CHAPTER 5
ANALYSIS AND DESIGN OF TOWER FOUNDATIONS

5.1 INTRODUCTION

The tower foundations cost 10 to 30% of the overall cost of the tower depending on the type of foundation, loading on the tower and the type of soil. The type of foundation adopted varies from location to location even for the same type of transmission tower. Experiences show that collapse of towers is often initiated by foundation failures. Moreover the towers will also have to be checked for permissible deflection at the top. The differential foundation settlement may lead to a large deflection at the top. Therefore if the total deflection at the top of the tower is to be restricted, the permissible deflection has to be carefully apportioned between the tower structure deflection and that caused by the differential foundation settlement. To design a safe and economical foundation, knowledge of soil behaviour, structural analysis of the foundation and a reasonable estimate of load are necessary. This chapter describes the design principles of transmission tower foundations based on the soil properties, soil-structure interaction studies and the settlement analysis of tower foundation.

5.2 LOADS, SAFETY FACTORS AND DEFLECTION LIMITATIONS

5.2.1 Loads: The load on the tower depends on the type of tower. The load on the foundation is arrived at, based on the structural analysis of the tower. Normally the foundation is designed to resist the following types of forces.

- uplift
- down thrust
- lateral load
- overturning moment
For a four legged lattice tower the foundation is to be designed for the forces uplift, down thrust and lateral load. However, the critical design force is generally the uplift. For two legged H framed towers, in addition to the uplift, down thrust and lateral load, overturning moment becomes a predominant criterion.

5.2.2 Factor of safety and deflection limitation: The foundation load should satisfy the Indian Electricity Rules 1956 (1980) which specifies the factor of safety of 2.0 for normal loading condition (NC) and 1.5 under broken wire condition (BC). These factors of safety apply only for the design of foundation structural members and for checking the stability of foundation. For the instantaneous settlement of tower, a factor of safety of 2.0 for normal loading condition is reasonable. For the consolidation settlement one has to take into account normal serviceability condition. A load factor of 1.0 may be assumed for this while calculating the load for finding the differential settlement of the foundation. This concept is in accordance with the load factor of 1.0 used in the limit state design of structures for serviceability condition (IS 456-1978). The towers need not be checked for deflection limitations under the broken-wire condition. While calculating settlements, only the down thrust is accounted for. Other types of loads are assumed either to cause insignificant settlement or to cause settlement which can be absorbed by the flexibility of the lower structure.

In (IS 802(Part I)-1977), there is no specific recommendation regarding the permissible tower deflection. However, in accordance with the practice followed in countries like the USSR, it is worthwhile to limit the tower deflection to \( \frac{H}{140} \) subject to the stability and performance of the tower. For transmission line structures, the lateral foundation movements which are caused by wind action, line deviation or broken wire condition, will not cause significant changes in the forces of tower members. Even if there is a differential
The total rotation permitted in towers is $\frac{1}{140}$.

Total rotation $\theta = \text{rotation due to tower deflection} + \text{rotation due to foundation settlement} = (\theta_t + \theta_s)$

Therefore the differential settlement should be limited to $\theta_s \leq B$, where $B$ is the base width of tower as shown in Figure 5.1.

It is preferable to limit the lateral deflection of foundation shaft to 12 mm to ensure a good soil-structure interaction. In addition, this deflection will have to be accounted for while designing the shaft reinforcement.

5.3 CLASSIFICATION AND PROPERTIES OF SOIL BASED ON TESTS

The design of tower foundation depends on the nature of loading and type of soil that supports the foundation. The soil is broadly classified as

- Sandy soil (loose, medium and dense)
- Clayey soil (soft, medium and stiff)
- Clay-silt-sand mixtures
- Rock (soft, medium and hard)

There are a number of methods for soil exploration like the in-situ test i.e. Standard Penetration Test (SPT) or Static Cone Penetration Test (SCPT) and laboratory tests such as
FIG. 5.1. PERMISSIBLE TOWER DEFLECTION

deflection due to structural
LOAD FOR FIXED BASE
DEFLECTION DUE TO SETTLEMENT OF FOUNDATION TOWER DEFLECTION
TOTAL PERMISSIBLE DEFLECTION

\[ \Delta = \frac{H}{100} \]

\( x \)
(i) Determination of strength parameters \((c, \phi)\), settlement characteristics such as rate of settlement \((\Delta/t)\), compression index \((a^c)\), etc.

(ii) Determination of elastic properties (modulus of compressibility \((K_s)\), coefficient of lateral subgrade reaction \((\gamma \text{ or } K)\), etc.

Among the field tests, SCPT and SPT are quite useful. SCPT gives point resistance \((q_u)\) and side friction \((f_s)\) and is quick and reliable. Whereas SPT, though reliable is costly and time consuming. The SPT value \((N)\) obtained from the field is corrected for over burden pressure in accordance with Chart (81) reproduced in Figure 5.2. The SPT \(N\) values and the SCPT \(q_u\) values are related as shown in Table 5.1. Table 5.2 gives the density \((\rho)\), relative density \((D_r)\) and the angle of internal friction \((\phi)\) for sandy soils for different \(N\) values. Table 5.3 shows the relationship between \(N\) value and the unconfined compressive strength \(C_u\) for clay soil. The elastic properties of soil can be estimated by referring to Table 5.4 to 5.6. Table 5.4 gives the modulus of compressibility \((E_s)\) and Poisson's ratio \((\mu)\) for the soil. Table 5.5 and Table 5.6 give the coefficient of the lateral subgrade modulus \(\gamma\) for sandy soil and \(K\) for clayey soils respectively. Table 5.7 classifies the type of rock based on its crushing strength obtained from laboratory tests on rock samples. Table 5.8 gives the ultimate bond strength of rock anchor interface for different types of rocks.

**TABLE 5.1 CORRELATION BETWEEN STANDARD PENETRATION TEST VALUE \(N\) AND STATIC CONE RESISTANCE \(q_u\) (82)**

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>(q_u)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clays</td>
<td>2.0</td>
</tr>
<tr>
<td>Silts, sandy silts and slightly cohesive</td>
<td>2.0</td>
</tr>
<tr>
<td>silt and mixtures</td>
<td></td>
</tr>
<tr>
<td>Clean sand to medium sands and slightly silty sands</td>
<td>3-4</td>
</tr>
<tr>
<td>Coarse sands and sands with little gravel</td>
<td>5-6</td>
</tr>
<tr>
<td>Sandy gravels and gravel</td>
<td>8-10</td>
</tr>
</tbody>
</table>
FIG. 5.2. CORRECTION FACTOR FOR OVER BURDEN PRESSURE FOR SPT VALUE
### Table 5.2: Relation between $N$, $\varphi$, $D_r$ and $y$ for Sandy Soil (81)

<table>
<thead>
<tr>
<th>Density ($\gamma$) gm/cc</th>
<th>Description</th>
<th>Relative Density</th>
<th>SPT Value</th>
<th>Angle of internal friction $\varphi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1 to 1.6</td>
<td>Very loose</td>
<td>0-15</td>
<td>0-4</td>
<td>Less than 28</td>
</tr>
<tr>
<td>1.45 to 1.85</td>
<td>Loose</td>
<td>15-35</td>
<td>4-10</td>
<td>28-30</td>
</tr>
<tr>
<td>1.5 to 2.1</td>
<td>Medium</td>
<td>35-65</td>
<td>10-30</td>
<td>30-36</td>
</tr>
<tr>
<td>1.75 to 2.25</td>
<td>Dense</td>
<td>65-85</td>
<td>30-50</td>
<td>36-41</td>
</tr>
<tr>
<td>2.1 to 2.4</td>
<td>Very dense</td>
<td>85-100</td>
<td>greater than 50</td>
<td>greater than 41</td>
</tr>
</tbody>
</table>

### Table 5.3: Relation between the Unconfined Compressive Strength ($C_u$), Cohesion ($c$) and N-Values for Clays (81)

<table>
<thead>
<tr>
<th>Consistency</th>
<th>SPT Value</th>
<th>Unconfined Compressive Strength $C_u$ kg/cm²</th>
<th>Cohesion ($c$) kg/cm²</th>
<th>Reduction Factor for Side Friction of Bored Piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft</td>
<td>0-4</td>
<td>0-0.5</td>
<td>0-0.25</td>
<td>0.7</td>
</tr>
<tr>
<td>Medium</td>
<td>4-8</td>
<td>0.5-1.0</td>
<td>0.25-0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Stiff</td>
<td>8-15</td>
<td>1.0-2.0</td>
<td>0.5-1.0</td>
<td>0.4</td>
</tr>
<tr>
<td>Very stiff</td>
<td>15-30</td>
<td>2.0-4.0</td>
<td>1.0-2.0</td>
<td>0.3</td>
</tr>
<tr>
<td>Hard</td>
<td>greater than 30</td>
<td>greater than 4</td>
<td>greater than 2.0</td>
<td>0.3</td>
</tr>
</tbody>
</table>
TABLE 5.4 MODULUS OF COMPRESSION AND POISSON'S RATIO FOR SOILS (83)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Modulus of compressibility ($E_s$) kg/cm²</th>
<th>Poisson's Ratio $\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft clay</td>
<td>3-30</td>
<td></td>
</tr>
<tr>
<td>Soft clay</td>
<td>20-40</td>
<td>0.1-0.5</td>
</tr>
<tr>
<td>Medium clay</td>
<td>45-90</td>
<td></td>
</tr>
<tr>
<td>Hard clay</td>
<td>70-200</td>
<td></td>
</tr>
<tr>
<td>Silt</td>
<td>20-200</td>
<td>0.3-0.35</td>
</tr>
<tr>
<td>Silty sand</td>
<td>50-200</td>
<td></td>
</tr>
<tr>
<td>Loose sand</td>
<td>100-250</td>
<td>0.2-0.4</td>
</tr>
<tr>
<td>Dense sand</td>
<td>500-1000</td>
<td></td>
</tr>
<tr>
<td>Loose gravel</td>
<td>500-1400</td>
<td>Reliable data not</td>
</tr>
<tr>
<td></td>
<td></td>
<td>available</td>
</tr>
<tr>
<td>Dense gravel</td>
<td>800-2000</td>
<td></td>
</tr>
</tbody>
</table>

TABLE 5.5 COEFFICIENT OF LATERAL SUBGRADE MODULUS $n_h$ FOR COHESIONLESS SOILS (SAND) Kg/cm³ (84)

<table>
<thead>
<tr>
<th>Soil Description</th>
<th>Loose sand</th>
<th>Medium sand</th>
<th>Dense sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry or moist</td>
<td>0.5</td>
<td>1.5</td>
<td>4</td>
</tr>
<tr>
<td>Submerged</td>
<td>0.26</td>
<td>1.0</td>
<td>2.4</td>
</tr>
</tbody>
</table>
### TABLE 5.6 COEFFICIENT OF LATERAL SUBGRADE MODULUS $k$ FOR COHESIVE SOILS (CLAY) (85)

<table>
<thead>
<tr>
<th>Description</th>
<th>Stiff clay</th>
<th>Very stiff clay</th>
<th>Hard clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined</td>
<td>1-2</td>
<td>2-4</td>
<td>greater than 4</td>
</tr>
<tr>
<td>compressive</td>
<td>kg/cm$^2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>strength</td>
<td>1.2-2.4</td>
<td>2.4-4.8</td>
<td>greater than 4.8</td>
</tr>
<tr>
<td>$k$ (kg/cm$^3$)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### TABLE 5.7 CLASSIFICATION OF ROCKS BASED ON COMPRESSIVE STRENGTH (81)

<table>
<thead>
<tr>
<th>Rock classification</th>
<th>Compressive strength kg/cm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very hard</td>
<td>greater than 2200</td>
</tr>
<tr>
<td>Hard</td>
<td>550-2200</td>
</tr>
<tr>
<td>Medium</td>
<td>140-550</td>
</tr>
<tr>
<td>Soft</td>
<td>Less than 140</td>
</tr>
</tbody>
</table>

### TABLE 5.8 ULTIMATE BOND STRENGTH OF ROCK ANCHOR INTERFACE Kg/cm$^2$ (86)

<table>
<thead>
<tr>
<th>Rock Classification</th>
<th>Ultimate Bond strength ($\tau_u$) kg/cm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft</td>
<td>0-5</td>
</tr>
<tr>
<td>Medium</td>
<td>5-15</td>
</tr>
<tr>
<td>Hard</td>
<td>15-30</td>
</tr>
<tr>
<td>Very Hard</td>
<td>30-40</td>
</tr>
</tbody>
</table>
5.4 TYPES OF FOUNDATION AND CLASSIFICATION

5.4.1 Types: Depending on the site condition and the forces acting on the tower legs, one of the following types of foundation is normally employed:

- drilled and/or belled shaft
- pad and chimney
- footing with undercut
- auger with reaming
- grillage
- special type

Perhaps the most commonly used one, is the drilled shaft shown in Figure 1.2.1. The response of this shaft to the uplift load depends largely on the friction between the ground and the shaft. Further improvement of this type is the belled shaft (Figure 1.2.2). Spread shallow foundations of various types with a broad base are placed in an excavated trench (pad and stem type) (Figure 1.3.2, 1.3.4). These may use anchors or augers (Figure 1.3.5) to resist uplift loads. When ground conditions require them, underreamed pile (Figure 1.3.7) or deep pile foundations are used. Steel grillage (Figure 1.3.6) are also provided but special precautions should be taken to protect the steel from corrosion. Where rock is encountered, anchors are embedded with cement grouts (Figure 1.3.8). In addition to these, special foundations such as mass concrete block, raft, shell, etc, are sometimes used.

Selection of the foundation needs judgement and experience and a careful study of all the parameters employed.

5.4.2 Classification: Foundations are classified (Figure 5.3) as shallow and deep based on $\frac{D}{B}$ ratio.
FIG. 5.3. CLASSIFICATION OF FOUNDATIONS

- HYBRID FOUNDATION
  - PILE RAFT
  - GRAILAGE
  - ROOTINGS ETC.
  - ANCHORS
  - UNDER-WATER PILES
  - BORED PILES, DRILLED SHAFT ETC.

\[
\frac{DL}{B} = x
\]

\[
x \leq 1
\]
where \( D_f \) - depth of foundation
\( B \) - breadth of foundation

If \( \frac{D_f}{B} < 1 \) then the foundation is considered as shallow and if \( \frac{D_f}{B} > 1 \) it is classified as deep foundation.

Pile foundation is classified as deep foundation. Even though footings may have more depth than breadth in some circumstances, it is treated as a shallow foundation for the bearing capacity analysis. This approximation leads to a conservative estimate of the factor of safety and hence adopted for convenience and ease in calculation.

5.5 FOUNDATION ANALYSIS

The foundation is analysed for

- bearing capacity
- settlement
- uplift resistance
- lateral resistance

5.5.1 Bearing capacity:

5.5.1.1 Shallow Foundation: Bearing capacity analysis is made on the basis of shear strength parameters \( \phi \) and \( c \). For a general \( c-\phi \) soil the ultimate bearing capacity for a shallow foundation \( (q_u) \) is obtained from the following equations.

\[
q_u = C N_c s_d c_d i_c + q (N_q-1) s_q d_q i_q + \frac{1}{3} Br N_r s_r d_r i_r W' \quad (5.1)
\]

(i) In case of general shear failure

\[
q_u = \frac{2}{3} C N_c s_d c_d i_c + q (N_q-1) s_q d_q i_q + \frac{1}{3} Br N_r s_r d_r i_r W' \quad (5.2)
\]
Where \( N_c, N_q, N_r, N_p, N_y \) and \( N_i \) are bearing capacity factors which can be obtained from Figure 5.4 and

- \( s_c, s_q, s_r \) - shape factors
- \( d_c, d_q, d_r \) - depth factors
- \( i_c, i_q, i_r \) - inclination factors.

These shape, depth and inclination factors may be obtained from Table 5.9.

\( q \) - surcharge \((r D_f)\)
\( B \) - width of foundation
\( w' \) - factor for water table

(i) If the water table is permanently at or below a depth of \((D_f + B)\) beneath the ground level surrounding the footing then \( w' = 1 \).

(ii) If the water table is located at a depth \( D_f \) or likely to rise to the base of the footing or above, then the value of \( w' \) shall be taken as 0.5.

(iii) If the water table is likely to be at \( D_f < (D_f + B) \), then the value of \( w' \) be obtained by linear interpolation. The relative density shall be used as a guide to determine the type of failure as given in Table 5.10.

**Table 5.9** FACTORS TO ACCOUNT THE SHAPE, DEPTH AND INCLINATION EFFECT OF SHALLOW FOUNDATION ON BEARING CAPACITY (87)

<table>
<thead>
<tr>
<th>Shape</th>
<th>Shape Factor</th>
<th>Depth Factor</th>
<th>Inclination Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circle</td>
<td>1.3 1.2 0.6</td>
<td>( d_c = 1 + 0.2 \frac{D_f}{B} \sqrt{N_p} ) ( 1_c = 1 - \left(1 - \frac{\alpha}{90}\right)^2 )</td>
<td>( i_c, i_q, i_r )</td>
</tr>
<tr>
<td>Square</td>
<td>1.3 1.2 0.8</td>
<td>( d_q = d_r = 1 ) for ( \beta &lt; 10^\circ ) ( i_r = \left(1 - \frac{\alpha}{90}\right)^2 )</td>
<td></td>
</tr>
<tr>
<td>Rectangle</td>
<td>1+0.2B ( \frac{B}{L} ) 1+0.4B ( \frac{B}{L} ) 1+0.4B ( \frac{B}{L} )</td>
<td>( d_q = d_r = 1 + 0.1 \frac{D_f}{B} \sqrt{N_p} )</td>
<td></td>
</tr>
</tbody>
</table>

For \( \beta > 90^\circ \)
Fig. 5.4. Terzaghi's Bearing Capacity Factor $(N_q)$ for Bored Piles

VALUES OF $N_c$ AND $N_q$

VALUES OF $N_r$

VALUES OF $N_c$ AND $N_q$

Fig. 5.5. Bearing Capacity Factor $(N_q)$ for Bored Piles
Where

- $B$ - breadth or diameter of foundation
- $L$ - Length of Foundation
- $\alpha$ - inclination of shaft to vertical
- $H_{ij}$ - $\tan^2 \left( \frac{45 + \alpha}{2} \right)$

### Table 5.10: Type of Failure Based on Relative Density (87)

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Description based on relative Density $D_r$</th>
<th>Type of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dense $D_r &gt; 70%$</td>
<td>General</td>
</tr>
<tr>
<td>2</td>
<td>Loose $D_r &lt; 20%$</td>
<td>Local</td>
</tr>
<tr>
<td>3</td>
<td>Medium $D_r = 20$ to $70%$</td>
<td>Interpolate between 1 and 2</td>
</tr>
</tbody>
</table>

#### 5.5.1.2 Deep Foundation

The ultimate bearing capacity of the deep foundation (bored and drilled shafts, piles) ($Q_u$) is determined as the sum of

1. the end bearing capacity and
2. the skin friction along the shaft.

#### 5.5.1.2.1 Normal Pile

The ultimate bearing capacity of the pile may be estimated using the following formula.

**In granular soils**

\[
Q_u = A_p \left( \sum_{i=1}^{n} (D_r N_r + P_D N_q) \right) + K P_{D1} \tan \theta A_{s1} \tag{5.3}
\]

where
- $A_p$ - cross sectional area of pile toe in $\text{cm}^2$
- $D_r$ - stem diameter in $\text{cm}$
- $r$ - effective unit weight of soil at pile toe in $\text{kg/cm}^2$
- $P_D$ - effective overburden pressure at pile toe in $\text{kg/cm}^2$
- $N_r$ and $N_q$ - bearing capacity factors
- $n$ - summation for $n$ layers in which pile is installed
- $i=1$
- $K$ - coefficient of earth pressure
\[ P_{Di} \text{ - effective overburden pressure in kg/cm}^2 \text{ for the } \]
\[ \text{i} \text{th layer where } i \text{ varies from 1 to } n. \]
\[ \delta \text{ - angle of wall friction between pile and soil, in } \]
\[ \text{degrees (may be taken equal to } 0 \text{ and } \]
\[ A_{di} \text{ - surface area of pile stem in cm}^2 \text{ in the } i \text{th } \]
\[ \text{layer where } i \text{ varies from 1 to } n. \]

\( N_r \) can be taken from Figure 5.4 and \( N_q \) will depend, apart from
\the nature of soil, on the type of pile and its method of
\construction. The values of \( N_q \) are given in Figure 5.5 for
\bored pile. The earth pressure coefficient \( K \) depends on the
\nature of soil strata, type of pile and its method of construction.
\For bored concrete piles, \( K \) value may vary from 0.5 to 3 from
\loose soils to dense soils.

(i) In cohesive soils

\[ Q_u = A_p \times N_r \times C_p + \alpha \times C \times A_s \] (5.4)

where \( A_p \) - cross sectional area of pile toe in cm\(^2\)
\( N_r \) - bearing capacity factor usually taken as 9
\( C_p \) - average cohesion at pile tip in kg/cm\(^2\)
\( \alpha \) - reduction factor (may be taken from Table 5.3)
\( c \) - average cohesion throughout the length of pile
\text{in kg/cm}\(^2\) and
\( A_s \) - surface area of pile shaft in cm\(^2\)

5.5.1.3.2 Under-reamed pile: The ultimate bearing capacity of
\under-reamed pile is determined taking into account the bearing
\resistance and shaft resistance of the under-ream also, as
\follows

(i) In sandy soils

\[ Q_u = \frac{\pi}{4} (D_u^2 - D_r^2) \left[ \frac{D_u}{2} n r H_r + r \times N_{q_r} \right]_{r=1}^{r=n} + \frac{\pi}{4} D_r^2 (\frac{1}{4} D_r H_r) \]
\[ + r d_r N_{q_r} + \frac{1}{4} \pi D_r K \tan \left( d_1^2 + d_r^2 - d_n^2 \right) \] (5.5)
where \( D_u \) - diameter of under-reamed bulb, in cm
\( D \) - diameter of stem in cm
\( n \) - number of under-reamed bulbs,
\( r \) - average field density of soil in kg/cm\(^3\)
\( d_r \) - depth of the centre of different under-ream bulbs in cm
\( d_f \) - total depth of pile in cm
\( d_1 \) - depth of the centre of the first under-ream bulb in cm
\( d_n \) - depth of the centre of last under-ream bulb in cm

\( K, N_r \) and \( N \) are as explained in equation (5.3).

(1) In Clayey soils

\[
Q_u = A_p N_c C_p + A_n N_c C'_a + A'_s + \alpha C_a A_s \tag{5.6}
\]

where

\[
A_p = \frac{C}{4} (D_u^2 - D^2)
\]

\( C'_a \) - average cohesion of soil around the under reamed bulbs, in kg/cm\(^2\)

\( A'_s \) - surface area of the cylinder circumscribing the under-reams, in cm\(^2\)

Other notations are as explained in Eq. (5.4)

5.5.2 Settlement: The estimation of foundation settlement requires, a knowledge of the stress distribution in the strata under the foundation and their compressibilities. The settlement may be computed as

Total settlement = Immediate settlement + consolidation settlement

(1) Immediate settlement \( S_i = I_p q B \left( \frac{1 - \mu^2}{E_s} \right) \tag{5.7} \)

where

\( I_p \) - influence factor for settlement (Table 5.11)
\( q \) - intensity of pressure kg/cm\(^2\)
\( \mu \) - Poisson's ratio of soil (Table 5.4)
\( E_s \) - modulus of elasticity of soil kg/cm\(^2\) (Table 5.4)
\( B \) - width or diameter of foundation, cm
### TABLE 5.11 INFLUENCE FACTORS FOR SETTLEMENT

<table>
<thead>
<tr>
<th>Shape of Loaded Area</th>
<th>$I_p$</th>
<th>Average value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At centre</td>
<td>At corner</td>
</tr>
<tr>
<td>Square and rectangle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$L/B = 1$</td>
<td>1.12</td>
<td>0.56</td>
</tr>
<tr>
<td>$L/B = 2$</td>
<td>1.52</td>
<td>0.76</td>
</tr>
<tr>
<td>$L/B = 5$</td>
<td>2.10</td>
<td>1.05</td>
</tr>
<tr>
<td>Circle</td>
<td>1.0</td>
<td>0.64</td>
</tr>
</tbody>
</table>

(ii) Consolidation settlement

$$S_c = \frac{C_c}{1+e_0} \cdot H_c \log 10 \quad \frac{P_o + \Delta P}{P_o}$$  \hspace{1cm} (5.8)

where
- $C_c$ = compression index
- $e_0$ = initial void ratio
- $H_c$ = thickness of compressible layer, cm
- $P_o$ = initial effective pressure at mid-height of larger, kg/cm²
- $\Delta P$ = pressure due to external loading (pressure increment kg/cm²)

Ground water movement and the possible danger of settlement should be carefully investigated. If a poor soil location is unavoidably chosen for the tower foundation, detailed soil investigation should be undertaken and a suitable type of foundation which will not cause excessive settlement should be selected.
5.5.3 Uplift Resistance: The resistance of soil in compression is reasonably well understood. However, the resistance to the uplift is uncertain and there are many theories reported for uplift resistance in the literature. These theories are generally based on either a slip surface rising vertically from the edge of the footing or a surface rising at 30° from the vertical forming a frustum (Figure 5.6). For the vertical slip surface, the shearing resistance along the sides of the plane or cylinder is calculated and added to the dead weight of the soil and footing. For the 30° cone theory the dead weight within the frustum is considered to provide resistance against the uplift. Test results showed that neither of these methods provides reliable results. The cone method is usually conservative (at shallow depth) but can be quite the opposite (at larger depth).

In addition to the uplift resistance, a foundation should be designed for its structural ability to carry tensile load. The tensile strain of soil is very small and is time dependent owing to the effects of pore water pressure equilibration and crack propagation. Hence the measurement of tensile strain and tensile strength properties of soil, is generally difficult.

Meyerhof and Adams considered this problem and proposed that footings should be considered as either shallow or deep since deep footings could develop only to some limiting pull-out force (Figure 5.7). Circular and rectangular footings were considered in both cohesive and cohesionless soils and equations were developed. These equations were developed neglecting the larger pull-out zone observed by tests using an approximation defined by line a b' in Figure 5.7. The cohesion developed on this cylinder circular footing is \( \frac{\pi}{4} B C D \). The passive earth pressure friction resistance developed in the sides of cylinder is \( \frac{\pi}{2} B_r D^2 K_u \tan \theta \) and should include a shape factor \( s_r \). The remaining resistance is the weight of the soil and footing materials within the cylinder \( W \), and the ultimate uplift resistance \( (T_u) \) is given by the equation
Fig. 5.6. Uplift of Footing in C - ϕ Soil (IS Code)

Fig. 5.7. Uplift of Footing in C - ϕ Soil (Meyerhof’s)

Fig. 5.8. Uplift of Footing in C - ϕ Soil (Proposed)
For deep footings where \( D > H \), Eq. (9) becomes

\[
T_u = \pi C B H + s_r \frac{\pi}{2} B (2 D-H) H K_u \tan \beta + W \tag{5.10}
\]

The shape factor is approximately

\[
s_r = 1 + \frac{H}{B} \tag{5.10a}
\]

With a maximum value for deep footings of

\[
s_r = 1 + \frac{H}{B} \tag{5.10b}
\]

Values of \( m \), \( s_r \) and \( \frac{H}{B} \) for various \( \beta \) values are given in Table 5.12.

### TABLE 5.12

<table>
<thead>
<tr>
<th>( \beta )</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>48</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limiting</td>
<td>2.5</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>7</td>
<td>9</td>
<td>11</td>
</tr>
<tr>
<td>( m )</td>
<td>0.05</td>
<td>0.1</td>
<td>0.15</td>
<td>0.25</td>
<td>0.35</td>
<td>0.5</td>
<td>0.6</td>
</tr>
<tr>
<td>Maximum</td>
<td>1.12</td>
<td>1.3</td>
<td>1.6</td>
<td>2.25</td>
<td>3.45</td>
<td>5.5</td>
<td>7.6</td>
</tr>
</tbody>
</table>

The passive earth pressure coefficient \( K_u \) is computed from \( K_p \) where \( K_p = \tan^2 (\frac{1}{2} \beta + \beta/2) \) and \( K_u = K_p \tan (\frac{2}{3} \beta) \). This value of earth pressure coefficient is used to account for the actual shape of the rupture surface in terms of the angle of inclination of the passive earth resistance and the resulting effect on the friction resistance.

For rectangular footings at shallow depths the uplift resistance can be computed as

\[
T_u = 2 C D (B+L)+2D^2(2 s_r B+B-L-B) K_u \tan \beta + W \tag{5.11}
\]

and for deep rectangular footings
The skin friction on the pile shaft is calculated as described in Sec. 5.5.1.b.

In IS 4091 (89) the uplift forces are assumed to be resisted by the weight of the footing plus the weight of an inverted frustrum of earth on the footing pad with sides inclined at an angle of up to 30° with the vertical. It does not include the shearing resistance along the failure plane.

However, Prof. John Douglas (90) has suggested that the shearing resistance along the failure plane may be reduced by a factor of safety of 2 for possible weakening of the soil due to construction. Therefore a mathematical equation is developed to account for the shearing resistance offered along the assumed failure plane shown in Figure 5.8 as

\[ Q_s = \int_0^D 2\pi \left[ R_s + (D-x) \tan \alpha \right] dx \cos \alpha (mC + rK_u \tan \beta) \]
\[ = 2\pi (mC + rK_u r \tan \beta) \cos \alpha \left[ \frac{R_s x^2}{2} + D \tan \alpha \frac{x^2}{2} - \frac{3}{3} \tan \alpha \right]_0^D \]

The vertical component of this is equal to

\[ Q_{sv} = 2\pi (mC + rK_u r \tan \beta) \left[ \frac{R_s x^2}{2} + D \tan \alpha \frac{x^2}{2} \right. \]
\[ \left. - \frac{3}{3} \tan \alpha \right]_0^D \]

where
- \( m \) = reduction factor for cohesion ( = 0.7)
- \( k \) = earth pressure coefficient as expressed above
- \( \alpha \) = 20° for sand and 30° for clay as per IS 4091-1979.

The equation derived above is applicable only for shallow footings. In the case of deep footings, the resistance to uplift should be calculated considering only the height \( H \) from the base
of the footing. Table 5.12 may be referred for fixing the height $H$.

This equation can be used for both circular and square footings.

Resistance to pull out may be contributed both by cohesion and friction. However most of the practical foundation construction methods may result in remoulding, leading to substantial reduction in cohesion. In addition to this, relaxation effects may, in the long run, reduce the value of cohesive resistance. The difference between Eq. (5.9) and Eq. (5.14) depends on approximation regarding the shape of failure surface. It may be noted that, in some instances, the pull out resistance given by equation (5.14) is twice that arrived from Eq. (5.9). Till the exact shape of failure surface is documented, based on tests, it is desirable that an estimate of uplift resistance is made making use of Eq. (5.9) and (5.14) and the least value be adopted in the design. The resistance contributed by cohesion may be completely omitted as this is uncertain and a suitable factor of safety be assured while calculating the contribution of frictional part.

Test results have shown more than twice the uplift resistance of the design value on well compacted soils after construction (90). It depends on the amount of compaction of the fill, its initial in-situ density, development of cohesion, etc. Belled shaft and footing with under cut in undisturbed soils constructed without disturbing the surrounding soil are very efficient in uplift resistance. In these cases a factor of safety less than 2 can be used with confidence for the shearing resistance along the assumed failure plane.
In piles, two cases should be considered separately, viz., piles with straight shaft and piles with under-reamed bulb. The ultimate uplift resistance of the pile is equal to the skin friction which can be mobilised along the effective length of the pile shaft plus the weight of the pile. The skin friction can be estimated as in eq. (5.3) for granular soil and as in eq. (5.4) for cohesive soil.

In under-reamed piles, the ultimate pull out resistance is generated by skin friction along the effective length of shaft and also uplift resistance developed by the underream in addition to the dead weight. The skin friction and uplift resistance may be calculated as described in Sec. 5.5.1. Further the ultimate pull out resistance of under-reamed pile may also be computed using Eq. (5.9), the least of the two may be taken as safe value against uplift.

The uplift resistance of a pile group is the lesser of the following two values

(i) the sum of the pull out resistances of the piles in a group

(ii) the sum of the shear resistance mobilised on the surface perimeter of the group plus the total weight of soil and piles enclosed in the perimeter.

5.5.4 Lateral resistance: When an individual pile is subjected to lateral loads, the movement of the pile is resisted by the passive pressures developed between the soil and pile. The amount and distribution of the reaction depends on the relative stiffness of the pile and the soil and on the boundary conditions of the pile.

The horizontal load capacity of vertical piles may be limited in three different ways.
(1) the ultimate capacity of the soil may be exceeded resulting in very large horizontal movement of the pile and the failure of foundation.

(ii) the bending moments may generate large bending stresses in the pile material resulting in a structural failure of piles.

(iii) the deflections of the pile heads may be too large to be compatible with the super structure.

Generally, the soil data are associated with a high degree of uncertainty. The methods for estimating lateral resistance, presently available for design, should be regarded as highly empirical. According to IS 2911 Part IV (91) the allowable horizontal load can be taken as 2 to 5% of the allowable vertical load. Most tower foundations satisfy this criterion.

In tower foundations, case (1) and case (11) are important. Case (iii) does not govern the design because the tower is assumed to be free standing. In case (1), the deflection due to working load which is only half the design load under normal condition, is normally within limiting value 0.75 inch (20 mm) (84). The movement of soil is time dependent and may rebound due to relaxation of load of short duration and hence the deflection is not large at design load. The deflection due to design load of short duration is easily accommodated by the tower because of adjustments of bolted joint and the tower is safe provided the foundation shaft is not subjected to severe stresses due to bending caused by excessive deflection. Hence limiting deflection with regard to structural safety of RCC shaft or designing for the structural safety of shaft against deflection is important. Hence it is clear that under lateral load conditions, the eventual criterion for the ultimate load is determined by the maximum moment developed in the shaft and the strength properties of the shaft which include the contact pressure on shaft.
From field tests it is seen that

(i) the load-deflection curve is non-linear (Figure 5.9.a). The soil adjacent to the pile at ground surface yields at the very small lateral load resulting in a nonlinear load-deflection behaviour.

(ii) the maximum moment occurs at a shallow level and it decreases to zero at some depth (Figure 5.9.b). Therefore the boundary condition at the pile tip plays no part in the pile behaviour under lateral load. Thus the pile can be considered to be semi-infinite.

A number of solutions to this problem have been suggested on the basis of different assumptions of distribution of lateral load and/or the variation and mobilisation of lateral soil pressure stiffness of the soil against lateral loads. The following two methods are commonly employed for lateral load analysis of piles.

1. Matlock and Reese Method (92)
2. Broms Method (92)

The piles may be classified as flexible, semi flexible and rigid based on the stiffness factor as per Table 5.13.

In most cases the shaft of a tower foundation behaves as a flexible member. In these circumstances, Matlock and Reese method can conveniently be used. Broms method cannot be applied to the semiflexible range as parameters cannot be easily selected in this range.

(a) Reese and Matlock method: Reese and Matlock developed a general solution to determine deflection, moment, etc., for the pile using wave theory. He has developed solutions (Figure 5.10)
FIG. 5.9. a. LATERALLY LOADED FLEXIBLE PILES, ARKANSAS RIVER TEST RESULTS. (i) LOAD VS DEFLECTION AT TOP FOR A NO. OF PILES (ii) MOMENT VS DEPTH IN PILE 2 AT DIFFERENT LOADS
for deflection, bending moments, etc. These solutions are based on the field test results (Figure 5.9.b). The procedure suggested is:

\[
\text{Compute } T = \left( \frac{E I}{k R} \right)^{\frac{1}{2}} \quad (5.15a)
\]

\[
R = 4 \sqrt{\frac{E I}{k B}} \quad (5.15b)
\]

where
- T - stiffness factor for cohesionless soil
- E - modulus of elasticity of concrete
- I - moment of inertia of concrete section
- \( \eta_h \) - coefficient of horizontal subgrade modulus for cohesionless soils
- R - stiffness factor for cohesive soil
- k - coefficient of horizontal subgrade modulus for cohesive soils
- B - width of pile shaft

The value of \( \eta_h \) is given in Table 5.5. The value of k is given in Table 5.6.

Compute the depth factors \( Z_{\text{max}} \) and Z using equations

\[
Z_{\text{max}} = \frac{1}{Z} \quad (5.15c)
\]

\[
Z = \frac{1}{Z} \quad (5.15d)
\]

For \( Z_{\text{max}} \) and various values of Z from charts (Figure 5.10) the coefficients \( A_y \) and \( B_y \) for deflection and \( A_m \) and \( B_m \) for moment are obtained. The equations for calculating the deflection and moment for a given lateral load and moment at the pile top are given below.
FIG. 5.9. b. DEFLECTION, SLOPE ETC. FOR ELASTIC CONDITIONS
FIG. 5.10 RESISTANCE TO LATERAL LOADS—GENERAL SOLUTION COEFFICIENTS

FIG. 5.11 SOIL REACTIONS AND BENDING MOMENTS FOR SHORT PILE UNDER HORIZONTAL LOAD IN COHESIVE SOIL.
Deflection \( y = \frac{A_H T^3}{E I} + \frac{B_H M_T}{E I} \) (5.16a)

Moment \( M = A_H H_T + B_H M_T \) (5.16b)

(b) Bromé Method: Brom has developed a solution based on the assumptions of two modes of failure:

1. Shear failure of the soil and
2. Bending failure of the pile

It is assumed that failure of the soil takes place in the case of rigid piles and that plastic bending failure of the pile occurs with flexible piles. In the later case, it is further assumed that the pile material has the capability of developing plastic hinges having sufficient rotational capacity to develop passive resistance of the soil along the height of the pile which will permit redistribution of bending moment along the height. For short term loading in uniform cohesive soils the method of Bromé is quick and convenient to use. For such soils Bromé has assumed that the reaction of the soil on the pile is represented by a simplified diagram (Figure 5.11).

**TABLE 5.13 PILE TYPE BASED ON STIFFNESS FACTOR (R or T) RELATED TO EMBEDDED LENGTH (L) (85)**

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Soil modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Linearly increasing</td>
</tr>
<tr>
<td>Rigid</td>
<td>( L &lt; 2 ) T</td>
</tr>
<tr>
<td>Semi-Flexible</td>
<td>( 4 ) T ( \geq ) ( 2 ) T</td>
</tr>
<tr>
<td>Flexible</td>
<td>( L &gt; 4 ) T</td>
</tr>
</tbody>
</table>
5.6 EXPERIMENTAL VERIFICATION

The analysis and design procedure of tower foundation suggested in section 5.5 is verified by conducting a nondestructive proto-test on under-reamed pile used for tower foundation. The following three cases are studied.

- Uplift
- Compression (down thrust)
- Lateral thrust

The details of the pile are given in Table 5.14. The load test is carried out as per the guidelines given in Appendix-A and the load settlement curve is plotted from the observed readings for uplift, compression and lateral thrust respectively in Figures 5.12 to 5.14.

5.6.1 Uplift: From the load-settlement curve (vide Figure 5.12), it is noticed that the settlement for 1.5 times safe load is only 3.2 mm and the increase in settlement is also not significant for small increase in load. The ultimate pull-out capacity is determined at 12 mm displacement. Thus the actual failure load is likely to be fairly large when this consideration is used. The ultimate uplift capacity values arrived by (1) the proposed method and (2) the method due to Meyerhof are compared with the test result in Table 5.15. The ultimate resistance of 35t by the proposed method is in close agreement with the 33.7t due to Meyerhof's method. Thus the actual failure load at 12 mm displacement could be about 35t when this is compared with the safe load of 11.25t as per IS 4091-1979. IS code is conservative since a factor of safety of more than 3 is indicated.

5.6.2 Compression: From the load-settlement curve of down thrust test (Figure 5.13) it is noticed that the settlement for 1.5 times safe load is 5 mm. The ultimate compression of the pile is theoretically calculated as 39.55t using eq.5.6. When this is compared with the safe load of 18t as per IS 4091-1979, a
### TABLE 5.14 DETAILS OF THE DOUBLE UNDER REAMED TEST PILE

<table>
<thead>
<tr>
<th>Pile Dia (Cm)</th>
<th>Bulb Dia (Cm)</th>
<th>Pile Spacing (Cm)</th>
<th>Dia of bulks (Cm)</th>
<th>Safe load (Kg)</th>
<th>Test load = 1.5 x safe load (Kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>62.5</td>
<td>440</td>
<td>90</td>
<td>11250</td>
<td>18000</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1800</td>
<td>16675</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>27000</td>
<td>2700</td>
</tr>
</tbody>
</table>

### TABLE 5.15 COMPARISON OF UPLIFT CAPACITY FROM TEST VALUES

<table>
<thead>
<tr>
<th>Sl.No.</th>
<th>Type of Foundation</th>
<th>Ultimate Resistance Proposed</th>
<th>Meyerhof's Load as per Max. Load IS:4091-1979 (1.5 x Safe Load)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Double Under Reamed Pile</td>
<td>35.0</td>
<td>33.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16.875</td>
<td>11.25</td>
</tr>
</tbody>
</table>

Factor of Safety as per proposed method = 3.1
Factor of Safety as per Meyerhof method = 3.0
FIG. 5.12. LOAD SETTLEMENT CURVE FOR UPLIFT TEST
FIG. 5.13. LOAD SETTLEMENT CURVE FOR DOWN THRUST TEST
factor of safety of 2.2 is indicated. This is closer to the expected factor of safety of 1.5 as for the broken wire condition.

5.6.3 Side thrust: The side thrust of 2700 kg (1.5 times safe load specified by IS 4091-1979) is applied at 75 cm above the cut-off level (Figure 5.15). The load displacement curve is plotted in Figure 5.14. The test pile is subjected to a moment at the cut-off level due to this. Therefore it is necessary to deduct the displacement caused by the moment from the total displacement caused by both moment and horizontal load to arrive at the displacement caused by lateral load alone. A theoretical load - displacement curve for lateral load and moment is drawn (vide Figure 5.15) at 1.5 times safe load using the Reese and Matlock method. The displacement at cut-off level due to lateral load is interpreted from the displacement at 75 cm above cut-off level from load test for 1.5 times safe load and the theoretical load-displacement curve. The steps involved in drawing the theoretical load-displacement curve and interpretation of displacement at cut-off level due to the safe load of 1800 kg are given below.

(i) Theoretical load-displacement curve: For M_{20} concrete and the soil property C_u = 1.52 kg/cm^2 using Table 5.6 and eq.5.15, the value of R is determined as 102. For the embedded length of pile (L=410 cm) from Table 5.13 it is identified that the pile is flexible. For Z_{max} (= \frac{L}{R}) and various values of Z, the coefficients A_x and B_y for displacements are obtained from Figure 5.10. Using equation 5.16 the theoretical load-displacement curve is plotted (vide Figure 5.15).

(ii) Determination of displacement at cut-off level at safe load:

Total displacement at cut-off level = (2.52+4.73) = 7.25 mm
(from Figure 5.15)

Total displacement at 75 cm above cut-off level

= Displacement at cut off level + displacement corresponding to the slope of pile at cut off level + the displacement due to free standing cantilever fixed at the ground
FIG. 5.14. LOAD DISPLACEMENT CURVE FOR SIDE THRUST TEST
FIG. 5.15. THEORETICAL LOAD DISPLACEMENT CURVE FOR SIDE THRUST
Displacement at 75 cm above cut-off level from load test = 7.54 mm.

\[
\text{Displacement at cut-off level due to } M \text{ and } B \text{ at safe load } = 7.54 \times \frac{7.25}{12.2} = 4.48 \text{ mm.}
\]

\[
\text{Displacement at cut-off level due to moment only } = 0.448 \times \frac{2.52}{4.73} = 1.55 \text{ mm.}
\]

\[
\text{Displacement at cut-off level due to } B \text{ only } = 4.48 - 1.55 = 2.93 \text{ mm.}
\]

This is less than 5 mm limitation at safe load.

5.7 ANCHORS

There are three basic types of ground anchor systems (Figure 5.16). The first type comprises of a cylindrical hole filled with grout or other fixing agent, depending on the load to be mobilised. The second type is a cylinder which is enlarged by a grout injected into the sides of the borehole under high but controlled pressure, the idea being to cause the grout to form a bulb of strengthened ground, which acts as the anchorage. The third type is a cylinder enlarged at one or more positions along its length by means of a special cutting device.

Within the fixed anchor length the applied load may be mobilised from the top and downwards or from the base upwards, depending on the method of attachment of the tendon to the grout column. With the former, the tendon is embedded in the grout and, on application of load, shear and normal stresses are developed at the grout, ground interface and progressively migrate along the fixed anchor length. This causes the grout surrounding the tendon to be put in tension with associated tension cracks occurring thereof. In corrosive ground and where the anchor may be required to function permanently, this form of load development is not favoured. A more attractive system is the compression anchor. With this anchor, force is transferred to the bottom end of the anchorage zone. This is achieved by
FIG. 5.16: ILLUSTRATION OF MAIN ANCHOR TYPES IN USE
means of a pressure pipe which isolates the tendon from the primary grout. Details of tension and compression anchors are shown in Figure 5.17.

The pull-out value of anchor bars grouted in drill holes depend on the bond strength developed between the steel rod and the surrounding grout. Assessment of the bond strength developed is very difficult except by full scale testing to failure.

5.7.1 Rock anchors: In the case of the anchor in bedrock, the pull-out resistance \( T_u \) is obtained from the following calculations:

Pull out resistance attainable from a single anchor

\[
T_u = \pi d L \cdot q_{ult}
\]  (5.17)

where
- \( d \) - diameter of anchor
- \( L \) - length of fixed anchor zone
- \( q_{ult} \) - ultimate rock/grout bond value (Ref. Table 8)

In the case of decomposed granite

\[
q_{ult} = 0.07 N + 1.24 \text{ kg/cm}^2
\]  (5.18)

where \( N \) - numerical value obtained from standard penetration test

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Earth pressure coefficient ( k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dense sandy gravel</td>
<td>1.4 to 2.3</td>
</tr>
<tr>
<td>Fine sands and sandy silts</td>
<td>0.5 to 1.0</td>
</tr>
<tr>
<td>Dense sand</td>
<td>1.4</td>
</tr>
</tbody>
</table>

TABLE 5.16 VALUES OF EARTH PRESSURE COEFFICIENT ON FIXED ANCHOR(86)
FIG. 5.17. DETAILS OF TENSION AND COMPRESSION ANCHOR

FIG. 5.18. UNDERREAM ANCHOR
5.7.2 Anchors in granular soils: Anchors in granular soil are installed by applying a low pressure grout into the soil in the vicinity of the fixed anchor length to permeate the voids and natural joints. The net effect is to solidify a bulk volume of soil whose diameter is considerably greater than that of the original anchor drill hole. Resistance to lateral load is mobilised by skin friction and the end bearing. The approach followed is to neglect the end bearing component and to rely on the skin friction load. Thus the safe ultimate anchor resistance may be expressed as

\[ P = k \pi D L \sigma_v' \tan \phi \]  
(5.19)

where \( k \) is an earth pressure coefficient on the anchor shaft (Table 15); and \( \sigma_v' \) is the average effective overburden pressure adjacent to the fixed anchor and \( \phi \)- angle of shaft friction.

5.7.3 Anchors in cohesive soils: For clay soil all the three types of anchors in Figure 5.16 are suitable. The failure of underreamed anchor (Figure 5.16c) may occur by the end bearing on the top under-ream or by a plug-like failure along the cylinder encompassing the main anchor length (Figure 5.18). The load carried by a plug failure mechanism will be \( \sigma \cdot C_u \pi D L \) and the load carried by end bearing is

\[ C_u N_c \pi (d^2 - d_s^2) \]  
(5.20)

where \( \sigma \)- a reduction factor for the reduction in average strength of the clay caused by the anchor hole (0.5 to 1)

\( C_u \) - undrained shear strength of clay

5.7.4 Group anchors: When groups of anchors are at close spacing, interaction occurs between them and the behaviour of an average anchor in the group is quite different from that of an isolated anchor and the efficiency of the group is always less than unity.
IS:4091-1979 suggests that the spacing of anchor rods should normally be one-half of the fixed length of rod. At present the state of knowledge of anchor grouping is far from satisfactory, even though it is used in practice.

5.8 STRUCTURAL DESIGN

The shaft is to be designed for the combined axial load and bending moment as per IS:456-1978. The shaft is checked for pull out resistance from development length criteria for the leg angle embedded in concrete and for the bearing in compression. The base is to be designed for the bending moment and shear due to contact pressure. In group piles, the pile cap is also designed using Reynolds Hand Book (93).

5.9 EXAMPLES

Typical examples for various types of soils and tower loadings are worked out and presented in Appendix B. The uplift resistance calculated based on the equations (5.9) and (5.14) for the five examples are compared in Table 5.17. The examples may be treated as a representative set and hence the conclusion arrived at could be assumed to be generally applicable. A cursory perusal of the table shows the validity of the two equations and the advantages it has over the recommendation of IS Code (89).
# TABLE 5.17 EXAMPLES ON UPLIFT CAPACITY OF TOWER FOUNDATIONS

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Type of Foundation</th>
<th>Meyerhof's Method</th>
<th>IS Code Method</th>
<th>Proposed Method</th>
<th>Governing Method for Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Pad Footing</td>
<td>58.32</td>
<td>44.39</td>
<td>59.3</td>
<td>Meyerhof</td>
</tr>
<tr>
<td>2</td>
<td>Bell Pier</td>
<td>52.37</td>
<td>24.12</td>
<td>48.72</td>
<td>Proposed</td>
</tr>
<tr>
<td>3</td>
<td>Footing with Under cut</td>
<td>144.84</td>
<td>76.90</td>
<td>125.0</td>
<td>Proposed</td>
</tr>
<tr>
<td>4</td>
<td>Footing with Under cut and anchor</td>
<td>401.6</td>
<td>237.8</td>
<td>265.2</td>
<td>Proposed</td>
</tr>
<tr>
<td>5</td>
<td>Under Reamed Pile</td>
<td>71.0</td>
<td>-</td>
<td>74.5</td>
<td>Meyerhof</td>
</tr>
</tbody>
</table>