Soil-Structure Interaction

FINAL YEAR

PROJECT UG 2013

By

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This is to certify that the

Final Year Project, titled

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“In the name of Almighty ALLAH, the Most BENEFICIENT, the Most MERCIFUL”.

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ABSTRACT

Foundations ,being the most critical and crucial part, serves as an interface between two completely different materials ,one being a material is combined form and serves as one body reinforced concrete ,or other one to support it is in particular form soil.

Our basis aim is to increase our understanding of interaction between a foundation and the soil, thus become able to not blindly follow the results of tests or analysis.

In our project we dealt with both types of foundations.

(a) Shallow foundations
  - Spread footing
  - Strip footing
  - Combined footing
  - Mat footing

(b) Deep Foundations
  - Drilled Shafts

Automated excel sheet is prepared which gives structural design of spread footing for axial load and biaxial moments and also for purely axial loading or with uniaxial moments.

Also automated excel workbooks for generating a design chart; a chart based on geotechnical data gives the footing size corresponding to applied load and required settlement.

Design of drilled shafts for both vertical and horizontal loading conditions using three Softwares in conjunction Oasys pile, Alp and AdSec.

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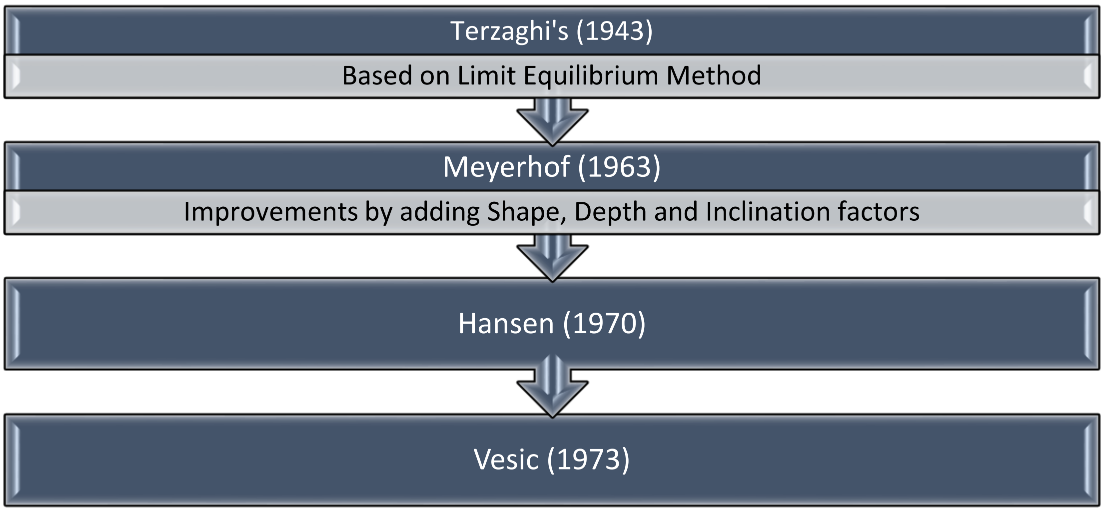
CHAPTER 1

Introduction

1.1 Historical Background

History of foundations is as old as humans start building structures. Importance of foundations was well known by builders. Although builders have recognized the importance of firm foundations for countless generations, but the discipline of foundation engineering did not begin until late nineteenth century.

Major breakthrough occurred in twentieth century when significant improvements have been done to define bearing capacity as settlement. First one method of bearing capacity to gain wide acceptance was that of Terzaghi, in which he used limit equilibrium method with certain assumptions that was comparatively much accurate of widely applicable.



1.2 Objectives and Limitations of Study

Basic aim of project is to understand the requirements of both the geo-technical and structural design of foundations and balancing to obtain optimum design. To serve the job of a foundation engineer, that must have its expertise in key areas of civil engineering like geotechnical engineering, structural engineering and construction engineering.

Since foundations provide basis for every civil engineering structure, so it is important to learn about various methods that are available to get bearing capacity in case of shallow foundations and skin friction or toe bearing capacity in case of deep foundation and also settlement of foundations. Then study the limitations and assumptions of each method and get a practical idea of where to apply a certain method.

We aim to study the effect of soil properties on structural design foundations and in return gat an effect of loaded foundation on soil .as it is well define that soil resistance define the amount of deformation of foundation and in return structural design properties define the deformation in soil.

End results will be the development of **design chart** for shallow foundations and automated excel work book for structural design of footing.

Using software to design single pile and also the comparison of field deflection results for pile with Plaxis 3D foundation software.

The whole Workflow for this project is mentioned below:

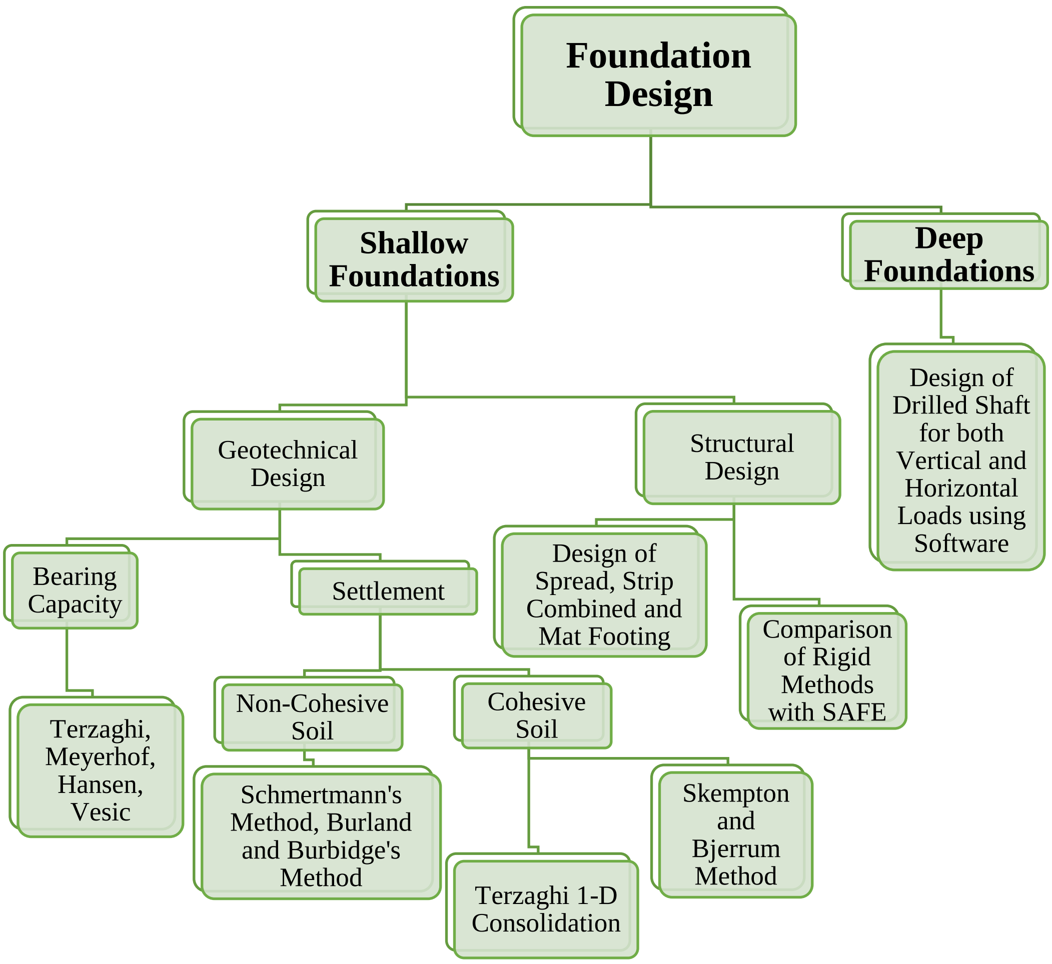


Figure 1: Project Scheme

1.2.2 Learning outcomes

Developing a keen insight into various aspects of foundation design of correlations factors at same time to judge for requirements of design.

In this project multiple Softwares were used as the developed expertise in using different Softwares e.g.

1. Excel
2. SAFE
3. Plaxis 3D foundation
4. Oasys pile
5. Oasys Alp
6. Oasys AdSec

CHAPTER 2

LITERATURE REVIEW

2.1. General

“A structure is no stronger than its connections” although this statement usually invokes images of connections between individual structural members it also applies to those between a structure and the ground that supports it. These connections are known as its foundations. Even the ancient builders knew that the most carefully designed structure can fail if they are not supported on suitable foundations for example tower of Pisa in Italy is perfect example.[1]

2.1.1 Foundation Engineering

Foundation engineer should have the knowledge of all the multiple disciplines of Civil Engineering. Multiple fields are covered in this one field of foundation engineering.[1]

2.1.2 Structural Engineering

The two factors that are to be kept in while designing of the foundation; the transmission of load to the soil and support to the structure. A foundation is a structure member which transmits the applied load to the soil. So that must be capable of transmitting the applied loads, so we must also understand the principles and practices of structural engineering .In addition, the foundation supports a structure .so we must understand the sources and nature of structural loads and the structure’s tolerance of foundation movements.[1]

2.1.3 Geotechnical engineering

“All foundations interact with the ground so design must reflect the engineering properties and behavior of adjacent soil and rock. Thus the foundation engineer must understand geotechnical engineering .Most foundation engineers also consider themselves to be geotechnical engineers”.[1]

2.1.4 Uncertainties

“In spite of many advances in foundation engineering theory, there are still loop holes in understanding. In general uncertainties are the result of our limited knowledge about soil conditions. Although many investigation and different techniques are used to define the properties of soil beneath the proposed foundation but it covers small portion of soil and rest is worked with interpolation and extrapolation .limitations in understanding of soil structure interaction also cause uncertainties for example how does side friction resistance is developed along the pile surface. How installation of pile affect the engineering properties of adjacent soil. Difficult to predict actual service loads that will act on foundation due to uncertainties, engineer does not follow the results blindly".[1]

2.1.5 Performance Requirements

"Foundation performance standards are not same for all structures or in all locations. Performance requirements for structural foundations are examined in each of the following categories".[1]

- Strength requirements
- Serviceability requirements
- Constructability requirements
- Economic requirements

There are basically two types of Foundations:

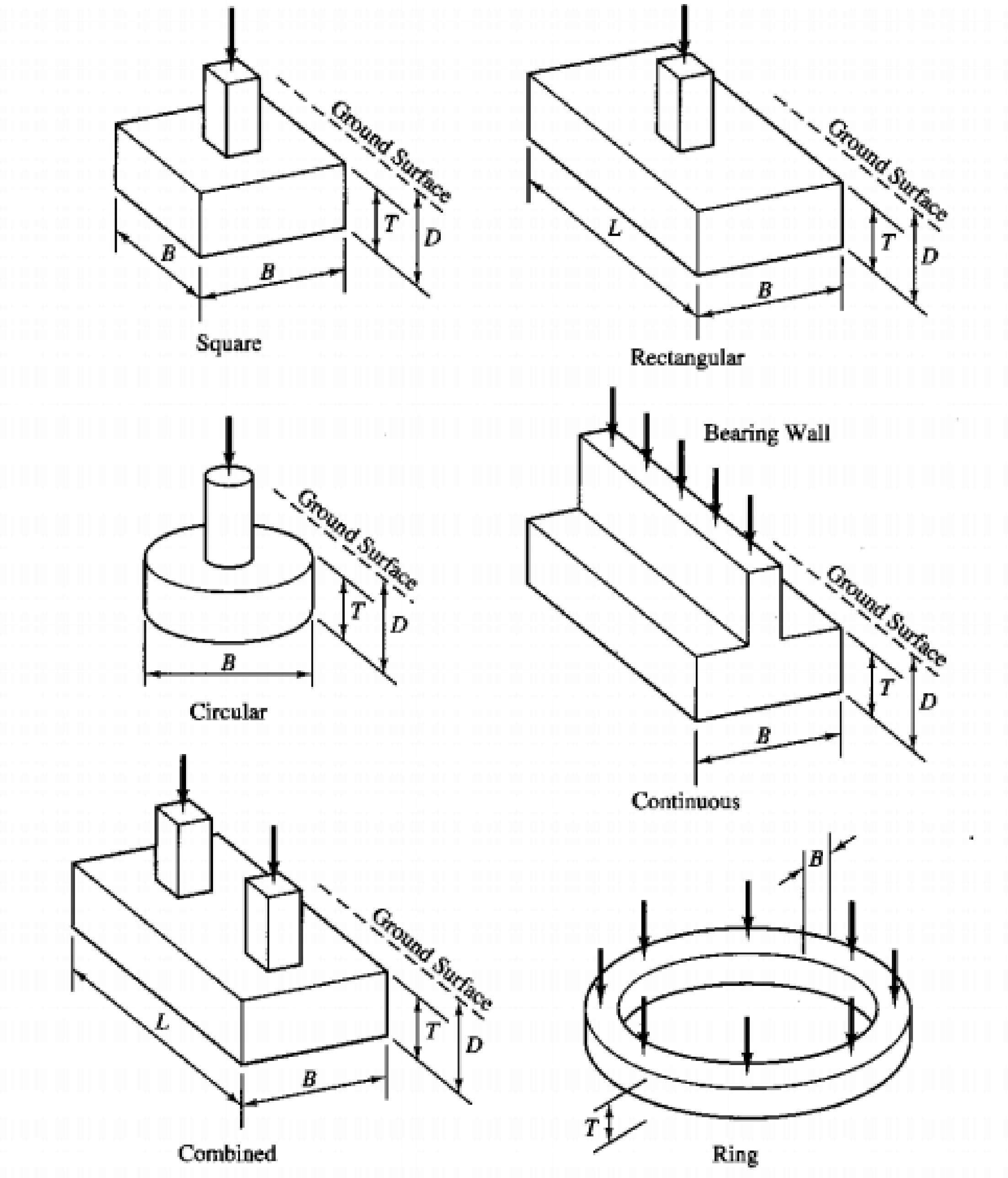
- ➢ Shallow Foundations
- ➢ Deep Foundations

2.2 Shallow Foundations

2.2.1 Introduction

"Shallow Foundations be defined as those which transfer load to near footing Soils and thus Terzaghi limits the criteria of defining shallow foundations as those, having D/B ratio varying from 0.25-1.0". Types of Shallow Foundations include:[1]

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


Ground Surface
T
D
B
B
Square
Ground Surface
L
B
T
D
Rectangular
Ground Surface
T
D
B
Circular
Bearing Wall
Ground Surface
T
D
B
Continuous
Ground Surface
L
B
T
D
Combined
B
T
Ring

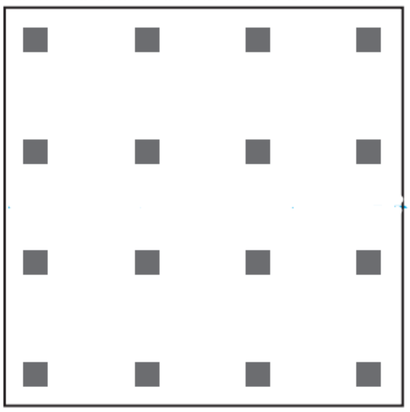


Figure 2: Types of Shallow Footings

2.2.1.1 Pressure Distribution

Simplified pressure Distribution is assumed for settlement calculations.

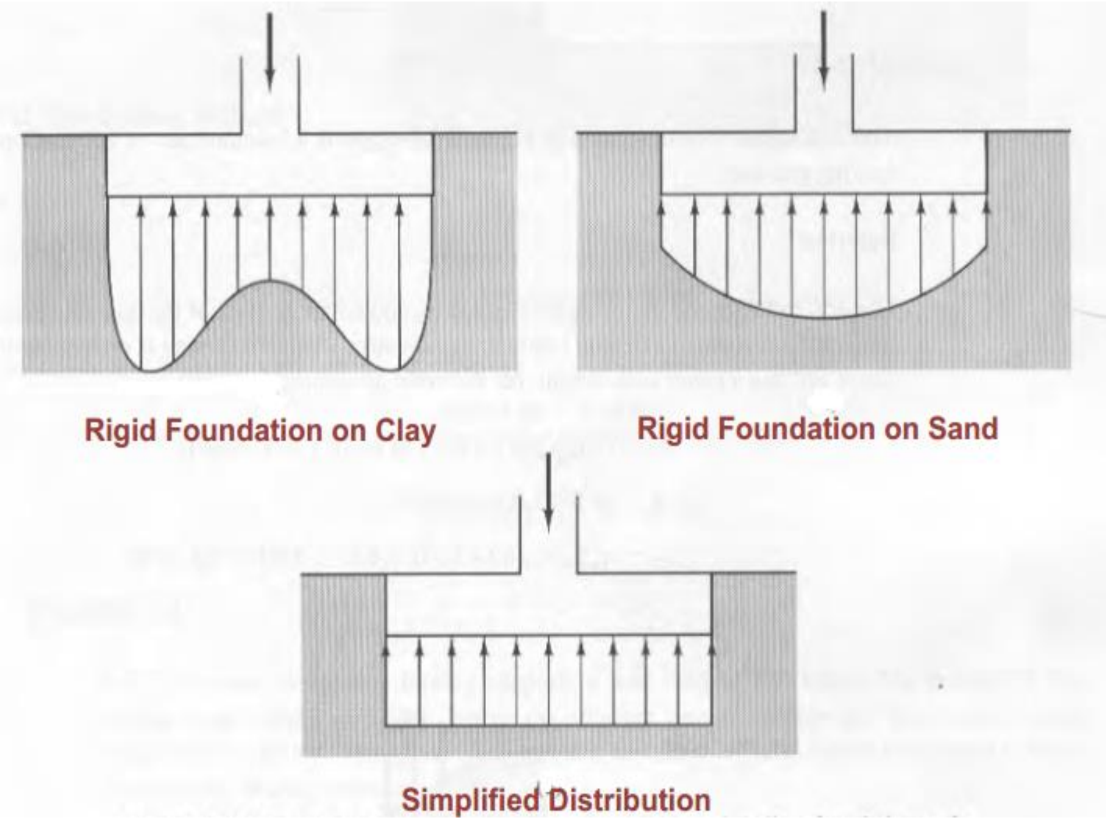


Figure 3: Pressure Distribution

Flowchart for Shallow Foundations is mentioned below:

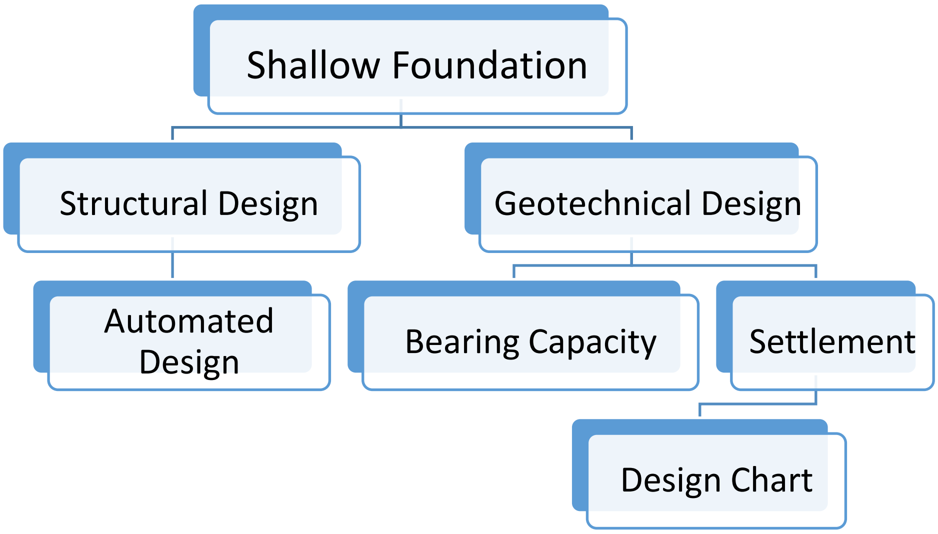


Figure 4: Scheme of Shallow Foundations

2.2.2 Geotechnical Design

As mentioned above, there are two requirements; Strength Requirement and Serviceability Requirement.

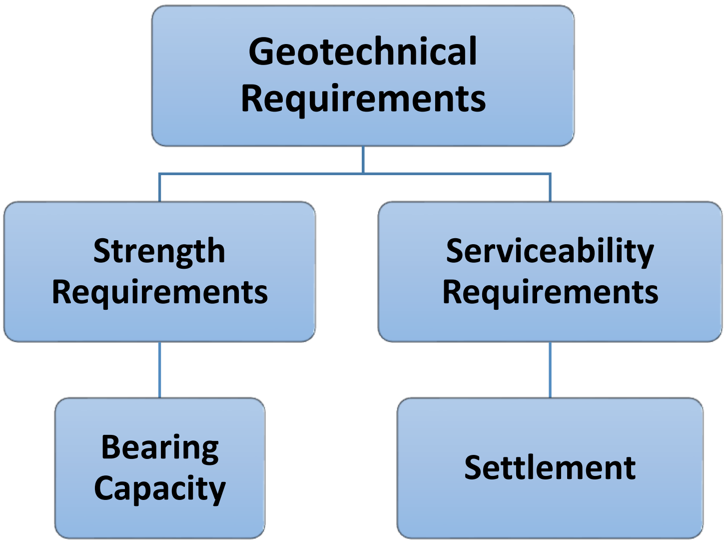


Figure 5: Scheme of Geotechnical Design

2.2.2.1 Strength requirement

“Geotechnical strength requirements are those that address the ability of the soil or rock to accept the loads imparted by the foundation without failing. The strength of soil is governed by its capacity to sustain shear stresses, so we satisfy geotechnical strength requirements by comparing shear stresses with shear strength and designing accordingly .In case of spread footing foundations, geotechnical strength requirement is expressed as the bearing capacity of the soil. If the load bearing capacity of the soil is exceeded, the resulting shear failure is called a bearing capacity failure”.[1]

General shear failure: “is the most common mode. It occurs in soils that arc relatively incompressible and reasonably strong. in rock, and in saturated, normally consolidated clays that are loaded rapidly enough that the undrained condition prevails”.[1]

Punching shear failure: “It occurs in very loose sands. Little or no bulging occurs at the ground surface and failure develops gradually”.[1]

Local shear failure: “is an intermediate case. The shear surfaces are well defined under the foundation, and then become vague near the ground surface. A small bulge may occur, but considerable settlement. The foundation just continues to sink ever deeper into the ground”.[1]

“Vesic' (1973) investigated these three modes of failure by conducting load tests on model circular foundations in a sand. The results, indicate shallow foundations (D/B less than about 2) can fail in any of the three modes, depending on the relative density. However, deep foundations (D/B greater than about 4) are always fail through punching shear failure”. [1]

2.2.2.1.1 Terzaghi’s Method of Bearing Capacity Computation:

Various limit equilibrium method of computing bearing capacity of soils were advanced in the first half of the twentieth century, but the first one to achieve widespread acceptance was that of Terzaghi [1943], his method includes the following assumption. [1]

1. The depth of the foundation is less than or equal to its width

2. The bottom of the foundation is sufficiently rough that no sliding occurs between the foundation and the soil.

3. The shear strength of the soil is describe by the formula

4. The general shear mode of failure governs

5. No consolidation of the soil occurs (i.e., settlement of the foundation is due only to the shearing and lateral movement of the soil)

6. The foundation is very rigid in comparison to the soil.

7. The soil between the ground surface and a depth D has nor shear strength and serves any as surcharge load

8. The applied load is compressive and applied vertically to the centroid of foundation and no applied moment loads are present.

"Terzaghi considered three zone in the soil. Immediately beneath the foundation is a wedge zone that remain intact and move downward with the foundation. Next, a radial shear zone extends from each side of wedge, where he took the shape of shear planes to be logarithmic spiral. Finally, the outer portion is linear shear zone in which the soil shear along planar surfaces".[1]

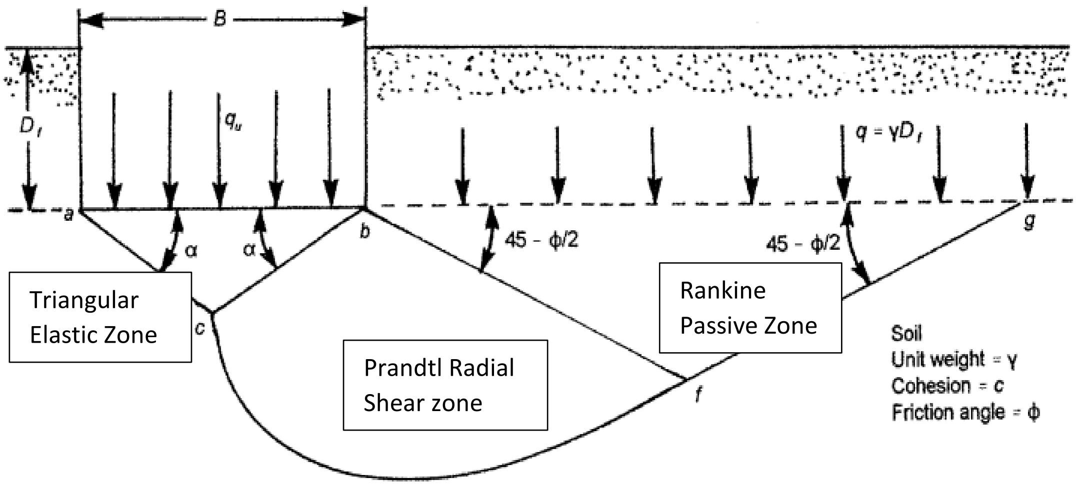


Figure 6: Terzaghi's Bearing Capacity Zones

"Since Terzaghi neglected the shear strength of soil between the ground surface and depth D. The shear surface stop at this depth and the overlying soil has been replaced with the surcharge pressure σ_{zD}. The approach is conservative, and is part of reason for limiting the value to relatively shallow foundation (D≤B)".[1]

"Terzaghi developed his theory for continuous foundation (i-e those with a base large L/B ratio). This is a simple case because it is two dimensional problem. He then extended it to square and round foundation by adding empirical coefficient obtained from model tests and produced the following bearing capacity formulas:[1]

For continuous Foundation:

$$q_{ult} = c'N_c + qN_q + 0.5B\gamma N_\gamma \qquad \text{(strip foundation; } 1<L/B\leq 10\text{)}$$

For square Foundation:

$$q_{ult} = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma \qquad \text{(square foundation; plan } B \times B\text{)}$$

For circular Foundation:

$$q_{ult} = 1.3c'N_c + qN_q + 0.3\gamma BN_\gamma \qquad \text{(circular foundation; plan } B \times B\text{)}$$

Where

q_{ult} = ultimate bearing capacity

C' = effective cohesion for soil beneath foundation

Φ' = effective friction angle for soil beneath foundation

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

γ' = effective unit weight of the soil (γ= γ' if groundwater table is very deep

D = depth of foundation below ground surface

B = width (or diameter) of foundation

Nc, Nq, Nγ = Terzaghi bearing capacity factor

Because of shape of failure surface, the values of c' and φ' only need to represent the soil between the bottom of the footing and depth B below the bottom. The soil between the ground surface and a depth D are simply as treated as overburden".[1]

The Terzaghi bearing capacity factors are:

Terzaghi's Formulae

1. $N_q = \frac{a_\theta^2}{2cos^2(45+\frac{\Phi'}{2})}$

2. $a_\theta = e^{\pi\left(0.75-\frac{\vartheta'}{360}\right)\tan\vartheta'}$

3. $N_c = 5.7$ for $\vartheta' = 0$

4. $N_c = \frac{N_q - 1}{tan\vartheta'}$ for $\vartheta' > 0$

5. $N_\gamma = \frac{tan\Phi'}{2}(\frac{K_{p\gamma}}{cos^2\vartheta'} - 1)$

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

Modification for Local Shear Failure includes:

$c' = 2c/3$ and $\vartheta' = \tan^{-1}(0.67\tan\vartheta)$

Vesic suggested a better mode to obtain ϑ' for estimating *Nc´* and *Nq´* for foundations on sand in the form:

$\vartheta' = \tan^{-1}(k \tan\vartheta)$

$k = 0.67 + Dr - 0.75\, Dr^2 \quad (where\ 0 \leq Dr \leq 0.67)$

2.2.2.1.2 Meyerhof

Meyerhof defined three zones below footing width B as elastic triangular shear zone, radial shear zone in the form of log spiral and third one zone being the mixed zone.[1][2]

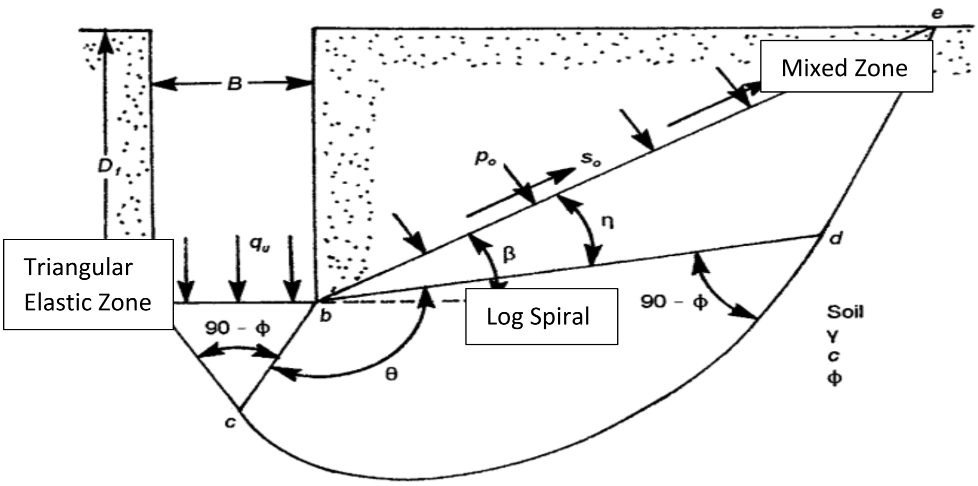


Figure 7: Meyerhof's Bearing Capacity Zones

Bearing Capacity Factors:

$$N_\gamma = \left(N_q - 1\right) tan 1.4\Phi'$$

$$N_q = \ tan^2(45 + \frac{\Phi'}{2})e^{\pi tan\Phi\prime}$$

$$N_c = \left(N_q - 1\right) cot\Phi'$$

Shape Factors:

For $\Phi' = 0°$

$$F_{cs} = 1 + 0.2(\frac{B}{L})$$

$$F_{qs} = F_{\gamma s} = 1$$

For $\Phi' \geq 10°$

$$F_{cs} = 1 + 0.2\left(\frac{B}{L}\right) tan^2(45 + \ \frac{\Phi'}{2})$$

$$F_{qs} = F_{\gamma s} = 1 + 0.1\left(\frac{B}{L}\right) tan^2(45 + \ \frac{\Phi}{2})$$

As shown, the value of $F_{\gamma s}$ will increase with the increase in the friction angle.

Depth Factors:

For $\Phi' = 0°$

$$F_{cd} = 1 + 0.2(\frac{D_f}{B})$$

$$F_{qd} = F_{\gamma d} = 1$$

For $\Phi' \geq 10°$

$$F_{Ds} = 1 + 0.2\left(\frac{D_f}{L}\right) tan^2(45 + \ \frac{\Phi'}{2})$$

$$F_{Ds} = 1 + 0.2\left(\frac{D_f}{L}\right) tan^2(45 + \ \frac{\Phi'}{2})$$”

2.2.2.1.3 Vesic's Theory

"Vesic formulae are based on both theoretical and experimental findings from these and other sources and is considered the best alternative to Terzaghi. Vesic has considered a number of additional loading and geometric conditions, which has increased its accuracy but at the same time it has added to its complexity.[1]

Vesic retained the Terzaghi's basic formulas and added more factors to it:

2.2.2.1.3.1 Shape, Depth and Inclination Factors

Shape Factors: A very board range of footing shapes were considered and defined:

$$s_c = 1 + (\frac{B}{L})(\frac{N_q}{N_c})$$

$$s_q = 1 + (\frac{B}{L})(tan\varphi')$$

$$s_\gamma = 1 - 0.4(\frac{B}{L})$$

Depth Factors; This method has no limitation of depth of the footing, there it can also be used for deep foundation.

$$d_c = 1 + 0.4k$$

$$d_q = 1 + 2k\ tan\varphi'(1 - sin\varphi')^2$$

$$d_\gamma = 1$$

For relatively shallow foundation (D/B$\leq$ 1), use k=D/B. For deeper footings D/B>1, use k=tan^{-1}(D/B).

Load Inclination Factors; The loads that don't act perpendicular to the base of the footing, but at a certain angle but still through its centroid. [1]

$$i_c = 1 - \frac{mV}{Ac'N_c} \geq 0$$

$$i_q = (1 - \frac{V}{\frac{Ac'}{tan\varphi'}})^m \geq 0$$

$$i_\gamma = (1 - \frac{V}{\frac{Ac'}{tan\varphi'}})^{m+1} \geq 0$$

P refers to the component of the load that acts perpendicular to the bottom of the footing and V refers to the component that acts parallel to the bottom.

For loads inclined in the B direction:

$$m = \frac{2 + B/L}{1 + B/L}$$

For loads inclined in the L direction:

$$m = \frac{2 + L/B}{1 + L/B}$$

Where,

V = applied shear load

P = applied normal load

A = base area of footing

c' = effective cohesion

Φ' = effective friction angle

B = foundation width

L = foundation length

Base Inclination Factor: If the applied load is at an angle from the vertical, it is better to incline the base of the footing to the same angle so the applied load acts perpendicular to the base. [1]

$$b_c = 1 - \frac{\propto}{147°}$$

$$b_\gamma = b_q = \left(1 - \frac{\propto \tan\varphi'}{57°}\right)$$

In most cases the footing is level, therefore the b factors is equal to 1.

Ground Inclination Factor; since the footing located near the top layer have lower bearing capacity than those on level ground. To account for this ground inclination factors. [1]

$$g_c = 1 - \frac{\beta}{147°}$$

$$g_q = g_\gamma = \{1 - tan\beta)^2$$

If the ground surface is level then g factors become equal to 1 and may be ignored.

2.2.2.1.3.2 Bearing Capacity Factors:

Following formulas for computing the bearing capacity factors N_x and N_q:

$$N_q = e^{\pi tan\varphi'}(45 + \frac{\varphi'}{2})$$

$$N_c = \frac{N_q - 1}{tsn\varphi'}$$

$$N_c = 5.14$$

N_γ value has a lot of disagreement in between the scientists. Relativily small changes in the geometry can change the value of N_γ". After a lot of consideration, this formula has been suggested:[1]

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

2.2.2.1.4 Effect of Water Table

When exploring the subsurface conditions, we determine the current location of the groundwater table and worst-case (highest) location that might reasonably be expected during the life of the proposed structure. We then determine which of the following three cases describes the worst-case field conditions: [1]

- Case 1: $D_w \leq D$
- Case 2:$D < D_w < D+B$
- Case 3: $D + B \leq D_w$

We account for the decreased effective stresses along the failure surface by adjusting the effective unit weight ϒ'. The effective unit weight is the value that, when multiplied by the appropriate soil thickness, will give the vertical effective stress. It is the weighted average of the buoyant unit weight, Υ_b, and the unit weight, ϒ, and depends on the position of the groundwater table. We compute ϒ' as follows: [1]

For Case 1 ($D_w \leq D$):

$\gamma' = \gamma_b = \gamma - \gamma_w$

For Case 2 ($D < D_w < D+B$):

$\gamma' = \gamma - \gamma_w (1 - (\frac{D_w - D}{B}))$

For Case 3 ($D + B \leq D_w$; no groundwater correction is necessary):

$\gamma' = \gamma$

In Case 1, the second term in the bearing capacity formulas also is affected, but the appropriate correction is implicit in the computation of σ_D'.

"If a total stress analysis is being performed, do not apply any groundwater correction because the groundwater effects are supposedly implicit within the values of c_T and φ_T. In this case, simply use $\gamma' = \gamma$ in the bearing capacity equations, regardless of the groundwater table position".[1]

2.2.2.1.5 Foundations with eccentric or moment loads

"Most foundations are built so the vertical load acts through the centroid ,thus producing a fairly uniform distribution of bearing pressure .however, sometimes it becomes necessary to accommodate loads that act through other points .these are called eccentric loads ,and they produce a non-uniform bearing pressure distribution." The eccentricity e, of the the bearing pressure is equal to :[1]

$$e = \frac{Pe_1}{P + W_f}$$

or for continuous footings

$$e = \frac{\left(\frac{P}{b}\right)e_1}{\left(\frac{P}{b} + \frac{W_f}{b}\right)}$$

Another similar condition occurs when moment loads are applied to foundations .these loads also produce non uniform bearing pressures .in this case, the eccentricity of the bearing pressure is:[1]

$$e = \frac{M}{P} + W_f \qquad e = M/P+W_f$$

$$e = M/b\ /(P/b + W_f/b)$$

Where:

e = eccentricity of bearing pressure distribution

P= applied vertical load

P/b= applied vertical load per unit length of foundation

M= applied moment load

M/b=applied moment load per unit length of foundation

e_1= eccentricity of applied vertical load

W_f= weight of foundation

W_f/b= weight of unit length of foundation

Two-way eccentric or moment loading: "If the resultant load acting on the base is eccentric I n both the B and L directions ,it must fall within the diamond shaped kern for the contact pressure to be compressive along the entire base of the foundation" .it falls within this kern only if the following condition is met :[1]

$$\frac{6e_b}{B} + \frac{6e_L}{L} \leq 1$$

Where:

$e_{B=}$= eccentricity in the B direction

e_L = eccentricity in the L direction

If condition is satisfied the magnitude of q at the four corners of a square or rectangular shallow foundation are:

$$q = (\frac{P+W_f}{A} - u_D)(1 \pm \frac{6e_b}{B} \pm \frac{6e_L}{L})$$

2.2.2.2 Serviceability Requirements

Foundations that study strength requirements will not collapse, but they still may not have adequate performance .for example, they may experience excessive settlement. Therefore we have the second category of performance requirements which are known as serviceability requirements.[1]

Settlement: The vertical downward load is usually the greatest load acting in foundations and the resulting vertical downward movement is usually the largest and most important movement. This vertical downward movement settlement. [1]

Settlement also occurs as a result of consolidation due to replacement of a fill. Although foundations with zero settlement old be ideal this is not an attainable goal. Stress and strain always go together, so imposition of loads from the foundations always cause some settlement in the underlying soils.so it is always tried to keep settlement in tolerable limits.[1]

2.2.2.2.1 Terzaghi 1-D Consolidation Settlement Computation

We compute the consolidation settlement by driving the soil beneath the foundation into layers, computing the settlement of each layer, and summing.

The soil beneath the foundation is divided into layers then settlement of each layer is computed and added. The top of first layer should be at the bottom of the foundation, and the bottom of last layer should be t a depth such that σ_z<0.10 σ_{zo}'.[1]

For normally consolidated soils ($\sigma_{z0}' = \sigma_c'$):

$$\delta_c = r \, \Sigma \frac{Cc}{1+e_0} H \log \left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right)$$

For over consolidated soil – case 1 ($\sigma_{zf}' < \sigma_c'$):

$$\delta_c = r \, \Sigma \frac{Cr}{1+e_0} H \log \left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right)$$

For over consolidated soil case 2 ($\sigma_{z0}' < \sigma_c' < \sigma_{zf}'$):

$$\delta_c = r \, \Sigma\left[\frac{Cr}{1+e_0} H \log \left(\frac{\sigma'_{c}}{\sigma'_{z0}}\right) + \frac{Cc}{1+e_0} H \log \left(\frac{\sigma'_{zf}}{\sigma'_{c}}\right)\right]$$

APPROXIMATE THICKNESS OF SOIL LAYERS FOR MANUAL COMPUTATION OF CONSOLIDATION SETTLEMNENT O SHALLOW FOUNDATIONS

	Approximate layer thickness	
Layer Number	square footing	continuous footing
1	B/2	B
2	B	2B
3	2B	4B

Where as

r = rigidity factor

C_c = compression index

Cr = recompression index

e_o = Initial void ratio

H = thickness of soil layer

σ_{z0}' = initial vertical effective stress at midpoint of soil layer

σ_{zf}' = final vertical effective stress at midpoint of soil layer

σ_{zf}' = preconsolidation stress at midpoint of soil layer

2.2.2.2.2 Skempton and Bjerrum method

The classical method is based on that settlement is one dimensional process in which all of strains are vertical. this assumption is accurate when evaluating settlement beneath the centre of wide fills, but it is less accurate when applied to shallow foundations, especially spread footing because their loaded area is much smaller, so Skempton and Bjerrum (1957) present method which account for three dimensional effect by dividing settlement into two components:[1]

- **Distortion Settlement,** (also called immediate settlement, initial settlement, or undrained settlement) is that cause by lateral distortion of soil beneath the foundation
- **Consolidation Settlement,** (also known as primary consolidation settlement) which because of change in effective stress which result in change in volume of soil.

In addition, Skempton and Bjerrum accounted for differences in such a way that when soil experience lateral strain, excess pore water pressure generated. This is reflected in parameter. [1]

Skempton and Bjerrum, computed settlement of shallow foundation as:

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

Where:

δ = settlement

δ_d= distortion settlement

ψ= three dimensional adjustment factor

δ_c=consolidation settlement

Based on elastic theory, the distortion settlement is:

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

2.2.2.2.3 Schmertmann's Method (for sands only)

Schmertmann's method was primarily developed to study the settlement of spread footing on sandy soils. Cone Penetration Test is the most commonly used method, but other in-situ methods are used to in the field. Schmertmann's method is based on physical model of settlement unlike other methods which are completely empirical. [1]

Schmertmann's method have the following steps:

1. Perform appropriate in-situ test.
2. The soil is considered from the base of the foundation to the influence zone. This zone is then divided into layers, around 5-10 layers.
3. Calculate peak strain influence factor
4. Calculate strain influence factor at mid-point of each layer
5. Calculate the correction factors.
6. Calculate the settlement.

2.2.2.2.3.1 Equivalent Modulus of Elasticity:

Unlike the classical method which use Compressive Index and Re-compressive Index, Schmertmann's uses the equivalent modulus of elasticity. This is a linear factor and hence the computations are simpler. But since soil is not a linear material i.e. the stress and strain, it is essential that E_S reflects that of an equivalent unconfined linear material such that the computed settlement will be the same as in real soil.[1]

The design value of E_S implicitly reflects the lateral strains in the soil, therefore it is larger than Young's Modulus but smaller than Confined Modulus. [1]

E_s from Cone Penetration Test (CPT) Results: Schmertmann developed empirical relations between E_s and the cone resistance q_c. CPT provides a continuous plot of q_c vs. depth, so our analysis can model E_s as a function of depth. The values of E_s/q_c vary 2.5 to 10, depending on the age and the consolidation of the sand or clay. It is usually considered that the soil is young and normally consolidated except when.[1]

E_s from Standard Penetration Test (SPT) Results: E_s computed form Standard Penetration test may also be used in Schmertmann's Method. These values may not be as precise as that of CPT because:

1. SPT has errors and is less accurate
2. Provides data of isolated point, CPT provides a continuous plot.

SPT data is adequate where soil conditions are good and loads are small.

After much experimentation and direct correlation between N_{60} and E_s this was the final relation that could produce an approximate value of E_s:

$$E_s = \beta_0\sqrt{OCR} + \beta_1 N_{60}$$

Where;

E_s = equivalent modulus of elastic
β_0, β_1 = correlation factors
OCR = over-consolidation ratio
N_{60} = SPT N-Value

Mostly OCR value is taken as 1, unless clear evidence is present.

E_s from Dilatometer Test Results: The dilatometer directly measures the modulus of elasticity and it can be used by Schmertmann analysis. Also it is proposed to combine the answers of CPT and Dilatometer and then use a modified version in Schmertmann.[1]

E_s from Pressure Test Results: This tests also provides data for the Schmertmann analysis. Special analysis methods intended specifically for Pressure test are also available. [1]

2.2.2.2.3.2 Strain Influence Factor:

After an extensive research on the distribution of vertical strain ε_2 below the spread footing, it was found that maximum strain value is not immediately below the footings, but at a depth of 0.5B to B, below the bottom of the footing, where B is the width of the footing. This distribution can be explained by strain Influence Factor I_ε. The peak value of I_ε is:[1]

$$I_{\varepsilon p} = 0.5 + 0.1\frac{\sqrt{q-\sigma zD}}{\sqrt{\sigma_{zP}}}$$

Where:

$I_{\varepsilon p}$ = peak strain influence factor

q = bearing pressure

σ_{zD} = vertical effective stress at a depth D below the ground surface

σ_{zP} = initial vertical stress at depth of peak strain influence factor. For square and circular footing (L/B= 1), compute at a depth of D+B/2 below the ground surface. For continuous footings, (L/B>10), computation should be done at D+B. [1]

Equation have been derived for the exact value of I_ε:

Square and Circular Footings

For z_f = 0 to B/2:

$$I_e = 0.1 + (z_f/B)(2I_{ep} - 0.2)$$

For z_f = B/2 to 2B:

$$I_e = 0.667I_{ep}(2 - z_f/B)$$

Continuous Footings

For z_f = 0 to B:

$$I_e = 0.2 + (z_f/B)(2I_{ep} - 0.2)$$

For z_f = B to 4B:

$$Ie = 0.333I_{ep}(4 - z_f/B)$$

Rectangular Foundations (1<L/B<10)

$$I_e = I_{es} + 0.111(I_{ec} - I_{es})(L/B - 1)$$

Where:

z_f = depth from bottom of foundation to midpoint of layer

I_e = strain influence factor

I_{ec} = I_e for continuous foundation

I_{ep} = peak I_e from equation

I_{es} = I_e for square foundation

In rectangular footings, to compute I_e under the foundations, first the I_e for each layer is calculated using the square foundations equation and then I_e for each layer using the combined footing formulas, and then the results are both used in the rectangular footing equation given above.[1]

2.2.2.2.3.3 Correction Factors

Schmertmann's method also provides the correction factor for depth of embankment, secondary creep in the soil and footing shape. They are:

$$C_1 = 1 - 0.5[\sigma'_{zD}/(q - \sigma'_{zD})]$$

$$C_2 = 1 + 0.2\log(t/0.1)$$

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

Where;

δ = settlement

C_1 = depth factor

C_2 = secondary creep factor

C_3 = shape factor

q = bearing pressure

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

I_E = 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

$$\delta = C_1C_2C_3 (q - \sigma'_{zD}) \Sigma (I_EH/ E_s)$$

2.2.2.2.4 METHOD OF BURLAND AND BURBIDGE (1985)

Burland and Burbidge (1985) proposed a method for calculating the elastic settlement of sandy soil using the field standard penetration number *N*60.[3]

Following Steps need to be followed:

1. Determination of Variation of Standard Penetration Number with Depth

Obtain the field penetration numbers (*N*60) with depth at the location of the foundation. The following adjustments of *N*60 may be necessary, depending on the field conditions:[3]

For gravel or sandy gravel,

$N\,60(a) \approx 1.25N\,60$

For fine sand or silty sand below the ground water table and *N*60 > 15,

$N_{60}(a) \approx 15 + 0.5(N60 - 15)$

Where $N_{60}(a)$ = adjusted *N*60 value

2. Determination of Depth of Stress Influence (z')

In determining the depth of stress influence, the following three cases may arise:

Case I: If N_{60} [or $N_{60}(a)$] is approximately constant with depth, calculate z' from

$$\frac{z'}{Br} = 1.4\left(\frac{B}{Br}\right)^{0.75}$$

Where B_R = reference width = 0.3 m

B = width of the actual foundation (m)

Case II: If *N*60 [or N_{60} (a)] is increasing with depth, use Eq. (41) to calculate z'.

Case III: If *N*60 [or $N_{60}(a)$] is decreasing with depth, calculate $z' = 2B$ and z' = distance from the bottom of the foundation to the bottom of the soft soil layer (= z''). Use $z' = 2B$ or $z' = z''$ (whichever is smaller) [3]

3. Determination of Depth of Stress Influence Correction Factor α

The correction factor α is given as

$$\alpha = \frac{H}{z'}\left(2 - \frac{H}{z'}\right) \leq 1$$

Where H = thickness of the compressible layer

4. Calculation of Elastic Settlement

The elastic settlement of the foundation *Se* can be calculated as:

A. For normally consolidated soil

$$\frac{S_e}{B_R} = 0.14\alpha\left\{\frac{1.71}{\left[\bar{N}_{60} or \bar{N}_{60(a)}\right]^{1.4}}\right\}\left[\frac{1.25\left(\frac{L}{B}\right)}{0.25+\left(\frac{L}{B}\right)}\right]^2\left(\frac{B}{B_R}\right)^{0.7}\left(\frac{q}{p_a}\right)$$

L = length of the foundation

pa = atmospheric pressure (≈100 kN/m2)

B. For over consolidated soil ($q \leq \sigma c'$; where $\sigma c'$ = over consolidation pressure)

$$\frac{S_e}{B_R} = 0.047\alpha\left\{\frac{0.57}{\left[\bar{N}_{60}\ or\ \bar{N}_{60(a)}\right]^{1.4}}\right\}\left[\frac{1.25\left(\frac{L}{B}\right)}{0.25+\left(\frac{L}{B}\right)}\right]^2\left(\frac{B}{B_R}\right)^{0.7}\left(\frac{q}{p_a}\right)$$

C. For over consolidated soil ($q > \sigma c'$)

$$\frac{S_e}{B_R} = 0.047\alpha\left\{\frac{0.57}{\left[\bar{N}_{60}\ or\ \bar{N}_{60(a)}\right]^{1.4}}\right\}\left[\frac{1.25\left(\frac{L}{B}\right)}{0.25+\left(\frac{L}{B}\right)}\right]^2\left(\frac{B}{B_R}\right)^{0.7}\left(\frac{q}{p_a}\right)$$

The procedure works reasonably well. However it may be difficult under normal working conditions to obtain the over consolidation pressure in the field. [3]

2.2.2.3 Design Chart

It is the combination of Bearing Capacity and Settlement for a particular site conditions.

For larger structures especially those with wide range of column loads require many spread footings, perhaps dozen of them so it is inconvenient to perform customer bearing capacity and settlement analyses for each one. So, geotechnical engineers develop generic design criteria that is applicable to the whole site, then structural engineer sizes each footing based on its load and these criteria. Its methodology is described in “Methodology” portion. [3]

2.2.3 Structural Design

Footings and other foundation units transfer loads from the structure to the soil or rock supporting the structure. Because the soil is generally much weaker than the concrete columns and walls that must be supported, the contact area between the soil and the footing is much larger than that between the supported member and the footing. The choice of foundation type is selected in consultation with the geotechnical engineer. Factors to be considered are the soil strength, the soil type, the variability of the soil typeover the area and with increasing depth, and the susceptibility of the soil and the building to deflections. 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.[4] Limit States for Design of Reinforced Concrete Foundations include:

- Limit States governed by the Soil
    1. a bearing failure of the soil under the footing
    2. a serviceability failure in which excessive differential settlement between adjacent footings causes architectural or structural damage to the structure
    3. Excessive total settlement
- Limit States Governed by the Structure
    1. Bearing Failure below column in footing
    2. Failure of Column footing Connection
    3. Failure of Reinforcement bond and hence slipping of reinforcement
    4. Flexural Failure of Footing

In our project till now we are dealing with shallow foundations. Shallow foundations include:

2.2.3.1 Wall Footings

Strip footings or *wall footings* display essentially one-dimensional action, cantilevering out on each side of the wall. It 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.[4]

A wall footing cantilevers out on both sides of the wall. The soil pressure causes the cantilevers to bend upward, and as a result, reinforcement is required at the bottom of the footing. The critical sections for design for flexure and anchorage are at the face of the wall (section *A–A*). One-way shear is critical at a section a distance *d* from the face of the wall (section *B–B*). The presence of the wall prevents two-way shear.[4]

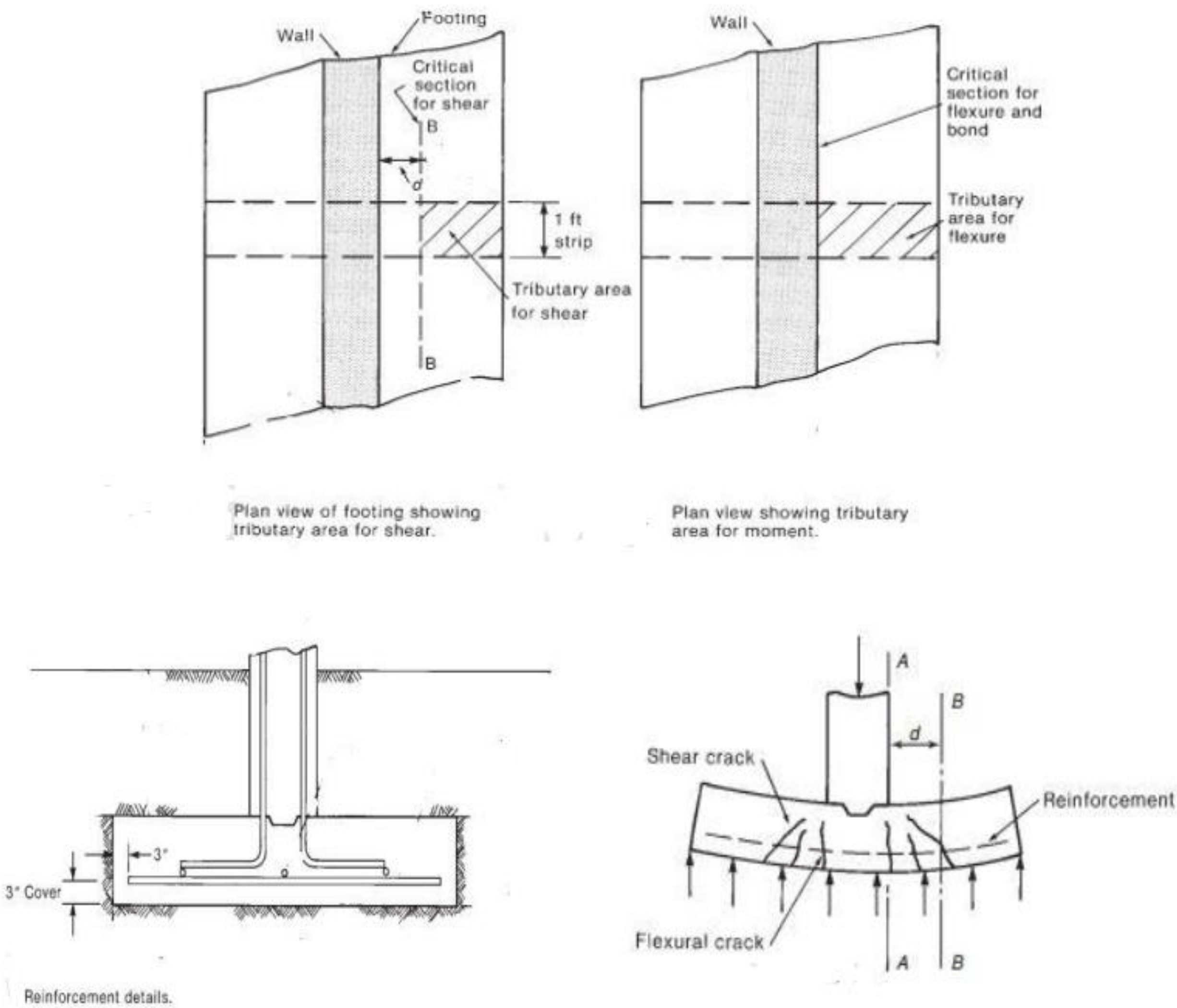


Figure 8: Strip Footing Structural Design

2.2.3.2 Spread Footing

Spread footings are square or rectangular pads that spread a column load over an area of soil large enough to support the column load. The soil pressure causes the footing to deflect upward causing tension in two directions at the bottom. As a result, reinforcement is placed in two directions at the bottom.[4]

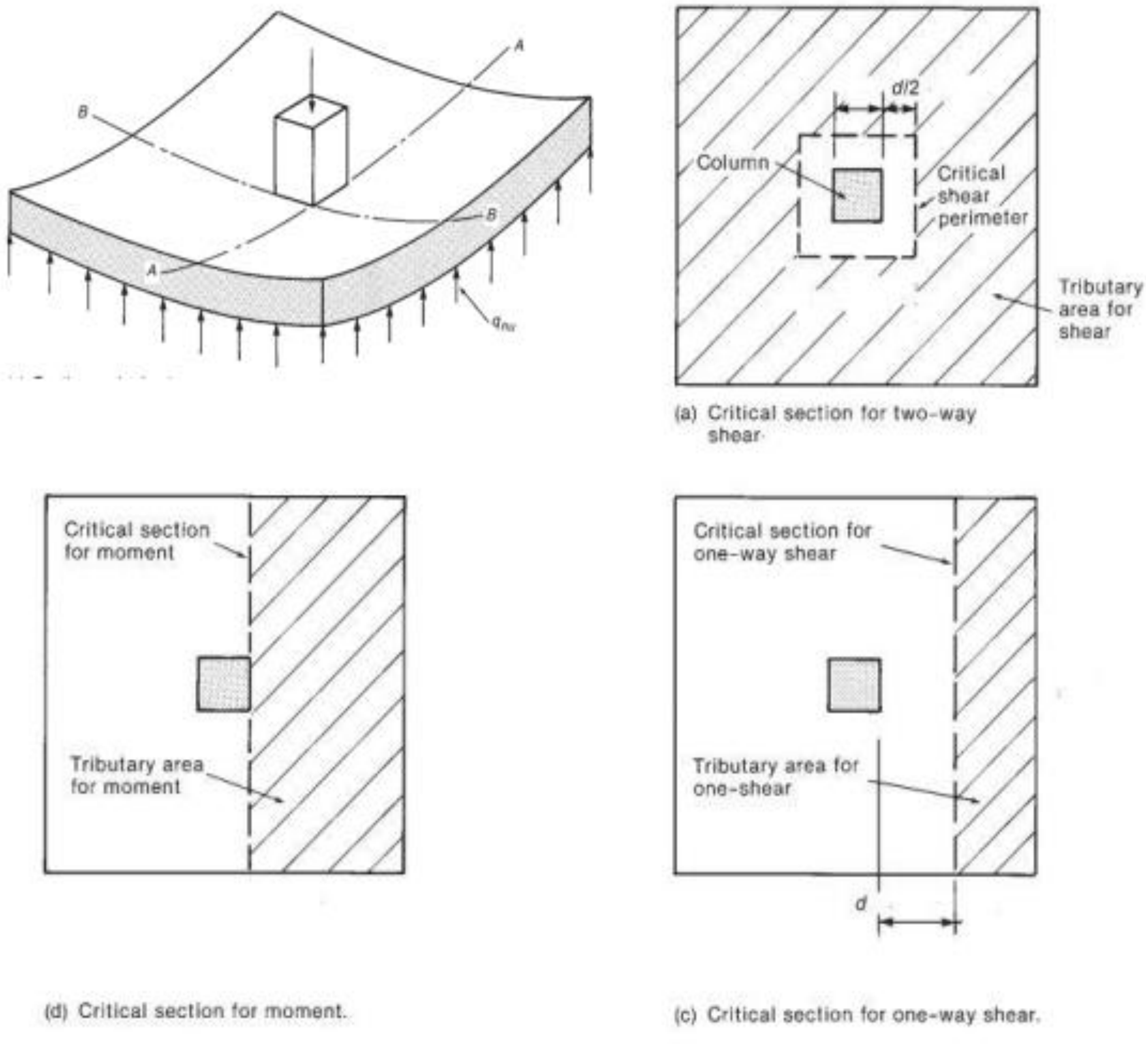


Figure 9: Spread Footing Structural Design

2.2.3.3 Combined Footing

Combined footings are used to support two or more column loads. They might be economical where two or more heavily loaded columns are so spaced that normally designed single-column footings would run into each other. Where one column is so close to a property line causing a usual isolated footing to extend across the line. Centroid of the footing should be made to coincide with the centroid of column loads to prevent uneven settlement.[4]

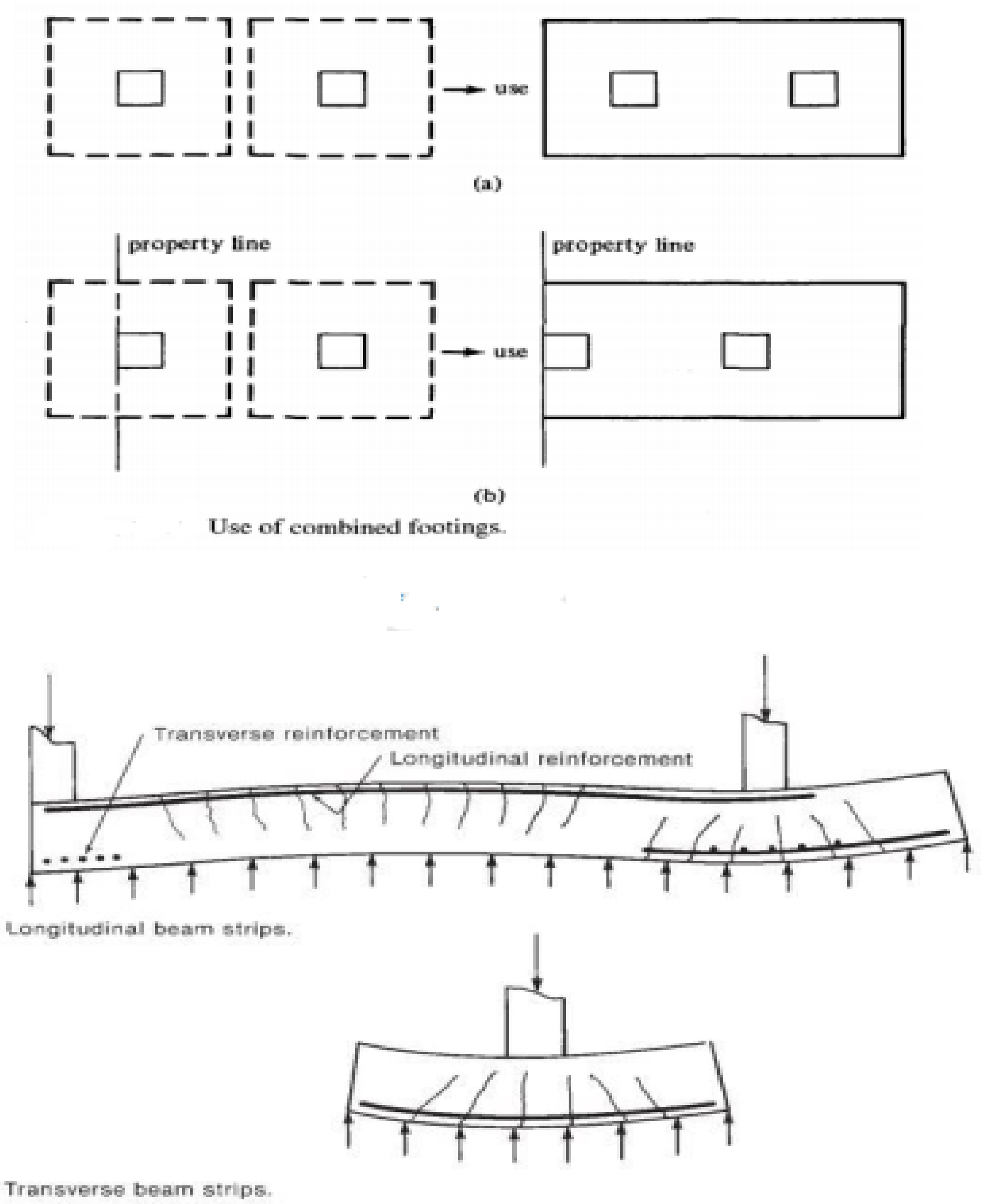


Figure 10: Combined Footing Structural Design

2.2.3.4 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.[4][1][5]

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).[1]

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.[1]

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.[1]

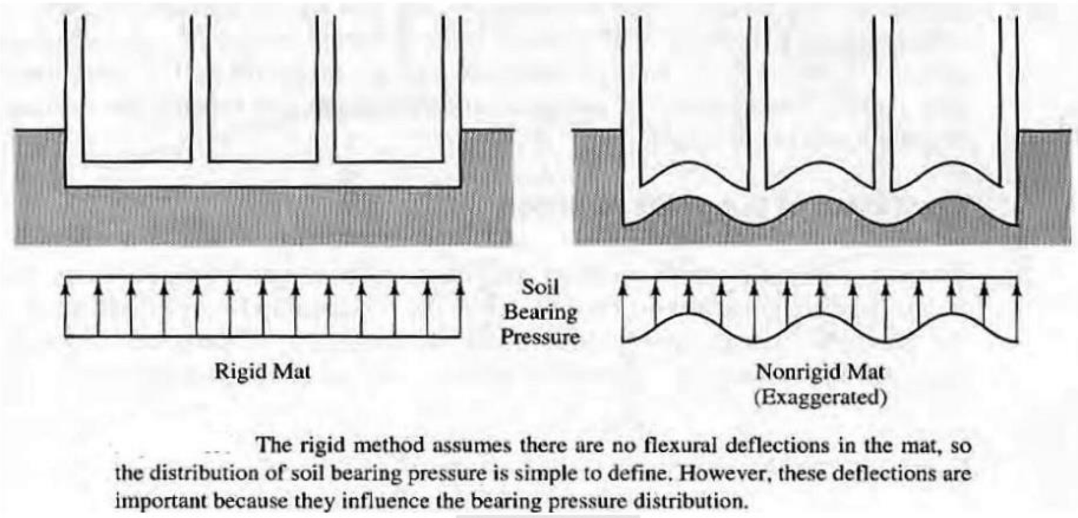


Figure 11: Mat Footing Design

2.2.4 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 Foundation

Spread footing is the most commonly used footings, but there are some situations where it is inadvisable to use a spread footing. Following are some cases:

1. If we encounter weak soil, or the structural load is too high that the area covered by spread footing is more.
2. If the upper soil is subjected to scour or undermining.
3. If the foundation is to be placed in water, e.g. piers.
4. The uplift of the spread footing is limited, so it can't be used if large uplift capacity is needed.
5. High lateral capacity is required

6. If any future excavation adjacent to the foundation is predicted, this can cause undermining of the footing.

In these cases, it is then advised to move towards **Deep Foundation**. A deep foundation is a type of foundation which transfers building loads to the earth farther down from the surface than a shallow foundation does, to a subsurface layer or a range of depths (50ft-150ft).[1]

Classification

1. **Piles:** Piles are constructed by using slender prefabrication members and driving or otherwise forcing them into ground.
2. **Drilled Shafts:** They are constructed by drilling a cylindrical hole into the ground, inserting reinforced steel and filling it by concrete.
3. **Caissons:** Caissons is one of the most confusing terms, since some engineers use this term for Drilled shafts. They are prefabricated boxes or cylinder that are sunk into the ground and then filled concrete.[1]

2.3.1 Axial Load Capacity

The most important geotechnical design requirement for most deep foundation is that they have enough axial load capacity to support the applied load. Ways have been devised to axial load capacity and use these to properly size the foundation.[1]
They have 3-main categories:

1. Full Scale static load tests on prototype foundations.
2. Analytical methods.
3. Dynamic methods.

Load Transfer: Axial load is applied to the ground by deep foundations by 2 methods, side friction and toe bearing. Skin friction is the result of sliding friction along the side of the foundation and adhesion between the soils and the foundation. Toe resistance is the result of compressive loading between the bottom of the foundation and soil. [1][6]

2.3.1.1 Downward Loads*:*

Side friction and toe bearing are fundamentally different modes of resistance, therefore they are evaluated separately.[1]

1. $P_a = \frac{P_{ult}}{F} = \frac{P_t + P_s - W_f}{F}$

P_a = allowable downward load capacity

P_{ult} = ultimate downward capacity

P_t = toe bearing resistance

P_s = side- friction resistance

W_f = weight of foundation

F = Factor of Safety

Simplifying the formula by sing net toe-bearing resistance:

1. $P_t' = P_t - W_f$
2. $P_a = \frac{P_t' + P_s}{F}$

In terms of unit toe bearing and side friction resistance gives:

1. $P_a = \frac{q_t' A_t + \sum f_s A_s}{F}$

The foundation must satisfy the following design criteria:

1. $P \leq P_a$

Where:

P = design downward load

P_a = allowable downward load capacity

q_t' = net unit toe-bearing resistance

A_t = toe bearing contact area

f_s = unit side-friction resistance

A_s = side friction contact area

F = Factor of safety

The toe-bearing and side-friction contact areas depends on the foundation geometry, and the net unit toe-bearing and side friction resistances primarily depend on the soil properties. Side friction varies in depth so the foundation is divided into sections and solved for each section.[1]

2.3.1.2 Upward Load

Due to the weight and the side friction in deep foundation, they are more effective in resisting upward loads. Deep foundation with expanded bases can resist additional uplift loads through bearing on top of the base.[1]

In case of upward forces, the foundation should satisfy:

1. $P_{upward} \leq (P_{upward})_a$

Where,

P_{upward} = applied tensile load.
$(P_{upward})_a$ = allowable upward load capacity.

Both the equation should be satisfied if the foundation is subjected to both upward and downward loading.

The allowable upward load capacity $(P_{upward})_a$ for straight deep foundation with D/B>6 is:

$$(P_{upward})_a = \frac{(Wf + \Sigma fsAs)}{F}$$

$(P_{upward})_a$ = allowable upward load capacity
W_f = weight of the foundation|
f_s = unit side friction resistance
A_s = side friction contact area
F = Factor of Safety

If the foundation is partially or fully submerged, the buoyancy effects should be considered in the computed value of W_f.

If D/B<6, a cone of soil may form around the foundation during an upward failure. This reduces the uplift capacity. But for deep foundation this problem is avoided because of their length.[1]

2.3.2 Lateral Load Capacity

Just like Axial Loads Soil has to bear certain lateral loads as well, so requiring the need for determining lateral load capacity.

2.3.2.1 Non-rigid Soil-Structure Interaction Analyses

Because of the shortcomings of the methods described thus far, engineers have developed more thorough lateral load analysis methods that consider the flexural rigidity of the foundation, the soil's response to lateral loads, and soil-structure interaction effects. This can be done using either of two methods: the finite element method or the p-y method.[1]

2.3.2.1.1 Finite Element Method

A finite element method (FEM) analysis consists of dividing both the foundation and the soil into a series of small elements and assigning appropriate stress—strain properties to each element The analysis then considers the response of these elements to applied loads, and uses this response to evaluate shears, moments, and lateral deflections in the foundation. Finite element analyses may be performed using either two-dimensional or three-dimensional elements. [1]

The accuracy of finite element analyses depends on our ability to assign correct engineering properties to the elements. This is easy to do for the foundation because the properties of structural materials are well-defined, but very difficult to do for the soil because it is more complex. For example, the stress-strain properties in the soil are definitely nonlinear. In addition, three-dimensional FEM analyses, which are more accurate, require more extensive computer resources.[1]

Finite element analyses have been used on specialized projects, and ultimately may become the preferred method of evaluating laterally-loaded deep foundations. However, they are still under development and require additional calibration with load test results before they are likely to be used on routine projects. [1]

2.3.2.1.2 p-y Method

The p-y method uses a series of nonlinear "springs" to model the soil—structure interaction, This is similar to the method used to analyse mat foundations, and is much simpler than the finite element method. Although the p-y method is not as rigorous as the finite element method, it has been extensively calibrated with full-scale load test results, and is easier to implement due to the widespread availability of commercial software. Therefore, this is the preferred method for most practical design problems, especially with "long" foundations.[1]

Numerical Model: The p-y method models the foundation using a two-dimensional finite difference analysis, It divides the foundation into n intervals with a node at the end of each interval, and the soil as a series of nonlinear "springs" located at each node, The flexural stiffness of each interval is defined by the appropriate EI, and the load-deformation properties of each spring is defined by a p-y curve. It also is necessary to apply appropriate boundary conditions, as described earlier.[1]

Using this information and applying the structural loads in increments, the software finds a condition of static equilibrium and computes the shear, moment, and lateral deflection at each interval. [1]

P-y Curves: The heart of the p-y method is the definition of the lateral load-deflection relationships between the foundation and the soil. These are expressed in the form of p-y curves, where p is the lateral soil resistance per unit length of the foundation (expressed in units of force per length), and y is the lateral deflection. The p-y relationship might first appear to be a nonlinear extension of the Winkler beam-on-elastic-foundation concept. However, there is an important difference between the two: The Winkler model considers only compressive forces between the foundation and the soil, whereas the lateral soil load acting on a deep foundation is the result of compression on the leading side, shear friction on the two adjacent sides, and possibly some small compression on the back side. Thus, it is misleading to think of the p-y curve as a compression phenomenon only (Smith, 1989), even though the numerical model appears to treat them as such.[1]

Evans and Duncan's Charts: Evans and Duncan (1982) developed a convenient method of expressing the lateral load-deflection behaviour in chart form. They compiled these charts from a series of p-y method computer analyses using the computer program **C0M624**.[1]

The advantages of these charts include the following:

• The analyses can be performed more quickly, and they do not require the use of a computer.

• The load corresponding to a given pile deflection can be determined directly, rather than by trial.

• The load corresponding to a given maximum moment in the pile can be determined directly, rather than by trial.

These charts are also a useful way to check computer output from more sophisticated analyses. The charts presented here are a subset of the original method and apply only to deep foundations that satisfy the following criteria:

• The stiffness, El, is constant over the length of the foundation.

• The shear strength of the soil, expressed either as s_u or φ, and the unit weight, Ɣ, are constant with depth.

• The foundation is long enough to be considered fixed at the bottom. For relatively flexible foundations, such as timber piles, this corresponds to a length of at least 20 diameters. For relatively stiff foundations, such as those made of steel or concrete, the length must be at least 35 diameters. [1]

We can idealize deep foundations that deviate slightly from these criteria, such as tapered piles, by averaging the El, s_u, φ, or γ values from the ground surface to a depth of 8 pile diameters.[1]

2.3.3 Pile Designing Suite

2.3.3.1 Oasys Pile (Version 19.6)

2.3.3.1.1 General

“**Oasys Pile** Pile load capacity and Settlement”[7]

“**Oasys Pile** calculates 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 undrained (cohesive) soils. Settlements are calculated for solid circular sections without under-ream.”[7]

2.3.3.1.2 Program Features

“The main features of **Oasys Pile** are summarised below.

Capacity analysis, settlement analysis, or both can be performed for a range of pile lengths and cross-sections in different soil profiles.”[7]

“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).”[7]

“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.”[7]

“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.”[7]

1. Working load approach
2. Limit-state approach, and
3. Code-based approach

2.3.3.1.3 API T-z curve

“API curve was used to calculate Settlement “

“Material Type - selection has to be made between two materials: sand and clay.”[7]

“z_c - the movement required to mobilise maximum stress. This is active only when the material type is sand.”[7]

“t_{RES}/t_{max} - the ratio of mobilised stress to maximum stress. This is active only when the material type is clay.”[7]

2.3.3.2 Oasys Alp

2.3.3.2.1 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.”[8] “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 behaviour can be modelled either assuming an Elastic-Plastic behaviour, 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."[8]

2.3.3.2.2 Program Features

"The main features of the problem analysed by **Alp** are summarised below and represented diagrammatically."[8]

"The **geometry** of 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

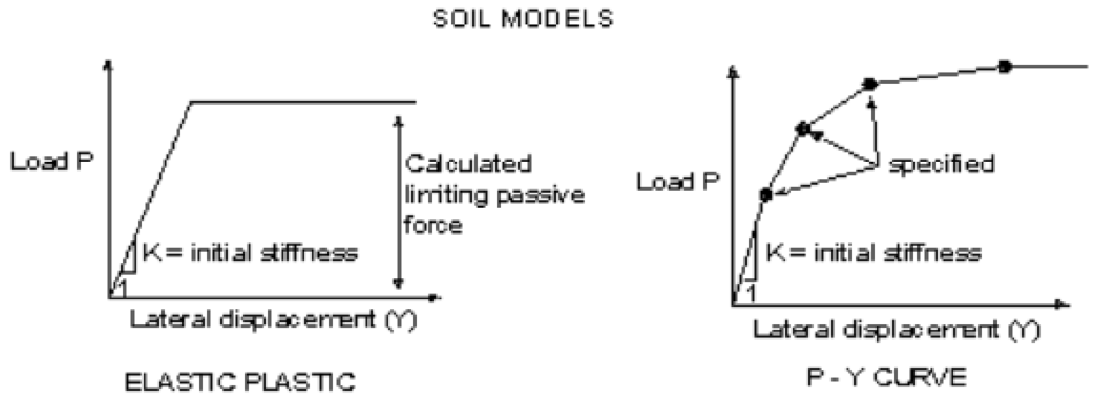


Figure 12: Soil Models for Lateral Load Analysis

boundaries, loads, restraints and displacements."[8]

"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."[8]

1. Elastic-Plastic2. Specified P-Y curves

3. Generated P-Y curves

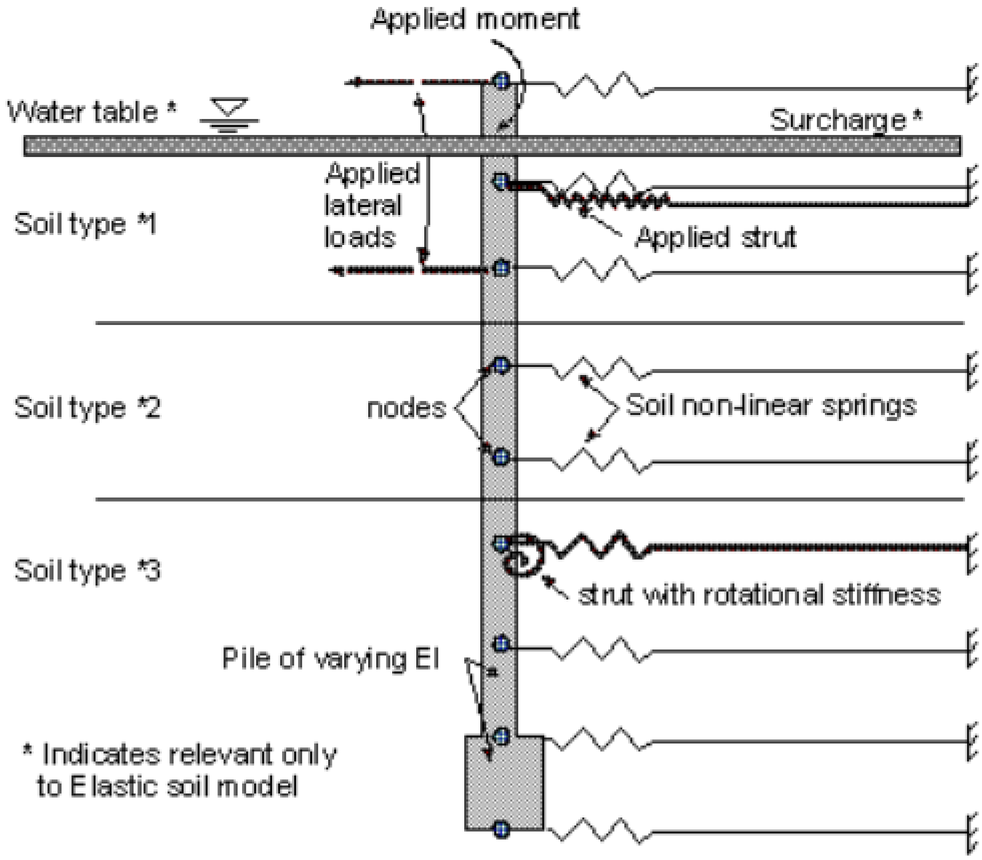


Figure 13: Concept of P-y curves

2.3.3.2.3 API RP2A 21st Edition (2000)

“From this software we have used this curve for calculations of lateral deflection in case of sand.”[8]

2.3.3.2.4 Soft Clay

“P-Y curves for soft clay are calculated using the method established by Matlock (1970).”[8]

2.3.3.3 Oasys AdSec

“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.”

2.3.3.3.1 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

METHODOLOGY

3.1 Design Chart Methodology

3.1.1 Input Data

In input portion you have to define:

Non-Cohesive Soil

- Shape
- Depth Limitation Criteria set as that of Terzaghi 0.25<D/B<1
- L/B ratio in the form of L and B
- Footing Depth “D”
- Depth of Water Table Dw
- Cohesion, c
- Relative Density Dr
- Unit weight γ
- SPT N_{60}
- OCR
- Soil type
- FOS for BC

Cohesive Soil

- Shape
- Depth Limitation Criteria set as that of Terzaghi 0.25<D/B<1
- L/B ratio in the form of L and B
- Footing Depth “D”
- Depth of Water Table Dw

- Foundation Rigidity Factor
- Friction Angle
- Organic Content
- SPT “N” values
- Unit weight γ
- Cc/1+e
- Cr/1+e
- $\boldsymbol{\sigma}$’m
- Su values can be entered directly or can be based on ‘N”, subjected to option selected

We must have to enter data of four soil layers, and maximum depth of input data is fixed and is based on the maximum influence depth for this maximum footing width. As we have a range of footing sizes available in the chart so data for settlement calculations for influence depth of each footing is extracted from the main input data, which maximum depth is based on maximum width available in chart.

3.1.2 Bearing Capacity Calculations

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

Four method were used.

- Terzaghi
- Meyerhof
- Hansen
- Vesic

Terzaghi used limit equilibrium method to compute bearing capacity. It was the first limit equilibrium method which was accepted by worldwide.

Later on Meyerhof (1963), Brinch Hansen (1970) and then Vesic’s formulation appeared in 1973 that is based on experimental results redefined the method.

Improvements in Terzaghi's Method are Foundation with depth, shape, load inclination or ground inclination factors.

3.1.3 Settlement Calculations

3.1.3.1 Non-Cohesive Soil

Schmertmann's method is preferred over other methods that are purely empirical, because it is based on physical model of settlement which has been calibrated using empirical data. Sivakugan and Johnson (2004) made a comparison between different methods: Schmertmann Et. Al. give relatively more accuracy as compared to that of "Burland and Burbidge" (empirical method). With improvement in defining Influence factors, accuracy of Schmertmann's method is improved.

3.1.3.2 Cohesive Soil

Analyses for cohesive soil generally require more input as compared t that for cohesion less soils, as this method requires, consolidation, settlement to be calculated and that requires greater number of parameter due to much more complexity in defining properties of clay as compared to sands.

Settlement calculated is based on Skempton and Bjerrum method, theoretically

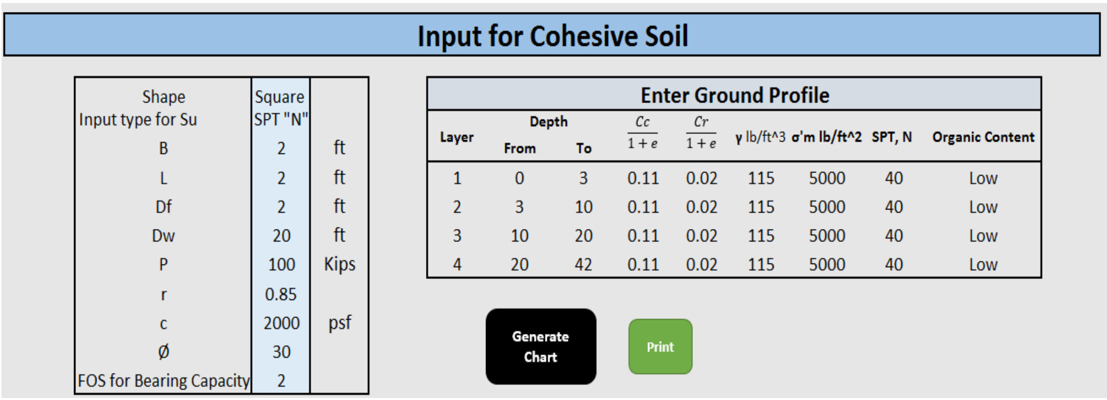
$$\delta = \delta_d + \Psi\delta_c$$

Where δc is obtained from Terzaghi 1-D consolidation.

Terzaghi 1-D consolidation method is good estimate for consolidation, countering small areas or loaded areas with less lateral extent and this method also gives good estimate for soils, which don't have much higher plasticity.

3.1.4 Using Automated Workbook

- Input require data
- Click on “generate chart button” (this click will calculate everything for you and chart will be generated).
- Click on “print chart” to get a print of design chart.



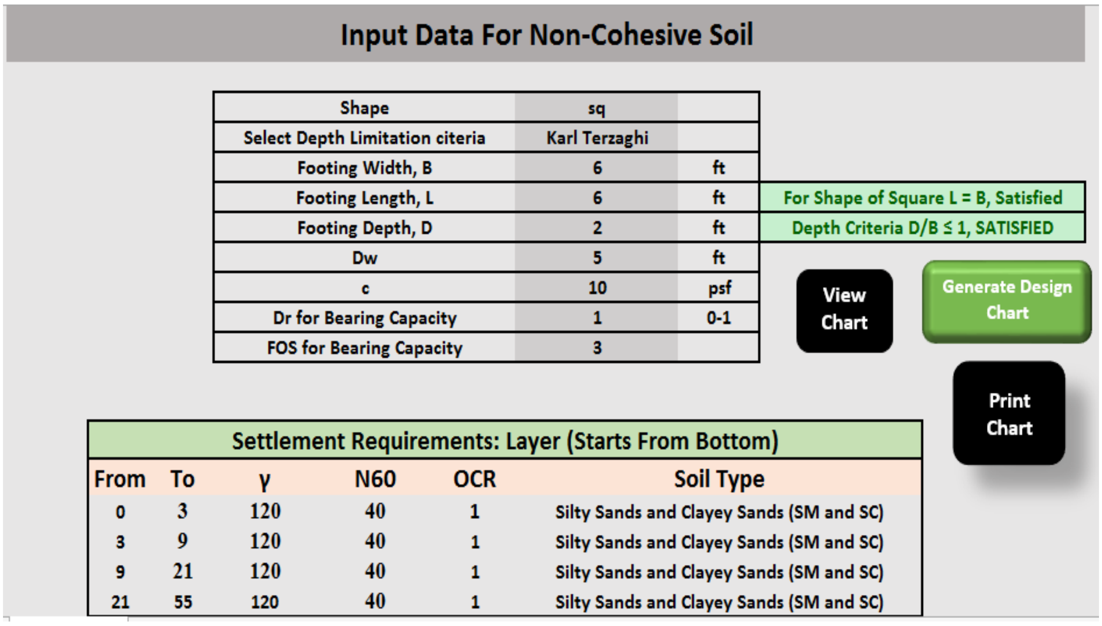
Input for Cohesive Soil

Shape	Square	
Input type for Su	SPT "N"	
B	2	ft
L	2	ft
Df	2	ft
Dw	20	ft
P	100	Kips
r	0.85	
c	2000	psf
Ø	30	
FOS for Bearing Capacity	2	

Enter Ground Profile

Layer	Depth From	To	$\frac{Cc}{1+e}$	$\frac{Cr}{1+e}$	γ lb/ft^3	σ'm lb/ft^2	SPT, N	Organic Content
1	0	3	0.11	0.02	115	5000	40	Low
2	3	10	0.11	0.02	115	5000	40	Low
3	10	20	0.11	0.02	115	5000	40	Low
4	20	42	0.11	0.02	115	5000	40	Low

Figure 14: Input for Cohesive Soil



Input Data For Non-Cohesive Soil

Shape	sq	
Select Depth Limitation citeria	Karl Terzaghi	
Footing Width, B	6	ft
Footing Length, L	6	ft
Footing Depth, D	2	ft
Dw	5	ft
c	10	psf
Dr for Bearing Capacity	1	0-1
FOS for Bearing Capacity	3	

Settlement Requirements: Layer (Starts From Bottom)

From	To	γ	N60	OCR	Soil Type
0	3	120	40	1	Silty Sands and Clayey Sands (SM and SC)
3	9	120	40	1	Silty Sands and Clayey Sands (SM and SC)
9	21	120	40	1	Silty Sands and Clayey Sands (SM and SC)
21	55	120	40	1	Silty Sands and Clayey Sands (SM and SC)

Figure 15: Input for non-Cohesive Soil

3.2 Spread Footing Structural Design

We decided to design spreadsheets that are fully automated and can design footings for a given loading. For this purpose, we considered the four footings spread, strip, combined and mat.

3.2.1 Automated Fixed Workbook for Spread footing Design for Biaxial Bending:

At the end we came out with an automated excel workbook for spread footing design, that can design that can design for biaxial moments also. Its significance is that this type of example is not available in any reinforced concrete book, however method to deal with this type of loading condition is present in Design of Concrete Structures by MacGregor.

So key concepts used for design of this sheet is,

1. Flat Plate action analogy
2. Combined Transfer action of moment of shear.
3. Polar moment of friction.

3.2.1.1 Inputs

In input enter the following data:

1. Colum Size
2. Footing dimensions
3. Loads and Moments
4. Concrete and reinforcement properties
5. Loading and Material over footing
6. Footing thickness
7. Allowable bearing pressure of soils
8. Concrete Cover
9. Specify Bar number

There are also checks and warnings given on input page.

3.2.1.2 How to Use

All the inputs are predefined however some parameters are added on trial parameters, like:

1. Footing Dimension
2. Footing thickness

These parameters can be changed until all of the checks are green and satisfied.

3.2.1.3 Working of Automated Sheets

Following steps are taken by sheets:

1. Factored Loads
2. Factored Net Soil Pressure
3. Eccentricity Check:
    a. Check for one-way eccentricity
    b. Check for two way eccentricity
    c. At the end check for $q_{equi} \leq N_a$
4. Check thickness of two way shear
5. Check bar combined action of moment and shear
6. Check thickness for one way shear
7. Flexural Design and Reinforcement calculation for long direction
8. Flexural design and reinforcement for short direction.
9. Calculation for development length.
10. At the end Design for transfer width and reinforced required "**Didn't get what was written"**
11. Insert formulas for polar moment of inertia, combined transfer of loads.

3.2.1.4 Output

Results of structural design contain the following things:

1. Footing Dimension:
    a. Width
    b. Length
    c. Thickness
2. Reinforcement details:

a. Bars #
b. Area of footing
c. # of bars required
d. Effective depth
e. Strain limit check
f. Required panel length of bars

3. Transfer Width
   a. Width of strip
   b. % of total area in strip
4. Stresses Generated and Strengths:
   a. Two Way Shear
   b. Combined action of moment and shear
   c. One Way Shear
   d. Moment in long dimension
   e. Moment in shorter dimension
5. Checks and Warnings:
   a. Eccentricity Check
   b. Thickness for 2-way shear
   c. Combined Action of moment and shear transfer
   d. Check thickness for one way shear
   e. Moment capacity check along longer direction.
   f. Moment capacity check along shorter direction
6. Eccentricity:
   a. E_b (ft)

b. E_l (ft)

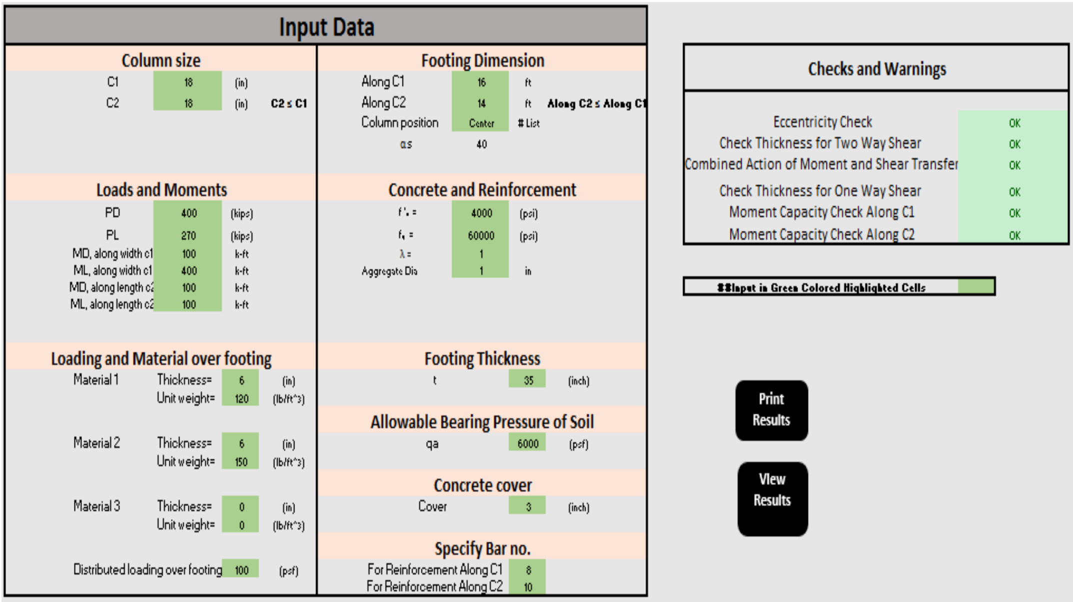


Figure 16: Input for Structural Design

3.2.2 Comparison with SAFE

After formulating excel sheet for footing Design, I decided to compare excel sheet with SAFE software. Its methodology is given below:

- Defined footing width and length same as that on excel
- 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 Formulating Methodology of Conventional Rigid Method

- Select dimension of footing which are L and B.
- Select number of columns in x and y direction.
- Decide center to center spacing and edge distances in x and y direction.
- Set all the columns dimensions on the footing length as well as width.
- Apply loads on all Columns and set allowable pressure of soil.
- Check eccentricity for loading conditions. Excel formulation is done for eccentricity check.
- Whole mat was divided into different strips.
- For all strips using provided data shear and bending moments diagrams are drawn automatically in Excel sheet.
- Using shear forces and given different properties of concrete thickness of mat is established automatically by formulation of excel sheet.
- Reinforcement for max positive and negative moments is designed.
- Checks are applied for safe design.

3.3.2 Comparison with SAFE

By observing Results Obtained from Structural Design of different Spread footings and Mat Footings of Different sizes, a trend was observed, So we Decided to compare Designs of Mat using Conventional Rigid Method and SAFE finite element method. Methodology Adopted is:

- Different Column Interspacing were considered for comparison.
- For given loads (already mentioned above) mat was designed by both Excel sheet
  and SAFE Finite Element Methods.
- By using both methods results were compared.

3.4 Design of Drilled Shaft using Oasys Pile, Alp and AdSec

First step is to define input data. The data was taken from "Basics of Foundation Design" (Benget. H. Fellenius, example 7.3) to design a single drilled shaft being used to support load from an electric pile.

Layer Name	Depth (m)	Unit Weight $\bar{\gamma}$ (kN/m^3)	β	N_q	Friction Angle Φ	Cohesion c	E_s (Kpa)	E_{50}
Sandy Silt	4	19.61	0.4	45	29	6	47780	NA
Soft Clay	17	16.67	0.3	5	12	15	9576	0.02
Silty Sand	6	20.6	0.5	55	32	4	95760	NA
Ablation Till	4	21.6	0.55	65	36	1	191521	NA

Figure 17: Design Data for Drilled Shaft

Applied loads were defined as;

1. Vertical Load 210kN
2. Horizontal Load 320kN
3. Applied Moment: 800kN-m

Design Procedure:

1. Use "Pile" to select the required length of pile to generate desired vertical load carrying capacity and also helps to calculate settlement induced due to the vertical load.
2. The second step is to use "Alp". It actually design for lateral loads and gives us pile head deflections. In this software, certain ratio of reinforcement is to be given and thus it will shape stiffness of pile and can control deflection.
3. After "Alp" we move towards "AdSec". This software actually design capacity of section and determine that whether related section is able to resist that loading combination or not.

If our design is not, then we will redesign reinforcement in section in “Alp” and then move toward “AdSec”, and if then the required reinforcement is exceeding 4%, then we to make a new selection.

4. We initially selected 600mm, 700mm and 800mm diameter piles, and as mentioned above in table. All vertical capacities calculated from “Pile” software are more than the desired values and also settlement is in tolerable limits.
5. By altering design and cyclic use of “Alp” and “AdSec” we observed that percentage of reinforcement in need to give to reaction to withstand applies loading is exceeding 4%, as reinforcement exceeding 4% is not desirable and we need to put structural steel shape in pile for design to economize the section and .
6. In selection section were of 700m and 800mm, and their pile head deflection calculates using “Alp” were also within tolerable limits.
7. After economic comparison, 800 diameter pile was selected.

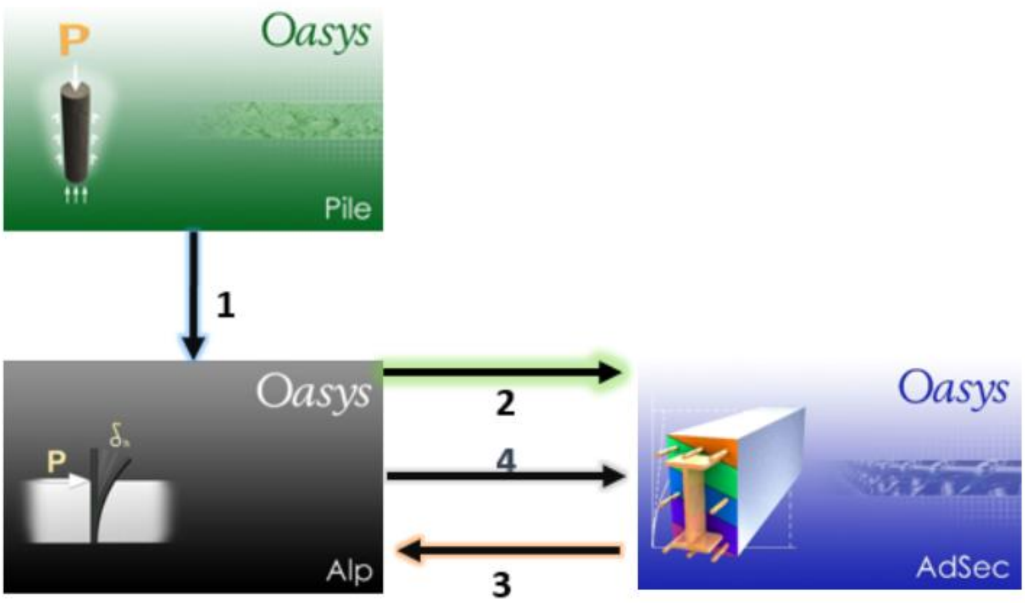


Figure 18: Process of Designing Pile using Softwares

CHAPTER 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:

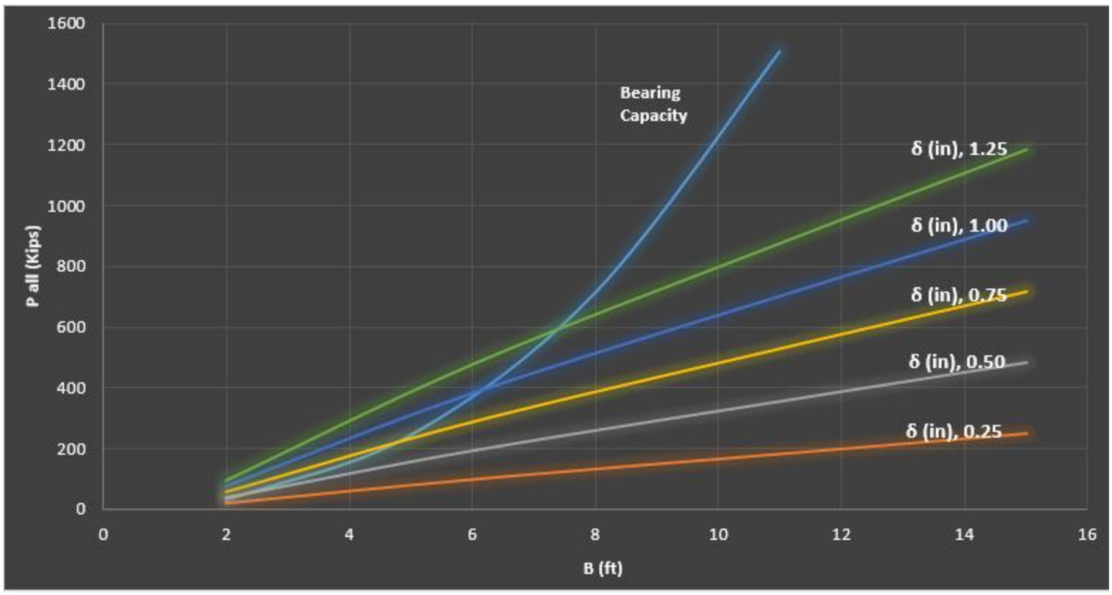


Figure 19: Design Chart Output

From Chart it can be seen that as we increase footing width "B" settlement increases but from chart it is observed that to decrease settlement footing size need to be increased, this is due to the fact that the higher the load higher the bearing Pressure, so higher will be the settlement.

In short, to make a balance between Bearing Pressure and Influence zone made due to increase in size of footing settlement curve makes a particular shape. Also this argument is supported in a manner that, settlement curve for 0.25 inch is having slope significantly lower than that of the settlement curve of 1.25 inch.

Concluding the whole Discussion in a nutshell, for higher loads small change in width "B" lead to greater effect reflected in settlement (See chart above in support of argument).

4.2 Spread Footing Structural Design

4.2.1 Automated Excel Sheet

You have to input the required parameters in Automated excel sheet and in return it will give you all the required parameters for Structural Design of Footing, including Dimensions and Reinforcement Requirements. This sheet also provides checks and warning to ensure the safety of the Design. These Checks and Warnings include:

- Eccentricity Check
- Check thickness for one way shear
- Check thickness for two way shear
- Check for Combined Transfer action of moment and shear
- Moment Capacity Checks

Results of Structural Design

Footing Dimensions		
Footing Width	14	ft
Footing Length	16	ft
Footing Thickness	35	in

Transfer Width	
Width of Strip	168
%age of Total rft	93.33

Reinforcement Details	Along Length	Along Width
Bar no #	8	10
Area of Reinforcement (in^2)	14.22	12.7
No of Bars Required	18	10
Effective Depth (d)	31.5	30.37
0.005 ≤ εt	0.051	0.063
Development Length of Bars (in)	47.4341649	60.24

Stresses Generated and Strengths	Stresses	Strength
Two Way Shear (Kips)	844.1145833	1152.839944
Combined Action of Moment and Shear	185	190
One Way Shear (Kips)	384.125	494
Moment in Longer Dimension (k-ft)	1825	1968
Moment in Shorter Dimension (k-ft)	1216.17	1702.00

Checks and Warnings	
Eccentricity Check	OK
Check Thickness for Two Way Shear	OK
Combined Action of Moment and Shear Transfer	OK
Check Thickness for One Way Shear	OK
Moment Capacity Check Along C1	OK
Moment Capacity Check Along C2	OK

Eccentricity	
eb (ft)	0.7152
eL (ft)	0.26349

Return to Input

Figure 20: Structural Design Results

4.2.2 Comparison of Automated Excel Sheet with SAFE

In order to check the accuracy of our designed sheet I chose SAFE for comparison purposes and for the same loading conditions as in excel.

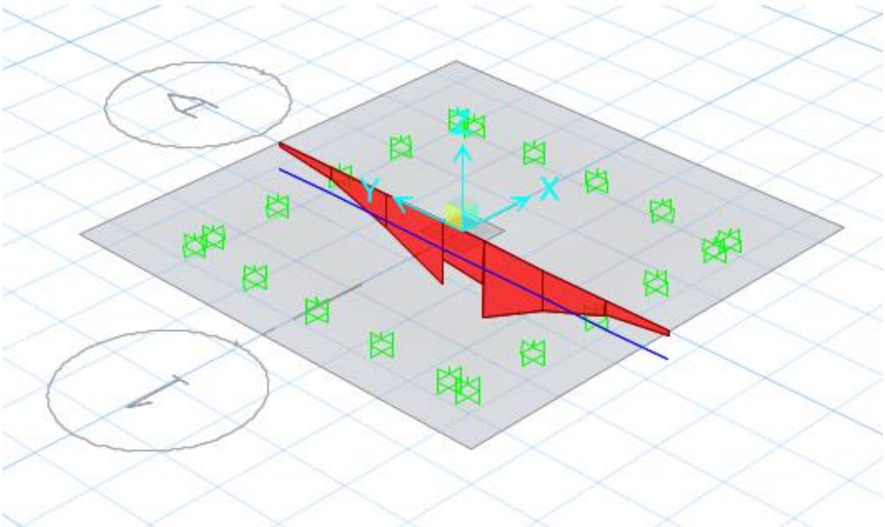


Figure 21: Spread Footing Designed

Comparison of Results is shown below:

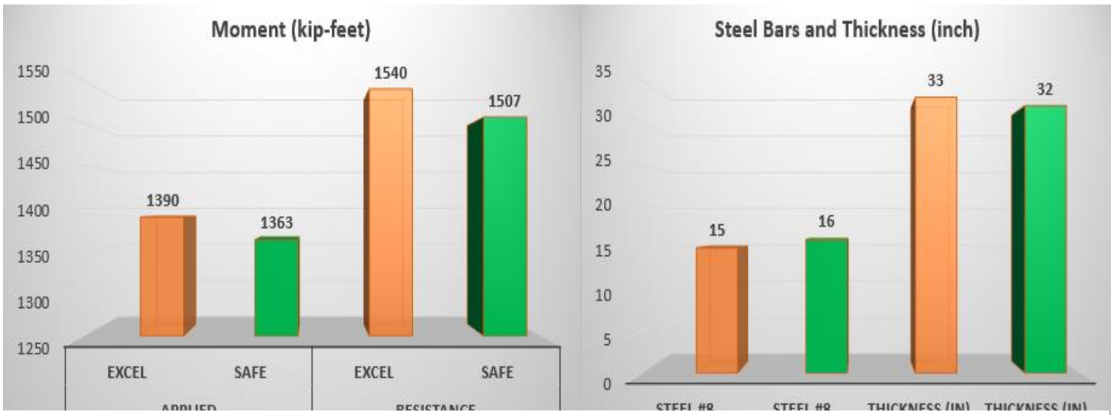


Figure 22: Comparison of Results of Excel and SAFE for Spread Footing

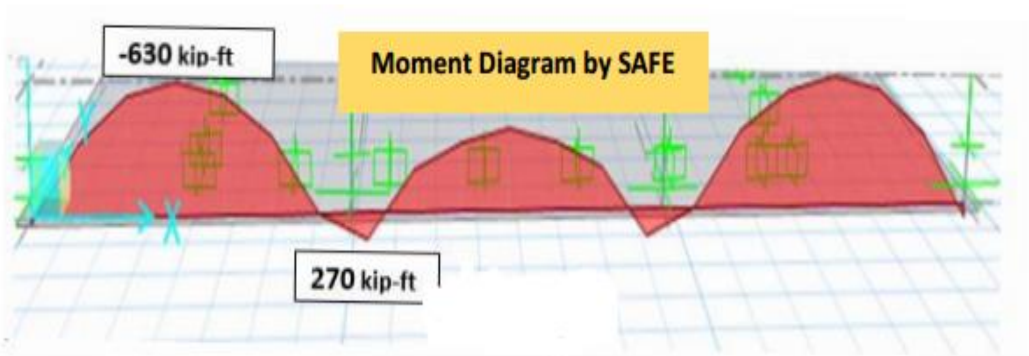
By comparing results it can be seen that difference is less than 2%. This difference is due to the fact of considering the deformations by SAFE.

4.3 Mat Footing Design

<u>Comparison of Conventional Rigid Method and SAFE Finite Element Method</u>

Mat was divided into Different Strips and one strip is taken to make a comparison between values obtained from Conventional Rigid Method and SAFE.

Results from 20 ft. column interspacing is shown below:



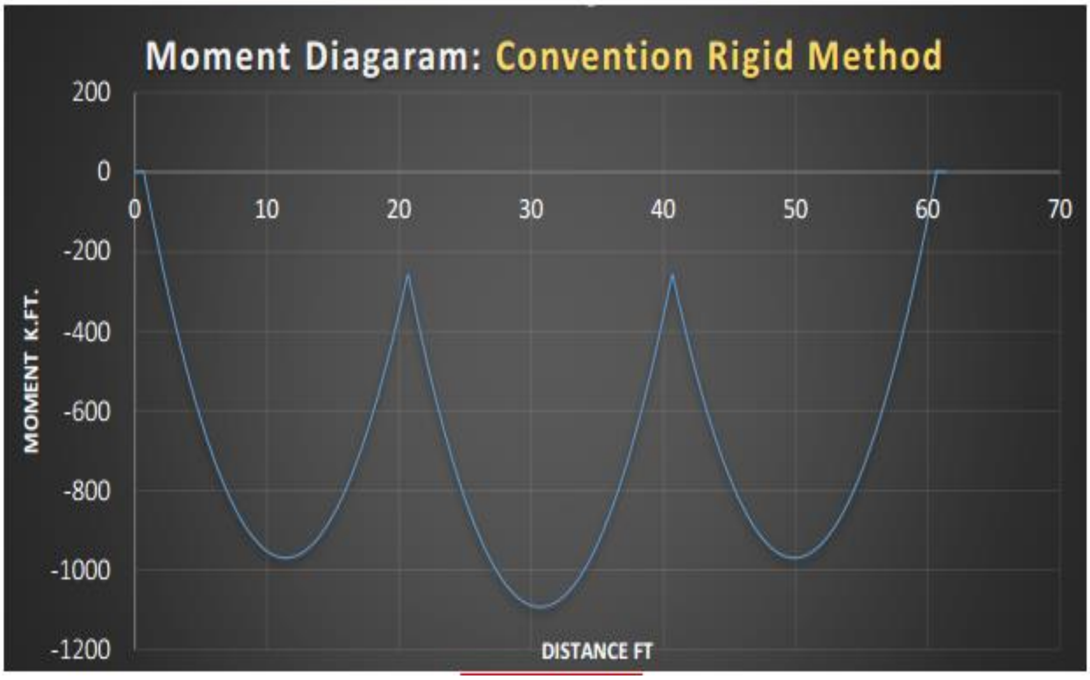
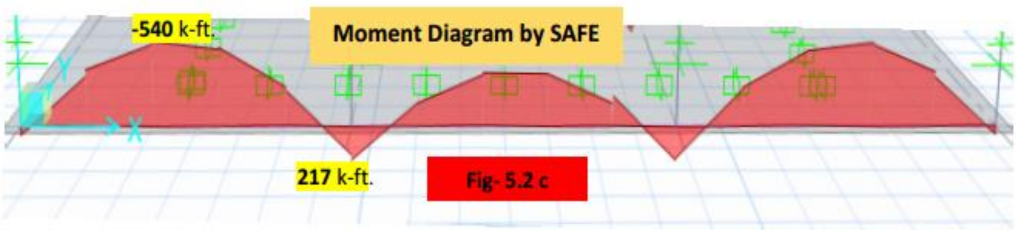


Figure 23: Comparison of Results of SAFE and Excel for MAT 20ft interspacing

Results for 15ft column Interspacing is shown below:



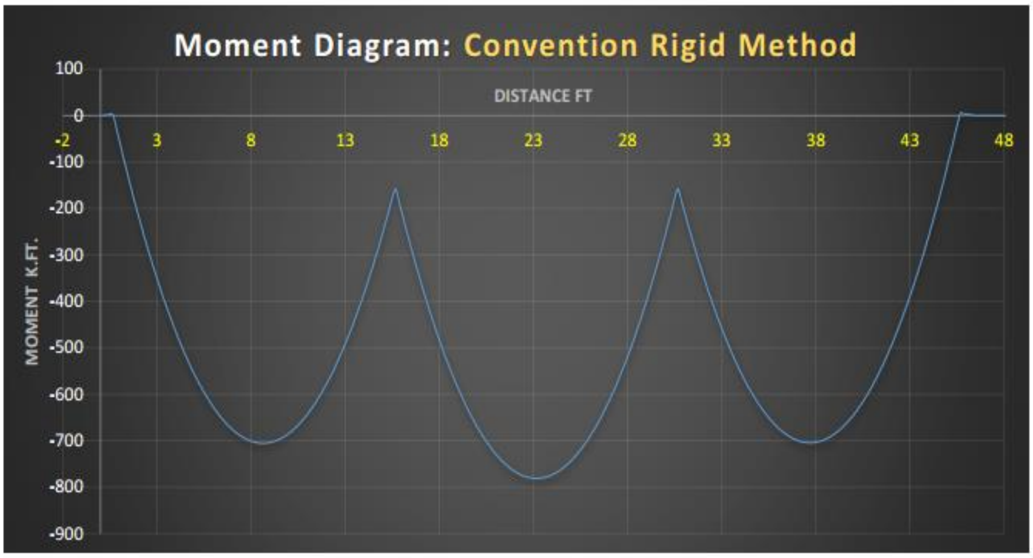


Figure 24: Comparison of results of SAFE and Excel for MAT 15ft interspacing

Discussion:

SAFE Finite Element Method is more accurate than conventional Rigid Method, because:

• Finite Element Method is based is based on a principal in which mat elements are considered to be connected to the ground through a series of "springs" which are defined using the coefficient of subgrade reaction.

• Typically one spring is located at each corner of each element.

• Can account for deflections in Mat and hence changes in soil reactions and redistribution of stresses.

You can see from graph "Comparison of thickness with increase in area of Footing", that as the footing area is increasing, thickness requirement by SAFE is also increasing and when area became much larger i.e. for mat, then there is a considerable difference in thickness requirement by SAFE i.e. 10 inch greater than Conventional Rigid Method, because the factor of deflection increases when area increase, thus for larger areas Finite Element Method is preferable.

4.4 Design of Pile

Pile is designed for given loading conditions using three Softwares in conjunction including Pile, Alp and AdSec.

Structural Tolerance Limits:

Settlement = 25mm

Pile Head Deflection = 20 mm

Result are Shown Below:

Applied Loads		
Vertical Load= 210 KN	Horizontal Load=320 KN	Applied Moment= 800 KN-m

Design Results								
Section	Diameter (mm)	Length (m)	Axial Load Capacity (KN)	Settlement (mm) ≤ 25	Deflection at head (mm) ≤ 20	Rebar	Rebar %	$\frac{Mapp}{Mres}$
1	600	13	254	17.6	19.4	N.A.	N.A.	N.A.
2	700	11	245	21.8	19.66	24#8	3%	0.799
3	800	11	292	19.17	19.60	17#8	1.66%	0.8

Figure 25: Pile Design Applied Loads and Results

Discussion:

As you can see, three sections are defined initially in Pile Software that calculate for the developed axial load capacity and settlement and hence determines the required length of pile. Then results from Pile are used in Alp to determine the deflection at pile head to check whether it is in tolerable limits or not. Then results are used in AdSec to determine that, whether the section reinforcement or section as whole is sufficient to resist the applied loading combination or not. If section is not able to withstand the applied loading then the loading point will lie outside the interaction diagram and then we need to redefine the section whether by increasing reinforcement or section diameter. It our case of 600 mm diameter section required reinforcement is increasing 4% that is not desired due to the additional cost of workmanship and also steel cost is much higher than concrete. So we dropped this section and selected 700 and 800 mm diameter sections for further calculations.

Let us see an example of 800mm section, whose reinforcement %age is increased from 1.2% to 1.66% in order to get an improved design that can withstand the applied loading conditions.

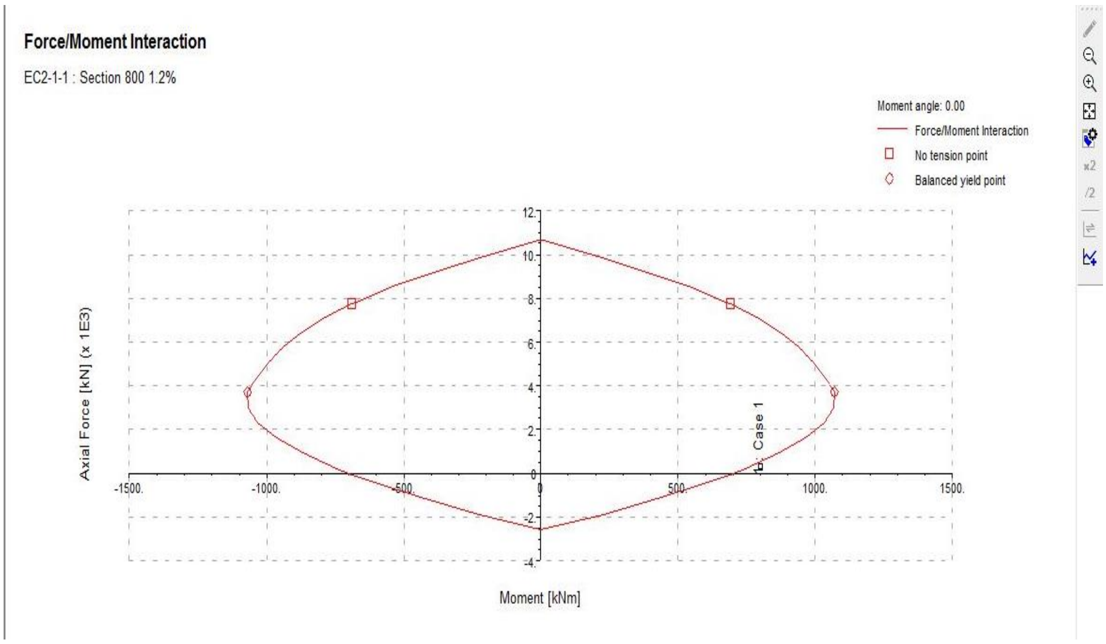


Figure 26: 800 mm Section of Pile with 1.2% Reinforcement

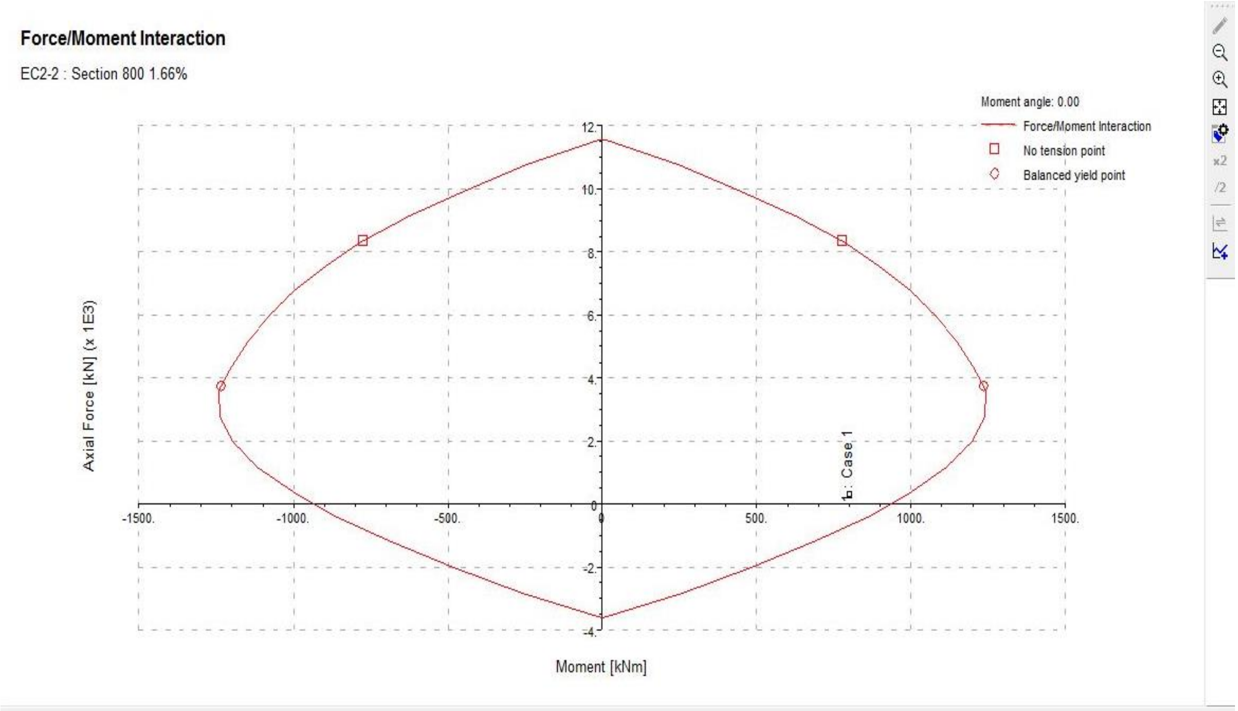


Figure 27: 800 mm Section of Pile with 1.66% reinforcement

At the end of design cost comparison is made between 800 and 700 mm diameter piles.

Section Dia.	Total Steel Cost	Total Concrete cost	Total Cost Rs
700mm	84000	27734	111734
800mm	60000	36223	**96223**

Figure 28: Cost Comparison of Designed Pile Sections

So from cost comparison it can be seen that 800 mm diameter pile section is a suitable option to use.

4.5 Comparison of Deflection of Laterally Loaded Drilled Shaft using PLAXIS 3D Foundation and FIELD Instrumentation

To make a comparison between the deformations predicted using PLAXIS 3D foundation and actual field measurements data was taken from a thesis of MS in Civil Engineering, presented by RICHARD S. WILLIAMMEE, JR at University of Texas at Arlington.

Data is given below:

Layer Name	Depth (m)	w	γ_d kN/m³	γ_{sat} kN/m³	γ_{unsat} kN/m³	Cohesion S_u	Φ	E kPa	V
Silty Sand	0.3	19%	14.85	18.8	17.67	0	21.6	4900	0.15
CH	0.6	21.2%	15.79	20.9	19.15	41.17	7.5	4070	0.15
CL1	0.6	16.9%	17.07	21.57	19.96	37.04	14.5	11108	0.2
CL2	1.5	15.7%	19.09	22.75	22.09	25.86	7.5	7757	0.2
CL3	3-onward	12.9%	17.8	20.82	20.07	21.1	28.6	6320	0.2

Figure 29: Data for Comparison of Lateral Deflection of Pile

Force were applied to short drilled shafts at an angle of about 16.1°.

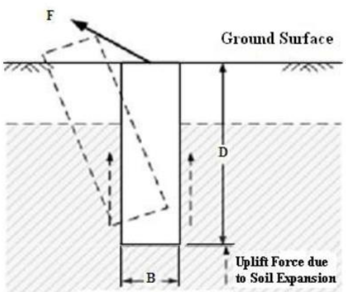


Figure 30: Direction of Application of Force on Pile

Shaft size Diameter x Depth	Load at 16.1^o **(Kips)**
1 x 6	16.57
1 x 10	20.2
2 x 10	45.8

Figure 31: Shaft Sizes and Applied Loadings

Following results were observed:

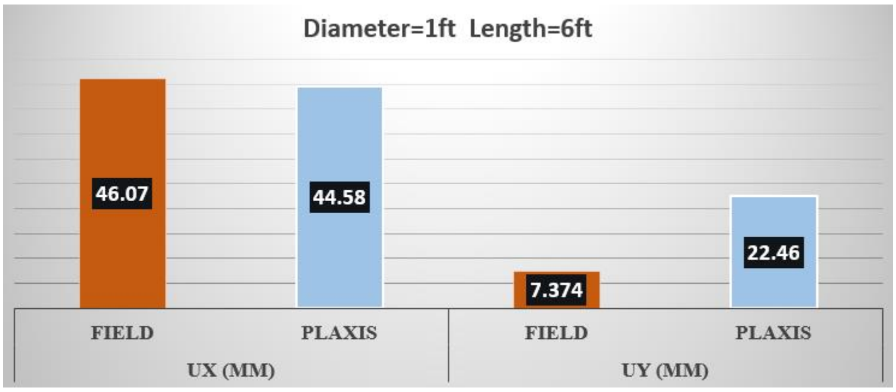


Figure 32: Result for 1ft x 6ft shaft

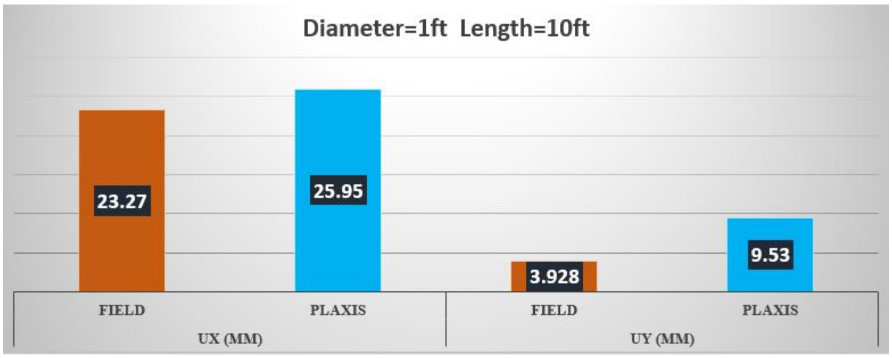


Figure 33: Result for 1ft x 10ft Shaft

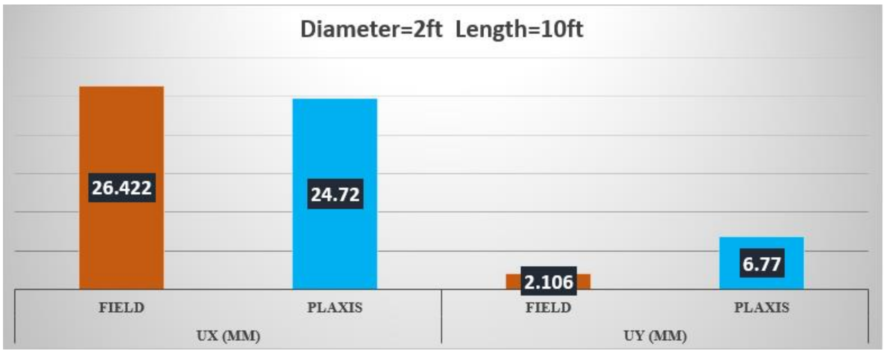


Figure 34: Result for 2ft x 10ft Shaft

Discussion:

From results it can be seen that there is much smaller difference in deflection prediction using PLAXIS 3D Foundation software but there is greater difference between that of predicted using software and actual field prediction.

CHAPTER 5

CONCLUSIONS

5.1 Effect of Friction Angle

- Effect of Friction Angle is more pronounced in Bearing Capacity Calculations and thus a major factor in defining load bearing capacity of soil. When friction angle value exceeds 20^{o} then value of Bearing Capacity exceeds exponentially.

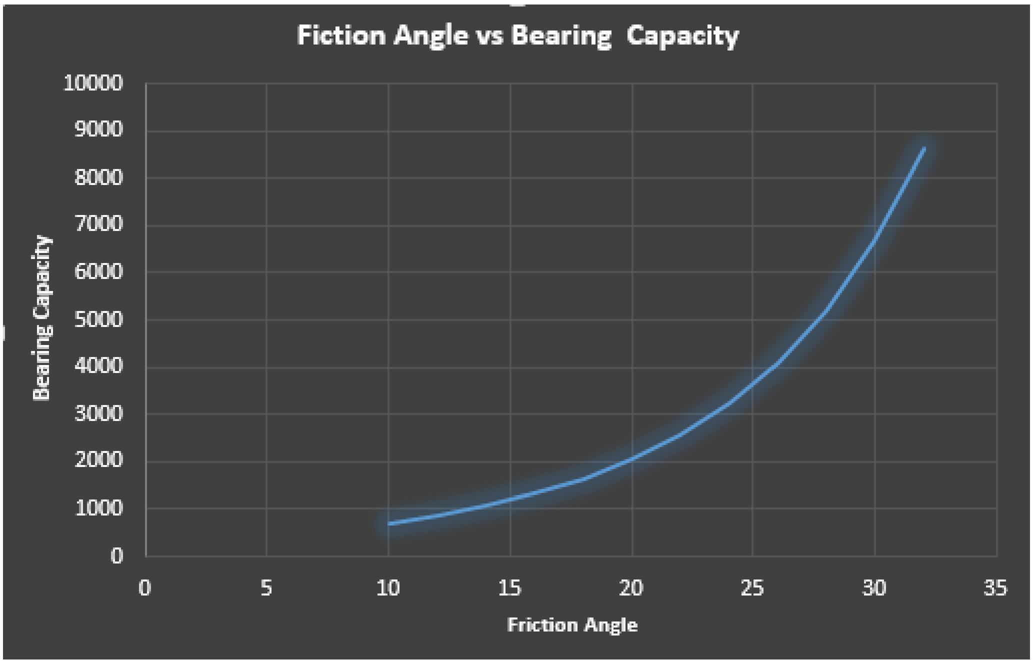


Figure 35: Effect of Friction Angle on Bearing Capacity

5.2 Limitation of Terzaghi 1-D Consolidation Method

- For settlement analysis of soil, using Terzaghi's 1D consolidation method must know your soil conditions in detail i.e. whether excess pore water pressure generated due to applied loading will be dissipated in the pattern of 1-D drainage or will be in form of 3D drainage, because for 3D drainage case, Terzaghi's 1D consolidation will not be a better option and the expected consolidation time will be much lesser than predicted by Terzaghi 1D consolidation method.

5.3 Soil Model Selection

- Behavior of Soil is very complex to model. For preliminary analysis we can use Mohr Coulomb's model but for detailed analysis needed for more important structures perform detailed analysis using soil models like Hardening Soil Model, Hardening Soil Model with small strain or Cam Clay Model.

5.4 Effect of Longitudinal Reinforcement in Pile Head Deflection

- Addition of reinforcement in pile has only a limited effect in restricting pile head deflections, so where deflections are excessive you have to select some pile head fixity options to meet structural tolerance limit.

5.5 Check Applicability of a Method

- Before using a certain Method search for its limitations and conditions for which the formula is derived.

5.6 Comparison of Designed Sheet for Biaxial Bending with SAFE

- By comparing results of Spread Footing designed for Biaxial Moments using Automated Excel sheet with SAFE software, give results having difference of less than 2%.

5.7 Use Softwares for Structural Design

- For Design of MAT foundation or Structural Design of other footings it is more accurate to use Softwares that consider deflections of mat due to applied loadings and hence the stress reversals such as SAFE, to get accurate Design.

5.8 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.

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14. RS. Pile Manual for Laterally Loaded Pile.

15. RS. Pile Manual for Axially Loaded Pile

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17. Plaxis 3D Foundation, Scientific Manual, Version 1.5

18. Oasys AdSec Manual

APPENDIX

PLAXIS 3D CALCULATIONS

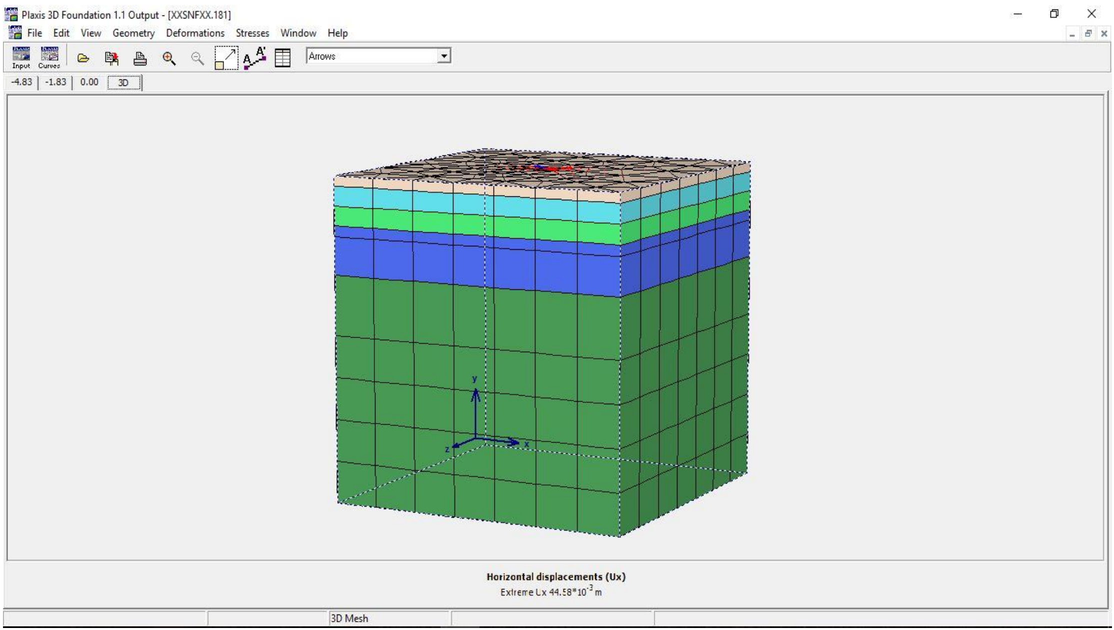

Figure 36: Drilled Shaft 1ft x 6ft

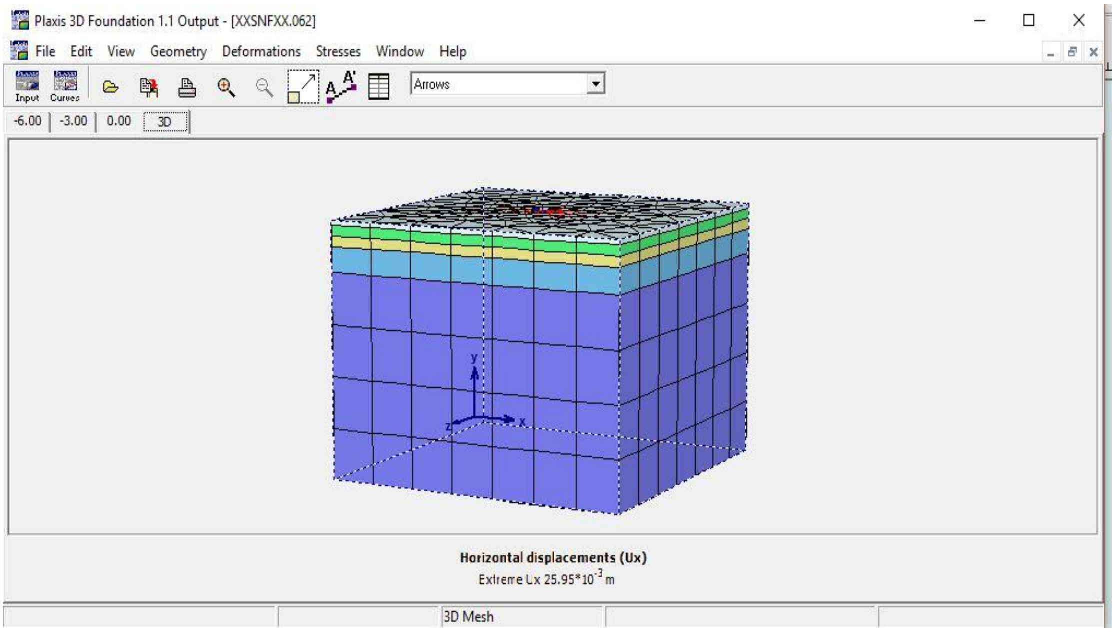

Figure 37: Drilled Shaft 1ft x 10ft

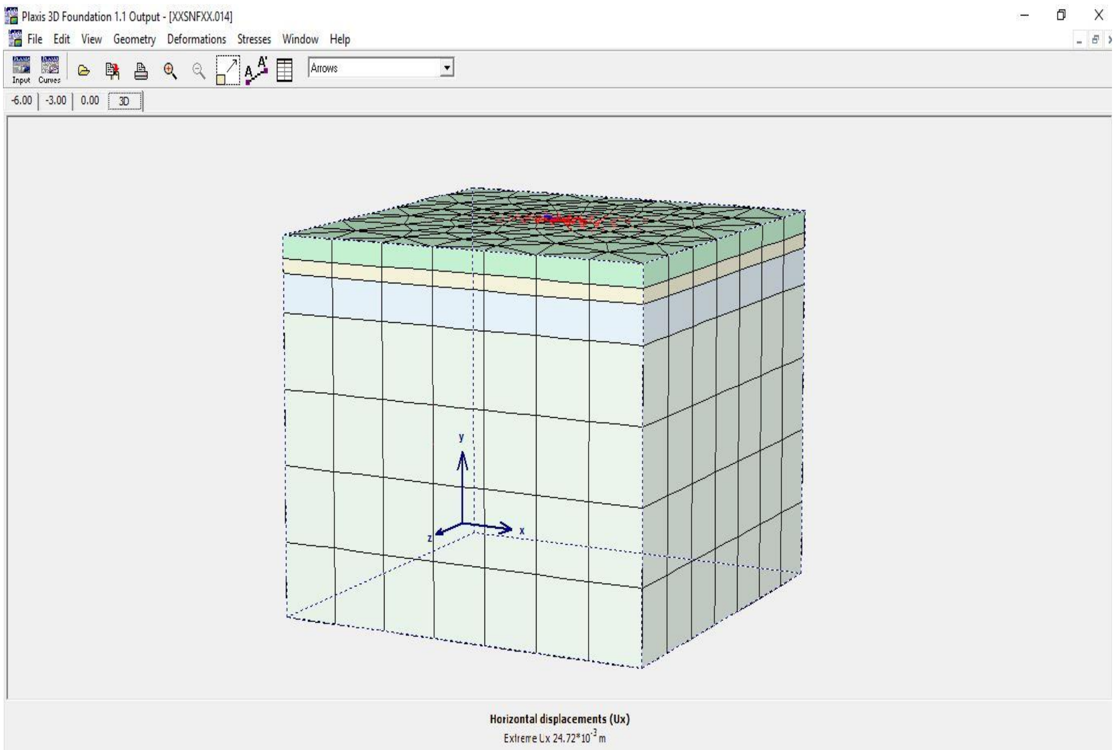

Figure 38: Drilled Shaft 2ft x 10ft

Pile, Alp, AdSec

Design

700 mm Diameter Section

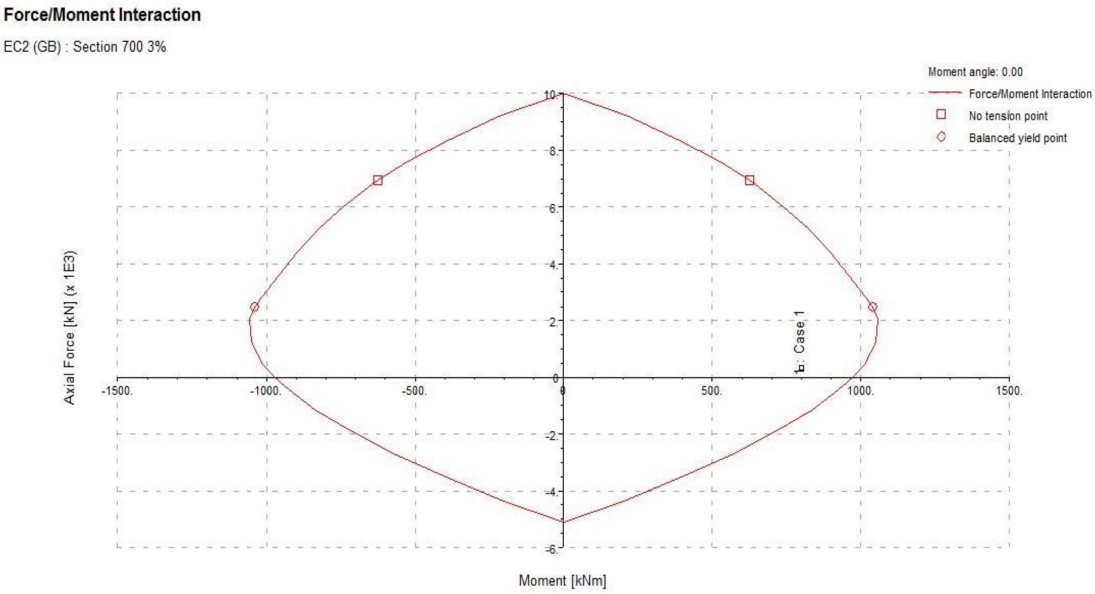

Figure 39: Load Moment Interaction Diagram for 700 mm Section with 3% Steel

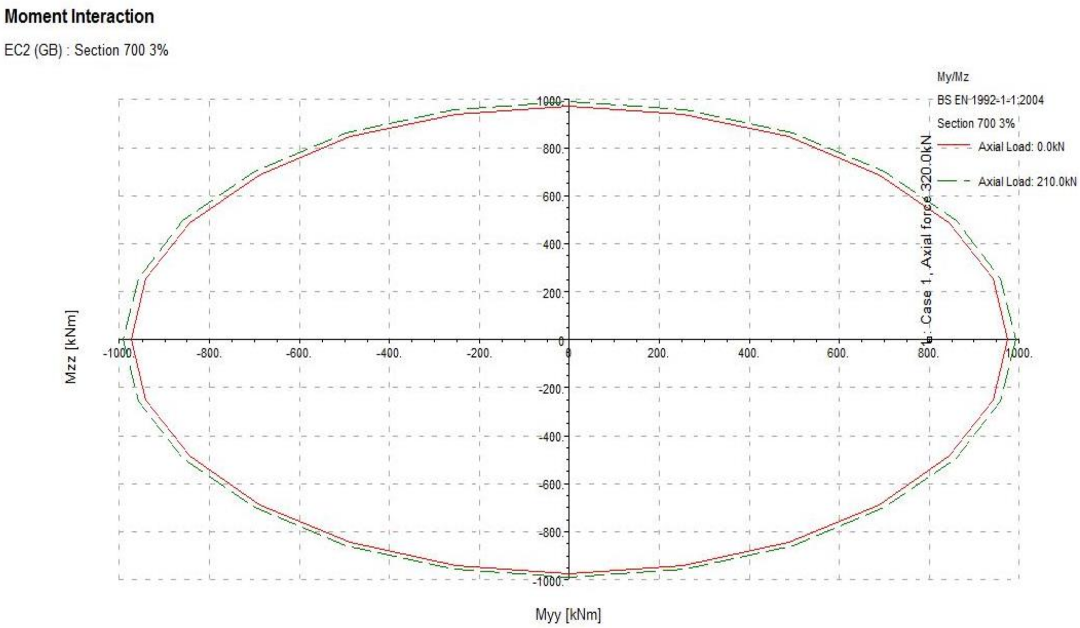

Figure 40: Moment Interaction Diagram, with and Without Axial Load

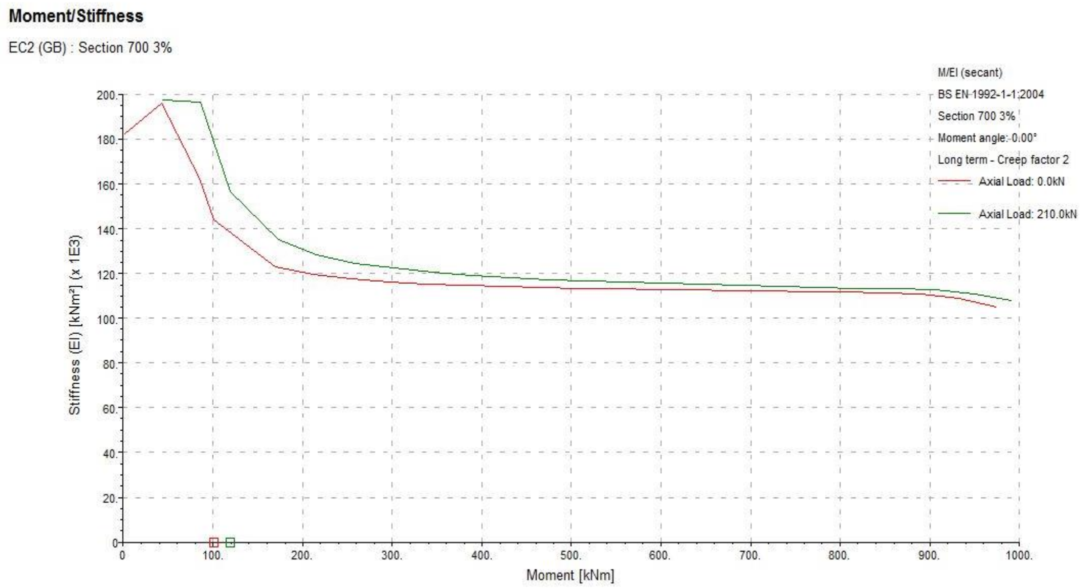

Figure 41: Serviceability Analysis for 700 mm diameter Section

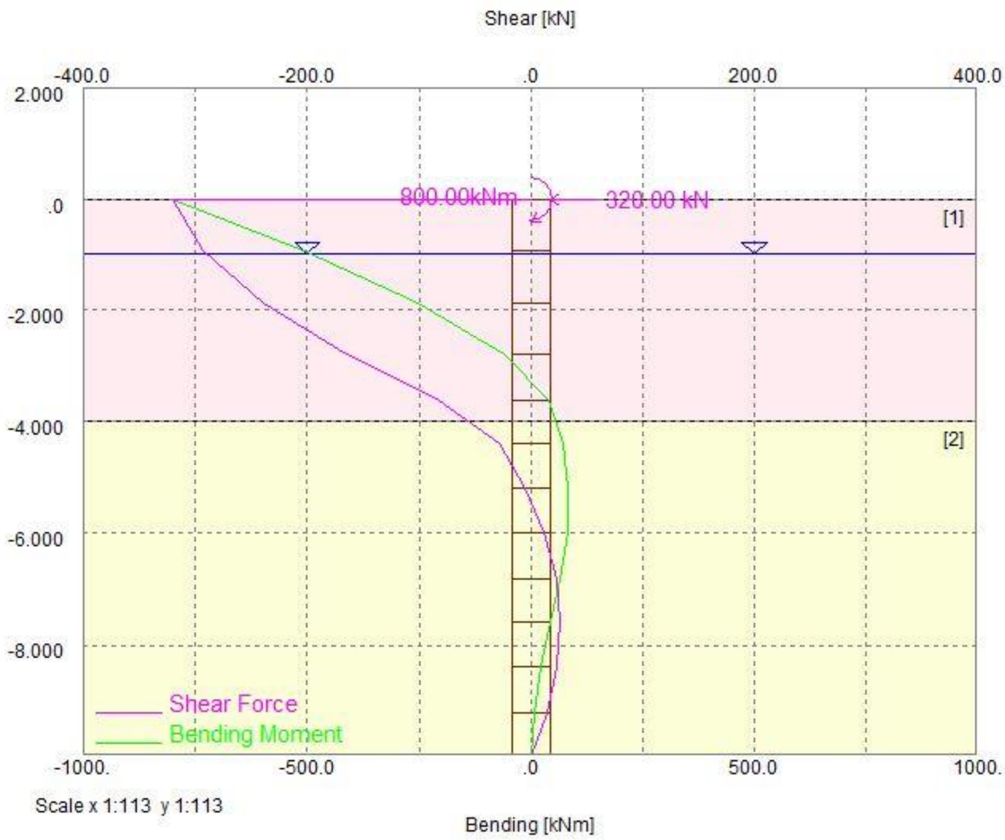

Figure 42: Lateral Load Analysis for 700mm diameter Section

800 mm Diameter Section

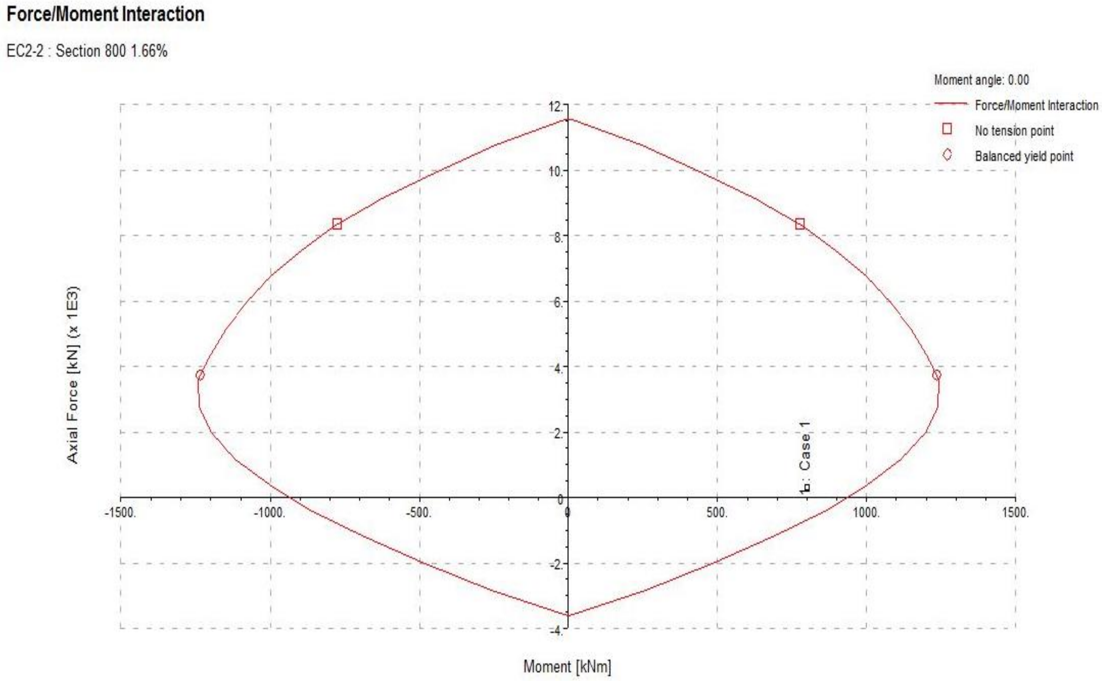

Figure 43: Load Moment Interaction Diagram for 800 mm section with 1.66% Steel

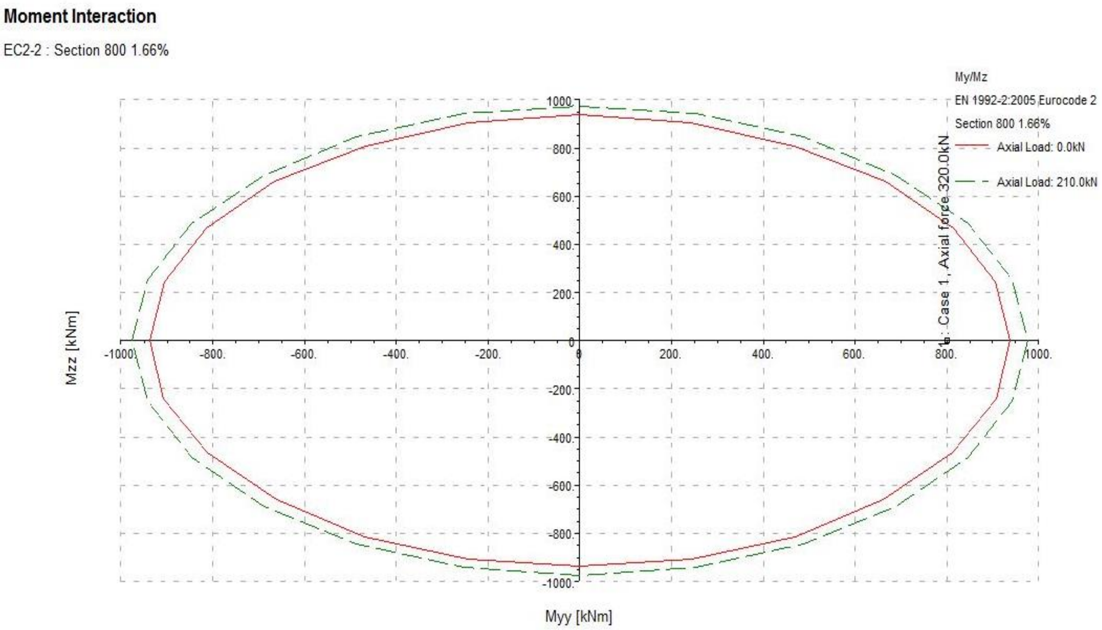

Figure 44: Moment Interaction Diagram for 800 mm section with 1.66% steel

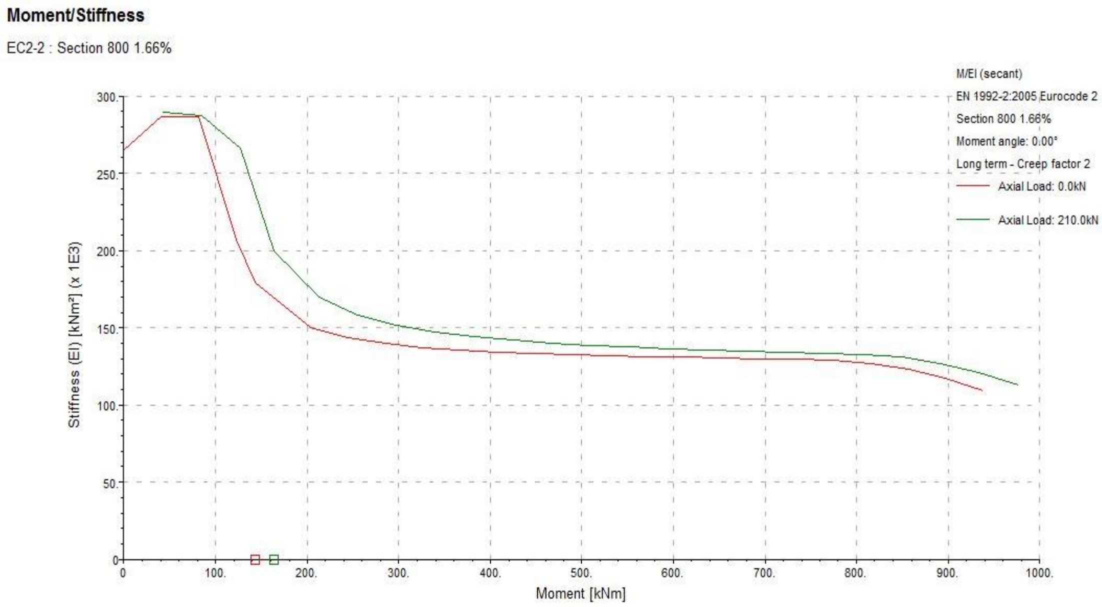

Figure 45: Serviceability Analysis for 800 mm section

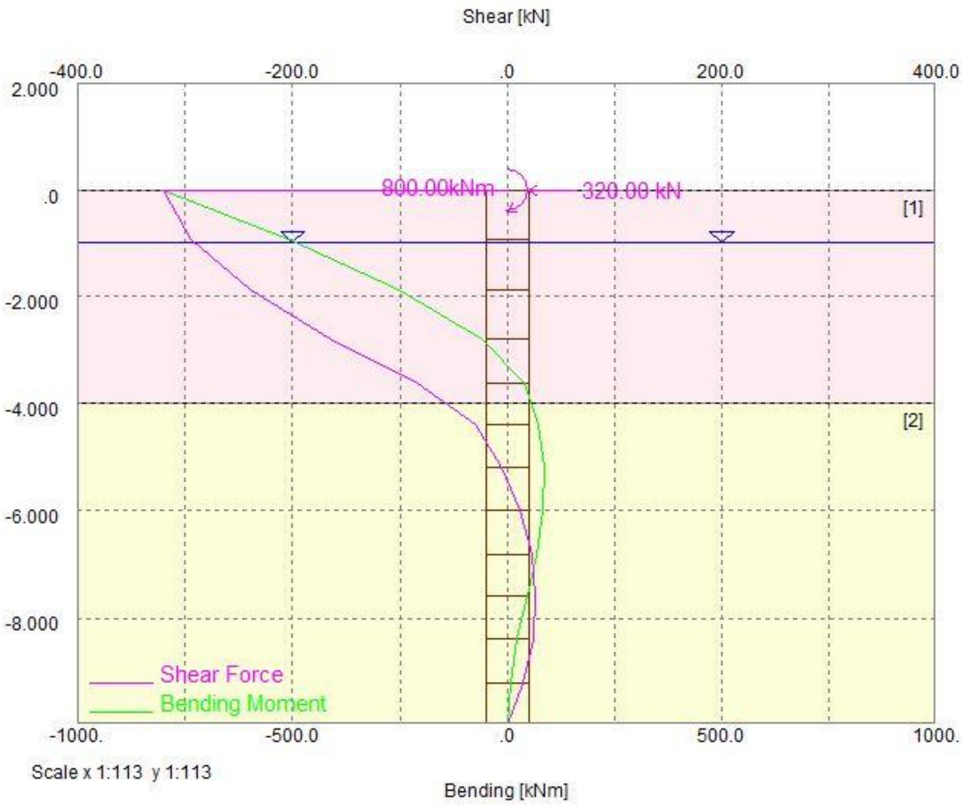

Figure 46: Lateral load Analysis for 800mm Section