.00	3.28	29.72	0.00	8.18	42.33	346.23	346.23	252.28
3.00	4.92	35.06	0.00	9.29	61.16	568.41	568.41	457.32
4.00	6.56	38.80	0.00	10.18	79.99	814.39	814.39	691.40
5.00	8.20	41.57	0.00	10.90	98.82	1077.56	1077.56	945.97
6.00	9.84	43.69	0.00	11.50	117.65	1353.56	1353.56	1215.56
7.00	11.48	45.38	0.00	12.01	136.48	1639.39	1639.39	1496.48
8.00	13.11	46.75	0.00	12.45	155.31	1932.91	1932.91	1786.15
9.00	14.75	47.89	0.00	12.82	174.14	2232.56	2232.56	2082.73
10.00	16.39	48.84	0.00	13.15	192.97	2537.18	2537.18	2384.87

Now find the point of rotation, for that assume various depths as point of rotation and take moment around point of application of lateral load with opposite signs above and below the point of rotation. Consider the value closest to zero as point of rotation. Note while calculating the point of rotation and carrying out calculation for ultimate loads passive resistance of top 2m strata below top of bed rock is neglected, this was suggested by Broms.

Following table gives calculations by taking different depths as point of rotation we will consider the depth with value closest to zero.

Calculator for Least Resultant of Forces (Approx. Zero)		
Location of Lateral Load Above Surface	13.82	m
Point of Rotation	Resultant of Forces	
1	-	
2	-	
3	-215917.19	
4	-191967.14	
5	-157306.65	
6	-110337.34	
7	-49520.56	
8	26640.79	
9	119613.95	
10	230844.17	

Point of rotation is at 8m depth so the embedment depth should be greater than that. Now find the value of ultimate load by taking its moment about point of rotation and equating it with moments of embedded depth portion about the point of rotation. Then the allowable load is calculated by dividing ultimate load by factor of safety.

Select Point of Rotation	8.00	m
Distance From Load Application To Point of Roatation	21.82	m
Moment About Point of Rotation	19732.83	KN-m
Ultimate Load (P-ult) per unit length	904.35	KN/m
Choose Factor of Safety (FOS)	2.00	
Allowable Load	**275.83**	**KN**
Enter Pile Efficiency	1.00	
FINAL PILE CAPACITY	**275.83**	**KN**
	28.13	**TONS**

Design Summary

Pile Diameter = 0.61 m

Pile Length = 15.85 + 10 = 25.85 m

Point of Rotation = 8 m below the bottom of excavation

F.O.S. = 2

Allowable Load = 275.83 KN

Design of Piles Installed along adjacent Building

In this case the surcharge load is 29.18 KN/m while the rest of parameters are same

Enter Soil Properties			
Cohesion (C)	0	KPa	
Friction Angle (ϕ)	30	Deg/Rad	0.52
Unit Weight (ɣ)	18.83	KN/m^3	
Surcharge (P)	29.18	KN/m	

Now the resultant earth pressure is calculated and results are shown below

Depth (Z)	Z/D	Kc	C x Kc	Kq	Po	Po x Kq	Pz	For Moment
0.00	0.00	7.00	0.00	4.80	29.18	140.06	140.06	
1.00	1.64	21.46	0.00	6.74	48.01	323.48	323.48	231.77
2.00	3.28	29.72	0.00	8.18	66.84	546.71	546.71	435.09
3.00	4.92	35.06	0.00	9.29	85.67	796.20	796.20	671.46
4.00	6.56	38.80	0.00	10.18	104.50	1063.93	1063.93	930.06
5.00	8.20	41.57	0.00	10.90	123.33	1344.82	1344.82	1204.38
6.00	9.84	43.69	0.00	11.50	142.16	1635.55	1635.55	1490.19
7.00	11.48	45.38	0.00	12.01	160.99	1933.80	1933.80	1784.68
8.00	13.11	46.75	0.00	12.45	179.82	2237.95	2237.95	2085.87
9.00	14.75	47.89	0.00	12.82	198.65	2546.79	2546.79	2392.37
10.00	16.39	48.84	0.00	13.15	217.48	2859.44	2859.44	2703.11

Now for calculation of point of rotation iterative approach is adopted and it comes out to be on 8m depth

Calculator for Least Resultant of Forces (Approx. Zero)		
Location of Lateral Load Above Surface	13.82	m
Point of Rotation	Resultant of Forces	
1	-	
2	-	
3	-253174.76	
4	-220957.33	
5	-176829.01	
6	-119248.17	
7	-46718.94	
8	42222.71	
9	149017.94	
10	275091.07	

Values of ultimate load and allowable load capacity are calculated and results are shown below

Select Point of Rotation	8.00	m
Distance From Load Application To Point of Roatation	21.82	m
Moment About Point of Rotation	24789.88	KN-m
Ultimate Load (P-ult) per unit length	1136.11	KN/m
Choose Factor of Safety (FOS)	2.00	
Allowable Load	**346.51**	**KN**
Enter Pile Efficiency	1.00	
FINAL PILE CAPACITY	**346.51**	**KN**
	35.33	**TONS**

Design Summary

Pile Diameter = 0.61 m

Pile Length = 15.85 + 10 = 25.85 m

Point of Rotation = 8 m below the bottom of excavation

F.O.S. = 2

Allowable Load = 346.51 KN

Wang and Reese Method

Design of Piles installed along the road side

Soil and pile properties are given in following table

UNIT WEIGHT OF SOIL (γ)	18.83	KN/m3	
HEIGHT OF PILE ABOVE FINAL EXCAVATION LEVEL (H)	15.85	m	
DRILLED SHAFT DIAMETER (b)	0.61	m	
PILE C/C SPACING (s)	1.22	m	
CLEAR SPACING BETWEEN DRILLED SHAFTS (sc)	1.83	m	
SOIL FRICTION ANGLE (Φ)	30.00	0.52	Deg/Rad
β=45+Φ/2	60.00	1.05	Deg/Rad
α=Φ (for dense sands)	30.00	0.52	Deg/Rad
SUBGRADE REACTION FORCE (R)	111.78	KN/m	
AT-REST EARTH PRESSURE CO-EFFICIENT (Ko)	0.50		
ACTIVE EARTH PRESSURE CO-EFFICIENT (Ka)	0.33		
PASSIVE EARTH PRESSURE CO-EFFICIENT (Kp)	3.00		

Values of $K_{o,}$ K_a and K_p are calculated using equations (3.2), (3.3), and (3.4) respectively. And subgrade reaction force is a function of depth of lowest anchor earth pressure due to retained soil and surcharge load.

$$R = \left(\frac{3H_L}{16}\right)P_E + \left(\frac{H_L}{2}\right)P_S$$

Surcharge Load	11.48	KN/m2

FOR SUBGRADE REACTION FORCE		
MAX EARTH PRESSURE (Pe)	80.70	
MAX SURCHARGE PRESSURE (Ps)	4.67	
HEIGHT OF LOWEST ANCHOR FROM BOTTOM OF PILE (HL)	6.40	
DEPTH OF HIEGHEST ANCHOR FROM TOP OF PILE (HT)	3.05	
45-Φ/2	30.00	0.52
SUBGRADE REACTION (R)	111.78	

While values of other soil parameters are given below

Θ	0.78
SEC Θ	1.41
TAN Θ	0.99
COT Θ	1.01
Reduction Factor (K)	0

TAN Φ	0.58	TAN (β-Φ)	0.58	COS β	0.50	TAN ^8 (β)	81.00
TAN β	1.73	SIN β	0.87	SIN ^2 (β)	0.75	SIN Φ	0.50
TAN α	0.58	COS α	0.87	TAN ^4 (β)	9.00		

A factor of safety of 2 is achieved with embedment depth of 3m which is considerably lesser than that of calculated using Brinch Hansen's method. But their comparison will be discussed in next chapter.

	eq (3.8)	eq. (3.11)	eq (3.15)	eq (3.16)		eq (3.6)		
TOE DEPTH (m)	WEDGE RESISTANCE SINGLE PILE (KN/m)	WEDGE RESISTANCE INTERSEC. (KN/m)	FLOW RESISTANCE (KN/m)	RANKINE CONTIN. RESIST. (KN/m)	MINIMUM RESIST.	TOTAL PASSIVE FORCE (KN)	LATERAL LOAD R-LOAD (KN)	FOS
0.50	34.57	-	179.53	68.92	34.57	7.00	167.20	0.04
1.00	103.81	103.66	359.05	137.84	103.66	38.78	198.98	0.19
1.50	207.73	200.54	538.58	206.75	200.54	111.49	231.71	0.48
2.00	346.32	321.58	718.10	275.67	275.67	241.30	265.41	0.91
2.50	519.59	466.80	897.63	344.59	344.59	444.37	300.06	1.48
3.00	727.54	636.18	1077.16	413.51	413.51	736.86	335.66	2.20
3.50	970.16	829.73	1256.68	482.42	482.42	1134.93	372.23	3.05
4.00	1247.45	1047.45	1436.21	551.34	551.34	1654.74	409.75	4.04
4.50	1559.43	1289.34	1615.73	620.26	620.26	2312.45	448.23	5.16
5.00	1906.07	1555.39	1795.26	689.18	689.18	3124.22	487.67	6.41

Design Summary

Max Earth Pressure = 80.70 KN/m^2

Max Surcharge Pressure = 4.67 KN/m^2

Subgrade Reaction = 111.78 KN/m

Embedment Depth = 3 m

Total Pile Length = 15.85 + 3 = 18.85 m

Calculations for Pile adjacent to Building

Now if we consider the side adjacent to building the only difference will be due to the surcharge load caused by the building which is given by

Surcharge Load	71.78	KN/m2

This surcharge load will change the value of subgrade reaction from that calculated in above case i.e.

FOR SUBGRADE REACTION FORCE		
MAX EARTH PRESSURE (Pe)	80.70	
MAX SURCHARGE PRESSURE (Ps)	29.19	
HEIGHT OF LOWEST ANCHOR FROM BOTTOM OF PILE (HL)	6.40	
DEPTH OF HIEGHEST ANCHOR FROM TOP OF PILE (HT)	3.05	
45-Φ/2	30.00	0.52
SUBGRADE REACTION (R)	190.25	

	eq (3.8)	eq. (3.11)	eq (3.15)	eq (3.16)		eq (3.6)		
TOE DEPTH (m)	WEDGE RESISTANCE SINGLE PILE (KN/m)	WEDGE RESISTANCE INTERSEC. (KN/m)	FLOW RESISTANCE (KN/m)	RANKINE CONTIN. RESIST. (KN/m)	MINIMUM RESIST.	TOTAL PASSIVE FORCE (KN)	LATERAL LOAD R-LOAD (KN)	FOS
0.50	34.57	-	179.53	68.92	34.57	7.00	262.93	0.03
1.00	103.81	103.66	359.05	137.84	103.66	38.78	294.71	0.13
1.50	207.73	200.54	538.58	206.75	200.54	111.49	327.45	0.34
2.00	346.32	321.58	718.10	275.67	275.67	241.30	361.14	0.67
2.50	519.59	466.80	897.63	344.59	344.59	444.37	395.79	1.12
3.00	727.54	636.18	1077.16	413.51	413.51	736.86	431.40	1.71
3.50	970.16	829.73	1256.68	482.42	482.42	1134.93	467.96	2.43
4.00	1247.45	1047.45	1436.21	551.34	551.34	1654.74	505.49	3.27
4.50	1559.43	1289.34	1615.73	620.26	620.26	2312.45	543.96	4.25
5.00	1906.07	1555.39	1795.26	689.18	689.18	3124.22	583.40	5.36

Design Summary

Max Earth Pressure = 80.70 KN/m^2

Max Surcharge Pressure = 29.19 KN/m^2

Subgrade Reaction = 190.25 KN/m

Embedment Depth = 3.5 m

Total Pile Length = 15.85 + 3.5 = 19.35 m

Detailed calculations for F.O.S and embedment (toe) depth are shown on above sheet.

Brom's Method

Results were calculated for same soil and there was a significant difference in pile capacity calculated using Broms method and that calculated using Brinch Hansen's method. Results are shown in in following tables.

Pile Data			
Material	Concrete		
Embedment Length (L)	5.00		m
Diameter (D)	0.61	0.00	m

Soil Density (γ)	18.83		KN/m^3
Friction Angle (Φ)	30.00	0.52	DEG/RAD
Passive Pressure (Kp)	11.12		KN/m
F.O.S	2.00		

Dimensionless length factor is calculated using equation (3.5 b). Horizontal subgrade reaction K_h is calculated using figure (3.2). value of K_h is checked against medium sand

Horizontal Subgrade Reaction		
Horizontal Subgrade Reaction (Kh) :	8143.00	KN/m^3
Corrected Value of Kh :	2714.33	KN/m^3

Section Modulus (Z)	0.0223	m^3
Dimensionless Factor	8.20	

Input :	E (MPa)	I (m^4)			
	24800.00	460.75			
Factor (η)			0.08	0.0014	Deg/Rad
Length Factor (ηxL)			1.71		

Figures (3.3) and (3.4) are used to calculate the ultimate loads and their values are given below. For calculating value of ultimate load we require $e - c/_L$ ratio. Where e-c is the eccentricity. Since the length factor (ηxL) is less than 2 so we have short pile.

For Free-Head Piles		
e-c	13.82	m
e-c/L	2.76	

For Short free or fixed headed piles		
Qu/Kp*D^3*γ	12.00	
Qu	285.27	KN

Design Summary

Embedment Depth = 5 m

Total Pile length = 15.85 + 5 = 20.85 m

Horizontal Subgrade Reaction = 2714.33 KN/m

Ultimate Load = 285.27 KN

Note: In Brom's method the design for Piles installed along the road side and the structural surcharge piles is the same due to exclusion of surcharge from the method of design.

5.1.2. DESIGN FOR FEDERAL COURTS, LAHORE

Design of Anchors

The building of federal courts is located near old state bank building Lahore. There are buildings on two sides and traffic lanes on other two sides. Total depth of excavation was 28 ft (8.54 m) and for that two rows of anchor were proposed. Depth of upper anchor level from NSL was 8 ft (2.44 m) while depth of lower anchor level was 18 ft (5.49 m) from NSL. Value of surcharge load was 800 lb/ft^2(38.28 KN/m^2).

Results of calculations carried out are shown in the following figures.

Soil and Anchor Properties		
Unit weight of soil (KN/m)	14.91	
Friction angle	20.00	0.35
Spacing between piles (m)	1.22	
Inclination angle of anchors	15.00	0.26

Soil and Anchor properties and depth of installation of anchors is given as

For anchors,	
Excavation depth	8.54
Number of anchors	2.00
Depth of first anchor from ground level	2.44
Spacing of anchors	3.05
Surcharge load from adjacent structures	38.28
Surcharge load from traffic	0.00

Note: All units are in metric system

Now the values of earth pressure due to retained soil mass and due to surcharge load are calculated using equations (4.3) and (4.4) respectively and results obtained are shown below.

Apparent Earth Pressure		
P_e	51.64	KN/m2

Earth Pressure due to Surcharge		
P_s	18.77	KN/m2

The values of horizontal anchor load moments and design load of anchors are calculated using equations (4.5 to 4.11). Maximum value of design load is selected for further calculations. The results are illustrated below

Anchor No.	Horizontal Anchor Load (KN/m)	Moment between Anchors (KN-m)	Design Anchor Load (KN)
1.00	237.20	129.90	299.59
2.00	211.49	65.51	267.12
		Max Design Load (KN)	299.59

To calculate the bond length and Anchor load capacity load transfer rate was selected for medium sand using table (4.1) and the values of anchor capacity and bond length was calculated using equation (4.12) and (4.13) respectively. While unbounded length can be selected from figure (4.1).

Reaction Force (KN/m)	58.12
Load transfer rate (KN/m)	100.00
F.O.S	2.00
Estimated bond length (m)	5.99
Assume bond length (m)	8.00

Allowable anchor capacity for assumed bond length (KN)	400.00

Final bond length (m)	8.00
Unbonded length (m)	5.00
Total length (m)	13.00

Design Summary

Anchor Design load Capacity = 400 KN

Un-bonded Length = 5 m

Bonded Length = 8 m

Angle of Anchor Installation = 15^{o}

Spacing between Adjacent Piles = 1.22 m (4 ft)

Design of Piles

Brom's Method

Soil and pile properties are shown in following table;

Pile Data			
Material	Concrete		
Embedment Length (L)	3.66		m
Diameter (D)	0.61	0.00	m

Soil Density (γ)	18.05		KN/m^3
Friction Angle (Φ)	33.00	0.58	DEG/RAD
Passive Pressure (Kp)	13.57		KN/m
F.O.S	2.00		

Dimensionless length factor is calculated using equation (3.5 b). Horizontal subgrade reaction K_h is calculated using figure (3.2). value of K_h is checked against medium sand

Horizontal Subgrade Reaction		
Horizontal Subgrade Reaction (Kh) :	8143.00	KN/m^3
Corrected Value of Kh :	2714.33	KN/m^3

Section Modulus (Z)	0.0223	m^3
Dimensionless Factor	6.00	

Input :	E (MPa)	I (m^4)		
	24800.00	92.31		
Factor (η)		0.12	0.0020	Deg/Rad
Length Factor (ηxL)		2.55		

Figures (3.3) and (3.4) are used to calculate the ultimate loads and their values are given below. For calculating value of ultimate load we require $e - c/L$ ratio. Since the length factor (ηxL) is between 2 and 4 so we have intermediate pile.

For Free-Head Piles		
e-c	6.91	m
e-c/L	1.89	

The calculated value of ultimate lateral load capacity of pile is given as

For Short free or fixed headed piles		
Qu/Kp*D^3*γ	10.00	
Qu	**277.89**	KN

Design Summary

Embedment Depth = 3.66 m

Horizontal Subgrade Reaction = 2714.33 KN/m

Ultimate Load = 277.89 KN

Brinch Hansen's Method

For same site conditions calculations were carried out using Brinch Hansen's method. In this case embedment depth is taken as 8 m. Surcharge pressure is 38.28 KN/m^2 (800 lb/ft^2) while other properties are shown below

Enter Soil Properties			
Cohesion (C)	0	KPa	
Friction Angle (ϕ)	33	Deg/Rad	0.58
Unit Weight (ɣ)	18.05	KN/m^3	
Surcharge (P)	38.28	KN/m	

Enter Pile Properties		
Diameter (D)	0.61	m
Length (L)	8	m

Now using the figure (3.8) find the values of coefficient of cohesion and coefficient of surcharge at the surface and at bottom of pile against angle of friction.

Enter Values From Graph	
Coeff. Cohesion at surface (Kc')	6
Coeff. Cohesion at depth (Kc")	83
Coeff. Surcharge at surface (Kq')	5.5
Coeff. Surcharge at depth (Kq")	25

Consequently calculate the values of α_q and α_c using equation (3.27) and (3.25). The results obtained are illustrated below.

Sin (45+ϕ/2)	0.88
Sin (ϕ)	0.54
Alpha c	0.14
Alpha q	0.08

Now the resultant earth pressure is calculated and results are shown below

Depth (Z)	Z/D	Kc	C x Kc	Kq	Po	Po x Kq	Pz	For Moment
0.00	0.00	6.00	0.00	5.50	38.28	210.54	210.54	
1.00	1.64	20.12	0.00	7.75	56.33	436.58	436.58	323.56
2.00	3.28	29.86	0.00	9.54	74.38	709.23	709.23	572.91
3.00	4.92	36.99	0.00	10.99	92.43	1015.37	1015.37	862.30
4.00	6.56	42.44	0.00	12.19	110.48	1346.39	1346.39	1180.88
5.00	8.20	46.73	0.00	13.20	128.53	1696.39	1696.39	1521.39
6.00	9.84	50.20	0.00	14.06	146.58	2061.21	2061.21	1878.80
7.00	11.48	53.06	0.00	14.81	164.63	2437.82	2437.82	2249.52
8.00	13.11	55.47	0.00	15.46	182.68	2823.96	2823.96	2630.89

Now for calculation of point of rotation iterative approach is adopted and it comes out to be on 6m depth

Calculator for Least Resultant of Forces (Approx. Zero)		
Location of Lateral Load Above Surface	6.67	m
Point of Rotation	Resultant of Forces	
1	-	
2	-	
3	-110867.11	
4	-86848.09	
5	-52860.28	
6	-7130.23	
7	52122.10	
8	126681.64	

Values of ultimate load and allowable load capacity are calculated and results are shown below

Select Point of Rotation	6.00	m
Distance From Load Application To Point of Roatation	12.67	m
Moment About Point of Rotation	14262.81	KN-m
Ultimate Load (P-ult) per unit length	1125.72	KN/m
Choose Factor of Safety (FOS)	2.00	
Allowable Load	**343.34**	**KN**
Enter Pile Efficiency	0.80	
FINAL PILE CAPACITY	**274.67**	**KN**
	28.01	**TONS**

Design Summary

Pile Diameter = 0.61 m

Pile Length = 8.54 + 8 = 25.85 m

Point of Rotation = 6 m below the bottom of excavation

F.O.S. = 2

Allowable Load = 343.34 KN

Wang and Reese Method

Soil and pile properties are given in following table

UNIT WEIGHT OF SOIL (γ)	14.92	KN/m3	
HEIGHT OF PILE ABOVE FINAL EXCAVATION LEVEL (H)	6.40	m	
DRILLED SHAFT DIAMETER (b)	0.60	m	
PILE C/C SPACING (s)	2.50	m	
CLEAR SPACING BETWEEN DRILLED SHAFTS (sc)	1.90	m	
SOIL FRICTION ANGLE (Φ)	20.00	0.35	Deg/Rad
β=45+Φ/2	55.00	0.96	Deg/Rad
α=Φ (for dense sands)	20.00	0.35	Deg/Rad
SUBGRADE REACTION FORCE (R)	157.63	KN/m	
AT-REST EARTH PRESSURE CO-EFFICIENT (Ko)	0.66		
ACTIVE EARTH PRESSURE CO-EFFICIENT (Ka)	0.49		
PASSIVE EARTH PRESSURE CO-EFFICIENT (Kp)	2.04		

Values of K_O K_A and K_P are calculated using equations (3.2), (3.3), and (3.4) respectively. And subgrade reaction force (R) is function of depth of lowest anchor earth pressure due to retained soil mass and due to the surcharge load.

$$R = \left(\frac{3H_L}{16}\right)P_E + \left(\frac{H_L}{2}\right)P_S$$

FOR SUBGRADE REACTION FORCE		
MAX EARTH PRESSURE (Pe)	30.43	
MAX SURCHARGE PRESSURE (Ps)	18.83	
HEIGHT OF LOWEST ANCHOR FROM BOTTOM OF PILE (HL)	0.00	
DEPTH OF HIEGHEST ANCHOR FROM TOP OF PILE (HT)	0.00	
45-Φ/2	35.00	0.61
SUBGRADE REACTION (R)	157.63	

Surcharge (KN/m2)	38.4

Other Soil Parameters are tabulated below

Θ	0.78
SEC Θ	1.41
TAN Θ	0.99
COT Θ	1.01
Reduction Factor (K)	0

TAN Φ	0.36	TAN (β-Φ)	0.70	COS β	0.57	TAN ^8 (β)	17.31
TAN β	1.43	SIN β	0.82	SIN ^2 (β)	0.67	SIN Φ	0.34
TAN α	0.36	COS α	0.94	TAN ^4 (β)	4.16		

F.O.S. of 1.75 is achieved with embedment depth of 5m. Detailes iterative result sheet is shown on next page.

Design Summary

Max Earth Pressure = 30.43 KN/m^2

Max Surcharge Pressure = 18.83 KN/m^2

Subgrade Reaction = 157.63 KN/m

Embedment Depth = 5 m

Total Pile Length = 8.54 +5 = 13.54 m

	eq (3.8)	eq. (3.11)	eq (3.15)	eq (3.16)		eq (3.6)		
TOE DEPTH (m)	WEDGE RESISTANCE SINGLE PILE (KN/m)	WEDGE RESISTANCE INTERSEC. (KN/m)	FLOW RESISTANCE (KN/m)	RANKINE CONTIN. RESIST. (KN/m)	MINIMUM RESIST.	TOTAL PASSIVE FORCE (KN)	LATERAL LOAD R-LOAD (KN)	FOS
0.50	14.57	-	45.41	38.04	14.57	3.09	408.66	0.01
1.00	40.00	36.14	90.82	76.08	36.14	15.58	424.35	0.04
1.50	76.31	75.71	136.23	114.12	75.71	42.30	441.14	0.10
2.00	123.49	123.32	181.64	152.15	123.32	88.10	459.02	0.19
2.50	181.54	178.99	227.05	190.19	178.99	157.81	478.01	0.33
3.00	250.47	242.71	272.46	228.23	228.23	256.26	498.09	0.51
3.50	330.26	314.48	317.87	266.27	266.27	388.29	519.26	0.75
4.00	420.93	394.30	363.28	304.31	304.31	558.73	541.54	1.03
4.50	522.47	482.17	408.69	342.35	342.35	772.43	564.91	1.37
5.00	634.88	578.09	454.10	380.39	380.39	1034.22	589.38	1.75

5.1.3. DESIGN OF LATERALLY LOADED PILE, JHIKA GALI, MURREE

After the landslide of Jhika Gali restoration process was carried out and piles were provided to cater the lateral loads. This topic describes the design procedure of pile and gives ultimate load that pile can bear.

Total pile length was 22 m with embedment depth of 12 m. Centroid of ultimate load acting on the pile was at a distance of 6.67 from top of bed rock. Pile Properties are given in following table.

Soil Properties			
Cohesion (C)	29	KPa	
Friction Angle (ϕ)	27	Deg/Rad	0.47
Unit Weight (γ)	21	KN/m^3	
Surcharge (P)	0	KN/m	

Pile Properties		
Diameter (D)	1.2	m
Embed. Depth (Z)	12	m

Values of coefficient of surcharge and coefficient of coefficient are obtained from figure (3.8) . Results are illustrated below

Enter Values From Graph	
Coeff. Cohesion at surface (Kc')	6
Coeff. Cohesion at depth (Kc")	45
Coeff. Surcharge at surface (Kq')	3.7
Coeff. Surcharge at depth (Kq")	13

Equations (3.25 and 3.27) and are used to find the values of α_q and α_c and they are shown below.

Sin (45+ɸ/2)	0.85
Sin (ɸ)	0.45
Alpha c	0.26
Alpha q	0.12

Following table shows the results and calculation carried out to find the value of resultant earth pressure P_z. Equation was used to find the resultant earth pressure.

Depth (Z)	Z/D	Kc	C x Kc	Kq	Po	Po x Kq	Pz	For Moment
0.00	0.00	6.00	174.00	3.70	0.00	0.00	174.00	
1.00	0.83	13.00	376.94	4.52	21.00	94.87	471.80	322.90
2.00	1.67	17.87	518.13	5.20	42.00	218.52	736.65	604.22
3.00	2.50	21.45	622.03	5.79	63.00	364.50	986.53	861.59
4.00	3.33	24.20	701.69	6.29	84.00	528.15	1229.84	1108.19
5.00	4.17	26.37	764.72	6.72	105.00	706.02	1470.74	1350.29
6.00	5.00	28.13	815.81	7.11	126.00	895.51	1711.33	1591.03
7.00	5.83	29.59	858.08	7.45	147.00	1094.62	1952.70	1832.01
8.00	6.67	30.81	893.63	7.75	168.00	1301.76	2195.39	2074.04
9.00	7.50	31.86	923.94	8.02	189.00	1515.70	2439.64	2317.51
10.00	8.33	32.76	950.09	8.26	210.00	1735.44	2685.53	2562.58
11.00	9.17	33.55	972.88	8.49	231.00	1960.16	2933.04	2809.28
12.00	10.00	34.24	992.92	8.69	252.00	2189.20	3182.11	3057.58
13.00	10.83	34.85	1010.67	8.87	273.00	2422.00	3432.67	3307.39
14.00	11.67	35.40	1026.52	9.04	294.00	2658.09	3684.61	3558.64
15.00	12.50	35.89	1040.75	9.20	315.00	2897.10	3937.85	3811.23

Moment is taken about the point of application of load while considering different depths as point of rotation. In this case the point of rotation is at a depth of 9 m (since we consider the depth which gives the moment value closest to zero). Sign of moments are opposite above and below the point of rotation so one should be careful while carrying out the calculations.

Calculator for Least Resultant of Forces (Approx. Zero)		
Location of Lateral Load Above Surface	6.67	m
Point of Rotation	Resultant of Forces	
1	-	
2	-	
3	-271717.13	
4	-249176.62	
5	-219011.12	
6	-180285.37	
7	-132030.13	
8	-73251.71	
9	-2938.32	
10	79935.59	
11	176406.33	
12	287518.65	
13	350921.37	
14	422699.20	
15	503382.96	

Now the final calculations for calculating Allowable load are shown in following table

Select Point of Rotation	9.00	m
Distance From Load Application To Point of Roatation	15.67	m
Moment About Point of Rotation	45329.28	KN-m
Ultimate Load (P-ult) per unit length	2892.74	KN/m
Choose Factor of Safety (FOS)	2.00	
Allowable Load	**1735.65**	**KN**
Enter Pile Efficiency	0.55	
FINAL PILE CAPACITY	**954.61**	**KN**
	97.34	**TONS**

Design Summary

Pile Diameter = 1.2 m

Pile Length = 10 + 12 = 22 m

Point of Rotation = 9 m below the bottom of excavation

F.O.S. = 2

Allowable Load = 1735.65 KN

CHAPTER 6: CONCLUSIONS AND REMARKS

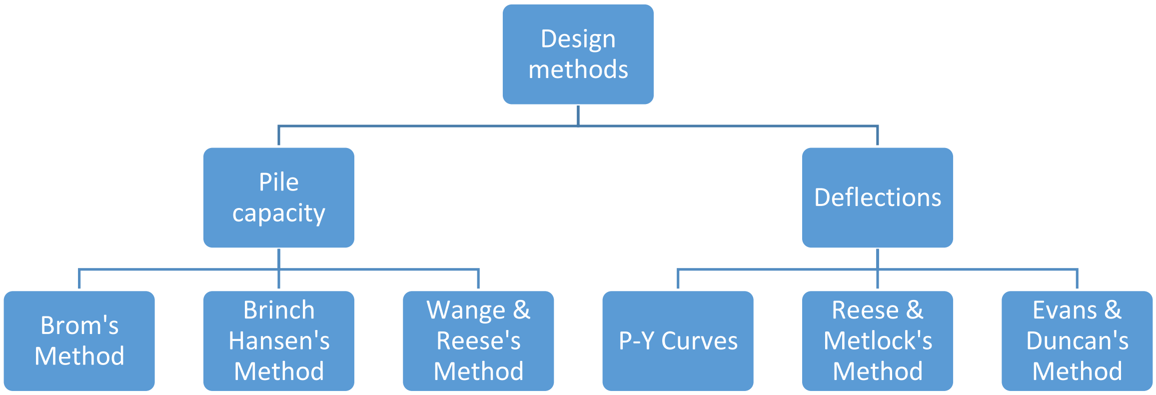
6.1. GENERAL REMARKS

For the overall design of the retention system, including both piles and anchors, the FHWA method has been generally more feasible. The anchor design is simpler and accurate, rendering quick and reliable design. However FHWA suggests Wange & Reese's method for pile design, which is precise and programmable, but has limitations such as ideal conditions and soil behavior. For the pile to be adequate, the site conditions and soil sample have to be studied thoroughly if Wange & Reese's method is to be applicable.

Generally, after evaluating pile parameters from different available methods in chapter 5, Brinch Hansen's method has been observed as more reliable. Where Brom's method yields similar results as Wange and Reese's method, Hansen's method always gives a more conservative design. This enables reduction in efforts of site investigation.

6.2. COMPARISON OF DIFFERENT METHODS OF PILE DESIGN

For comparison purpose we can divide the available methods is two categories with respect to output.



6.2.1. COMPARISON OF PILE CAPACITY METHODS

We consider design results of case study: Haly Towers, piles along roadside, for illustrating the initial comparison;

Design Summary: Brinch Hansen's Method

Pile Diameter = 0.61 m

Pile Length = 15.85 + 10 = 25.85 m

Point of Rotation = 8 m below the bottom of excavation

F.O.S. = 2

Allowable Load = 275.83 KN

Design Summary: Wange & Reese's Method

Pile Diameter = 0.61 m

Pile Length = 15.85 + 3 = 18.85 m

Allowable Load =335.66 KN

F.O.S = 2.20

Design Summary: Brom's Method

Pile Diameter = 0.61 m

Pile Length = 15.85 + 5 = 20.85 m

Horizontal Subgrade Reaction = 2714.33 KN/m

Ultimate Load = 285.27 KN

Remarks

Wange & Reese’s method gives the least embedment depth for the same diameter. This is due to the inclusion of pile spacing and soil flow resistance in the design. This means that this method designs a continuous wall of equally spaced piles. As the wedges are considered to resist the loads acting on the piles, their effect is reduced when the piles are spaced too close. However for small retention systems, this method may not be adequate, however this is sufficient for larger systems.

Brom’s method is a hand calculation tool for quick design, and therefore is often considered as inaccurate and unreliable. Notice that it gives a relatively high pile capacity for a mere 5m embedment depth. This method is suitable for small buildings and it is always a good practice to increase the factor of safety when using this method as it ignores the surcharge and only considers passive force of retained soil.

Brinch Hansen’s method yields the most comprehensive results, suitable for small or large projects. It considers the surcharge loads and evaluates a point of rotation using the acting lateral load and the resistance provided by the soil. Moreover most importantly, this method ignores top 2 meters of the embedment depth when calculating moments for the point of rotation. This is due to excavation and backfill of top 2 meters, which reduce its stiffness. Also sometimes the soil gets eroded. This produces a more conservative design, which when coupled with the factor of safety, is not only reliable but also immune to unfavorable site conditions.

6.2.2. COMPARISON OF PILE DEFLECTION METHODS

Using the final pile design of case study: Haly Towers, piles along roadside;

Pile Diameter = 0.61 m

Pile Length = 15.85 + 10 = 25.85 m

Allowable Load = 275.83 KN

(Brinch Hanen’s Method)

Evaluation of deflections by available methods to facilitate comparison is done on spreadsheets made for each method. Spreadsheets can be seen on appendix A. For purpose of completeness, computations of Evans & Duncan’s method is shown in this section.

Pile Properties		
Diameter (D)	0.61	m
Elastic Modulus (E)	24800.00	Mpa
Moment of Inertia (Ip)	878.07000	m4

Ratio of Moment of Inertia of Pile to the Moment of Inertia of Soil	
Moment of Inertia of soil (Is)	0.00679
Ri (Ip/Is)	129258.80

Surcharge (Ps)	80.70	KN/m2
Eccentricity (e)	13.82	m
Moment (Me)	1115.27	KN-m

Friction Angle	30	0.52	Deg/Rad
Rankines Passive (Kp)	2.99		
Submerged Unit Weight of Soil	18.83 KN/m3		
Characteristic Load (Pc)	373.136 KN		
Ps/Pc	0.0002		

Ys/D is read from figure 6.1 and Ys is calculated :

Free-Headed	
Ys/D	0.0100
Deflection (Ys)	**0.0061 m**

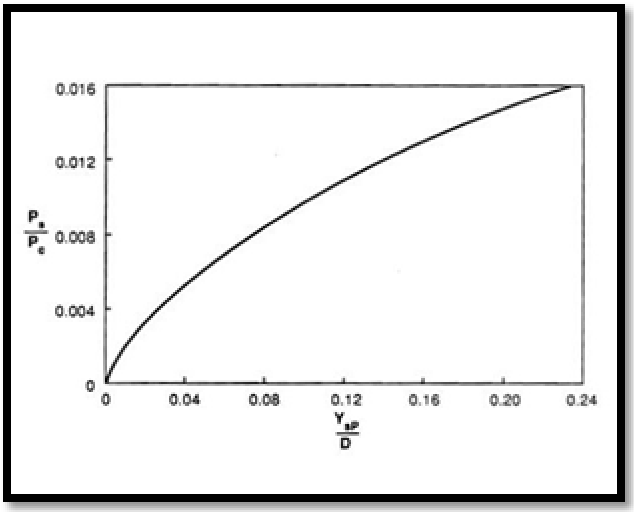


Figure 6. 1

P-Y method and Reese & Metlock's method largely depends on P-Y curves and hence is often in-accurate. The iterative nature of Reese & Metlock's method is also a negative aspect of the method. Hence these methods are not opted for deflection checks.

6.2.3. FINAL COMPARISON

Brom's Method

Features

- Initially developed for short, rigid and unfixed piles in cohesive soils
- In 1964 it's scope was extended to long piles with fixed heads and cohessionless soils
- It assumes that for short piles ultimate resistance is governed by passive earth pressure of surrounding soil
- For long piles ultimate resistance is governed by yield resistance of pile.

Limitations

- It does not consider the time dependent behaviour of soils.
- Valid for homogenous soils only
- It is not valid for cyclic loadings
- It does not take into acount the axial loads.

Brinch Hansen's Method

Features

- It is a simple method for calculating the lateral load capacity for short piles using lateral earth pressure coefficients
- It separates the soil resistance at different depths
- It also consider the cohesion factor in calculations
- Main feature of this method is that it is applicable even if the conditions are not favourable because it adopts conservative approach.

Limitations

- Only capable to find the ultimate resistance of the soil
- Calculations are not possible under working loads
- It does not consider the non-linear soil behaviour
- Does not take into account the time dependent behaviour of soils.

Wang & Reese's Method

Features

- This method is used to evaluate ultimate passive resistance for piles embeded in cohessive and cohessionless soils.
- To calculate ultimate passive resistance of cohessionless soil this method consider three potential failure mechanisms
  i. Wedge failure infront of an individual shaft
  ii. Overlapping wedge failure for deep or closely-spaced shafts
  iii. Plastic flow around the shaft

Limitations

- The active earth pressure acting on the wall as it moves away from the retained soil mass is considered only for cohessionless soils and not for cohessive soils
- This method was developed for stiff clays at relatively shallow depths, therefore the active earth pressures are negative.

P-Y Curves

Features

- This method is extensively used to take into account the soil-structure interaction and non-linear resistance of soils.
- For ultimate soil resistance P_u is a function of the pile diameter

Limitations

- Soil is idealized as a series of independent non-linear springs represented by P-Y curves. Therefore, the continuous nature of the soil is not explicitly modeled.
- The results are very sensitive to the p-y curves used. The selection of adequate p-y curves is the most crucial problem when using this methodology to analyze laterally loaded piles (Reese and Van Impe 2001).
- Sellecting appropriate p-y modulus and p-y curves is a difficult task.
- P-y curves and modulus are influenced by several pile-related factors, such as
    - Pile type and flexural stiffness
    - Type of loading
    - Pile geometry
    - Pile cap condition
    - Pile installation conditions

Evans and Duncan's Method

Features

- This method was developed for homogenous soils.
- This method can be used to determine ground line deflections, maximum moments and the location of the maximum moment.
- There are separate design graphs for cohesive and non-cohesive soils

Limitations

- The soil should be modeled as homogenous layer
- This method over-estimates the deformations in some situations.

Reese and Metlock's Method

Features

- This method is independent of pile size and depends primarily on soil properties
- It satisfies the bending properties of pile
- It is used to find set of elastic deflections of pile

Limitations

- A set of p-y curves is needed so limitation of p-y method are also incorporated in it
- It uses trial and error method for estimating the depth T. (Trials depths are assumed)
- This method can not be computerized, (because it used number of graphs so it is difficult to incorporate their values).

6.3. CONCLUSIVE REMARKS

For design of a reliable retention system, with the ability to withstand diverse site conditions, the conclusive remarks are as follows:

Best method for Anchor design	**FHWA method**
Best method for lateral capacity design of pile	**Brinch Hansen's**
Best method for deflection evaluation of pile	**Evan's & Duncan**