DESIGN OF SHALLOW AND DEEP FOUNDATION

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By

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This is to certify that the

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"In the name of Almighty ALLAH, the Most BENEFICIENT, the Most MERCIFUL".

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Thank you.

ABSTRACT

Foundations are structural elements which transfers load to the underlying soil. Being the most critical and crucial part, serves as an interface between two completely different materials.

Our basic aim is to understand the geotechnical and structural aspects of foundations to get the optimum design.

In our project we dealt with both types of foundations.

(a) Shallow foundations

- Spread footing
- Strip footing
- Combined footing
- Mat footing

(b) Deep Foundations

Excel sheet are prepared which gives design charts.

Design of drilled shafts for both vertical and horizontal loading conditions using two softwares in conjunction Oasys pile, Alp.

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Chapter 1

1 INTRODUCTION

1.1 Historical Background

Builders have realized the need of having stable foundations since structures began to rise above the ground. Builders at the time of Greeks and Romans understood the need for an adequate foundation because many of their structures have remain unyielding for centuries.

1.2 Objectives of Study

Our objective is to understand the geotechnical and structural design of foundation to obtain a design having both strength and economical aspect. Foundation engineering is the fundamental part which is used in every structure for its stability.

For foundation design there are different approaches for its analysis and design. Throughout our project we looked for different methods and checked their precision, applicability and comparison with other method. Use of modern software enabled to compare the results.

End results were the development of design chartsfor shallow foundations and excel work book for structural design of footing.

Using software to design single pile and also the comparison of field deflection results for pile.

1.3 Learning Outcomes

Understanding of various key concepts of foundation design for getting a design having strength and serviceability along with economical point of view.

In this project multiple softwarehad been used for the purpose of comparison of results and getting expertise.

- Excel
- SAFE

- Plaxis 3D foundation
- Oasys pile
- Oasys Alp
- OasysAdSec

CHAPTER 2

2 Literature Review

2.1 General

A Foundation is the key element of any architectural structure which transfers its loads to the underlying soil.

Foundation engineer should have knowledge of multiple fields because it needs knowledge from different fields of civil engineering e.g. structural engineering.

Two factors must be kept in mind while designing the foundation; support to the structure and transmission of loads. Type of foundation is categorized by transmission of its loads to underlying soil. Since foundation supports the structure so we must understand the nature and source of loads of structure and structure's tolerance to foundation movement.

2.1.1 Uncertainties

Soil being the natural element is mixture of different elements. It can have different composition and properties at the same site. Its properties can differ in vertical as well as horizontal direction. Geotechnical engineering is the new-born field in comparison to other fields. It is based on correlation parameters. So inspite of advance technology there is always chance of uncertainties in the results which is catered for by factor of safety.

2.1.2 Performance Requirements

Performance requirements need to be covered are as follows

- Strength requirements
- Serviceability requirements
- Economic requirements

2.2 Shallow Foundations

Shallow foundations are those which transfer loads to the shallow depth. Terzaghi set the criteria for defining shallow foundation as those having D/B ratio varying between 0.25-1.00. Types of shallow foundations include:

- Strip footing
- Spread footing
- Mat footing
- Combined footing

Pressure Distribution

We assume simplified uniform pressure distribution under foundation.

2.2.1 Geotechnical Design

For geotechnical design two requirements needs to be met:

- Strength requirements
- Serviceability requirements

2.2.1.1 Strength requirements

Strength requirement is governed by capacity of soil to sustain shear stresses generated due to loads. So for strength design shear strength is compared with shear stresses and design accordingly.Geotechnical strength requirement is expressed as bearing capacity of soil. If the load exceeds the bearing capacity, it is termed as bearing capacity failure. It has further three types.

General shear failure

When sudden failure happens in the stiff cohesive soil or dense sand and the failure happen to reach to ground surface.

Local Shear Failure

When the failure happen in the clayey or sandy soil of medium take place and the failure surface reaches gradually outward.

Punching Shear Failure

When the soil is loose enough that it cannot bear the shear forces, causing the soil beneath to collapse, and the shear zone progresses vertically downward.

2.2.1.2 Serviceability Requirements

Foundations satisfying strength requirements will not collapse, but they may not have serviceability performance. For example, they may happen to have excessive settlement.Therefore, we have the secondary of performance requirements known as serviceability requirements. These are intended to produce foundations that perform well when subjected to the service loads.

2.2.2 Bearing Capacity Calculation Methods

2.2.2.1 Terzaghi's Method of Bearing Capacity Computation:

Terzaghi (1943) took into consideration the roughness of strip footing and soil weight present on the horizontal plane through the base of the footing and modified the Prandtl expression.

His method includes the following assumptions:

- Depth of foundation less or equal to its width.
- Foundation has roughness which prevents it from sliding.
- Formula for calculation of shear strength is $s = c^{'} + \sigma' tan\varphi'$.
- Bearing capacity failure is general.
- The foundation has very rigidity as compare to underlying soil.
- Soil above the base of foundation takes no part in shear strength and only serve as a surcharge.
- No moments loads are present and the load is vertically to the centroid of foundation.

For squarefoundations:

$$q_{ult} = 1.3c'N_c + \sigma'_{zD}N_q + 0.4\gamma'BN_\gamma$$

For continuousfoundation:

$$q_{ult} = c'N_c + \sigma'_{zD}N_q + 0.5\gamma'BN_\gamma$$

For circularfoundations:

$$q_{ult} = 1.3c'N_c + \sigma'_{zD}N_q + 0.3\gamma'BN_\gamma$$

The Terzaghi bearing capacity factors are:

$$N_q = \frac{a_\theta^2}{2_{cos}^2(45 + \frac{\varphi'}{2})}$$

$$a_\theta = e^{\pi(0.75 - \frac{\varphi'}{2})tan\varphi'}$$

$$N_c = 5.7 \quad for\ \varphi' = 0$$

$$N_c = \frac{N_q - 1}{tan\varphi'} \quad for\ \varphi' > 0$$

$$N_\gamma \approx \frac{2(N_q + 1)tan\varphi'}{1 + 0.4\sin(4\varphi')}$$

2.2.2.2 Meyerhoff

Unlike Terzaghi, Meyerhof (1951) considered thefailure surfaceupto the ground surface Meyerhof equation for strip footing is similar inform to that of Terzaghi, but N_c, N_q and $N_\gamma$are different. He also included the shape factor sq, depth factors di and inclination factors ii. This method overestimates bearing capacity in case of sand soil. However gives reasonable results for clay soil.

2.2.2.3 Vesic's Theory

The failure surface is similar to that of Terzaghi's with difference that he considered zone I below the footing is in active Rankine state, with inclined faces of the wedge at $(45 + \varphi/2)$ to the horizontal.

It gives more accurate values and it also applies to a vast range of load and geometry condition. But it is complex to apply.

N_γ given by Vesic, is a simplified form of that given by Caqrot and Kerisal (1948).

$$N_\gamma = 2(N_q + 1)tan\varphi$$

For shape, depth and inclination factors, it usesVesic(1973).

2.2.2.4 Hansen

Brinch Hansen (1957, 1970) give followingequation for bearing capacity taking consideration of effects of depth and shape of footing and applied load inclination:

$$q_d = cN_c s_c d_c i_c + qN_q s_q d_q i_q + 0.5\gamma BN_\gamma s_\gamma d_\gamma i_\gamma$$

Where s_c,s_q and s_γ are shape factors.

d_c,d_c and d_γ are depth factors.

i_c,i_c and i_γ are inclination factors.

The recommendations of Hansen for $N_c$and N_q, are identical to those of Meyerhof, and are the result of those of Prandtl (1921) and Reissner (1924).

These are :

$$N_q = (e^{xtan\varphi})tan^2(45^\circ + \varphi/2)$$
$$N_c = (N_q - 1)cot\varphi$$
$$N_q = 1.5(N_q - 1)tan\varphi$$

2.2.2.5 Skempton

Skempton (1951) give following equation for calculation of bearing capacity of footings in saturated clay.

$$q_d = cN_c + \gamma D_f$$

For strip footings:

$$N_c = 5\left(\frac{1 + 0.2D_f}{B}\right), for\ N_c\ \leq 7.5$$

For rectangular, square or circular footings:

$$N_c = 6\left(1 + 0.2\frac{D_f}{B}\right)\left(1 + 0.2\frac{B}{L}\right), for\ N_c\ \leq 9$$

where, D_f = footing depth

B = width (or diameter) or footing

L = Length of footing

2.2.3 Settlement Calculation Methods

2.2.3.1 Terzaghi 1-D Consolidation Settlement Computation

Consolidation settlement is computed by dividing the soil under the foundation into layers, computing the settlement of each layer, and summing. The first layer top should start from the bottom of the foundation, and the last layer bottom should be at a depth such that $\Delta\sigma_z < 0.10\sigma'_0$.

For normally consolidated soils ($\sigma'_{z0} \approx \sigma'_c$):

$$\delta_c = r\sum\frac{C_c}{1 + e_0}Hlog\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right)$$

For over consolidated soils-Case I ($\sigma'_{zf} < \sigma'_c$):

$$\delta_c = r\sum\frac{C_r}{1 + e_0}Hlog\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right)$$

For over consolidated soils-Case II $(\sigma'_{z0} < \sigma'_c < \sigma'_{zf})$:

$$\delta_c = r\sum\frac{C_r}{1+e_0}Hlog\left(\frac{\sigma'_{zc}}{\sigma'_{z0}}\right) + r\sum\frac{C_c}{1+e_0}Hlog\left(\frac{\sigma'_{zf}}{\sigma'_c}\right)$$

Where:

$r = \text{rigidity factor}$

$$\delta_c = \text{ultimate consolidation settlement}$$

$$C_c = \text{kcompression index}$$

$$C_r = \text{recompression indexk}$$

$$H = \text{thickness of the soil layer}$$

$$e_0 = \text{initialkvoid ration}$$

$$\sigma'_{z0} = \text{initialkvertical effective stress at midpoint of soil layer}$$

$$\sigma'_c = \text{preconsolidationkstress at midpoint of soil layer}$$

$$\sigma'_{zf} = \text{finalkvertical effective stress at midpoint of soil layer}$$

2.2.3.2 Skempton and Bjerrum method

Skempton and Bjerrum presented method of computing the total settlement of shallow foundations. The settlement is divided into two components:

Distortion settlement iskthat caused by the lateral distortionkof the soil under the foundation.

Consolidation settlement is caused by the change in the volume resulting from change in effective stress.

This methods accounts for differences in generation of excess pore water pressures when soil happens to experiences lateral strain. This is reflected in the parameter ψ.

Following formula is used for computation of settlement in shallow foundations.

$$\delta = \delta_d + \psi\delta_c$$

Where:

$\delta = settlement$i

$$\psi = \text{three dimensional adjustment factor}$$

$$\delta_d = distortion\ settlement$$

$$\delta_c = consolidation\ settlement$$

the distortion settlement on the Base of elastic theory is:

$$\delta_d = \frac{(q - \sigma'_{zD})B}{E_u} I_1 I_2$$

2.2.3.3 Burland and Burbidge

Burland and Burbidge(1985) method for calculation of elastic settlement of sandy soil from the *standard penetration number*,N_{60}.

Variation of Standard Penetration Number with Depth

Obtaining the field penetration numbers (N_{60}). $N_{60(a)}$may be necessary depending on the field conditions with following adjustments:

For sandy gravel or gravel,

$$N_{60(a)} \approx 1.25\ N_{60}$$

For silty sand or fine sand below the groundwater table and N_{60}>15,

$$N_{60(a)} \approx 15 + 0.5(N_{60} - 15)$$

where$N_{60(a)}$ =adjusted N_{60} value.

Depth of Stress Influence (*z*')

To determine the depth of stress influence, these three cases may come:

Case I.If N_{60} [$or\ N_{60(a)}$] happens to be approximately constant with the depth, calculate *z*` from

$$\frac{z`}{B_R} = 1.4\left(\frac{B}{B_R}\right)^{0.75} \qquad eq.1$$

Where

B_R =reference width = 0.3 m

B =width of actual foundation

Case II.If N_{60} [or $N_{60(a)}$] happens to increase with depth, calculate z' from $eq.1$.

Case III.If N_{60}[or $N_{60(a)}$] happens to be decrease with depth, $z` = 2B$or till the bottom of soft soil layer measured from the bottom of the foundation (whichever is smaller).

Calculation of Elastic Settlement S_e

The elastic settlement of the foundation is calculated from

$$\frac{S_e}{B_R} = \alpha_1\alpha_2\alpha_3\left[\frac{1.25\left(\frac{L}{B}\right)}{0.25+\left(\frac{L}{B}\right)}\right]^2 \left(\frac{B}{B_R}\right)^{0.7}\left(\frac{q`}{p_a}\right)$$

Summary of $q`$, α_1 ,$\alpha_2$andα_3

Soil type	$q`$	α_1	α_2	α_3
Normally consolidated sand	q_{net}	0.14	$\frac{1.71}{[\bar{N}_{60}\ or\ \bar{N}_{60(a)}]^{1.4}}$	$\alpha_3 = \frac{H}{z`}\left(2-\frac{H}{z`}\right)$ (if $H \le z`$) or $\alpha_3$= 1( if H > z`)
Overconsolidated sand ($q_{net} \le \sigma`_c$)	q_{net}	0.047	$\frac{0.57}{[\bar{N}_{60}\ or\ \bar{N}_{60(a)}]^{1.4}}$	
Overconsolidated sand ($q_{net} > \sigma`_c$)	$q_{net} - 0.67\sigma`_c$	0.14	$\frac{0.57}{[\bar{N}_{60}\ or\ \bar{N}_{60(a)}]^{1.4}}$	where$H$ = depth of compressible layer

Where

α_1 =constant

α_2 = compressibility index

α_3 = correction for the depth of influence

L = length of the foundation

p_a = atmospheric pressure = 100 KN/m^2 ($\approx 2000\ lb/ft^2$)

2.2.3.4 Meyerhoff

Meyerhoff (1956) provided a correlation for calculating the*net bearing pressure* for foundations by standard penetration resistance, *N*60. Net pressure is defined as

$$q_{net} = \bar{q} - \gamma D_f$$

where$\bar{q}$ = stress at the level of the foundation.

According to theory, for 25 mm of maximum settlement,

$$q_{net}\left(\frac{kip}{ft^2}\right) = \frac{N_{60}}{4} \qquad (for\ B \le 4\ ft)$$

And

$$q_{net}\left(\frac{kip}{ft^2}\right) = \frac{N_{60}}{4}\left(\frac{B+1}{B}\right)\left(\frac{B+1}{B}\right)^2 \qquad (for\ B\ > 4\ ft)$$

Researchers have observed that results are conservative. Later, Meyerhof (1965) suggested that net allowable bearing pressure should increaseby about 50%. Later Bowles (1977) proposed the modified form of bearing equations

$$S_e(in.) = \frac{2.5q_{net}}{N_{60}F_d} \quad (for\ B \le 4\ ft)$$

And

$$S_e(in.) = \frac{4q_{net}}{N_{60}F_d}\left(\frac{B}{B+1}\right) \quad (for\ B > 4\ ft)$$

Where

$$S_e = \text{Settlement, in inches}$$

$$F_d = \text{depth factor} = 1 + 0.33\left(\frac{D_f}{B}\right)$$

$$B = \text{foundation width, in feet}$$

2.2.3.5 Schmertmann's Method (for sands only)

Schmertmann's method was developed primarily for computing settlement on sandy soils. It is mostly used with cone penetration test (CPT) results, but can be used with other in-situ tests. This method was developed from field and laboratory tests.

Equivalent Modulus of Elasticity

Schmertmann's method uses equivalent modulus of elasticity, E_s, which simplifies the computations because it is linear function. However, soil is not a linear material (i.e., stress and strain are not proportional), so the value of Es must reflect that of an equivalent unconfined linear material such that the computed settlement will be the same as in real soil.

The design value ofE_s implicitly reflects the lateral strains in the soil. Thus, it is larger than the *modulus of elasticity*, *E*, but smaller than the *confined modulus*, *M*.

$E_s$from Cone Penetration Test (CPT) Results

Schmertmann developed empirical correlations between the cone resistance, q_c, and E_s. This method is especially useful because the CPT provides a continuous plot ofq_c vs. depth.

E_s VALUES FROM CPT RESULTS		
Soil Type	USCS Group Symbol	E_s/q_c
Young, normally consolidated clean silica sands (age < 100 years)	SW or SP	2.5-3.5
Aged, normally consolidated clean silica sands (age > 3000 years)	SW or SP	3.5-6.0
Over consolidated clean silica sands	SW or SP	6.0-10.0
Normally consolidated silty or clayey sands	SM or SC	1.5
Over consolidated silty or clayey sands	SM or SC	3

While interpreting the CPT data for use in Schmertmann's method, overburden correction to q_c is notused.

$\boldsymbol{E_s}$ From Standard Penetration Test (SPT) Results

Schmertmann's method can be used with E_s values based on the standard penetration test. These values are not as precise as those obtained from the cone penetration test because:

- The standard penetration test is more prone to error, and is a less precise measurement.
- The standard penetration test provides only a series of isolated data points, whereas the cone penetration test provides a continuous plot.

Nevertheless, SPT data is adequate for many projects, especially those in which the loads are small and the soil conditions are good.

The following relationship should produce approximate, if somewhat conservative, values of E_s:

$$E_s = \beta_\circ\sqrt{OCR} + \beta_1 N_{60}$$

Where:

$$E_s = equivalent\ modulus\ of\ elasticity$$

$$\beta_\circ, \beta_1 = correlation\ factors$$

$$OCR = overconsolidation\ ratio$$

$$N_{60} = SPT\ N - value\ corrected\ for\ field\ procedures$$

Factors $\beta_\circ, \beta_1$				
	$\beta_\circ$		β_1	
Soil Type	lb/ft^2	KPa	lb/ft^2	KPa
Clean sands (SW and SW)	100000	5000	24000	1200
Silty sands and clayey sands (SM and SC)	50000	2500	12000	600

Strain Influence Factor

Schmertmann conducted extensive research on the distribution of vertical strain, ε_z, below the spread footings. He founded that the greatest strains is notunder the footing but at a depth of 0.5B to B under the bottom of the footing. This is presented by the strain influence factor, I_ε. The distribution of I_ε with depth has been idealized as two straight lines.

The peak value of the strain influence factor, $I_{\varepsilon p}$ is:

$$I_{\varepsilon p} = 0.5 + 0.1\sqrt{\frac{q - \sigma'_{zD}}{\sigma'_{zp}}}$$

Where:

$I_{\varepsilon p}$ =peakstrain influence factor

$q =$ Bearing pressure

σ'_{zD} = Vertical effective stress at a depth D below the ground surface

σ'_{zD} = Initial vertical effective stress at depth of peak strain influence factor (for square and circular foundations ($L/B = 1$), compute σ'_{zp} at a depth of $D + B/2$ below the ground surface; for continuous footings ($B/2 \geq 10$), compute it a depth of $D + B$.

The exact value of I_{ε} at any given depth may be computed using the following equations:

Square and circular foundations:

For $z_f = 0\ to\ B/2 : I_{\varepsilon} = 0.1 + \left(z_f/B\right)(2I_{\varepsilon p} - 0.2)$

For $z_f = B/2\ \ to\ 2B: I_{\varepsilon} = 0.667 I_{\varepsilon p}\left(2 - z_f/B\right)$

Continuous foundations ($B/2 \geq 10$):

For $z_f = 0\ to\ B: I_{\varepsilon} = 0.2 + \left(z_f/B\right)(I_{\varepsilon p} - 0.2)$

For $z_f = B\ to\ 4B: I_{\varepsilon} = 0.333 I_{\varepsilon p}\left(4 - z_f/B\right)$

Rectangular foundations ($1 < L/B < 10$):

$$I_\varepsilon = I_{\varepsilon s} + 0.111(I_{\varepsilon c} - I_{\varepsilon s})(L/B - 1)$$

Where:

z_f =depth from bottom of foundation to midpoint of layer

I_ε =strain influence factor

$I_{\varepsilon c} = I_\varepsilon$ for a continuous foundation

$I_{\varepsilon p}$ =peak I_ε

$I_{\varepsilon s} = I_\varepsilon$ for a square foundation ≥ 0

$$C_1 = 1 - 0.5\left(\frac{\sigma'_{zD}}{q - \sigma'_{zD}}\right)$$

$$C_2 = 1 + 0.2\, log\left(\frac{t}{0.1}\right)$$

$$C_3 = 1.03 - 0.03\, L/B \geq 0.73$$

Where:

δ =settlement of footing

C_1 =depth factor

C_2 =secondary creep factor

C_3 =shape factor = 1 for square and circular foundations

q =bearing pressure

σ'_{zD} = effective vertical stress at a depth below the ground surface

I_ε =influence factor at midpoint of soil layer

H = thickness of soil layer

E_s = equivalent modulus of elasticity in the soil layer

t =time since application of load (in years)

B = foundation width

L =foundation length

Using the following formula to compute the settlement δ:

$$\delta = C_1 C_2 C_3 (q - \sigma'_{zD}) \sum \frac{I_\varepsilon H}{E_s}$$

2.2.4 Structural Design

Footings are structural members used to support columns and walls and transmit their loads to the underlying soils. Reinforced concrete is a material admirably suited for footings and is used as such for both reinforced concrete and structural steel buildings, bridges, towers, and other structures. The permissible pressure on a soil beneath a footing is normally a few tons per square foot. The compressive stresses in the walls and columns of an ordinary structure may run as high as a few hundred tons per square foot. It is, therefore, necessary to spread these loads over sufficient soil areas to permit the soil to support the loads safely. Not only is it desired to transfer the superstructure loads to the soil beneath in a manner that will prevent excessive or uneven settlements and rotations, but it is also necessary to provide sufficient resistance to sliding and overturning. To accomplish these objectives, it is necessary to transmit the supported loads to a soil of sufficient strength and then to spread them out over an area such that the unit pressure is within a reasonable range. The design of a footing must consider bending, development of reinforcement, shear, and the transfer of load from the column or wall to the footing

Soil Failure Limitations are as follows:

- Soil bearing capacity failure.
- Differential settlement at different locations.
- Total Settlement.

Soil Failure Limitations are as follows:

- Column footing junction failure.
- Slipping of the reinforcement due to bond development failure.

- Bearing failure of footing just beneath column.

2.2.4.1 Wall Footing

A *wall footing* is simply an enlargement of the bottom of a wall that will sufficiently distribute the load to the foundation soil. Wall footings are normally used around the perimeter of a building and perhaps for some of the interior walls.

Design of Wall Footing:

The theory used for designing beams is applicable to the design of footings with only a few modifications. The upward soil pressure under the wall footing of tends to bend the footing into the deformed shape. The footings will be designed as shallow beams for the moments and shears involved. In beams where loads are usually only a few hundred pounds per foot and spans are fairly large, sizes are almost always proportioned for moment. In footings, loads from the supporting soils may run several thousand pounds per foot and spans are relatively short. As a result, shears will almost always control depths.

One-way shear and Two-way shear or Punching shear is checked against a trial value of effective depth to see its adequacy. For one-way shear, shear strength is calculated at a distance of d or effective depth from the face of the footing. For Two-way shear, shear strength of the footing is calculated at critical perimeter. Distance d is added to column dimensions on each side and are summed to get critical perimeter. Design shear strength must be greater than actual or applied shear strength for the footing thickness to prevent shear failure.

It appears that the maximum moment in this footing occurs under the middle of the wall, but tests have shown that this is not correct because of the rigidity of such walls. If the walls are of reinforced concrete with their considerable rigidity, it is considered satisfactory to compute the moments at the faces of the walls (ACI Code 15.4.2). Should a footing be supporting a masonry wall with its greater flexibility, the code states that the moment should be taken at a section halfway from the face of the wall to its center.

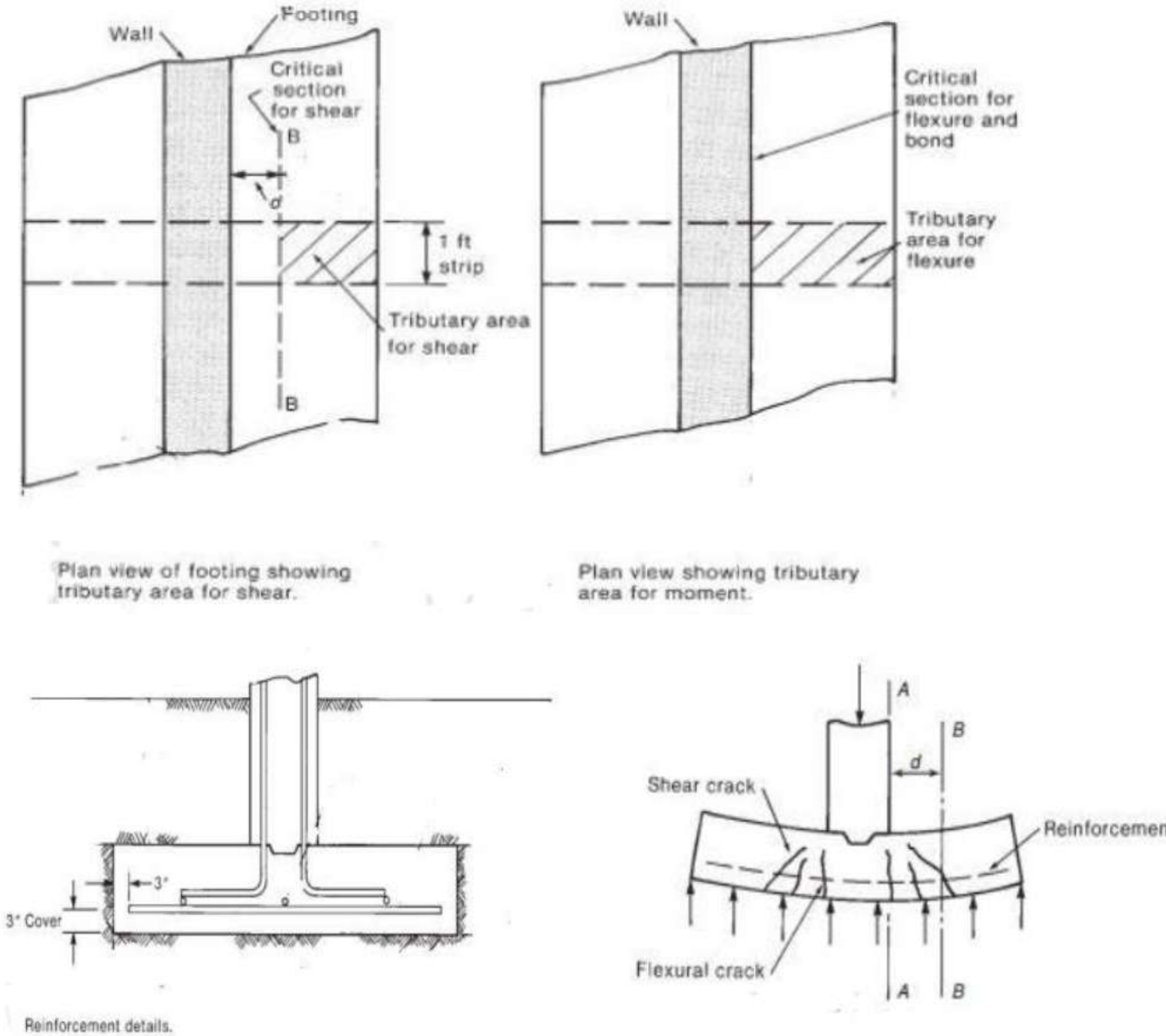


Figure 1:Wall footing behavior under bending

2.2.4.2 Spread Footing

Spread footings are *isolated or single-column square, rectangular, circular or even octagonal footing* used to support the load of a single column. These are the most commonly used footings, particularly where the loads are relatively light and the columns are not closely spaced. Upward soil pressures cause biaxial bending in the footing, for which reinforcement should be provided along both the axes respectively. Shear checks and moments are calculated as briefed in wall footing section. Moment is calculated at the face of the column.

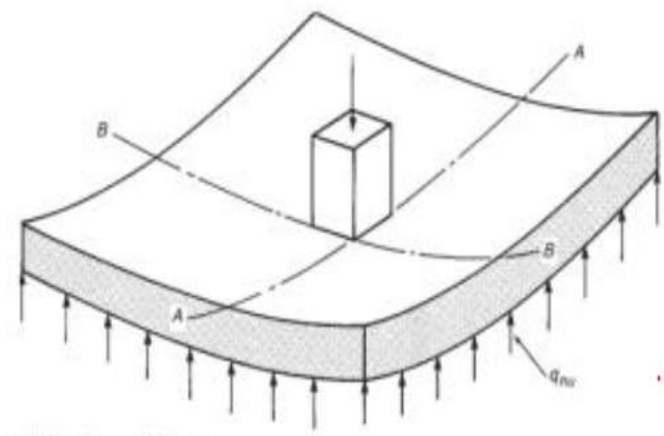


Figure 2: Two-way bending

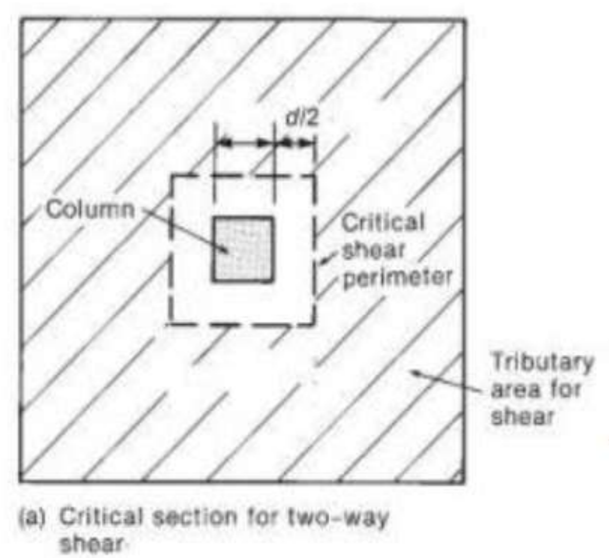


Figure 3: Square Footing Two-way shear

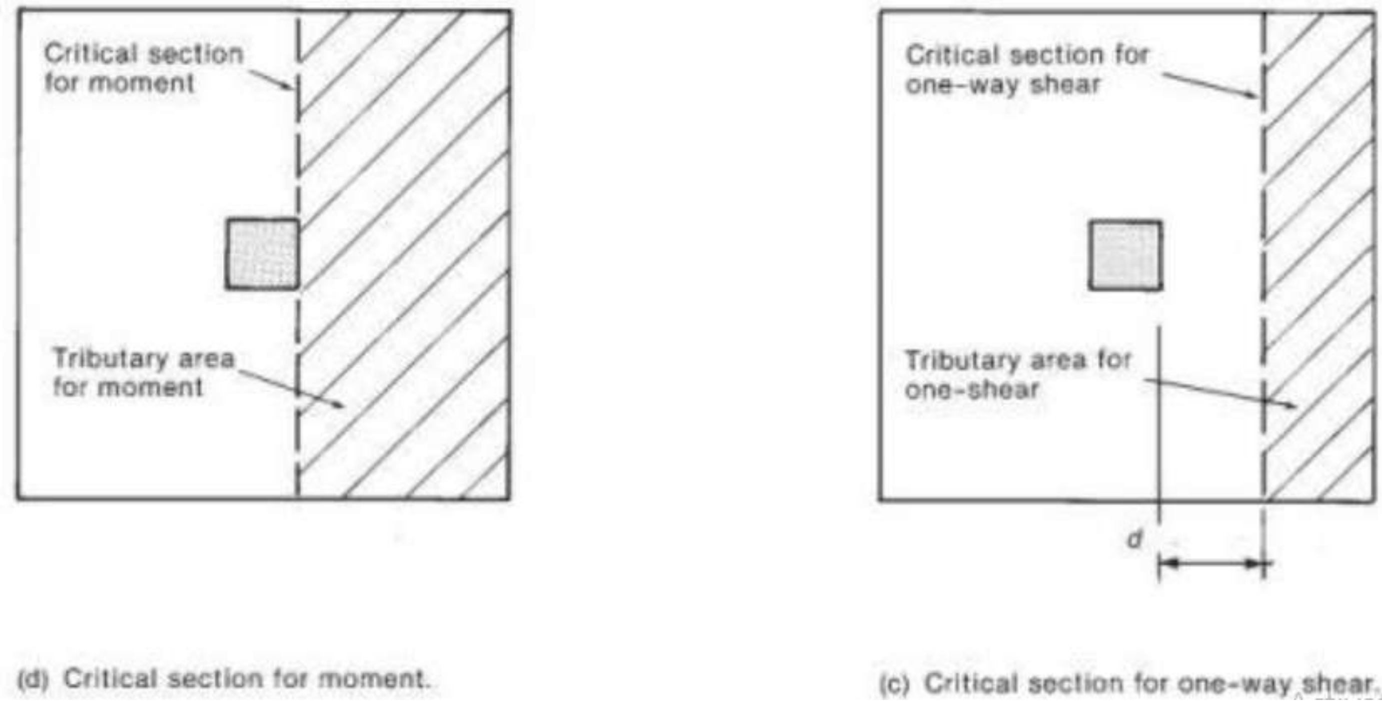


Figure 4: Critical section for moment and one-way shear

2.2.4.3 Combined Footing

Combined footings are used to support two or more column loads. A combined footing might be economical where two or more heavily loaded columns are so spaced that normally designed single-column footings would run into each other. Single-column footings are usually square or rectangular and, when used for columns located right at property lines, extend across those lines. A footing for such a column combined with one for an interior column can be designed to fit within the property lines.

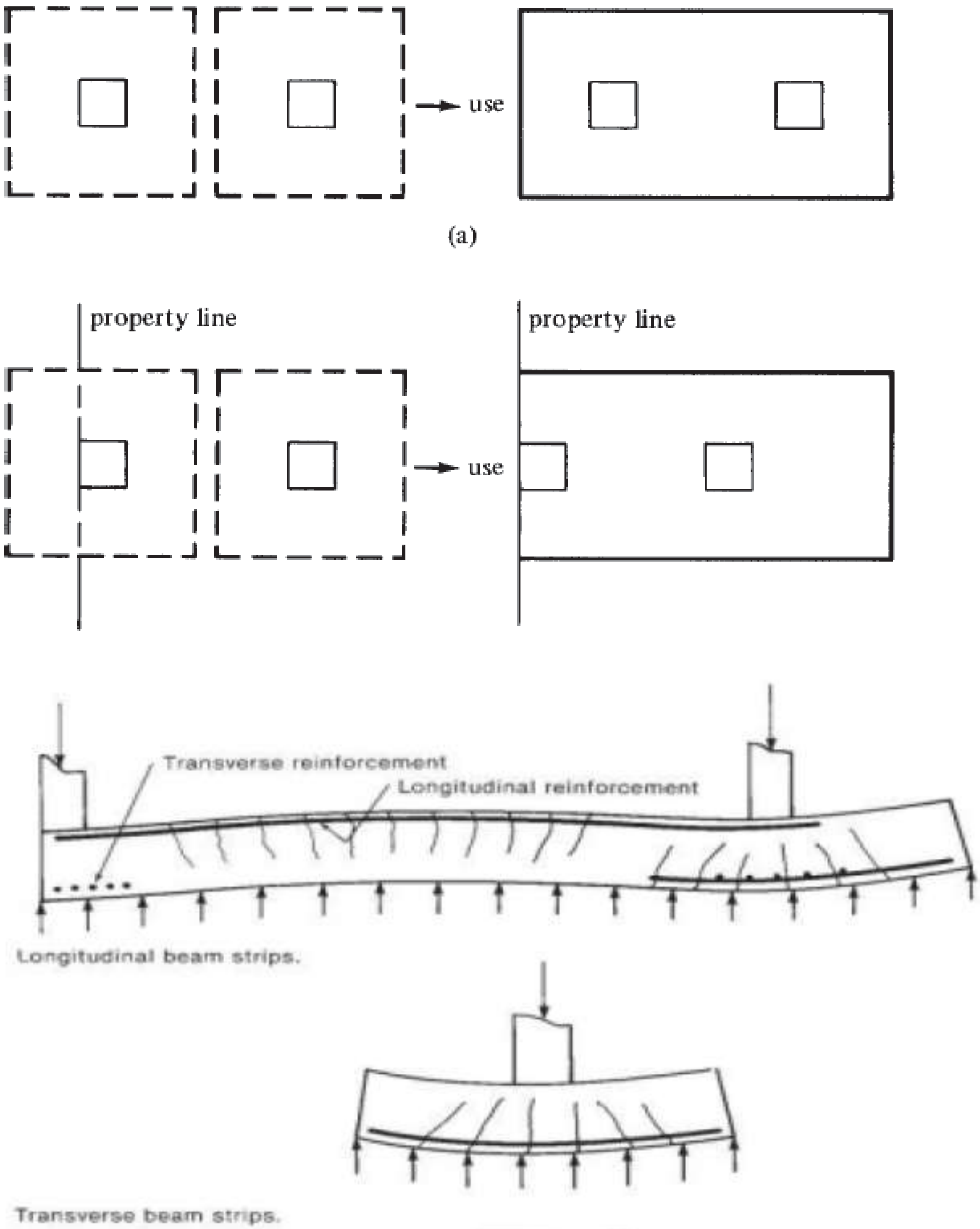


Figure 5: Combined Footing Structural Design

2.2.4.4 Mat Footing

A mat foundation supports all the columns in a building. A mat foundation would be used when buildings are founded on soft or irregular soils in locations where pile foundations cannot be used. Design is carried out by assuming that the foundation acts as an inverted slab. The distribution of soil pressure is affected by the relative stiffness of the soil and foundation, with more pressure being developed under the columns than at points between columns. Use of Mat foundation is pronounced under conditions, like; Low bearing capacity of soil, As Water Barrier to excessive uplift pressures, Soil is expansive and collapsible, Tolerable total and differential settlement.

There are various methods to design mat foundations, including; Rigid Method, Nonridged Methods (Coefficient of Subgrade Reaction, Winkler Method, Coupled Method, Pseudo Coupled Method, Multiple-Parameter Method, Finite Element Method.

Rigid Method: This method assumes that mat is much more rigid than underlying soils which means any distortions in the mat are too small to significantly affect the distribution of soil pressure. Finite Element Method (Non rigid Method): This analysis method divides the soil into a network of small elements, each with defined engineering properties and each connected to adjacent elements in a specified way. The structural and gravitational loads are applied and then elements are stressed and deformed accordingly. Thus in principal, should be an accurate representation of mat and should facilitate a precise and economical design.

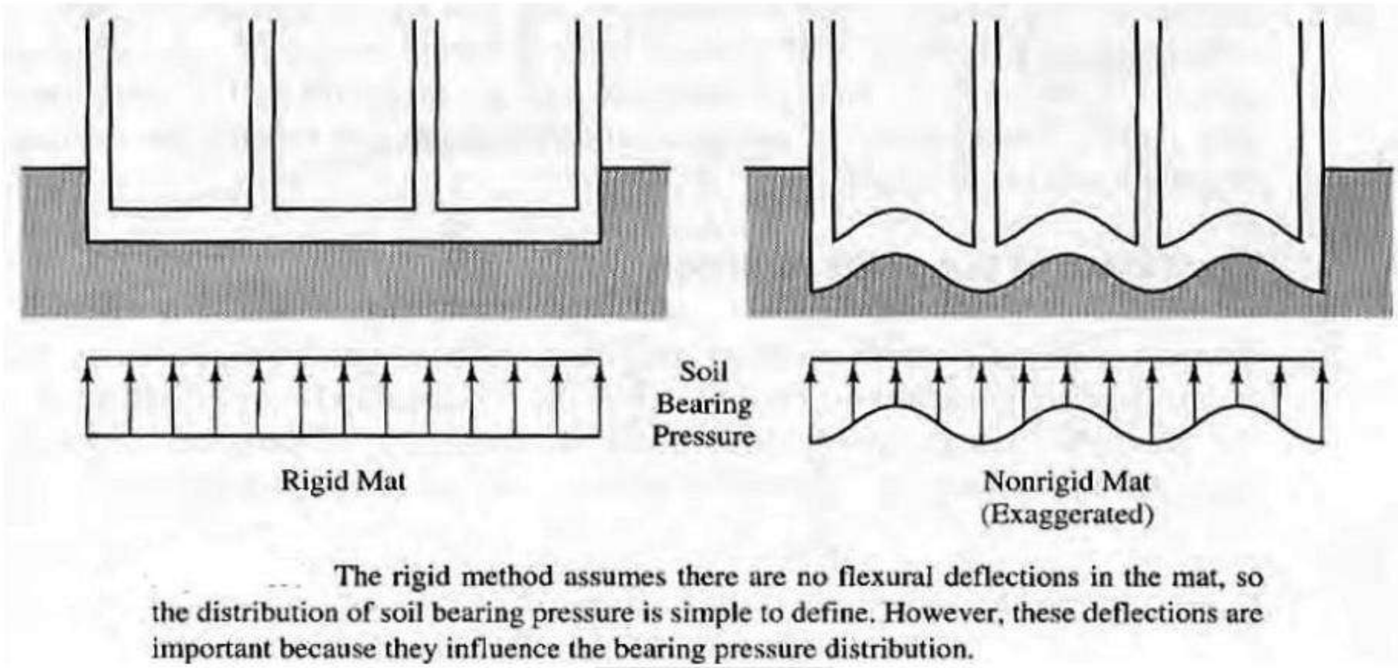


Figure 6: Rigid vs Non-Rigid MAT Behavior

2.2.4.5 SAFE

Safe is a tool for designing concrete floor and foundation system. It subdivides a large problem into smaller, simpler parts that are called finite elements. The simple equations that model these finite elements are then assembled into a larger system of equations that models the entire problem.

It translates the object-based model into an optimal finite-element model. Diagrams, contour plots, and animations, available in 2D and 3D views, display deformed configuration, component response, and min/max value of response data.

2.3 Deep Foundations

Deep Foundations were studied for lateral load capacity and axial load capacity and design charts were established for both.

2.3.1.1 Lateral Load Capacity

2.3.1.2 Single vertical piles subjected to lateral loads

The ultimate resistance of a vertical pile to a lateral load and the deflection of the pile as the load builds up to its ultimate value are complex matters involving the interaction between a semi-rigid structural element and the soil, which deforms partly elastically and partly plastically. Taking the case of a vertical pile unrestrained at the head, the lateral loading on the pile head is initially carried by the soil close to the ground surface. At a low loading the soil compresses elastically but the movement is sufficient to transfer some pressure from the pile to the soil at a greater depth. At a further stage of loading the soil yields plastically and transfers its load to greater depths. A short rigid pile unrestrained at the top and having a length to width ratio of less than 10 to 12 (Figure 6.18a) tends to rotate, and passive resistance develops above the toe on the opposite face to add to the resistance of the soil near the ground surface. Eventually the rigid pile will fail by rotation when the passive resistance of the soil at the head and toe are exceeded. The short rigid pile restrained at the head by a cap or bracing will fail by translation in a similar manner to an anchor block which fails to restrain the movement of a retaining wall transmitted through a horizontal tied rod (Figure 6.18b). The failure mechanism of an infinitely long pile is different. The passive resistance of the lower part of the pile is infinite, and thus rotation of the pile cannot occur, the lower part remaining vertical while the upper part deforms to a shape shown in Figure 6.19a. Failure takes place when the pile fractures at the point of maximum bending moment, and for the purpose of analysis a plastic hinge capable of transmitting shear is assumed to develop at the point of fracture. In the case of a long pile restrained at the head, high bending stresses develop at the point of restraint, for example, just beneath the pile cap, and the pile may fracture at this point (Figure 6.19b).

The pile head may move horizontally over an appreciable distance before rotation or failure of the pile occurs, to such an extent that the movement of the structure supported by the pile or pile group exceeds tolerable limits. Therefore, having calculated the ultimate load and divided it by the appropriate safety factor, it is still necessary to check that the permissible deflection of the pile is not exceeded. There are many inter-related factors which govern the behavior of laterally loaded piles. The dominant one is the pile stiffness, which influences the deflection and determines whether the failure mechanism is one of the rotation of a short rigid element, or is due to flexure followed by the failure in bending of a long pile. The type of loading, whether sustained (as in the case of earth pressure transmitted by a retaining wall) or alternating (say, from reciprocating machinery) or pulsating (as from the traffic loading on a bridge pier), influences the degree of yielding of the soil. External influences such as scouring around piles at sea-bed level, or the seasonal shrinkage of clay soils away from the upper part of the pile shaft, affect the resistance of the soil at a shallow depth. Methods of calculating ultimate resistance and deflection under lateral loads are presented in the following sections of this chapter. No attempt is made to give their complete theoretical basis. Various simplifications have been necessary in order to provide simple solutions to complex problems of soil–structure interaction, and the limitations of the methods are stated where these are particularly relevant. Most practical calculations are processes of trial and adjustment, starting with a very simple approach to obtain an approximate measure of the required stiffness, and embedment depth of the pile. The process can then be elaborated to some degree to narrow the margin of error, and to provide the essential data for calculating bending moments, shearing forces and deflections at the working load. Very elaborate calculation processes are not justified, because of the non-homogeneity of most natural soil deposits and the disturbance to the soil caused by installing piles. None of these significant factors can be reproduced in their entirety by the calculation methods. Failure mechanisms to be considered are failure of a short rigid pile by rotation or translation, and failure of a long slender pile in bending with local fracture and displacement of the soil near the pile head. Pile load tests, when undertaken as a means of determining the transverse resistance, need not necessarily be taken to the stage of failure, but the magnitude and line of action of the test load should conform to the design requirements.

The effects of interaction between piles in groups and fixity at the pile head are required to be considered. Where transverse resistance is determined by calculation, the method based on the concept of a modulus of horizontal sub grade reaction is permitted. The structural rigidity of the connection of the piles to the pile cap or substructure is to be considered as well as the effects of load reversals and cyclic loading. For any important foundation structure which has to carry high or sustained lateral loading, it is advisable to make field loading tests on trial piles having at least three different shaft lengths, in order to assess the effects of embedment depth and structural stiffness. For less important structures, or where there is previous experience of pile behavior to guide the engineer, it may be sufficient to make lateral loading tests on pairs of working piles by jacking or pulling them apart. These tests are rapid and economical to perform and provide a reliable check that the design requirements have been met.

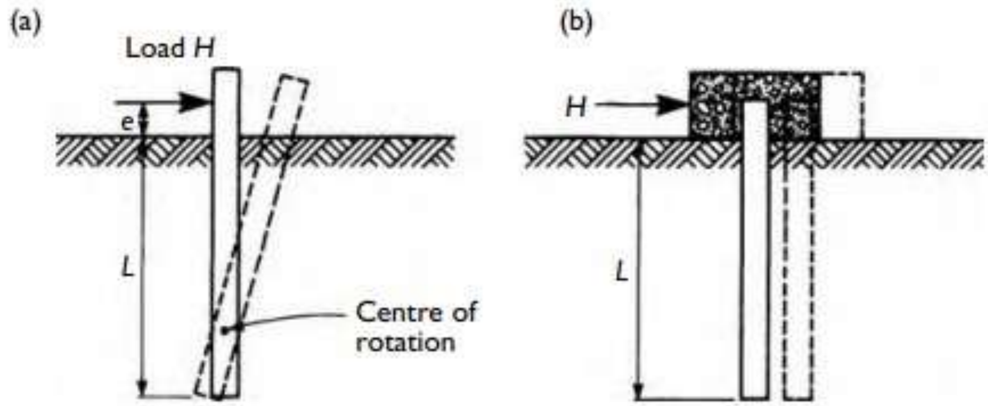


Figure 6.18 Short vertical pile under horizontal load.

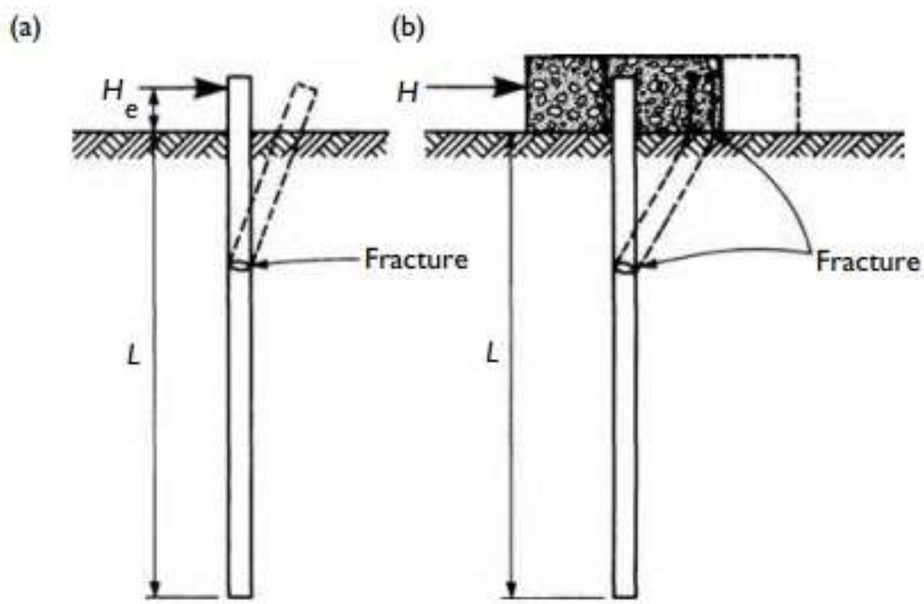


Figure 6.19 Long vertical pile under horizontal load (a) Free head (b) Fixed head.

Figure 7: Short and Long Pile under horizontal load

.

Calculating the ultimate resistance of short rigid piles to lateral loads, the first step is to determine whether the pile will behave as a short rigid unit or as an infinitely long flexible member. This is done by calculating the stiffness factors R and T for the particular combination of pile and soil. The stiffness factors are governed by the stiffness (EI value) of the pile and the compressibility of the soil. The latter is expressed in terms of a 'soil modulus', which is not constant for any soil type but depends on the width of the pile B and the depth of the particular loaded area of soil being considered. The soil modulus k has been related to Terzaghi's concept of a modulus of horizontal subgrade reaction. In the case of stiff over-consolidated clay, the soil modulus is generally assumed to be constant with depth. For this case

$$\text{stiffness factor } R = \sqrt[4]{\frac{EI}{kB}} \text{ (in units of length)}$$

For short rigid piles it is sufficient to take k in the above equation as equal to the Terzaghi modulus k1, as obtained from load/deflection measurements on a 305 mm square plate. It is related to the undrained shearing strength of the clay, as shown in Table 6.5

Table 6.5 Relationship of modulus of subgrade reaction (k_1) to undrained shearing strength of stiff over-consolidated clay

Consistency	*Firm to stiff*	*Stiff to very stiff*	*Hard*
Undrained shear strength (c_u) kN/m^2	50–100	100–200	>200
Range of k_1 MN/m^3	15–30	30–60	>60

For most normally consolidated clays and for granular soils the soil modulus is assumed to increase linearly with depth, for which

$$\text{stiffness factor } T = \sqrt[5]{\frac{EI}{n_h}} \text{ (in units of length)}$$

where soil modulus $K = n_h \times x/B$

Values of the coefficient of modulus variation *nh* were obtained directly from lateral loading tests on instrumented piles in submerged sand at Mustang Island, Texas. The tests were made for both static and cyclic loading conditions and the values obtained, as quoted by Reese et al., were considerably higher than those of Terzaghi. The investigators recommend that the Mustang Island values should be used for pile design. Other observed values of *nh* are as follows: Soft normally-consolidated clays: 350 to 700 kN/m3 , soft organic silts: 150 kN/m3 having calculated the stiffness factors R or T, the criteria for behavior as a short rigid pile or as a long elastic pile are related to the embedded length L as follows:

Pile type	*Soil modulus*	
	Linearly increasing	*Constant*
Rigid (free head)	$L \leq 2T$	$L \leq 2R$
Elastic (free head)	$L \geq 4T$	$L \geq 3.5R$

2.3.1.3 Brinch Hansen's method

Brinch Hansen's method can be used to calculate the ultimate lateral resistance of short rigid piles. The method is a simple one which can be applied both to uniform and layered soils. It can also be applied to longer semi-rigid piles to obtain a first approximation of the required stiffness and embedment length to meet the design requirements. The resistance of the rigid unit to rotation about point X is given by the sum of the moments of the soil resistance above and below this point. The passive resistance diagram is divided into a convenient number n of horizontal elements of depth L/n. The unit passive resistance of an element at a depth z below theground surface is then given by:

$$p_z = p_{oz}K_{qz} + cK_{cz}$$

where *poz* is the effective overburden pressure at depth z, c is the cohesion of the soil at depth z, and *Kqz* and *Kcz* are the passive pressure coefficients for the frictional and cohesive components respectively at depth z. Brinch Hansen has established values of *Kq* and *Kc* in relation to the depth z and the width of the pile B in the direction of rotation, as shown in Figure 6.22.

The total passive resistance on each horizontal element is and, by taking moments about the point of application of the horizontal load:

$$\sum M = \sum_{z=0}^{z=x} p_z \frac{L}{n}(e + z)B - \sum_{z=x}^{z=L} p_z \frac{L}{n}(e + z)B$$

The point of rotation at depth x is correctly chosen when the passive resistance of the soil above the point of rotation balances that below it. Point X is thus determined by a process of trial and adjustment. If the head of the pile carries a moment M instead of a horizontal force, the moment can be replaced by a horizontal force H at a distance e above the ground surface where M is equal to He. Where the head of the pile is fixed against rotation, the equivalent height e1 above ground level of a force H acting on a pile with a free head is given by:

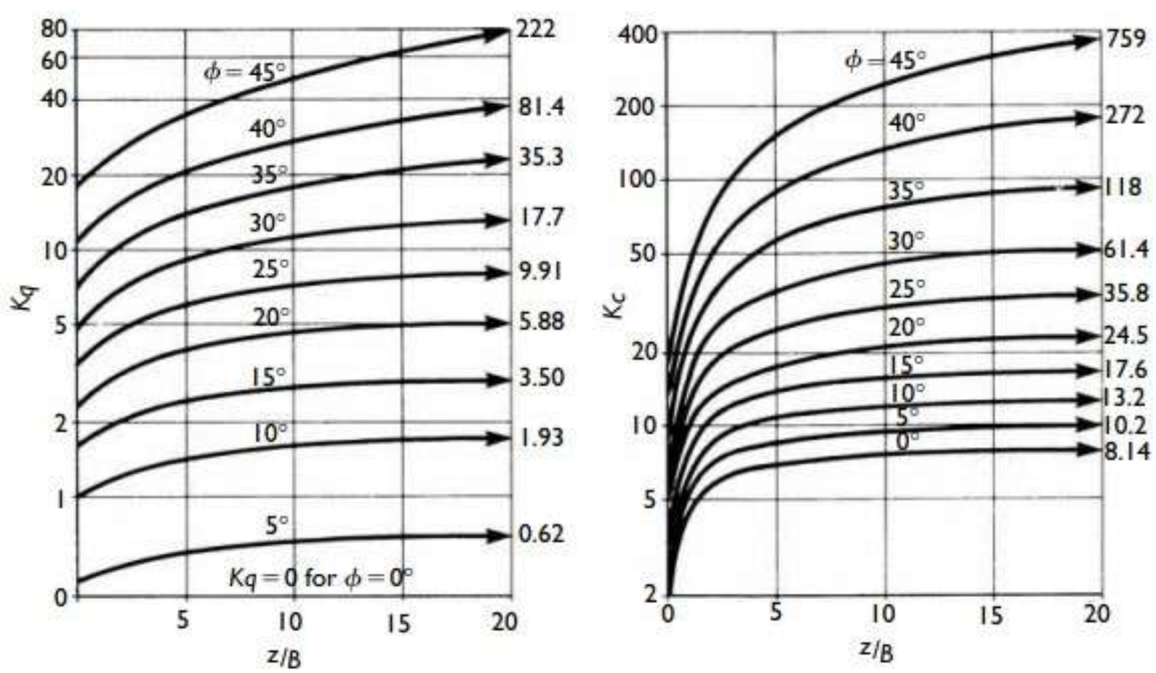


Figure 8:Brinch Hansen's Co-efficients Kc and Kq

where e is the height from the ground surface to the point of application of the load at the fixed head of the pile (Figure 6.21a), and zf is the depth from the ground surface to the point of virtual fixity. The depth zf is not known at this stage but for practical design purposes it can be taken as 1.5 m for a compact granular soil or stiff clay (below the zone of soil shrinkage in the latter case), and 3 m for a soft clay or silt. The American Concrete Institute recommends that zf should be taken as 1.4R for stiff, over-consolidated clays and 1.8T for normally consolidated clays, granular soils and silt, and peat. Having obtained the depth to the center of rotation, the ultimate lateral resistance of the pile to the horizontal force Hu can be obtained by taking moments about the point of rotation, when:

$$H_u(e + x) = \sum_{0}^{x} p_z \frac{L}{n} B(x - z) + \sum_{x}^{x+L} p_z \frac{L}{n} + B(z - x)$$

The final steps in Brinch Hansen's method are to construct the shearing force and bending moment diagrams (Figure 6.21b and c). The ultimate bending moment, which occurs at the point of zero shear, should not exceed the ultimate moment of resistance Mu of the pile shaft. The appropriate load factors are applied to the horizontal design force to obtain the ultimate force Hu.

When applying the method to layered soils, assumptions must be made concerning the depth z to obtain *Kq* and *Kc* for the soft clay layer, but z is measured from the top of the stiff clay stratum to obtain *Kc* for this layer, as shown in Figure 6.23

The un-drained shearing strength cu is used in equation 6.14 for short-term loadings such as wave or ship-berthing forces on a jetty, but the drained effective shearing strength values are used for long-term sustained loadings such as those on retaining walls. A check should be made to ensure that there is an adequate safety factor for undrained conditions in the early stages of loading.

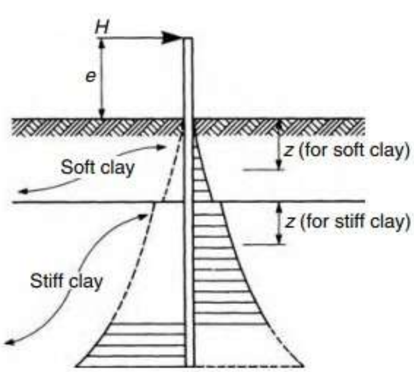


Figure 9: Reactions in layered soil on vertical pile under horizontal load

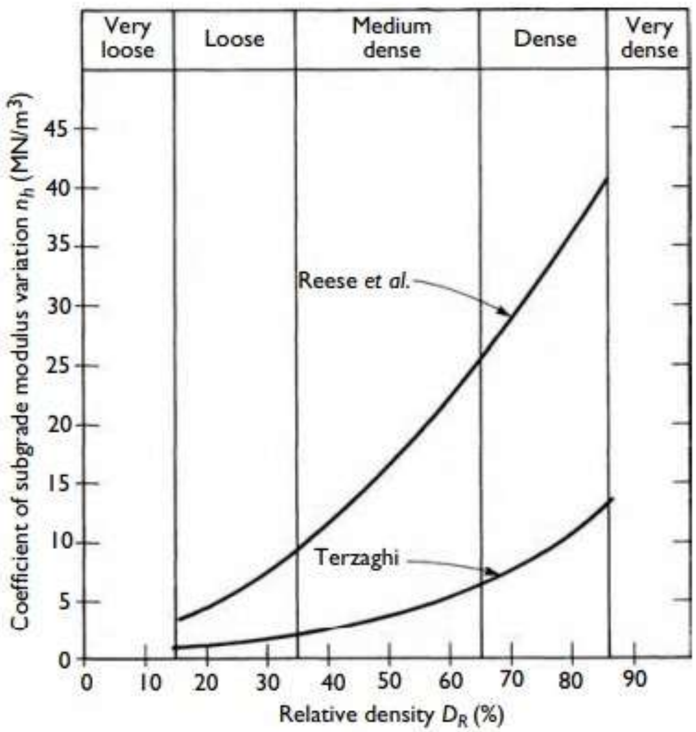


Figure 10: Relationship between coefficient of modulus variation and relative density

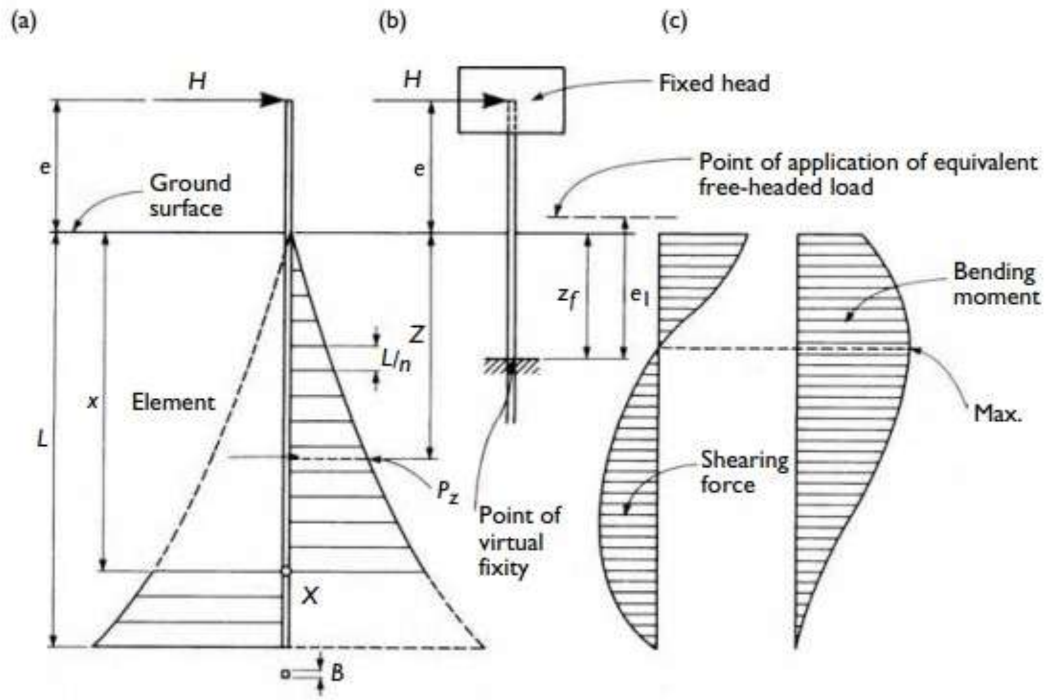


Figure 11: Brinvh Hansen's method for calculating ultimate resistance of short piles (a) Soil reactions (b) Shearing force diagram (c) Bending moment diagram

2.3.1.4 Calculating the ultimate resistance of long piles

The passive resistance provided by the soil to the yielding of an infinitely long pile is infinite. Thus the ultimate lateral load which can be carried by the pile is determined solely from the ultimate moment of resistance Mu of the pile shaft. A simple method of calculating the ultimate load, which may be sufficiently accurate for cases of light loading on short or long piles of small to medium width, for which the cross-sectional area is governed by considerations of the relatively higher compressive loading, is to assume an arbitrary depth zf to the point of virtual fixity. Then from Figure 6.24: ultimate lateral load on free-headed pile Hu Mu/(e+zf), ultimate lateral load on fixed-headed pile Hu 2Mu /(e+zf).

Arbitrary values for zf which are commonly used are given in the reference to the Brinch Hansen method. It has already been stated that vertical piles offer poor resistance to lateral loads. However, in some circumstances it may be justifiable to add the resistance provided by the passive resistance of the soil at the end of the pile cap and the friction or cohesion on the embedded sides of the cap. The pile cap resistance can be taken into account when the external loads are transient in character, such as wind gusts and traffic loads, but the resulting elastic deformation of the soil must not be so great as to cause excessive deflection and hence overstressing of the piles.

2.3.1.5 The deflection of vertical piles carrying lateral loads

A simple method which can be used to check that the deflections due to small lateral loads are within tolerable limits and as an approximate check on the more-rigorous methods described below, is to assume that the pile is fixed at an arbitrary depth below the ground surface and then to calculate the deflection as for a simple cantilever either free at the head, or fixed at the head but with freedom to translate

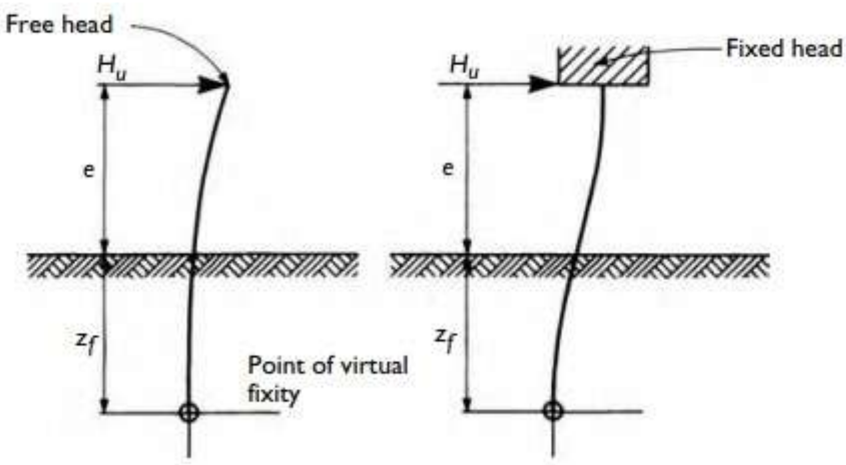


Figure 12: Piles under horizontal load considered as simple cantilever

Thus from Figure 6.24

$$\text{deflection at head of free-headed pile } y = \frac{H(e + z_f)^3}{3EI} \tag{6.20}$$

and

$$\text{deflection at head of fixed-headed pile } y = \frac{H(e + z_f)^3}{12EI} \tag{6.21}$$

where E is the elastic modulus of the material forming the pile shaft, and I is the moment of inertia of the cross-section of the pile shaft. Depths may be arbitrarily assumed for zf.

2.3.1.6 Elastic analysis of laterally loaded vertical piles

The suggested procedure for using this is to first calculate the ultimate load Hu for a pile of given cross-section (or to determine the required cross-sections for a given ultimate load) and then to divide Hu by an arbitrary safety factor to obtain trial working load H. The alternative procedure is to calculate the deflection y0 at the ground surface for a range of progressively increasing loads H up to the value of Hu. The working load is then taken as the load at which y0 is within the allowable limits. As a first approximation, Hu can be obtained by the Brinch Hansen method. It may be necessary to determine the bending moments, shearing forces, and deformed shape of a pile over its full depth at a selected working load. These can be obtained for

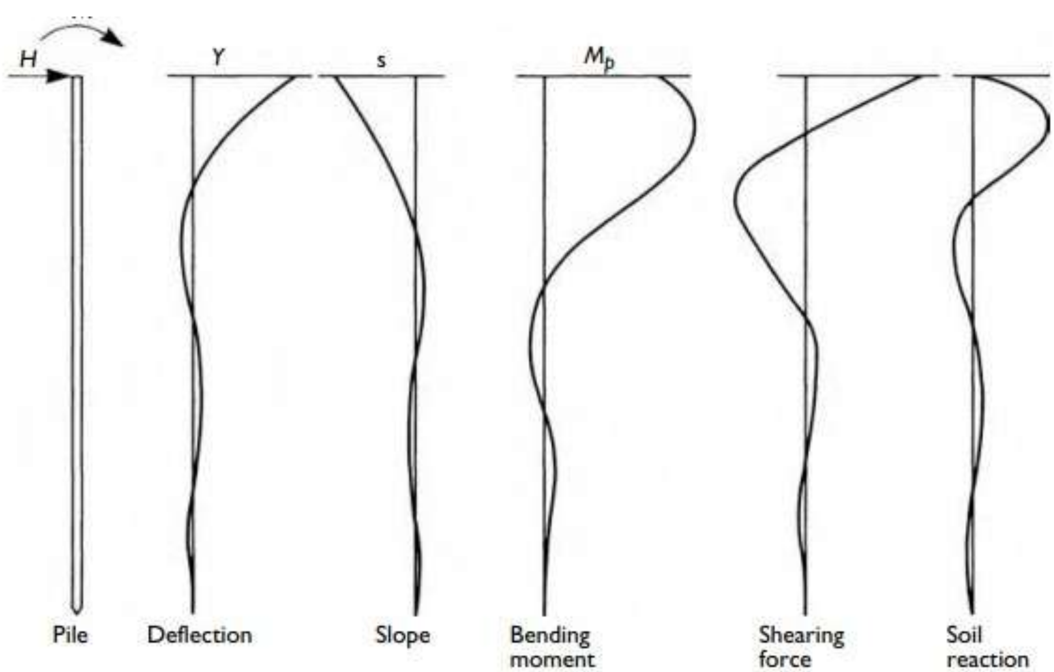


Figure 13: Deflections, slopes, bending moments, shearing forces and soil reactions for elastic conditions

workingload conditions on the assumption that the pile behaves as an elastic beam on a soil behaving as a series of elastic springs. Calculations for the bending moments, shearing forces, deflections, and slopes of laterally loaded piles are necessary when considering their behavior as energy absorbing members resisting the berthing impact of ships, or the wave forces in offshore platform structures. Reese and Matlock have established a series of curves for normally consolidated and cohesion-less soils for which the elastic modulus of the soil Es is assumed to increase from zero at the ground surface in direct proportion to the depth. The deformed shape of the pile and the corresponding bending moments, shearing forces, and soil reactions are shown in Figure 6.25. Coefficients for obtaining these values are shown for a lateral load H on a free pile head in Figure 6.26a to e, and for a moment applied to a pile head in Figure 6.27a to e. The coefficients for a fixed pile head are shown in Figure 6.28a to c. For combined lateral loads and applied moments the basic equations for use in conjunction with Figures 6.26 and 6.27 are as follows:

$$\text{Deflection } y = y_A + y_B = \frac{A_y H T^3}{EI} + \frac{B_y M_t T^2}{EI}$$

$$\text{Slope} = s_A + s_B = \frac{A_s H T^2}{EI} + \frac{B_s M_t T}{EI}$$

$$\text{Bending moment} = M_A + M_B = A_m HT + B_m M_t$$

$$\text{Shearing force} = V_A + V_B = A_v H + \frac{B_v M_t}{T}$$

$$\text{Soil reaction} = P_A + P_B = \frac{A_p H}{T} + \frac{B_p M_t}{T^2}$$

For a fixed pile head, following equations are used

$$\text{Deflection} = y_F = \frac{F_y H T^3}{EI}$$

$$\text{Bending moment} = M_F = F_m HT$$

$$\text{Soil reaction} = P_F = F_p \frac{H}{T}$$

In equations 6.22 to 6.29, H is the horizontal load applied to the ground surface, T (a stiffness factor) (as equation 6.12), Mt is the moment applied to the head of the pile, Ay and By are deflection coefficients (Figures 6.26a and 6.27a), As and Bs are slope coefficients (Figures 6.26b and 6.27b), Am and Bm are bending-moment coefficients (Figures 6.26c and 6.27c), Av and Bv are shearing-force coefficients (Figures 6.26d and 6.27d), Ap and Bp are soil resistance coefficients (Figures 6.26e and 6.27e), Fy is the deflection coefficient for a fixed pile head (Figure 6.28a), Fm is the moment coefficient for a fixed pile head (Figure 6.28b), and Fp is the soil resistance coefficient for a fixed pile head (Figure 6.28c). In Figures 6.26 to 6.28 the above coefficients are related to a depth coefficient Z for various values of Zmax, where Z is equal to the depth x at any point divided by T (i.e. Z x/T) and Zmax is equal to L/T. The use of curves in Figure 6.28 is illustrated in Example 6.6. The case of a load H applied at a distance e above the ground surface can be simulated by assuming this to produce a bending moment Mt equal tob H*e, this value of Mt being used in equations 6.22 to 6.29. The moments Ma produced by load H applied at the soil surface are added arithmetically to the moments Mb produced by moment Mt applied to the pile at the ground surface. This yields the relationship between the total moment and the depth below the soil surface over the embedded length of the pile. The deflection of a pile due to a lateral load H at some distance above the soil surface is calculated in the same manner. The deflections of the pile and the corresponding slopes due to the load H at the soil surface are calculated and added to the values calculated for moment Mt applied to the pile at the surface. To obtain the deflection at the head of the pile, the deflection as for a free-standing cantilever fixed at the soil surface is calculated and added to the deflection produced at the soil surface by load H and moment Mt , together with the deflection corresponding to the calculated slope of the pile at the soil surface. This procedure is illustrated in Example 8.2. Davisson and Gill(6.15) have analysed the case of elastic piles in an elastic soil of constant modulus. The bending moments and deflections are related to the stiffness coefficient R (equation 6.11) but in this case the value of K is taken as Terzaghi's subgrade modulus k1, using the values shown in Table 6.5. The dimensionless depth coefficient Z in Figure 6.29 is equal to x/R. From these curves, deflection and bending moment coefficients are obtained for free-headed piles carrying a moment at the pile head and zero lateral load (Figure 6.29a) and for free-headed piles with zero moment at the pile head and carrying a horizontal load (Figure 6.29b). These curves are valid for piles having an embedded length L greater than 2R and different moment and deflection curves are shown for values of Zmax L/R of 2, 3, 4, and 5. Piles longer than 5R should be analysed for Zmax

5. The equations to be used in conjunction with the curves in Figure 6.29 are as follows:

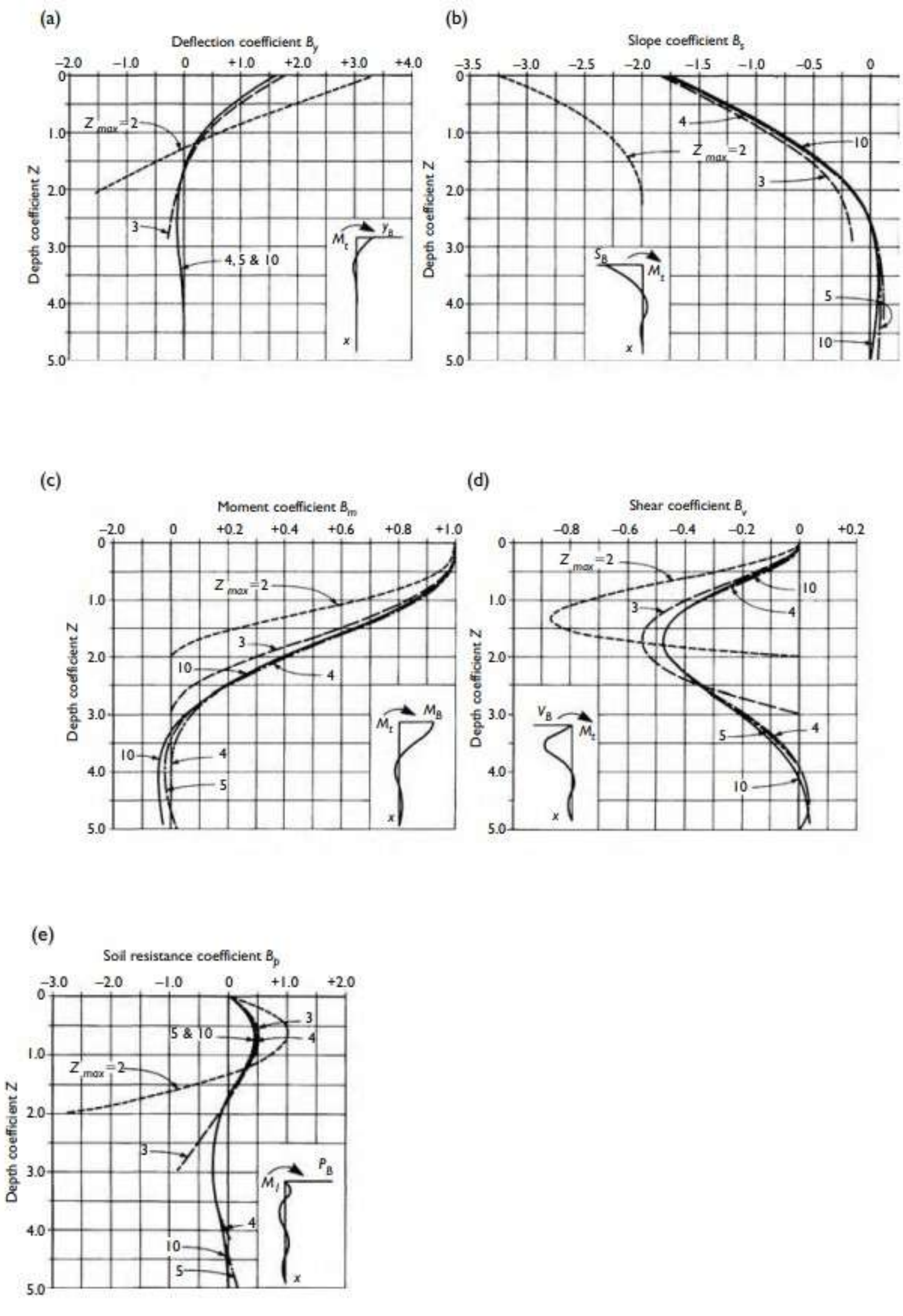


Figure 14: Coefficients for piles with moment at free head in soil with linearly increasing modulus

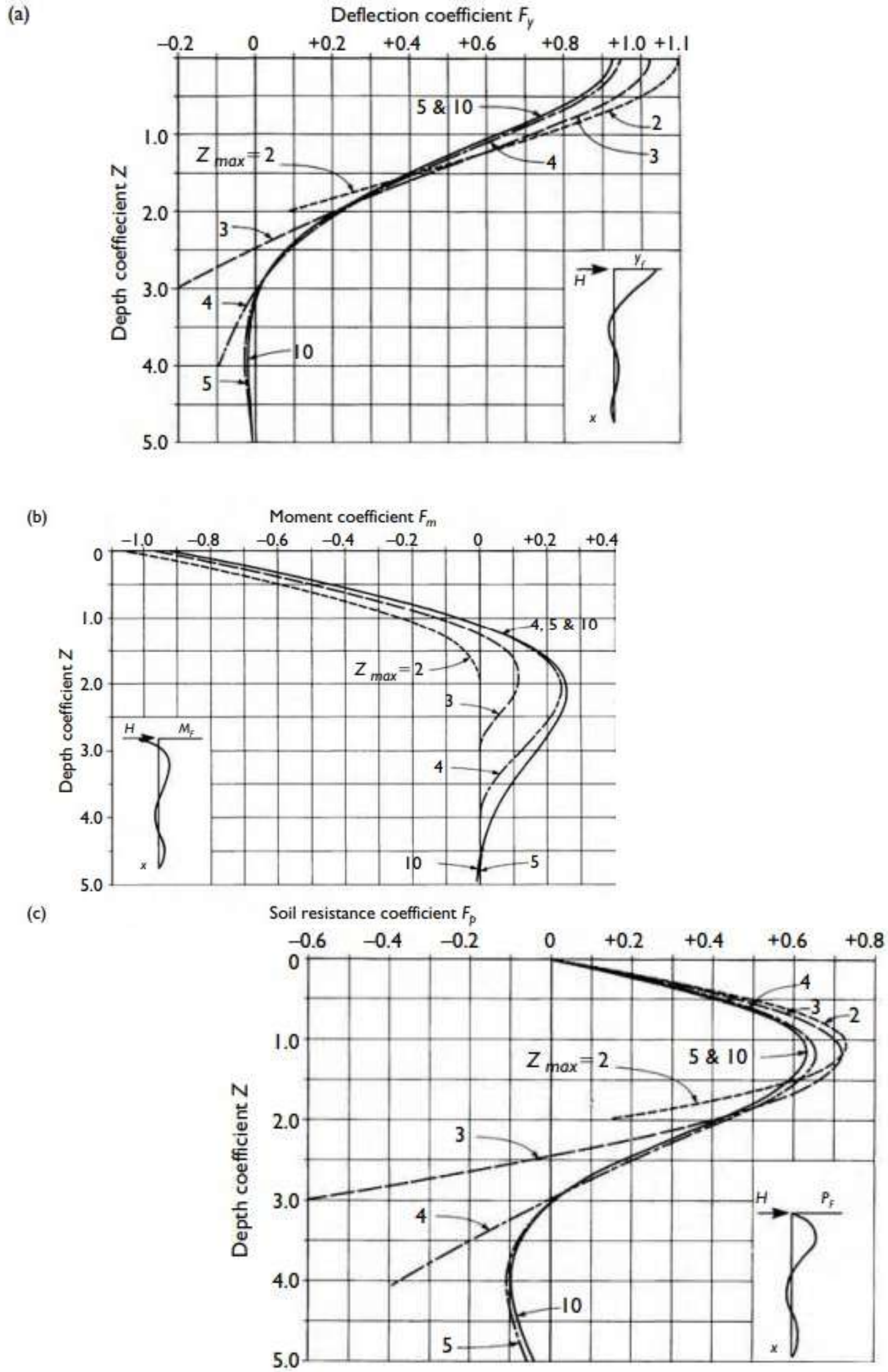


Figure 15: Coefficients for fixed headed piles with lateral load in soil with linearly increasing modulus

Load on pile head		*For free-headed pile:*
Moment	M;	Bending moment $= MM_m$
Moment	M;	Deflection $= My_m\frac{R^2}{EI}$

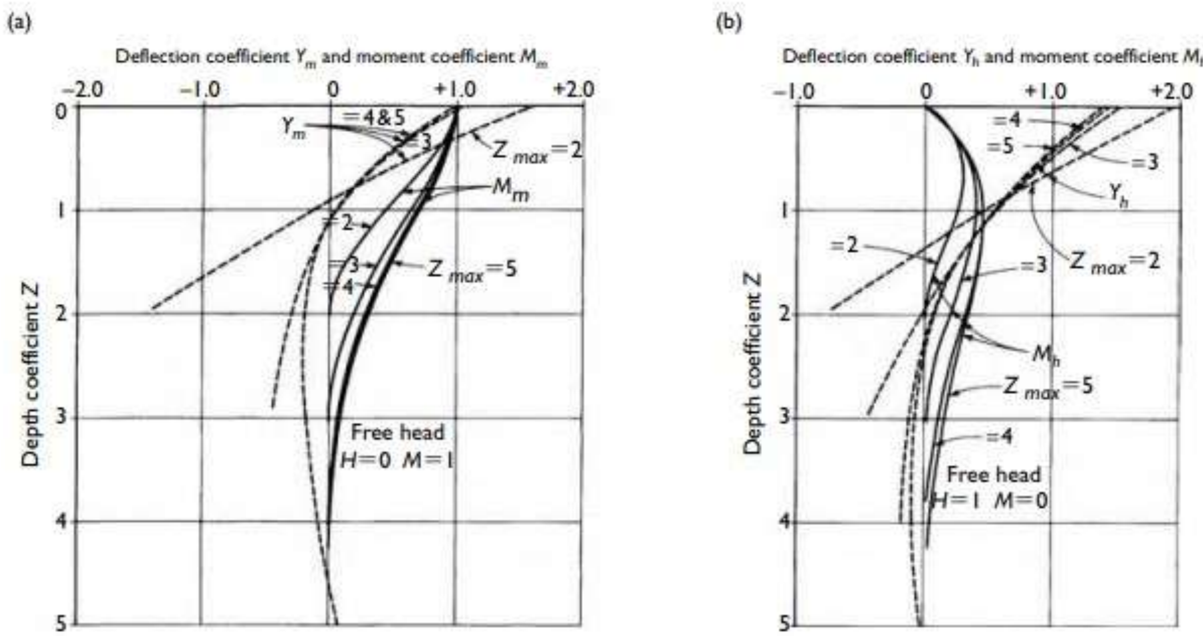


Figure 16: Coefficients for free headed piles carrying lateral load or moment at pile in soil of constant modulus

Horizontal load	H;	Bending moment $= HM_hR$
Horizontal load	H;	Deflection $= Hy_h\frac{R^3}{EI}$

The effect of fixity at the pile head can be allowed for by plotting the deflected shape of the pile from the algebraic sum of the deflections and then applying a moment to the head which results in zero slope for complete fixity, or the required angle of slope for a given degree of fixity. The deflection for this moment is then deducted from the calculated value for the free-headed pile. Conditions of partial fixity occur in jacket-type offshore platform structures where the tubular jacket member only offers partial restraint to the pile that extends through it to below sea-bed level. Where marine structures are supported by long piles (L/4T), Matlock and Reese have simplified the process of calculating deflections by re-arranging equation to incorporate a deflection coefficient Cy. Then

$$y = C_y \frac{HT^3}{EI}$$

where

$$C_y = A_y + \frac{M_t B_y}{HT}$$

Values of Cy are plotted in terms of the dimensionless depth factor Z(=x/T for various values of Mt/HT in Figure 6.30. Included in these curves are the fixed-headed case (i.e. Mt/HT =-0.93) and the free-headed case (i.e. Mt =0)

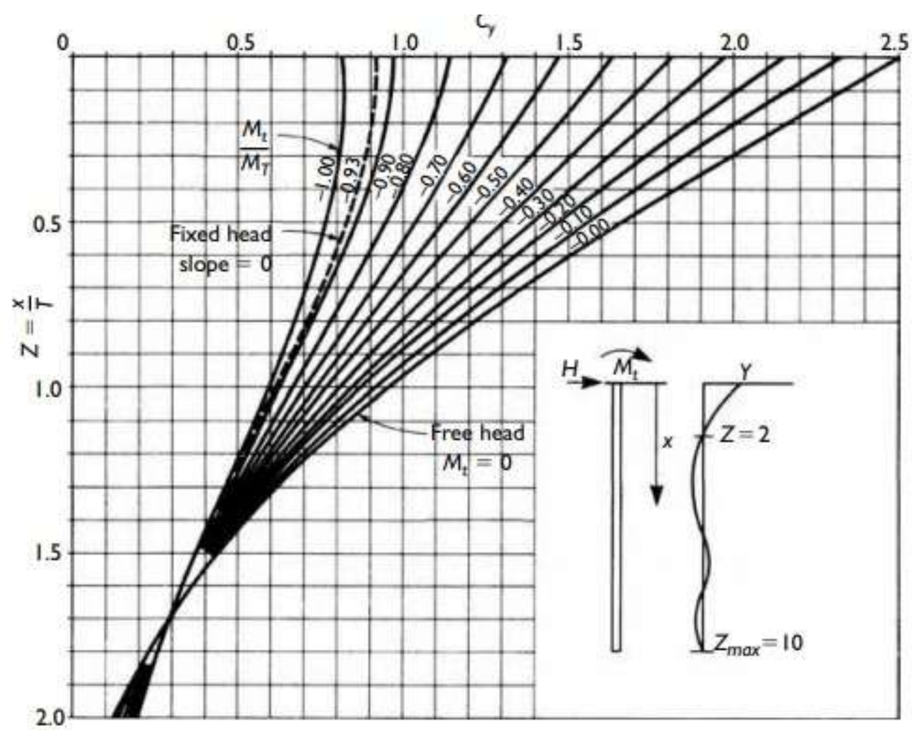


Figure 17: Coefficients for calculating deflection of pile carrying both moment and lateral load

The elastic deflections of piles in layered soils, each soil layer having its individual constant modulus, have been analysed by Davisson and Gillwho have produced design charts for this condition.

2.4 Literature Review for OASYS

2.4.1 Pile Designing Suite

Oasys Pile (Version 19.6)

General

Oasys Pilecalculates the vertical load carrying capacities and vertical settlements of a range of individual piles in a layered soil deposit. The theory is based on both conventional and new methods for drained (frictional) and un-drained (cohesive) soils. Settlements are calculated for solid circular sections without under-ream.

Program Features

Capacity analysis, settlement analysis, or both can be performed for a range of pile lengths and cross-sections in different soil profiles.

Settlements are calculated for only solid circular cross-sections without under-ream. The soil is specified in layers. Each layer is set to be drained (frictional) or undrained (cohesive) and appropriate strength parameters are specified. Maximum values can be set for ultimate soil/shaft friction stress and end bearing stress within each layer. Levels may be specified as depth below ground level; or elevation above ordnance datum (OD).

Pore water pressures within the soil deposit can be set to hydrostatic or piezo metric. Pile capacities may be calculated for a range of pile lengths and a range of cross-section types such as circular, square and H-section. The circular and square cross-sections may be hollow or solid, whereas the H-section is only solid. Under-reams or enlarged bases may be specified.

Pile settlements may be calculated for a range of pile lengths and a range of solid circular cross-sections without under-ream. There are three approaches available to calculate the capacity of the pile.

1. Working load approach

2. Limit-state approach and

3. Code-based approach

API T-z curve

API curve was used to calculate Settlement "
Material Type- selection has to be made between two materials: sand and clay.

zc- the movement required to mobilize maximum stress. This is active only when the material type is sand.

tRES/tmax- the ratio of mobilized stress to maximum stress. This is active only when the material type is clay.

Oasys Alp

General

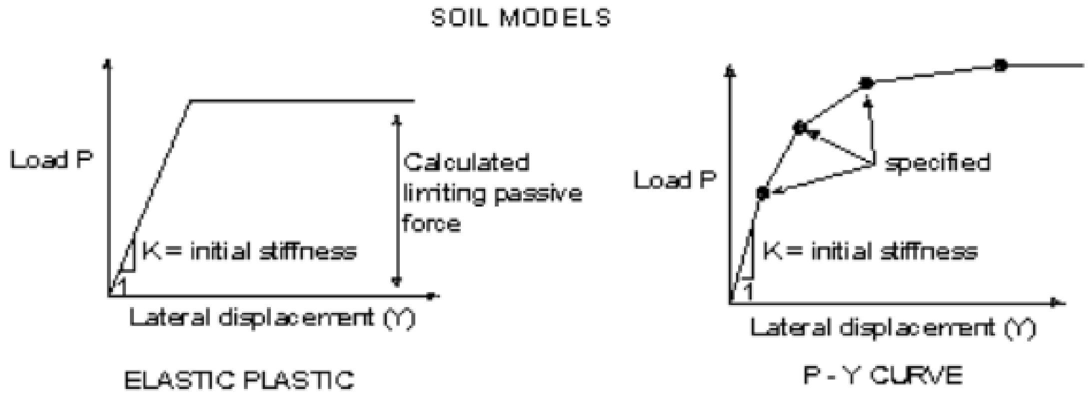
Alp(Analysis of Laterally Loaded Piles) is a program that predicts the pressures, horizontal movements, shear forces and bending moments induced in a pile when subjected to lateral loads, bending moments and imposed soil displacements.

The pile is modelled as a series of elastic beam elements. The soil is modelled as a series of non-interactive, non-linear "Winkler type" springs. The soil load-deflection behavior can be modelled either assuming an Elastic-Plastic behavior, or by specifying or generating load-deflection (i.e. P-Y) data. Two stiffness matrices relating nodal forces to displacements are developed. One represents the pile in bending and the other represents the soil.

Program Features

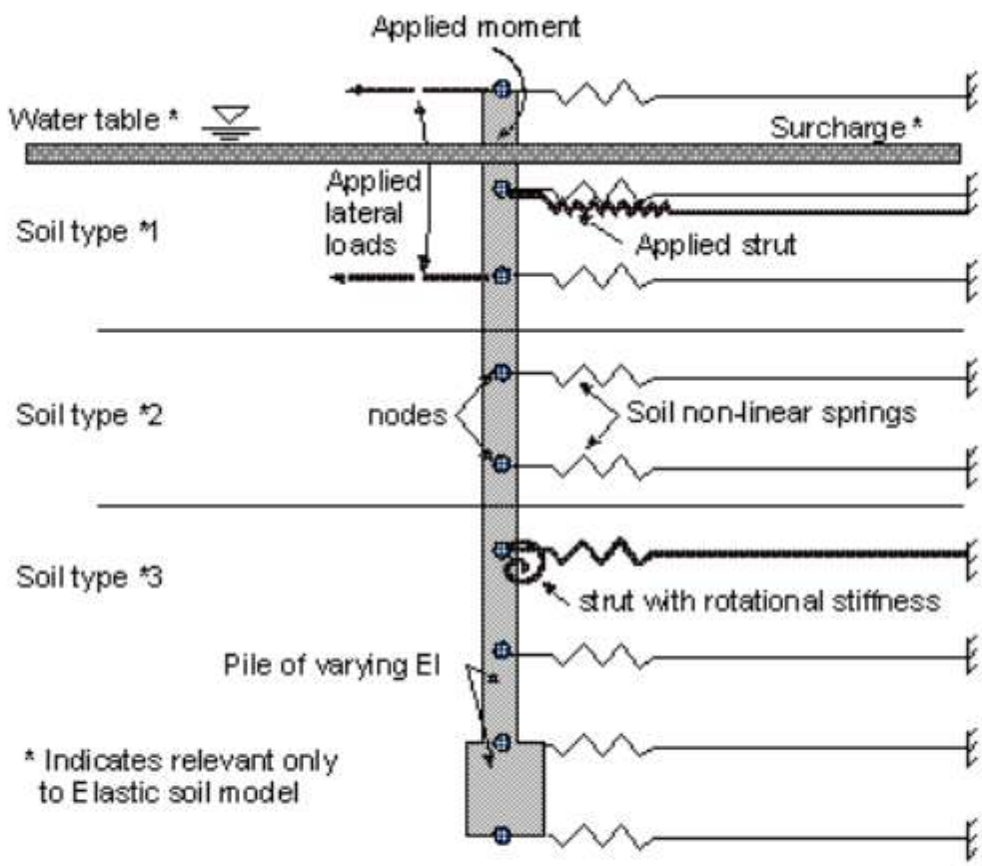
The main features of the problem analyzed by Alpare summarized below and represented diagrammatically.

The geometryof the pile is specified by a number of nodes, which may be specified directly by the user or generated automatically based on the elevation of soil boundaries, loads, restraints and displacements.



The positions of these nodes are expressed in terms of reduced level. Pile stiffness is constant between nodes, but may change at nodes. Three methods of modelling the soil are available.

1. Elastic-Plastic
2. Specified P-Y curves
3. Generated P-Y curves



API RP2A 21st Edition (2000)

From this software we have used this curve for calculations of lateral deflection in case of sand.

Soft Clay

P-Y curves for soft clay are calculated using the method established by Matlock (1970).

OasysAdSec

It is a program used for non-linear analysis of sections, particularly concrete sections. Analysis can be carried out by selected concrete design codes i.e. ACI, Eurocode. It also carries serviceability analysis of sections.

Analysis Type

ULS: For the Ultimate Limit State (ULS) the section analysis options are:

1. the ultimate moment capacity of the section

2. stresses from the ultimate applied load

3. ultimate axial force/moment (N/M or P/M) interaction charts

4. ultimate moment (Myy/Mzz) interaction chart (for biaxial bending only)

SLS: For the Serviceability Limit State (SLS) the program calculates:

1. cracking moment

2. stresses, strains, stiffness and crack widths for each applied loading and strain

3. moment-curvature and moment stiffness charts.

CHAPTER 3

3 METHODOLOGY

3.1 Design Chart Methodology

3.1.1 Input Data

3.1.2 Bearing Capacity inputs

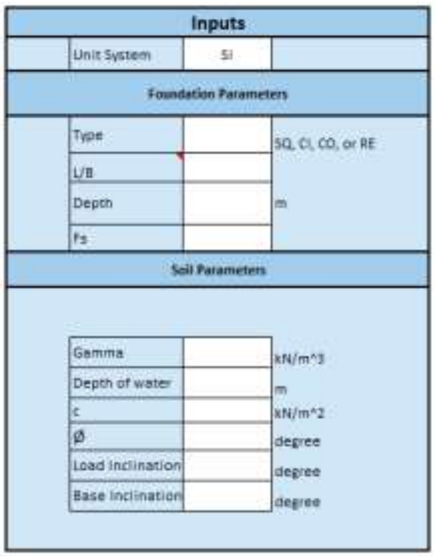


Figure 18: Bearing Capacity Inputs

Depending on settlement method, input needed are

3.1.3 Burland and Bubidge Inputs

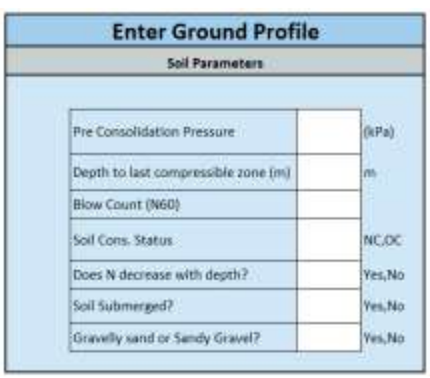


Figure 19:Burland and Burbidge Inputs

3.1.4 Terzaghi 1-D Consolidation Inputs

Enter Ground Profile						
Layer	Depth		$\frac{Cc}{1+e}$	$\frac{Cr}{1+e}$	γ lb/ft^3	σ'm lb/ft^2
	From	To				
1	0	2				
2	2					
3	5					
4	11					
5	23					

Figure 20:Terzaghi 1-D Consolidation Inputs

3.1.5 Meyerhof Inputs

Enter Ground Profile			
Soil Parameters			
	Blow Count (N60)		

Figure 21:Meyerhoff Inputs

3.1.6 Schmertmann Inputs

Enter Ground Profile			
Layer	Depth		Es (kPa)
	From	To	
1			
2			
3			
4			
5			

Figure 22:Schmertmann Inputs

3.1.7 Bearing Capacity Calculations

Some data given in input were utilized to get bearing capacity for that particular soil.

Five methods were used. We took the average of values from all listed methods.

- Terzaghi

- Meyerhoff
- Hansen
- Vesic
- Skempton

3.1.8 Settlement Calculations

Non-Cohesive Soil

Following methods were used for non-cohesive soils:

- **Schmertmann**
- **BurlandAnd Burbidge**
- **Meyerhof**

Cohesive Soil

- Terzaghi 1-D Consolidation
- Skempton and Bjerrum

3.1.9 Using Automated Workbook

After entering all the inputs, a button is available which when clicked generates the desired chart after taking some time for calculations.

3.2 Spread Footing Structural Design

We decided to design Excel spreadsheets for all spread footings (Rectangular, Circular, Square and Wall Footing), Combined Footing and Mat Footing. After the design of all these footings, we compared their results with SAFE software.

3.2.1 Inputs:

Excel workbooks require following input data

- Service Loads(Dead & Live)
- Column Size
- Allowable Bearing Pressure of Soil
- Unit Weight of Soil
- Footing Thickness
- Bar #
- Concrete Cover
- Concrete Stress Requirements
- Steel Grade

Input Data		
lambda	1	
fc'	3000	psi
fy	60000	psi
column Size	16	in
Dead Load	200000	lbs
Live Load	160000	lbs
Allowable Bearing Pressure(q)	5000	psf
Unit Weight of Soil	100	pcf
Unit Weight of Concrete	150	pcf
Footing Depth	5	ft
Bar #	8	
Effective Depth	29	in
Alpha	40	

Figure 23: Inputs for Isolated Footings Structural Design

3.2.2 How to use:

Following parameter has to be entered on trial basis until all checks are satisfied.

- Footing Thickness

3.2.3 How Auto mated sheet works:

It works in the following manner

- Factored Loads Calculation
- Factored Ultimate Soil Bearing Pressure
- Eccentricity Checks

- One-Way Eccentricity
- Two-Way Eccentricity

- One-Way Shear Check
- Two-Way Shear Check
- Flexure Design and Steel Calculation for Longer and Shorter Direction
- Development Length Calculation
- Check for Strain

3.2.4 Outputs:

Results of Structural Design are as follows

- Footing Dimensions
  - Width
  - Length
  - Thickness
- Details of Steel
  - Required Bar Number
  - Total Numbers of Bars
- Checks
  - Eccentricity Check
  - One-Way Shear Check for Thickness
  - Two-Way Shear Check for Thickness
  - Strain Check
  - Moment Capacity Check Along Long Direction
  - Moment Capacity Check Along Shorter Direction

REQUIRED STEEL AREA			
As Required	As min		
As=Mn/ϕfy(d-a/2)	As min=3*√fc'*	8.65 sq. in	
3.25	As min=200*bw	10.52 sq. in	
	above two Areas	10.52 sq. in	
Use Bottom Bars(Both Sides)	14.00	#	8.00

REQUIRED STEEL AREA			
a) Along longer side			
Mu	252.03		
R	52		
Steel ratio	0.0099		
steel area	2.14 sq. in		
Use Bars	4.00	#	8.00
b) Along shorter side			
Mu	650		
R	269		
steel ratio	0.02		
steel area	4.86 sq. in		
Use Bars	7.00	#	8.00

Figure 24: Steel Areas for square and rectangular footing

One-Way Shear Check		
Factored Shear Force Acting on Section ef=Vu2	79407	lbs
Nominal Shear Strength=Vc	345806	lbs
Design Shear Strength=ϕVc	259354	lbs
Effective Depth Is Adequate		

Two-Way Shear Check		
Critical Perimeter=bo	180	in
Perimeter=Vu1	411234	lbs
Nominal Shear Strength=Vc	571822	lbs
Design Shear Strength=ϕVc	428867	lbs
Effective Depth Is Adequate		

Figure 25: One-way and Two-way shear Checks

3.2.5 Comparison with SAFE:

Results of Structural design of Excel Workbooks are compared with SAFE software to get a better understanding of what we achieved and to see how our design were close to SAFE software. The procedure to design footings on SAFE software is as follows

- Same Footing Dimensions as those In Excel Workbook
- Defined same material properties as that on excel
- Loading was also kept the same
- Modulus of Sub-grade reaction was defined by dividing Bearing Capacity with allowable settlement of 1 inch.
- Footing thickness was kept as a trial Dimension and varied until Shear check is satisfied.
- At the end results were drawn and then compared with that of excel

3.3 MAT Footing Structural Design:

3.3.1 Conventional Rigid Method

- Input footing dimensions along x and y direction
- Input no. of columns in x and y direction
- Input size of column in x and y direction
- Input center to center spacing and edge distances in x and y direction
- Apply loads and moments on columns and set allowable bearing pressure of the soil
- SFD and BMD are drawn automatically by Excel
- Negative and positive reinforcement is designed by the back end calculations by Excel for each strip
- Eccentricity is checked for applied loading conditions on all columns
- Other Checks are applied for safe design

3.3.2 Comparison with SAFE

Exact replica of MAT footing is also designed on SAFE, which uses subgrade reaction approach, to understand differences in results. Finite Element Method (FEM) gives conservative results as compared to Conventional Rigid Method (CRM).

3.4 Axial load capacity

3.4.1 Bearing Capacity and Settlement Chart Methodology

Input Data

For the input data you have to define the following parameters to calculate the bearing capacity and settlement of pile:

Non-Cohesive Soil

- Shape
- Depth of foundation up to which we want to calculate the capacity for pile
- Depth of Water Table Dw
- Diameter of pile D or width B of pile based on the shape of pile
- Unit weight γ each 1ft depth of soil
- Standard Penetration number SPT blow count N
- Soil type
- FOS for BC
- Internal Friction Angle Ø
- Constant Factor m to calculate the modulus of elasticity of soil Es`
- Atmospheric Soil Pressure 100 kN/m^2
- Modulus of elasticity of pile material Ep
- Settlement required to mobilize ultimate toe bearing
- Parameter g required to calculate the settlement due to toe bearing
- Settlement required to mobilize ultimate skin friction
- Parameter h required to calculate the settlement due to skin friction

Cohesive Soil

- Shape
- Depth of foundation up to which we want to calculate the capacity for pile
- Depth of Water Table Dw
- Diameter of pile D
- Unit weight γ each 1ft depth of soil
- Un-drained shear strength for each 1 ft depth of soil
- Standard Penetration number SPT blow count N
- Soil type
- Cohesion of Soil c'
- FOS for BC
- Internal Friction Angle Ø
- Constant Factor m to calculate the modulus of elasticity of soil Es`
- Atmospheric Soil Pressure 100 kN/m^2
- Modulus of elasticity of pile material Ep
- Settlement required to mobilize ultimate toe bearing
- Parameter g required to calculate the settlement due to toe bearing
- Settlement required to mobilize ultimate skin friction
- Parameter h required to calculate the settlement due to skin friction

We must have to enter the data for each 1 ft depth for specific parameters like effective unit weight, un-drained shear strength for cohesive soil and similarly values of SPT blow count for non-cohesive soil check the effect of bearing capacity for each depth.

The purpose to calculate the bearing capacity for each 1 ft depth is that, based on the given allowable area for foundation ,we can calculate the number of piles that can be given under structure against the allowable bearing capacity of one pile of specific depth.

For the calculation of allowable settlement , allowable load against toe bearing and skin friction in case of Vesic method while ultimate toe bearing resistance and ultimate skin friction resistance is taken in case of Fellenius method.

3.4.2 Bearing Capacity Calculation:

3.4.2.1 Toe Bearing Capacity (Non-Cohesive soil)

For the calculation of allowable toe bearing resistance we used following five methods:

- Mayerhoff
- De Beer
- Berezantzev
- Janbu
- Vesic

The difference in the toe bearing capacity values is due to difference in the bearing capacity factors used by different scientists. The different bearing capacity factors are based on different assumed failure patterns at the toe of the pile. Janbu is old method and underestimates the toe resistance while De beer overestimates the value of toe resistance. Meyerhof and Berzantzev gives reasonable value for the toe resistance while Vesic uses the theory of expansion of cavity and take the factor of rigidity to calculate the allowable toe resistance.

For design toe bearing capacity we take the average of all the values obtained by janbu ,Mayerhof and Berezantzev Methods.

3.4.2.2 Skin Resistance Capacity (Non-Cohesive soil)

For the calculation of allowable skin resistance we used following method:

- Soil Earth Pressure Method

After finding the ultimate skin resistance capacity and ultimate toe bearing capacity and divide them factor of safety to get the allowable pile capacity.

3.4.2.3 Toe Bearing Capacity (Cohesive Soils)

For the calculation of allowable toe bearing resistance we used following methods:

- Meyerhof
- Vesic
- O'neil and Reese

All the above mentioned methods give different values for ultimate toe bearing capacity. The difference is due to the taken different bearing capacity factor Nc. Meyerhoff takes the value of Nc as 9 throughout. Vesic and O'neil and Reese says that bearing capacity factor Nc depends on the rigidity index Ir. Vesic presented his theorey of expansion of cavity to find the value of bearing capcity factor Nc while O'neil and Reese presented the relationship between undrained cohesion and rigidity index to find the value of bearing capacity factor Nc.

Mayeroff gives the lower value while Vesic,O'neil and Reese give reasonable higher values for the ultimate toe bearing capacity. For design purposes we take the average of all values obtained from different methods.

3.4.2.4 Skin Resistance Capacity (Cohesive soil)

For the calculation of allowable skin resistance, we used following method:

- API Method (Alpha Method)
- Burland (Beta Method)

Alpha method depends on the adhesion factor α. For the calculation of adhesion factor α different experts give different functions to compare the values of α with that obtained from static load tests. It appeared that API function gives the most appropriate values of α. While Beta Method assumes the drained condition in case of driven pile. Beta Method assumes that clay remoulds after the dissipation of excess pore water pressure which results in zero cohesion. For the calculation of skin resistance Beta Method calculates the earth pressure coefficient K.

Alpha Method gives higher values while Beta Method gives lower values for ultimate skin resistance. For design purposes we take the average of all values obtained from different methods.

3.4.3 Settlement Calculations

For the calculation of settlement we used following methods:

- Vesic 1970
- Vesic 1977
- Fellenius 1999

All the above mentioned methods calculated settlement for cohesive and non-cohesive soil. Vesic 1970 is an empirical method and mainly depends upon the pile diameter and ad settlement due to pile length increase is no considerable while Vesic 1977 is semi-empirical method and calculates the settlement based on the semi-empirical coefficients. Vesic1977 calculates the settlement based on the working load condition and total settlement is calculated in three s mention following:

- Settlement due to Axial Deformation Ss
- Settlement due to pile base Sp
- Settlement due to pile shaft Sps

Fellenius 1999 assumes the value of settlement required to mobilize ultimate toe bearing and ultimate skin friction based on the diameter of pile and generate the load settlement curve on different working loads.

Since in case of pile accurate settlement cannot be calculated due to different phenomenon happening while driving pile into the ground and possible only with the help of full scale static load tests. Therefore for design purposes we took average of all values obtained from above mentioned methods.

3.5 lateral load capacity

3.5.1 Bearing Capacity and deflection Chart Methodology

Input Data

For the input data you have to define the following parameters to calculate the bearing capacity and deflection of pile:

Brinch Hansen

- Height of application of load above the ground level
- Diameter of pile
- Length of pile
- Depth of pile about which rotation of pile occurs

- Unit weight γ
- FOS for BC
- Internal Friction Angle Ø
- Cohesion of soil c'
- Rate of change of cohesion c'

Brom method

- Diameter of pile
- Length of pile
- Modulus of elasticity of pile material Ep
- Unit weight γ
- FOS for BC
- Internal Friction Angle Ø

- Constant of modulus of horizontal sub-grade reaction nh
- Tensile strength or rupture strength of pile material fy
- Define the condition that pile is either fixed or free headed from top

3.5.2 Bearing Capacity Calculation:

For the calculation of allowable bearing resistance we used following methods:

- Brinch Hansen Method
- Brom Method

For the calculation of bearing capacity certain parameters are entered in the input. In case of Brinch Hansen method total pile is divided into number of slices and the actual point about which rotation of piles occurs is determined by trial and error approach. After finding the depth about which rotation of pile occurs moment is taken of all slice forces about that point and resultantl ultimate lateral force is found out by the division of the sum of moments by the total depth of pile. For the allowable capacity ultimate lateral resistance is divided by the factor of safety. The disadvantage of this method is that this cannot be used for short rigid piles.

While in case of Brom method certain parameters are entered in the input and characteristic length against specific diameter is found out to check that either given pile of specific length is short pile or long pile. Short pile capacity depends on the length and diameter of pile while long pile mainly depends on diameter of pile. After check ultimate lateral resistance and resultant moment is find out.

We generated the bearing capacity charts against different diameter of piles which are helpful in determining the allowable capacity against specific diameter and length of pile and are helpful in design purposes for short and long piles.

3.5.3 Deflection Calculations:

For the calculation of defection, we used following method:

- Sub grade Reaction Approach

For the calculation of deflection, certain parameters are entered in the input data and value of deflection against the specific load and moment is calculated. For the design purposes we generated charts having diameter on x-axis and allowable load on the y-axis for specific deflections range, which are helpful in design purposes and are helpful to determine the allowable load against specific diameter and deflection.

CHAPTER 4

4 RESULTS AND DISCUSSION

4.1 Design Chart

We have obtained Design Chart from Automated Excel Sheet that we have generated. The Design Chart is applicable to the whole site for which the particular site conditions are input.

Let us see the view of Design Chart:

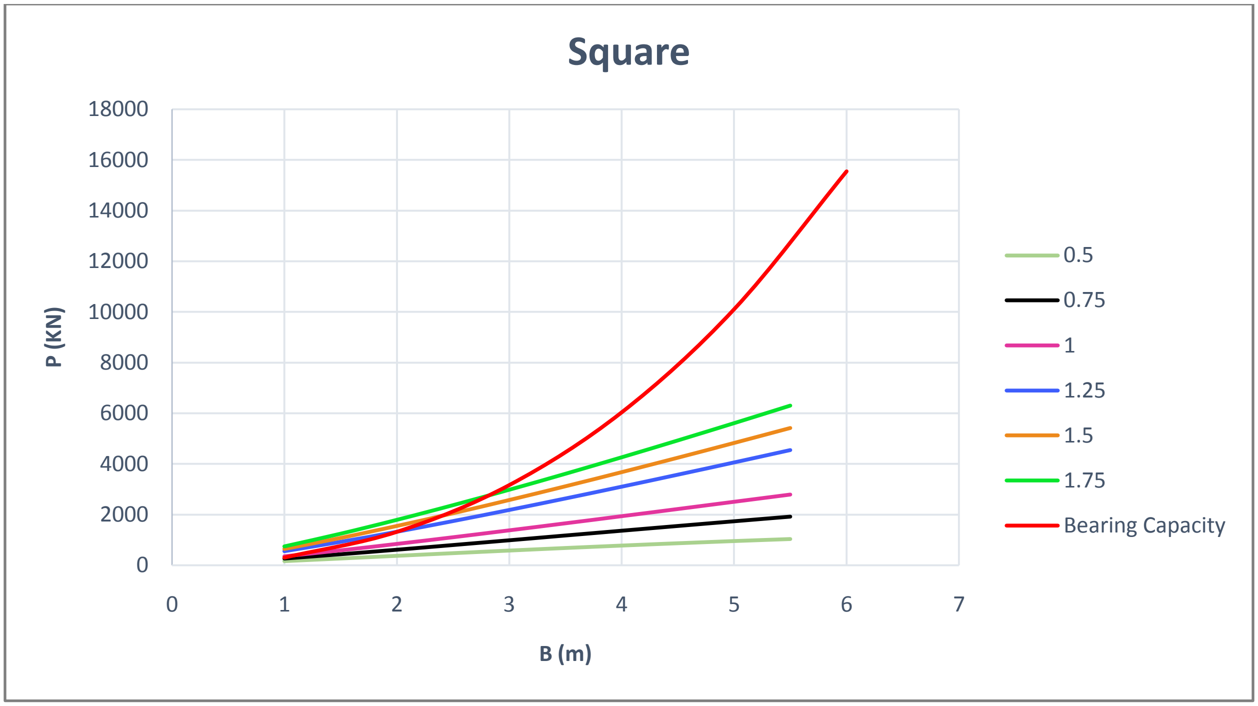


Figure 26: Design Chart Output

Chart is having one bearing capacity curve and other are different curves for constant settlement. From chart we can instantly get the desired width of footing for respective load and settlement.

4.2 Shallow Footings Structural Design

4.2.1 Combined Footing Bending Moment Comparison Excel vs SAFE

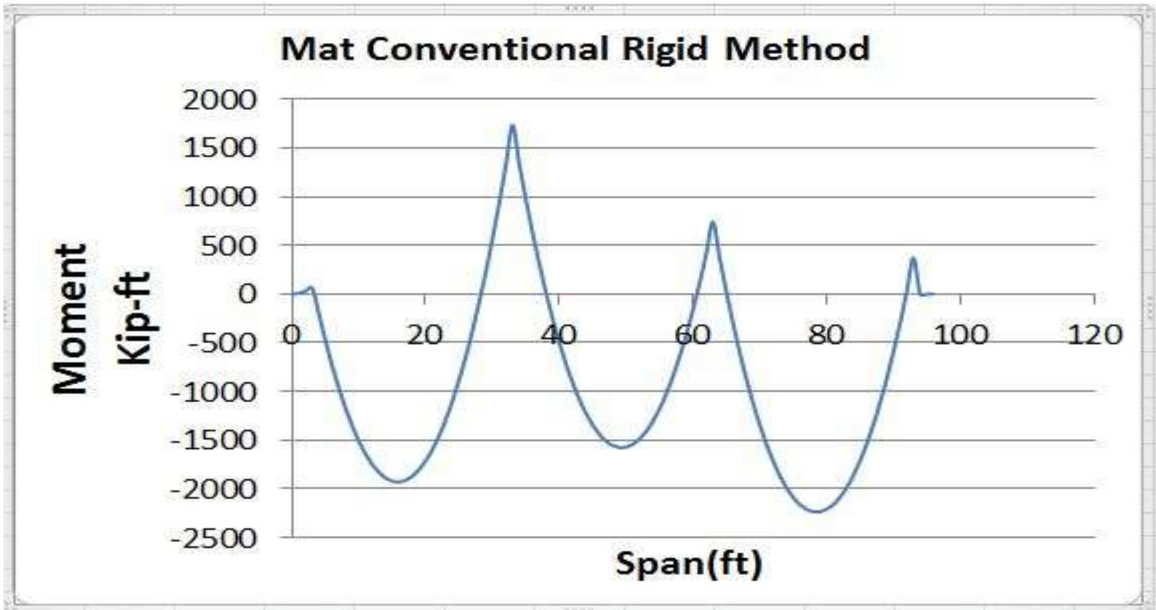


Figure 27: Combined footing bending moment on Excel

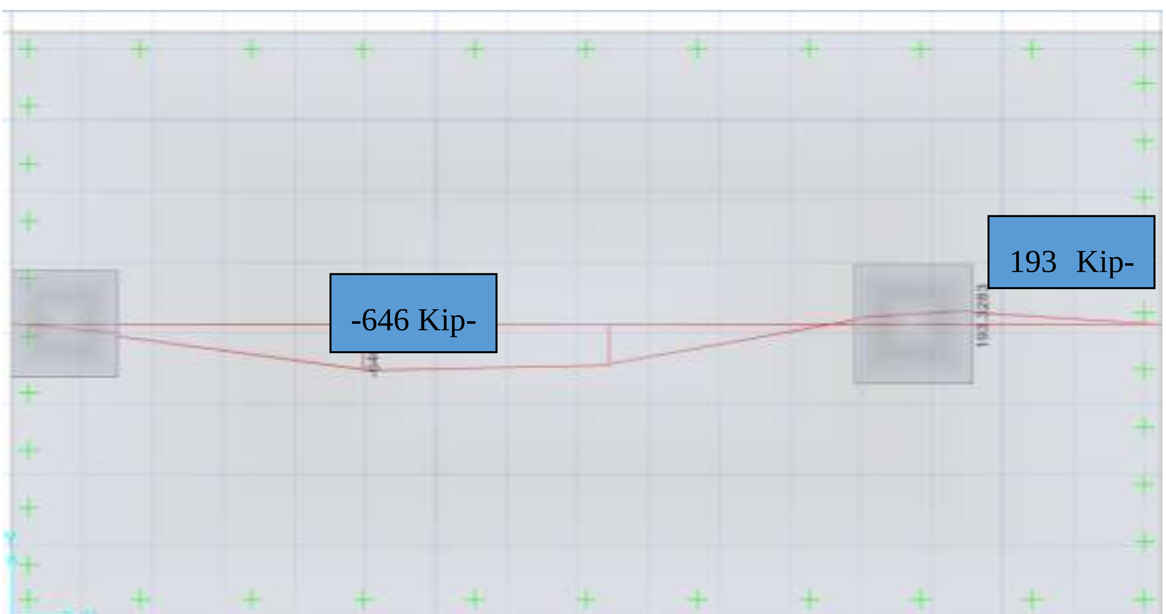


Figure 28: Combined footing bending moment using SAFE

4.2.2 MAT Footing Bending Comparison Excel vs SAFE

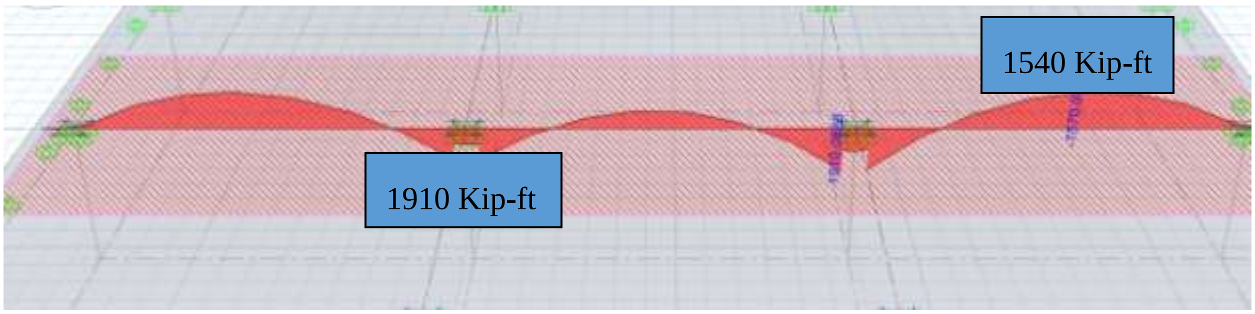


Figure 29: MAT footing bending moment using SAFE

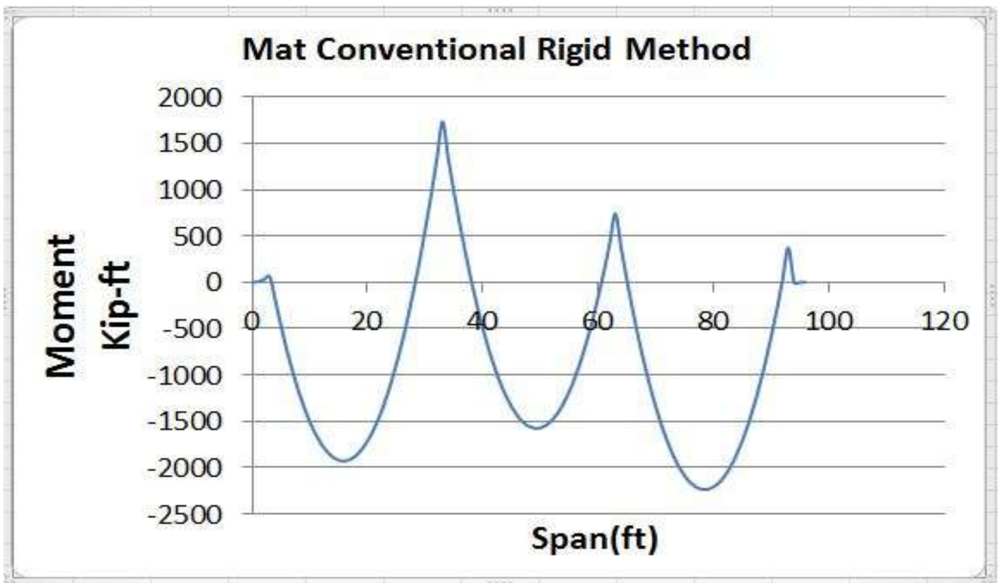


Figure 30: MAT footing bending moment using Conventional Rigid Method

4.3 Pile Foundations Design Charts

4.3.1 Allowable Axial Capacity for Un-drained

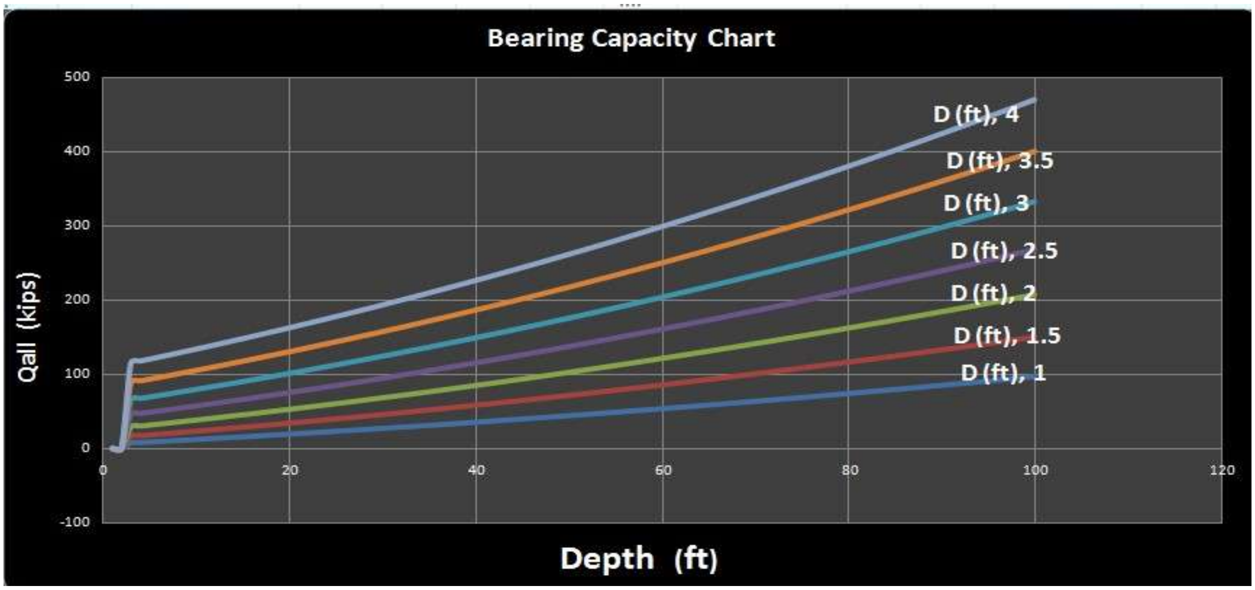


Figure 31: Allowable axial load capacity for undrained

4.3.2 Allowable Axial Capacity for Drained

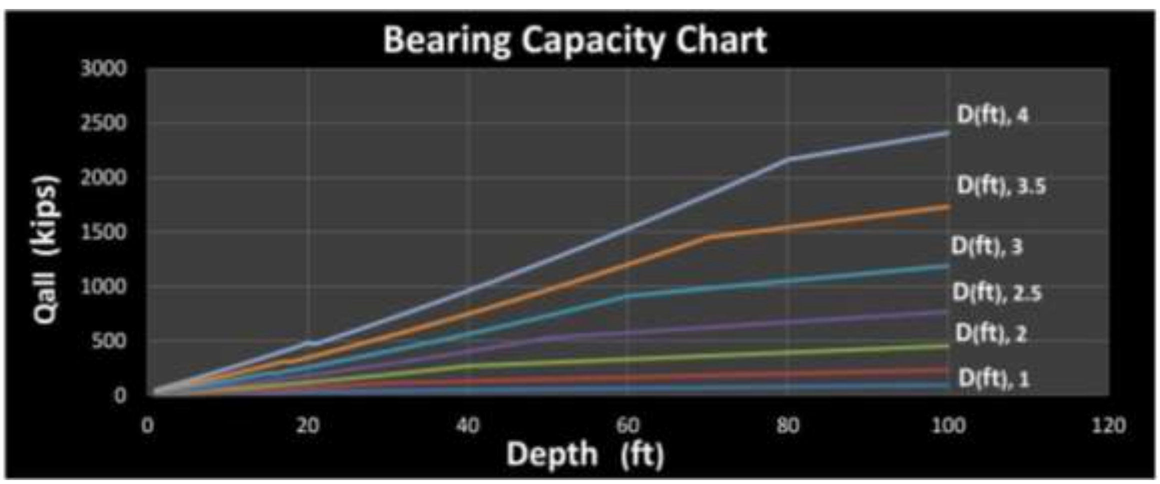


Figure 32:Allowable axial load capacity for drained

4.3.3 Load Settlement Response by Vesic

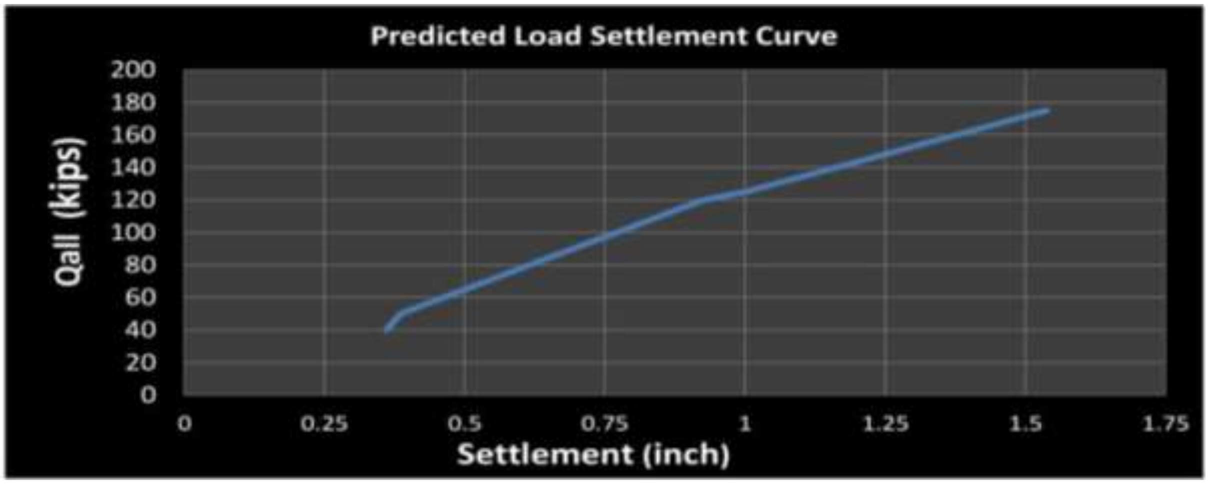


Figure 33: load settlement response by Vesic

4.3.4 Load Settlement Response by Fellenius

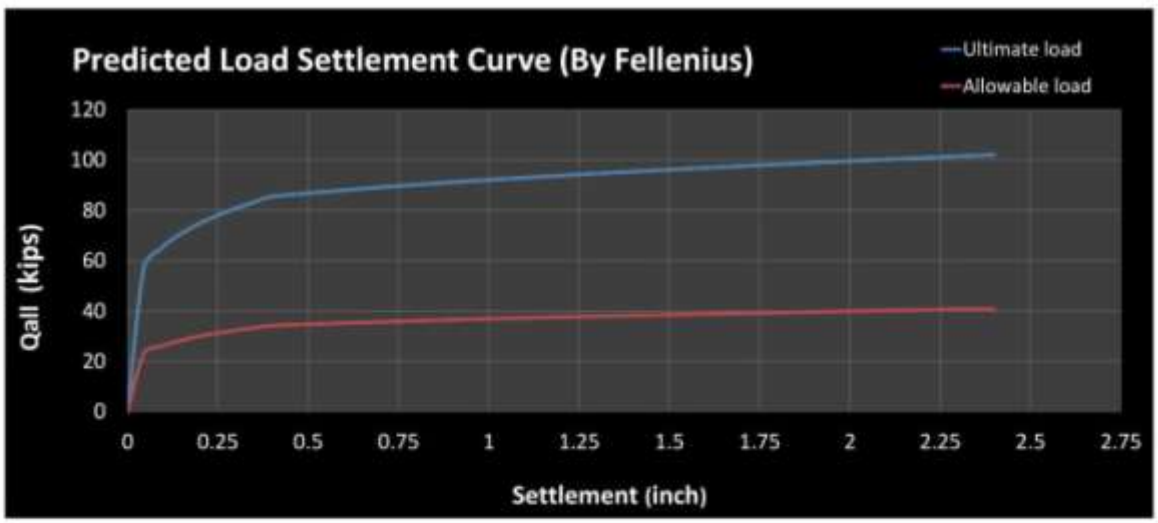


Figure 34: load settlement response by Fellenius

4.3.5 Lateral Capacity of Short/Rigid Pile (Restrained)

This design chart enables us to find out allowable bearing capacity of short or rigid piles for Diameters and Depth ranges as depicted in the chart.

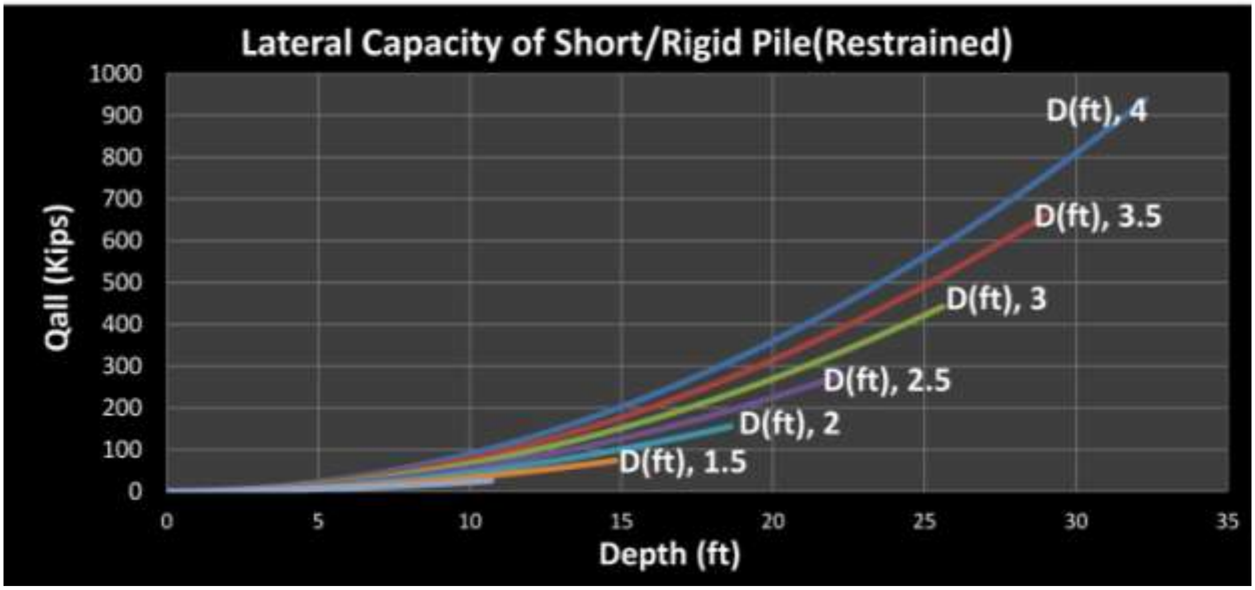


Figure 35: Lateral Capacity of Short/Rigid Pile (Restrained)

4.3.6 Lateral Capacity of long/flexible Pile

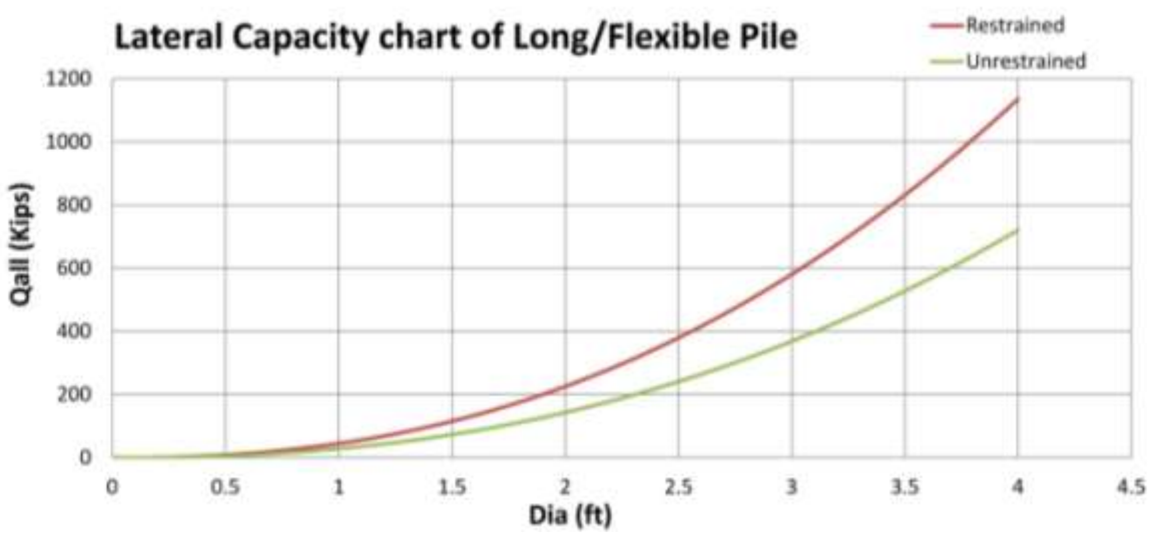


Figure 36:Lateral Capacity of Long/Flexible Pile

4.3.7 Load Deflection Curve of Pile

For a fixed settlement of pile (0.25-1 in), we can find Allowable Bearing Capacity for any diameter of pile.

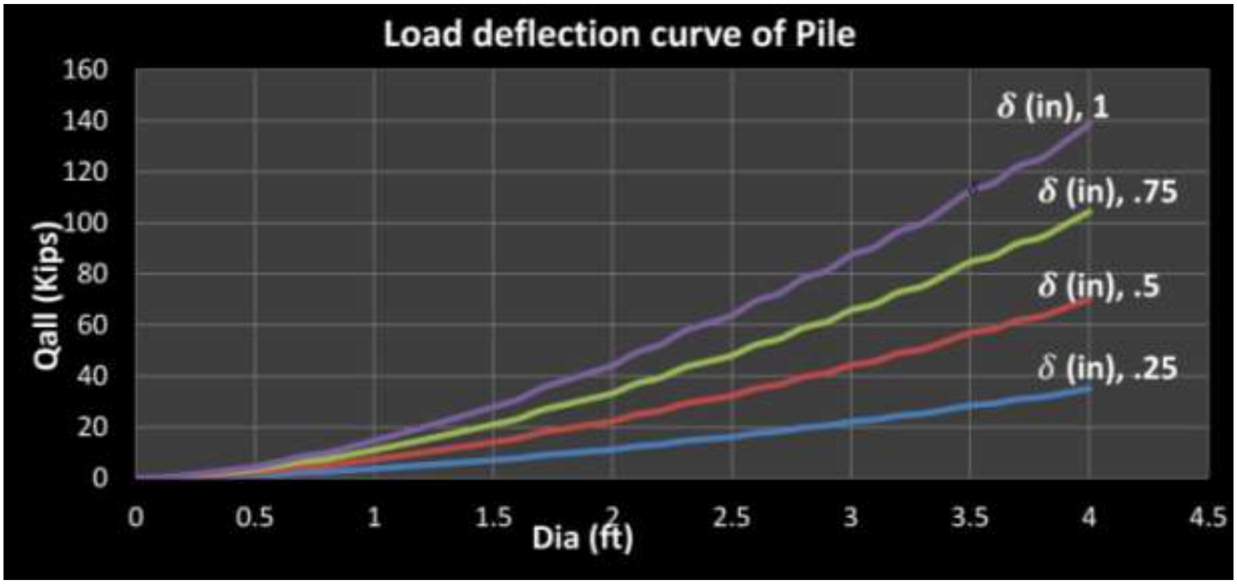


Figure 37: Load Deflection Curve of Pile

4.3.8 Load Deflection Curve Checked on OasysAlpile

Pile Deflections and Bending Moments were compared with results of OasysAlpile software.

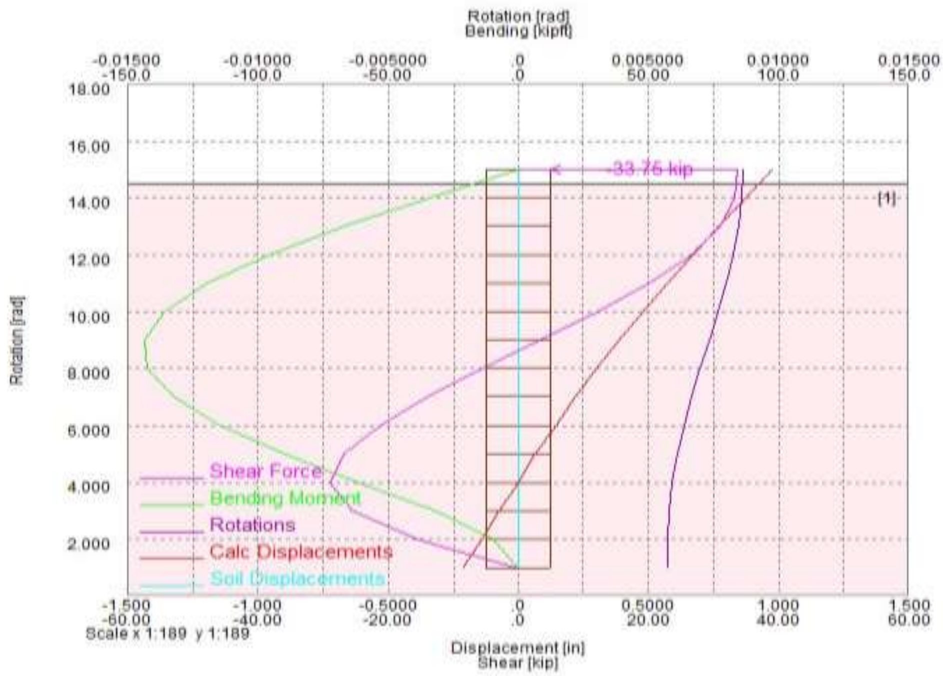


Figure 38: Load Deflection Curve Checked on OasysAlpile

4.3.9 Bending Moment Comparison of Pile

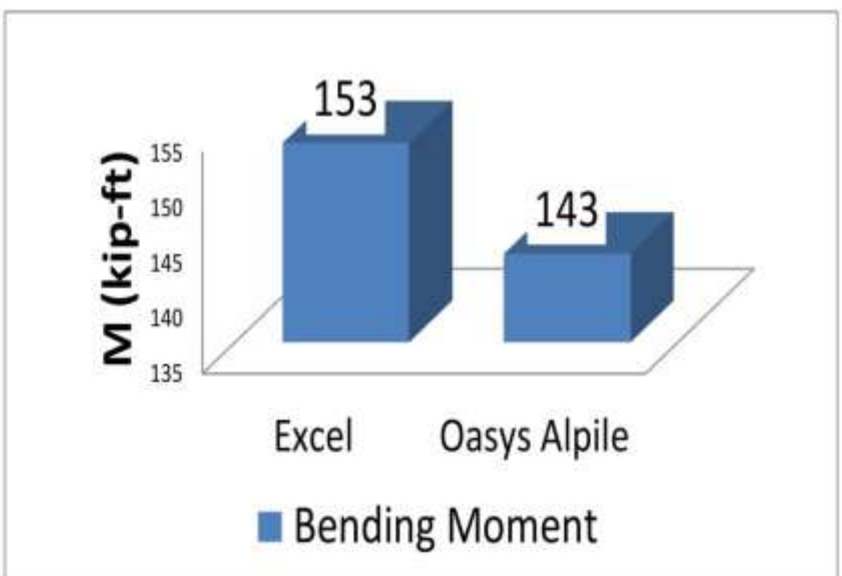


Figure 39: bending moment comparison of pile

4.3.10 Driven Pile Design ByOasys Pile

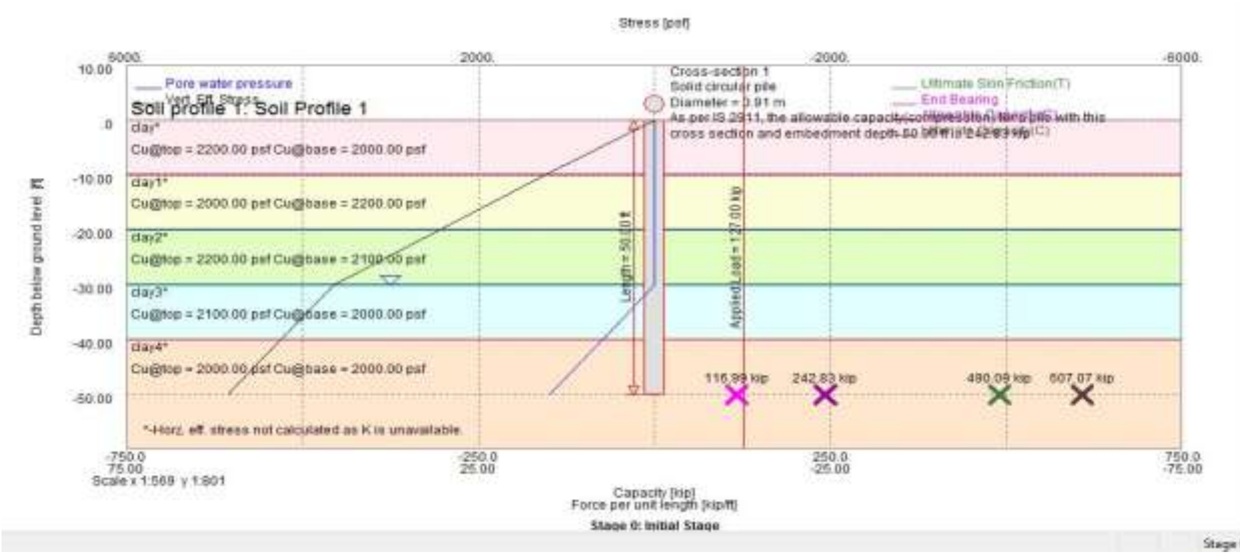


Figure 40: Driven pile design by Oasys Pile

CHAPTER 5

5 CONCLUSIONS

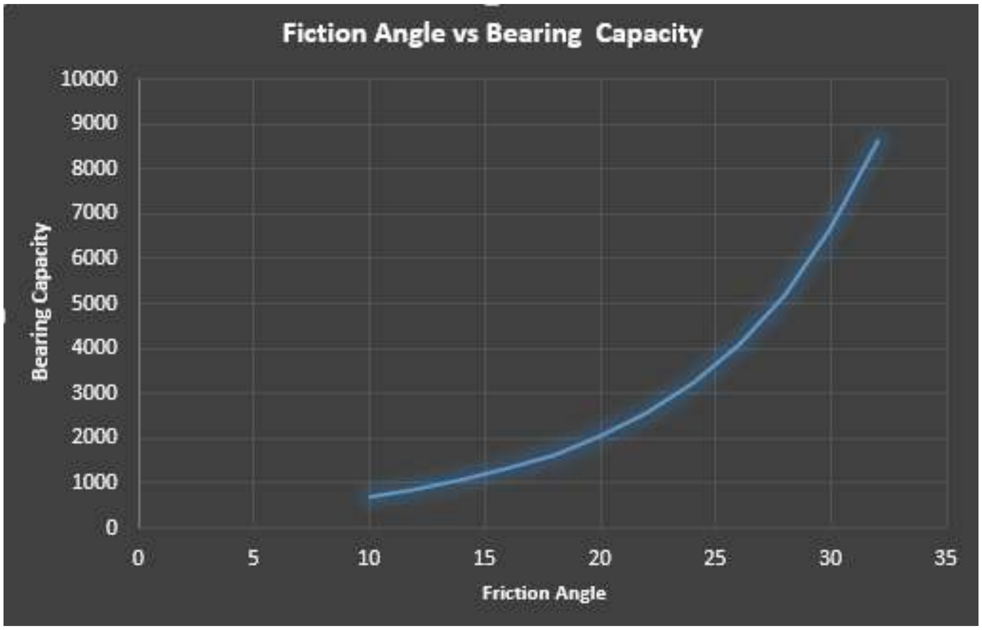
1. Comparison of different Methods

By comparison of different methods for calculation of bearing capacity of shallow foundation graphs shows that Terzaghi gives the lowest value while Vesic gives the highest value. The difference in the values is due to different assumed failure pattern of shallow foundation.

Soil type	Footing Shape	c	γ (lb/ft^3)	B	Depth
φ-soil	square	0	118 (lb/ft^3)	7.5′	6′

2. Effect of Friction Angle

For ϕ soil, at lower value of angle of friction for instance 0 to 20, the ultimate bearing capacity by all methods are approximately similar to each other but difference in bearing capacity calculation increases exponentially with increase of friction angle. Effect of friction angle is more important in case of bearing capacity calculation and thus is major factor in defining load bearing capacity of soil. When friction angle exceeds 20 degree the value of bearing capacity exceeds exponentially.



3. Limitation of Terzaghi Equation

Terzaghi's (1943) equation is not suited for footing with moments and/or horizontal loads or for foundation on slopey ground. We need to calculate the bearing capacity by Hansen and Mayerhof equation.

4. Applicability of different Methods for c - ϕ soil

For c - ϕ soil, Meyerhof's (1963) equation is not much different from Tarzaghi's (1943) equation up to a depth of D/B ≤1,but Hansen (1970), Vesic (1973), highly differ from Tarzaghi's (1943) equation up to depth of D/B ≤1.

5. Limitation of Terzaghi 1-D Consolidation Method

For settlement analysis of soil we first have to analyse the soil condition in detail i.e whether excess pore water pressure generated due to applied loading will dissipate in the pattern of 3-D drainage or will dissipate in the pattern of 1-D drainage. The expected consolidation time will be much lesser than predicted by Terzaghi 1D consolidation method. For the accurate analysis schmertmann method is more better option.

6. Soil Model Selection

Behaviour of Soil is very complex due to fact that application of load on the foundation induces settlement in the foundation. For preliminary analysis we can use Mohr Coulomb's model but for detailed analysis needed for complex structures and high rise buildings detailed analysis using soil models like Hardening Soil Model, Hardening Soil Model with small strain or Cam Clay Model is the better option.

7. Effect of Longitudinal Reinforcement in Pile Head Deflection

In case of excessive settlement longitudinal reinforcement in pile has only a limited effect in restricting pile head deflections, so for the excessive deflections you have to select some pile head fixity options to meet structural tolerance limit.

8. Check Applicability of a Method

Before using a certain Method make sure you check the limitations and conditions for which the formula is derived. Otherwise Improper analysis can lead to improper results.

9. Comparison of Designed Spread Sheet with SAFE

By comparing the results of isolated footing , combined footing, continuous footing and Spread Footing design for Biaxial Moments using Automated Excel sheet with SAFE software, give results having difference of less than 3%.

10. Use Softwares for Structural Design

For Structural Design of MAT foundation and other footings such as isolated footings and Combined footings it is better to use Software such as SAFE that consider deflections due to applied loadings and use the finite element analysis which results in accurate Design.

11. Applicability of a Spread Sheets for deep foundation

Spread sheets for deep foundation are helpful in calculating the bearing capacity and settlement as long as strata is same and even having different soil parameters such as un-drained shear strength and unit weight of soil but are not helpful in case of different strata due to fact that the behaviour of deep foundation and resistance to the applied loading in case of clay and sand is different which makes analysis difficult through spread sheets in case of seep foundation.

12. Comparison of Designed Spread Sheet with OASYS

For Geotechnical Design of deep foundations such as piles it is better to use Software such as OASYS PILE for axial capacity and settlement while OASYS LPILE for lateral capacity and deflection and use the finite element analysis which results in accurate Design. Also Software makes easier the analysis for deep foundation even in case of different soil strata.

13. Accepting results of Geotechnical Analysis

It is difficult to model soil conditions accurately for Geotechnical Design, so any result of analysis must be accepted by proper utilizing engineering knowledge and judgement.

CHAPTER 6

6 References

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