

8.7. Design procedure for foundation

The design of any foundation consists of following two parts.

8.7.1 Stability analysis

Stability analysis aims at removing the possibility of failure of foundation by tilting, overturning, uprooting and sliding due to load intensity imposed on soil by foundation being in excess of the ultimate capacity of the soil. The most important aspect of the foundation design is the necessary check for the stability of foundation under various loads imposed on it by the tower, which it supports. The foundation should remain stable under all the possible combinations of loading, to which it is likely to be subjected under the most stringent conditions. The stability of foundations should be checked for the following aspects.

Check for bearing capacity

The total downward load at the base of footing consists of compression per leg derived from the tower design, buoyant weight of concrete below ground level and weight of concrete above ground level.

While calculating over weight of concrete for checking bearing capacity of soil, the position of water table should be considered at critical location i.e., which would give maximum over weight of concrete. In case of foundation with chimney battered along the slope of leg, the center line of chimney may not coincide with the center of gravity of base slabs/pyramid/block. Under such situation, axial load in the chimney can be resolved into vertical and horizontal components at the top of the base slabs/pyramid/block. The additional moments due to the above horizontal loads should be considered while checking the bearing capacity of soil.

Further even in cases where full horizontal shear is balanced by the passive pressure of soil, the horizontal shears would cause moment at the base of footing as the line of action of side thrusts (horizontal shears) and resultant of passive pressure of soil are not in the same line. It may be noted that passive pressure of soil is reactive forces from the soil for balancing the external horizontal forces and as much mobilized passive pressure in soil adjoining the footing cannot be more than the external horizontal shear.

Thus the maximum soil pressure below the base of the foundation (toe pressure) will depend up on the vertical thrust (compression load) on the footing and the moments at the base level due to the horizontal shears and other eccentric loadings. Under the action of down thrust and moments, the soil pressure below the footing will not be uniform and the maximum toe pressure 'p' on the soil can be determined from the equation.

$$P = \frac{W}{B \times B} + \frac{M_T}{Z_T} + \frac{M_L}{Z_L}$$

Where

'W' is the total vertical down thrust including over weight of the footing,

'B' is the dimension of the footing base;

M_T & M_L are moments at the base of footing about transverse and longitudinal axes of footing and

Z_T & Z_L are the section module of footing which are equal to $(1/6)B^3$ for a square footing.

The above equation is not valid when minimum pressure under the footing becomes negative. The maximum pressure on the soil so obtained should not exceed the limit bearing capacity of the soil.

Check for uplift resistance

In the case of spread foundations, the resistance to uplift is considered to be provided by the buoyant weight of the foundation and the weight of the soil volume contained in the inverted frustum of cone on the base of the footing with slides making an angle equal to the angle of earth friction applicable for a particular type of the soil.

Referring to the figure 8.8 the ultimate resistance to uplift is given by:

$$U_p = W_s + W_f$$

Where W_s is the weight of the soil in the frustum of cone

W_f is the buoyant weight / overload of the foundation.

Depending up on the type of foundation i.e., whether dry or wet or partially submerged or fully submerged, the weights W_s & W_f should be calculated taking into account the location of ground water table.

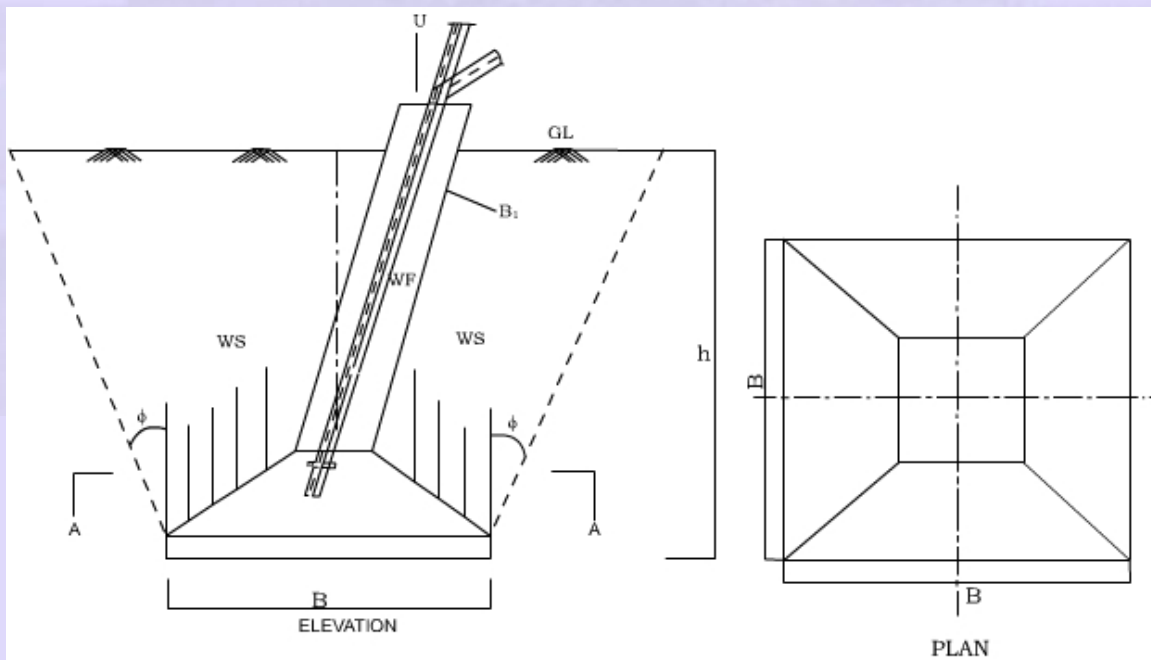


Figure 8.8

Under-cut type of foundation offers greater resistance to uplift than an identical footing without under-cut. This is for the simple reason that the angle of earth frustum originates from the toe of the under-cut and there is perfect bond between concrete and the soil surrounding it and there is no need to depend on the behavior of back filled earth. Substantial additional uplift resistance is developed due to use of under-cut type of foundation. However, to reflect advantage of additional uplift resistance in the design the density of soil for under-cut foundation has been increased as given in Table of Annexure.

In cases where frustum of earth pyramid of two adjoining legs overlap, the earth frustum is assumed truncated by a vertical plane passing through the center line of the tower base.

Check for side thrust

In towers with inclined stub angles and having diagonal bracing at the lowest panel point, the net shearing force of the footing is equal to the horizontal component of the force in the diagonal bracing whereas in towers with vertical footings, the total horizontal load on the tower is divided equally between the numbers of legs. The shear force causes bending stresses in the unsupported length of the stub angles as well as in the chimney and tends to overturn the foundation.

When acted upon by a lateral load, the chimney will act as a cantilever beam free at the top and fixed at the base and supported by the soil along its height. Analysis of such foundations and design of the chimney for bending moments combined with down thrust uplift is very important. Stability of a footing under a lateral load depends on the amount of passive pressure mobilized in the

adjoining soil as well as the structural strength of the footing in transmitting the load to the soil. (Refer figure 8.9)

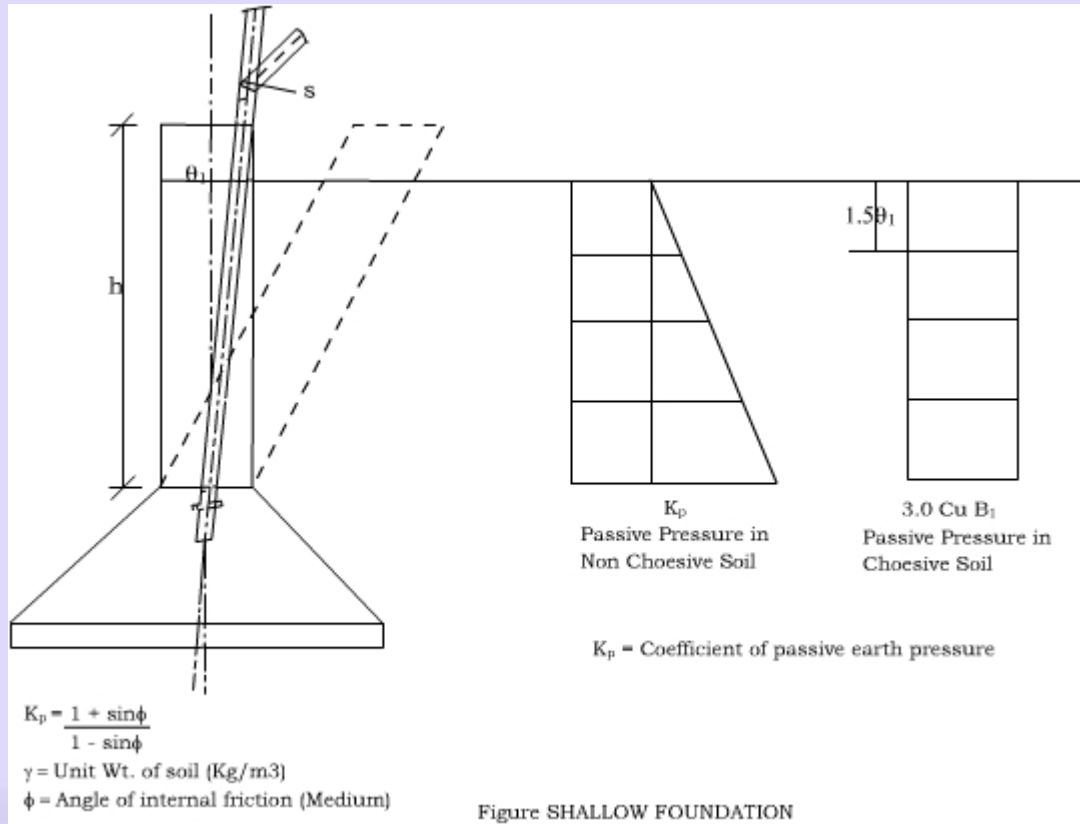


Figure 8.9

Check for over-turning

Stability of the foundation against overturning under the combined action of uplift and horizontal shears may be checked by the following criteria as shown in Figure 8.10.

- i The foundation over-turns at the toe
- ii The weight of the footing acts at the center of the base and
- iii Mainly that part of the earth cone which stands over the heel causes the stabilizing moment. However, for design purposes this may be taken equal to the

half of the cone of earth acting on the base. It is assumed to act through the tip of the heel.

For stability of foundation against overturning, factor of safety shall not be less than 1.5 (DL + LL + WL) (IS: 1904-1986)

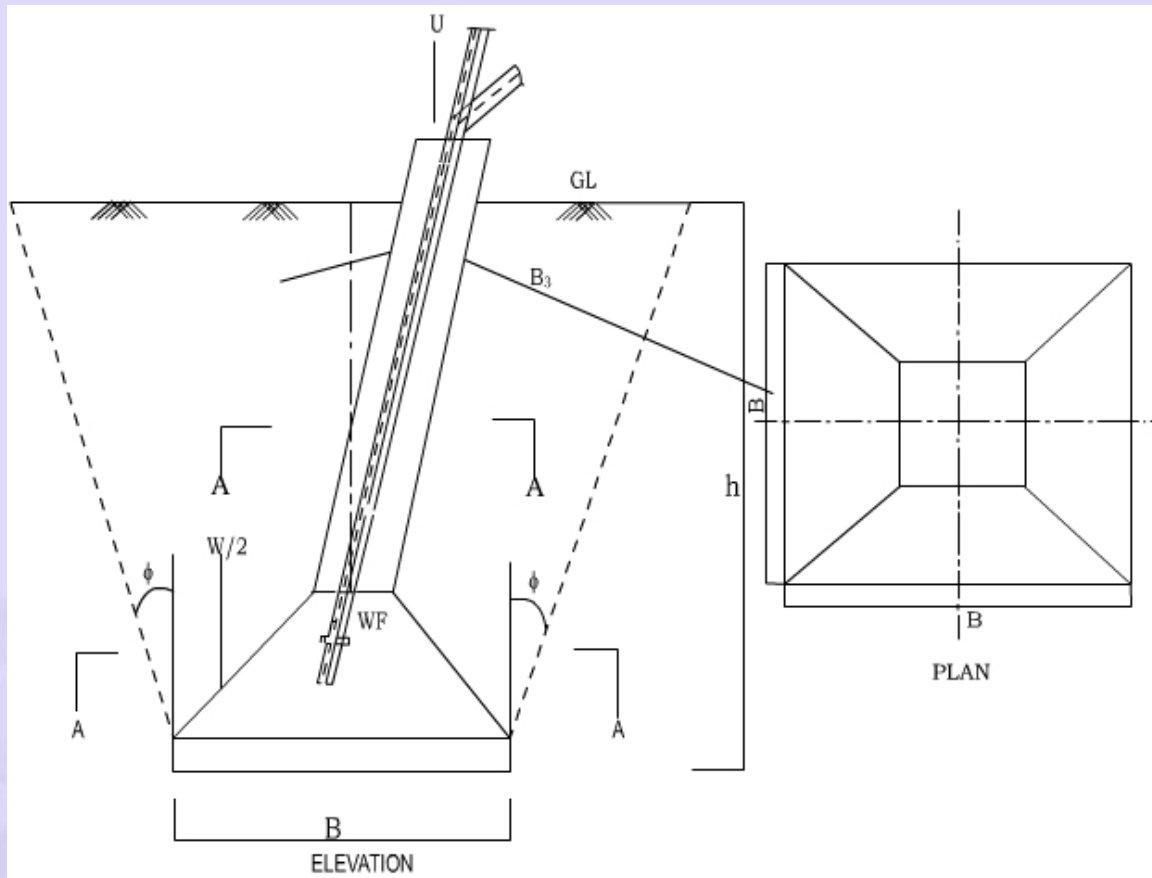


Figure 8.10

Check for sliding

In the foundation of towers, the horizontal shear is comparatively small and possibility of sliding is generally negligible. However, resistance to sliding is evaluated assuming that passive earth pressure conditions are developed on

vertical projections above the toe of foundations. The friction between bottom of the footing and soil also resist the sliding of footing and can be considered in the stability of foundation against sliding. The coefficient of friction between concrete and soil can be considered between 0.2 and 0.3. However, the frictional force is directly proportional to vertical downward load and as such may not exist under uplift condition. For cohesive soil the following formula can be applied for calculating the passive pressure to resist sliding.

$$P_p = 2C \tan \theta + \gamma h \tan^2 \theta$$

Where C = Cohesion

$$\theta = 45^\circ + 1/2 \text{ of angle of earth frustum}$$

H = height of foundation

γ = unit weight of soil

For stability of foundation against sliding. Factor of safety shall not be less than 1.5(DL + LL + WL) (IS: 1904-1986)

8.7.2 Structural design of foundation

Structural design of concrete foundation comprises the design of chimney and the design of base slab/pyramid/block. The structural design of different elements of concrete foundation is discussed below.

Structural design of chimney

The chimney should be designed for maximum bending moments due to side thrust in both transverse and longitudinal direction combined with direct pull (Tension)/ direct down thrust (compression).

Usually, combined uplift and bending will determine the requirement of longitudinal reinforcement in the chimney. When the stub angle is embedded in the chimney to its full depth and anchored to the bottom slab/pyramid/block the chimney is designed considering passive resistance of soil leaving 500mm from ground level. This is applicable for all soils - cohesive, non-cohesive and mixture of cohesive and non-cohesive soils. In hilly areas and for fissured rock, passive resistance of soils will not be considered. Stub angles will not be considered to provide any reinforcement.

In certain cases, when stub is embedded in the chimney for the required development length alone and same is not taken up to the bottom of foundation of leg of the tower is fixed at the top of the chimney/pedestal by anchor bolts, chimney should be designed by providing reinforcement to withstand combined stresses due to direct tension/down thrust (compression) and bending moments, due to side thrust in both transverse and longitudinal direction. The structural design of chimney for the above cases should comply with the procedures given in IS: 456-1978 and SP-16 using limit state method of design.

Case 1 when stub angle is anchored in base slab/pyramid/block

When the stub is anchored in base slab/pyramid/block reinforcement shall be provided in chimney for structural safety on the sides of the chimney at the periphery.

$$\frac{P_u}{f_{ck} B_1^2} = 0.36k + \sum [(P_i / 100)(f_{si} - f_{ci})] / f_{ck} \quad (8.1)$$

$$\frac{M}{f_{ck} B_1^2} = 0.36k(0.5 - 0.416k) + \sum [(P_i / 100)(f_{si} - f_{ci})] / f_{ck} (Y_i / D) \quad (8.2)$$

$$K = \frac{m \sigma_{cbc}}{m \sigma_{cbc} + \sigma_{st}}$$

Where

A_{si} = cross sectional area of reinforcement in it row

$$P_i = 100 A_{si} / B_1^2$$

f_{ci} = stress in concrete at the level of ith row of reinforcement

f_y = Stress in the ith row of reinforcement, compression being positive and tension being negative

Y_i = distance from the centroid of the section to the ith row of reinforcement: positive towards the highly compressed edge and negative towards the least compressed edge

n = Number of rows of reinforcement

f_{ss} = stress in stubs

f_{cs} = stress in concrete

f_{ck} = characteristic compressive strength of concrete

m = modular ratio

σ_{cbc} = permissible bending compressive stress in concrete

σ_{st} = permissible tensile stress in steel

Case 2 when stub is provided in chimney only for its development length

When stub is provided in chimney only for its development length, chimney has to be designed for and reinforcement provided for combined stresses due to direct puit (tension) thrust (compression) and bending moments. The requirement of longitudinal reinforcement should be calculated in accordance with IS: 456-1978 and SP: 16 as an independent concrete column.

In this case, from the equilibrium of internal and external forces on the chimney section and using stress and strains of concrete and steel as per IS:456-1978 the following equations are given in SP:16 are applicable.

$$\frac{P_u}{f_{ck} B_1} = 0.36k(0.5 - 0.416k) + \sum \left[(P_i / 100)(f_{si} - f_{ci}) + (p_s / 100)(f_{ss} - f_{ci}) \right] / f_{ck} \quad (8.3)$$

$$\frac{P_u}{f_{ck} B_1} = 0.36k(0.5 - 0.416k) + \sum \left[(P_i / 100)(f_{si} - f_{ci}) \right] / f_{ck} (Y_i / D) \quad (8.4)$$

In each of the above cases, for a given axial force compression or tension, and for area of reinforcement, the depth of neutral axis $Y_u = KB^1$ can be calculated from Equations using stress strain relationship for concrete and steel as given in IS: 456-1978. After finding out the value of 'K' the bending capacity of the chimney section can be worked out using equation. The bending capacity of the chimney section should be more than the maximum moment caused in the chimney by side thrust (horizontal shear). Chimney is subjected to biaxial moments i.e., both longitudinal and transverse. The structural adequacy of the chimney in combined stresses due to axial force (tension/compression) and bending should be checked from the following equation:

$$\left[\frac{M_T}{M_{ut}} \right]^{un} + \left[\frac{M_L}{M_{ul}} \right]^{un} < 1.0$$

Where,

M_T and M_L are the moments about transverse and longitudinal axis of the chimney

M_{ut} and M_{ul} are the respective moment of resistance with axial loads of P_u about transverse and longitudinal axis of chimney which would be equal in case of square chimney with uniform distribution of reinforcement on all four faces.

N is an exponent whose value would be 1.0 when axial force is tensile and depends on the value of P_u / P_{uz} when axial force is compressive where:

$$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_{ys} A_s + 0.75 f_{ys} A_{ss}$$

In the above equation

A_c is the area of concrete

A_s is the area of reinforcement steel

A_{ss} is the cross sectional area on stub to be taken as zero

f_y is the yield stress of reinforcement steel and

f_{ys} is the yield stress of stub steel to be taken as zero.

P_u / P_{uz}	N
0.2	1.0
0.3	2.0

For intermediate values, linear interpolation may be done.

The solution of equations for case-2 is given in SP-16 in the form of graphs for various grades of concrete and steel and these can be readily used

Important codal provision F

While designing the chimney, the important codal provisions as given below should be followed:

- (a) In any chimney that has a larger cross sectional area than that required to support the load the minimum percentage of steel shall be based on the area of concrete required to resist the direct stress and not on the actual area.
- (b) The minimum number longitudinal bars provided in a column shall be four in square chimney and six in a circular chimney.
- (c) The bars shall not be less than 12mm in diameter.

- (d) In case of a chimney in which the longitudinal reinforcement is not required in strength calculation, nominal longitudinal reinforcement not less than 0.15% of the cross sectional area shall be provided.
- (e) The spacing of stirrups/ lateral ties shall be not more than the least of the following distances:
 - i. The least lateral dimension of the chimney
 - ii. Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied.
 - iii. Forty-eight times the diameter of the transverse stirrups / lateral ties.
- (f) The diameter of the polygonal links or lateral ties shall be not less than one-fourth of the diameter of the largest longitudinal bar and in no case less 6mm.
- (g) Structural design of base slab

The base slab in R.C.C spread foundations could be single stepped or multisteped. The design of concrete foundations shall be done as per limit state method of design given in IS: 456-2000.

Important codal stipulations for R.C.C foundations

The important provisions applicable for concrete foundations which are necessary and should be considered in the design are explained below:

- (a) Footings shall be designed to sustain the applied loads moments and forces and the included reactions and to ensure that any settlement which may occur shall be as nearly uniform possible and the bearing capacity of the soil is not exceeded.

(b) Thickness at the edge of footing in reinforced concrete footing shall not be less than 15cm (5cm lean concrete plus 10cm structural concrete). In case of plain concrete footing thickness at the edge shall not be less than 5cm.

(c) Bending moment

i. The bending moment at any section shall be determined by passing through the section of a vertical plane which extends completely across the footing and computing the moment of the forces acting over the entire area of the footing on the side of the said plane.

ii. The greatest bending moment to be used in the design of an isolated concrete footing which supports a column / pedestal shall be the moment computed in the manner prescribed in c(i) above at section located as follow:

- a. At the face of the chimney.
- b. At the sections where width / thickness of the footing changes.

(d) Shear and bond

The shear strength of footing is governed by the more severe of the following two conditions:

i. The footing acting essentially as a wide beam with a potential diagonal crack extending in a place across the entire width; the critical section for this condition should be assumed as a vertical section located from the face of the chimney at a distance equal to the effective depth of the footing in case of footings on soils.

ii. Two-way action of the following with potential diagonal cracking along the surface of truncated cone or pyramid around the concentrated load.

(e) Critical section

The critical section for checking the development length in a footing shall be assumed at the same plane as those described for bending moment in para (c) above and also at all other vertical planes where abrupt changes of section occurs.

When a plain concrete pyramid and chimney type footing is provided and pyramidal slopes out from the chimney at an angle less than 45 from vertical, the pyramid is not required to be checked for bending stresses. Thus, in such cases the footing is designed to restrict the spread of concrete pyramid of slab block to 45 with respect to vertical.

8.7.3 Concrete technology for tower foundation designs

While designing the various types of concrete footings it is better to know about certain aspects of concrete technology which are given below:

Properties of concrete

The grade of the structural concrete used for tower foundations should not be leaner than M15 having a 28-day cube strength of not less than 15 N/mm² and concrete shall conform to IS: 456, for special foundations like pile foundations richer concrete of grade of M20 having a 28-day cube strength of not less than 20 N/mm² should be used. M15 grade concrete shall have the nominal strength of not less than 15 N/mm² at the end of 28 days as ascertained from the cube test. Such strength at the end of 7 days shall not be less than 10 N/mm².

The density of the concrete will be 2300 kg/m³ for plain concrete and 2400 kg/m³. For R.C.C other properties of concrete shall be as given in IS: 456-2000.

Properties of steel

The high yield stress cold deformed reinforcement bars used in the R.C.C shall conform to IS: 1786-1979 and shall have yield stress of not less than 415 N/mm². When mild steel reinforcement bars are used in R.C.C., they shall conform to IS:432 (part-I) and shall have yield stress of not less than 26 N/mm² for bars of size up to 20mm diameter and 24 N/mm² for bars above 20mm diameter.

Pull-out tests on tower foundation

The pull out tests conducted on foundations help in determining the behaviors of the soil while resisting the uplift forces.

The feed from this pull out test results in a particular type of soil can be conveniently used in the design of foundations. The procedure of pull out tests, equipments and results are discussed in detail below.

Selection of site

Trial pits of size 1.0 x 1.0 x 3.0 meter are made and the strata of the soil are observed. It is ascertained that the strata available at the location is one in which we are interested (i.e., a particular type of soil or combination of soils is available). Soil samples are taken from and around the site and subjected to various tests. Particularly relating to the density of soil, bearing capacity of soil, cohesion and angle of internal friction etc.

Design of foundation for pull-out test

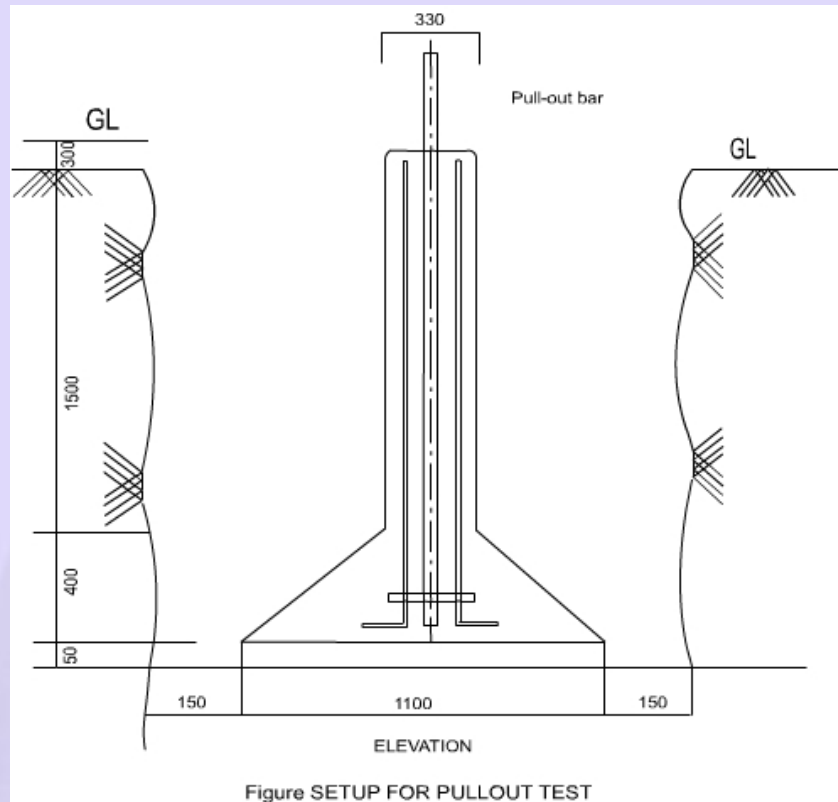


Figure 8.11

Design of foundations for pull-out test is carried out with a different view point as compared to the design of actual foundations for tower. This is due to the fact that the pull-out tests are conducted to measure the pull-out resistance and the pull-out bars should be strong so that these do not fail before the soil/rock fails.

Based on the actual tower foundation loadings (down thrust, uplift and side thrust) and the soil parameters obtained from the tests, a foundation design is developed. The design has a central rod running from the bottom of the footing up to a height of about 1.5m to 2.0m above ground, depending on the jacking requirements. The central rod is surrounded by a cage of reinforcement bars.

A typical design development for the pull-out test is shown in Figure 8.11.

Casting of foundation

The pits are excavated accurately. The concrete mix, reinforcement, from boxes etc. are exactly as per the design. The pouring of the concrete is done such that voids are minimized. The back filling of the soil should be carried out using sufficient water to eliminate voids and loose pockets. The foundation should be cured for 14 days (minimum) and thereafter left undisturbed for a period not less than 30 days.

Investigation of foundation towers

Normally it is believed that once the foundation is cast and the tower is erected, the foundations can not be re-opened for investigation or repairing.

If the foundations of the tower have to be investigated, certain locations are selected at random in such a fashion that foundations for various types of soils are covered one by one. Out of the four individual footings of selected tower, two diagonally opposite foundations are selected and one of the four faces of each of these two foundations is excavated in slanting direction from top to bottom.

After the investigation is over and corrective measures have been chalked out it is advisable to backfill the excavating mixing earth with fight cement slurry, particularly when the soil is non-cohesive such as soft murrum / hard murrum, softrock / hard rock etc., (say one cement bag for every three to four cubic meter of earth). This will ensure good bond and safeguard the foundation against uplift forces, even if corrective repairs of the foundations are delayed.

Repair of foundation of a tower

After it is established that the foundation is unhealthy, it is better to take the corrective steps as early as possible. The methods would be different for rectifying isolated location/locations (one to two) and for rectifying complete line/line sections including a number of towers. These are discussed below.

(a) Rectification of isolated locations (one or two) is done on individual basis. Any one of the four footings is taken up first. It is opened up from all the four sides. The tower legs connected to this footing are guyed. After rectifying the foundation backfilling is done. A minimum of seven-days time is allowed for curing of the repaired foundation before excavating the second leg for repairs. In the similar way the other legs also repaired.

Foundation defects and their repairs

The main possible defects in the cast concrete can be as follows:

(a) Under sizing of foundation due to wrong classification of soil; for example, the soil may be dry black cotton but the foundation cast may be that for normal dry soil, if the corrective measures are not taken, the foundation can fail. An R.C.C collar is designed for the type of soil and tower loadings to remedy such a defect.

(b) Improper foundation of pyramid/chimney etc., due to improper concrete laying:

If the concrete is simply poured from the top of the form box, without taking care to fill the voids (using crow bar, vibrator etc.) the concrete does not reach to the corners of the form and thus the foundation is not completely formed.

Foundation for roof top communication towers

In urban areas where the land is very costly communications towers are placed on the rooftops of the buildings with the added advantage of the altitude. For placing the tower on the buildings, the stability of the building for the additional loads envisaged on the building due to the placing of his tower etc., shall be checked and certified.

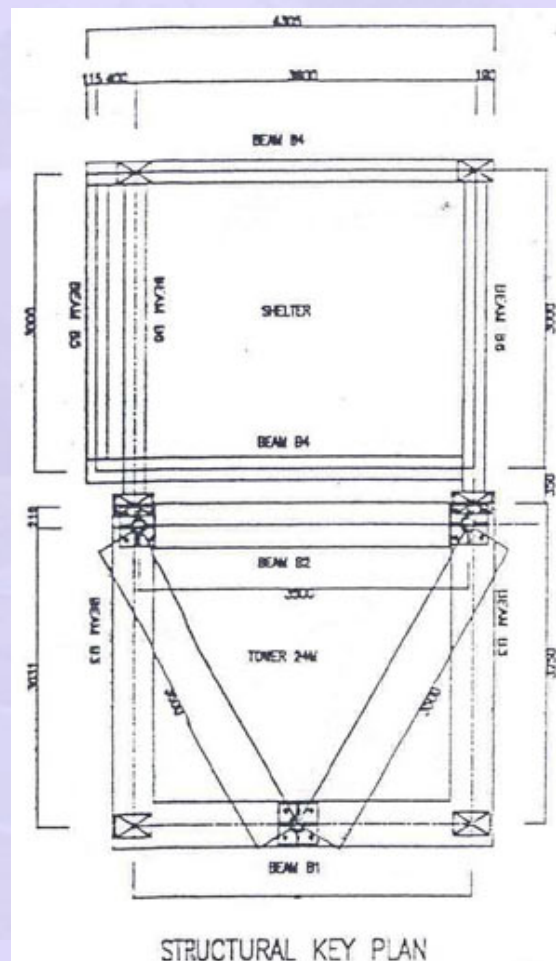


Figure 8.12

The foundation for the tower shall be designed in such a way that the loads are directly transferred on to the columns only, following ways attains this.

- (a) The column rods of the building are exposed and the reinforcement required for the tower pedestal is welded to the exposed rods.
 - (b) Where the exposing of the rods is not possible the pedestal rods can be anchored by drilling holes vertically on top of the columns and grouting them with the chemical grouts.
 - (c) Welding the pedestal rods to the beam rods at the column beam junctions.
- Typical details are shown in the figure 8.12

The for the roof top communication tower consists of the pedestal and beam arrangement. The tower base plate rests on the beam. The beam shall be designed for both down thrust and uplift forces coming from the tower. Figure 8.13 shows the typical foundation detail for the roof top communication tower.



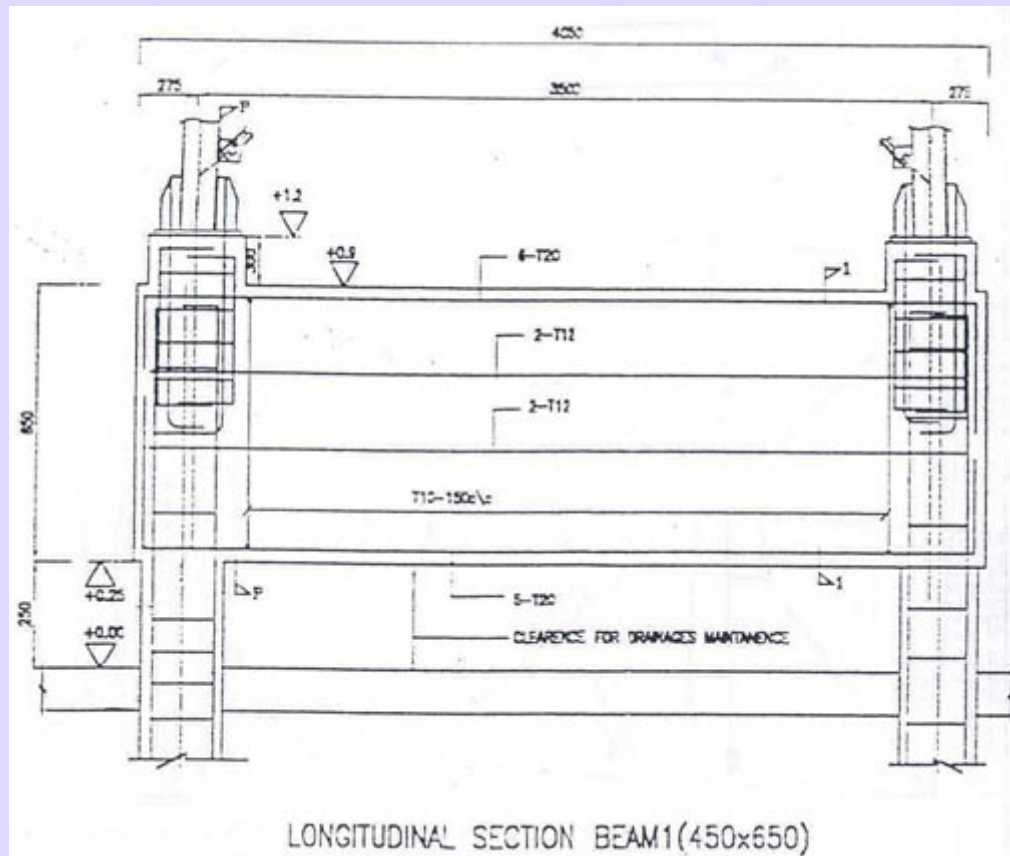


Figure 8.13

Spacing of the columns in the existing building governs the economy of the foundation design. The size of base plate governs the width of the beam, width of the beam has to be at least 50mm more than the base plate and the depth governed by the anchor bolt, the depth of the beam shall be more than the anchor bolt length. For the proper distribution of the concentrated force coming from the tower leg in the beam, it is suggested to place the base plate on the pedestal on the beam minimum 250mm height limiting to 400mm.

Example 1

Design forces on tower leg

Ultimate compression: 81,400 kg

Ultimate uplift : 58,250 kg

Ultimate shear : 2,250 kg

Tower data

Base width = 4m

Height = 36m

Soil data

Soil type : medium dense sand

Site location : Manali (Madras)

Average SPT value: 12

In-situ density $\gamma = 1.79 \text{ t/m}^3$

$$\gamma_{\text{sub}} = 1.0 \text{ t/m}^3$$

$$\phi = 32^\circ$$

Water table at 1.5m below ground level.

$N_{\text{corrected}}$ for overburden upto 5m depth = 16

$N_q = 23$, $N_\gamma = 30$

Type of foundation

Select a pad footing of size 2.5m x 2.5m at 2.5m depth

Check for uplift

Uplift resistance is calculated using different methods discussed in the text.

1. From equation (9.7) of Mayerhof,

$$T_u = \pi CBD + S_J \frac{\pi}{2} B \gamma D^2 K_u \tan \phi + W \quad (1)$$

$$= 58.32 \text{ t}$$

2. From conventional method (IS: 4091 - 1979) for 20° dispersion in cohesionless soils,

$T_u = 44.39 \text{ t}$ (weight of frustum of earth + weight of concrete as show Figure) (2)

3. From equation (9.13), uplift resistance along the shearing plane

$$Q_{\phi r} = 2\pi(C + K_{\phi} \gamma \tan j) \left[\frac{R_e x^2}{2} + D \tan \alpha \frac{x^2}{2} - \frac{x^3}{3} \tan \alpha \right]_0^D$$

$$= 30.12 \text{ t}$$

Since the footing is a pad, assume a factor of safety of 2 for possible weakening of soil due to excavation and refilling.

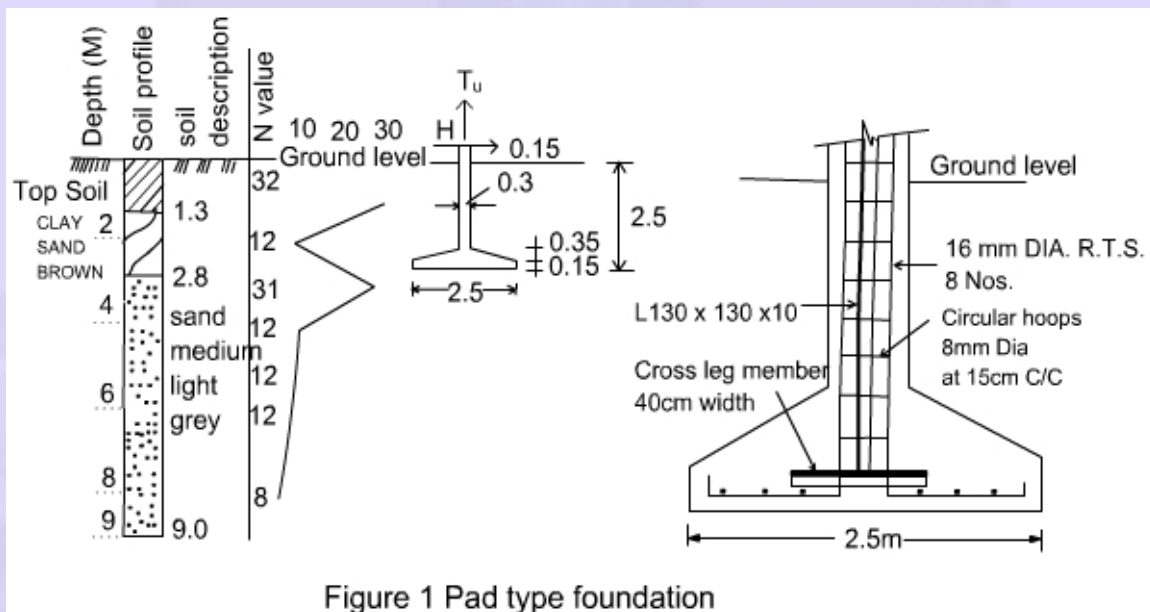


Figure 1 Pad type foundation

$$Q_s \text{ allowable} = 30.12 / 2 = 15.06 \text{ t} \quad (3)$$

Adding equation 2 and 3, the total uplift resistance,

$$T_u = 44.39 + 15.06 = 59.45 \text{ t} \quad (4)$$

Design uplift resistance = 58.32 t (Least of equation 1 and 4)

Design uplift force = 58.25 t, hence, safe.

Check for bearing capacity

From equation 9.1,

$$\begin{aligned} Q_u &= CN_s d_i + q (N_s - 1) s_q d_{qs} + \frac{1}{2} B \gamma N_s d_i w \\ &= 94.8 \text{ t/m}^2 \end{aligned}$$

Where values of s , d and i are chosen from Table 9.9.

$$\text{Area of footing} = 2.5 \times 2.5 = 6.25 \text{ m}^2$$

$$\text{Ultimate bearing capacity of footing} = 6.25 \times 94.8$$

This is greater than the ultimate compression 81.4 t, hence safe.

Generally, bearing capacity is not the governing criterion.

Check for settlement

From equation (9.20) instantaneous settlement

$$s_i = I_p q B \left(\frac{1 - \nu^2}{E_\theta} \right)$$

Choosing ν and E_s from table 9.4 and I_p from table 9.14

$$s_i = 0.313 \text{ cm}$$

Since the soil is sandy, settlement due to consolidation does not arise.

$$\text{Base width of tower} = 4 \text{ m}$$

Maximum possible rotation

$$\theta = 0.313 / 400 = 0.00078$$

Check for lateral capacity by Reese and Matlock Method

$$\text{From Table 9.5, } \eta_n = 1.5$$

$$\text{Therefore } T = \left(\frac{EI}{\eta_n} \right)^{1/5} = 89.8$$

$$Z_{\max} = L/T = 2.78$$

$$Z = x / T \text{ at } x = 0, Z = 0;$$

From Figure,

$$A_y = +3, B_y = 2.0, A_m = 0.66, B_m = 0.64$$

$$H = H_u / 2 = 1,125 \text{ kg}$$

$$M_t = 1,125 \times 15 = 16,875 \text{ kg cm}$$

$$Y_{\max} = \frac{A_y H T^3}{EI} + \frac{B_y M_t T^2}{EI}$$

Y_{\max} at service load = 0.31 cm < 2 cm, hence safe

$$M_{\max} = A_m H T + B_m M_t$$

$$M_{\max} \text{ at ultimate load} = 154,856 \text{ kg cm.}$$

Structural design

$$\text{Deflection at ultimate load} = 2 \times 0.31 = 0.62 \text{ cm.}$$

Check for compression and bending

$$\text{Moment due to eccentricity} = 81,400 \times 0.62$$

$$= 50,468 \text{ kg cm}$$

$$\text{Design moment} = 154,856 + 50,468$$

$$= 205,324 \text{ kg cm}$$

$$\text{Design compressive load} = 81,400 \text{ kg}$$

Use M 150 concrete and Fe 415 steel.

$$f_{ck} = 150 \text{ kg/cm}^2, f_y = 4,150 \text{ kg/cm}^2$$

$$\text{Let } d' / D = 0.1$$

Use chart 56 of IS SP: 16 (S and T) - 1980,

$$\frac{M_u}{f_{ck} D^3} = 0.0467$$

$$\frac{P_u}{f_{ck} D^2} = 0.6$$

$$\frac{P}{f_{ck}} = 0.15, p = 0.15 \times 15 = 2.25 \text{ percent}$$

$$A_c = 15.9 \text{ cm}^2$$

Provide 16mm diameter ribbed twisted steel (RTS) 8 numbers.

Use 8mm hoops at 15 cm c/c which satisfy the code requirements.

Check for tension

Design uplift = 58,250 kg

Let the leg section be L 130 x 130 x 10

Perimeter = 52

Design bond stress 1.0 N/mm² (IS: 456 - 1978)

$$\begin{aligned} \text{Development length required} &= \frac{\text{load}}{\text{perimeter} \times \text{stress}} \\ &= \frac{58,250}{52 \times 10} = 112 \end{aligned}$$

Hence safe.

Anchor the leg member in the chimney part with 40cm cross member as shown in Figure 1 for additional safety