There has been an increasing trend of construction of microwave communication and transmission line towers all over the country. The foundations of such towers constitute nearly 20 to 40 percent of the total cost of towers. However, very little information is available on the design procedure of tower foundation. The paper presents the underlying concepts for designing tower foundations efficiently and economically. A computer program in BASIC, which uses these concepts, is also presented. This program may be used to optimally proportion the tower foundations.

N. Subramanian and V. Vasanthi

Transmission line towers, antenna towers, towers used for oil well derricks and mine-shaft equipment, beacon supports, and observation platform, etc., are examples of self-supporting towers. Out of these various types of towers, transmission line towers are subjected to torsional forces, in addition to other forces.

Normally, the tower foundation constitutes about 20 to 40 percent of the total cost of tower. A rough idea about this cost could be obtained from the relative weights of the foundation and tower. It was observed that for a 100-m high microwave tower, the weight of the foundation concrete was around 410t, while the weight of the structural steel of the tower was only 65t. From the engineering point of view, the foundation design of towers poses a serious problem due to different types of soils encountered and also due to the various forces acting on the foundation. Thus, the structural engineer is faced with a difficult task of producing economical and reliable design. A very little information is available for the design of such foundations.

The design of tower foundation is basically an interative procedure. Since the uplift force is predominant, the design poses a number of problems, and hence, is amenable to computerization. However, till now, no program is available in India for the design of these foundations. In this paper a computer program is presented, based on the provisions of the recent Indian Codes of Practice. A brief outline of the procedure to be used for the design of tower foundations is also described. Salient features of the package developed, based on this procedure are enumerated. The package has been developed using the BASIC language for use on an IBM PC or compatible machine based on the working stress method of design. Both unreinforced and reinforced concrete sections could be designed by using this program. Different types of soil conditions, viz., normal dry, wet, submerged, partially submerged, black cotton, wet black cotton, soft rock, and hard rock, are considered. Based on this procedure, the authors have designed a 100-m microwave tower foundation which was executed by the Indian Telephone Industries in Rajasthan.

Design

There are two parts in the design. They are: stability analysis, and strength design. Stability analysis aims at removing the possibility of failure by overturning, uprooting, sliding and tilting of the foundation due to soil pressure being in excess of the ultimate capacity of the soil. The strength design consists of proportioning the components of the foundation to the respective maximum moment, shear, pull and thrust or combination of the same.

The type of loading that controls the foundation design depends mainly on the kind of towers being designed. The controlling design loads for four-legged lattice towers are vertical uplift, compression and side thrust.

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

(i) drilled and/or belled shaft
(ii) pad and chimney
(iii) footing with undercut
(iv) auger with reaming
(v) grillage
(vi) special type.

Selection of foundation type needs judgement and experience and a careful study of all the parameters. However, in India, pad and chimney type of foundation is employed for a majority of towers, and hence, in this paper, the same type of foundation is considered, Fig. 1. The concrete used for the
**Design for uplift resistance**

Apart from resisting the vertical compression, the soil surrounding a tower foundation has also to resist a considerable amount of upward pull and side thrust. As a matter of fact, the available uplift resistance of the soil is the deciding factor in selecting the size of the footing. However, unfortunately, adequate theory has not yet been established for the accurate assessment of the uplift resistance of the soil mass. It is generally considered that the resistance to uplift is provided by the shear strength of the soil and the weight of the foundation. Various empirical relationships linking ultimate holding power to the physical properties of the soil, as well as the dimensions of the footing have been proposed on the basis of experimental results.

Calculations of the ultimate uplift capacity obtained by these methods show a wide fluctuation as shown in Fig 2. The method proposed by Meyerhof and Adams, Matsuo and Balla have been found to be in close agreement for sands 

\((C = 0)\), but differ significantly for cohesive soils.

Hence, in the Indian Code\(^2\), the traditional method of assessment of uplift resistance by computing the weight of earth in the inverted frustum of cone/pyramid, whose sides

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**Fig 1 Different types of tower foundation**

- Chimney-pad type foundation
- Reinforced concrete foundation
- Anchor type foundation
- Benching of hard rock

**Fig 2 Comparison of uplift methods**

**Fig 3 Assumptions and variables used in the computer program**
make an angle with the vertical equal to the angle of internal friction of the soil, has been considered. In practice, many designs done on the basis of weight frustum of cone/pyramid with sides making an angle 20° in the case of non-cohesive soils and 30° in the case of cohesive soils have been found to be satisfactory. Though this method is found to give results within ±15 percent range of the experimental values, the results are generally found to be on the conservative side. It is because the earth cone method neglects adhesion or friction along the failure surface.

Referring to Fig 3, the ultimate resistance to uplift will be given by
\[ U_r = W_r + W_f \]
where, \( W_r \) = weight of soil in frustum of pyramid and \( W_f \) = buoyant weight of the foundation.

For square footing, which is common for the type of foundation shown in Fig 1(a) and (b), the volume of earth in the frustum of pyramid for dry normal soil as per IS code is
\[ V = D_D / 6 + 2 tan \beta D_D / 3 + 4 / 3 \tan \beta D_D^2 \]
where, \( D_D \) = depth of pyramid = \( D \cdot H \), \( B \) = breadth of footing, \( \beta \) = angle of repose of soil.

It should be noted that, apart from being a function of the properties of soil (\( \beta \), \( C \), etc.), the effective uplift resistance is also affected by the degree of compaction of the soil, and the groundwater table at the location of the foundation.

In this program, the earth cone method (i.e., the IS code method) as well as the method suggested by Meyerhof and Adams are considered.

For both these methods, equations were derived for finding the volume of earth resisting the uplift considering the following:

(i) normal dry soil and soft rock
(ii) wet soil
(iii) submerged or partially-submerged soil
(iv) if reinforced concrete foundation is chosen, \( H_2 \) is assumed as zero (refer Fig 3)
(v) in case where the frustum of earth pyramid of two adjoining legs superimpose each other, the earth frustum is assumed to be truncated by a vertical plane passing through the centre line of the tower base.

Since the program requires the values of \( B \), \( H_1 \), and \( H_2 \), (which will be known only after designing the foundation), it gives some approximate values as guidance, which may be input as the initial values. The program prints the values of factor of safety against uplift resistance. The user has to check whether the values of \( B_1 \), \( H_1 \), and \( H_2 \) can be changed in order that he can reduce the factor of safety to nearly 1.00 so that the optimum design is achieved.

It has to be noted that two values of factor of safety against uplift are printed, one without considering passive pressure (IS code method) and the other considering the same. The user can choose any one of them for optimization, depending upon his needs.

**Check for sliding**

The shear force acting on the foundation causes bending stresses in the unsupported length of the stub angle as well as in chimney/shaft of the foundation and tends to overturn the foundation. When the lateral resistance of the adjoining soil is small or totally neglected as uncertain, the bending and overturning actions will be more.

However, in tower designs, it is a common practice to consider the side thrust on the foundation to be resisted by the passive earth pressure mobilized in the adjoining soils due to rotation of the footing. Because of the somewhat larger lateral movement tolerated in the foundation of common self-supporting, bolted towers, it is permissible to depend on the mobilization of the passive pressure even when the foundation construction involves excavation and provision of backfill; but when such passive pressure is relied upon, it is mandatory to compact the backfill with special care.

Stability of a footing under a lateral load will be dependent upon 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. Solution of this problem involves the study of the soil structure interaction and assessment of the soil pressure for the allowable lateral displacement. A very little information is available on the soil structure interaction of tower foundations.

Hence, in the program for the unreinforced foundation, the following method is adopted. Referring Fig 4,

\[ \text{Factor of safety against sliding} = \frac{\mu C + \sum P_i}{\text{Side thrust}} \]

where, \( P_i \) is the sum of passive pressure components of the soil, \( C \) is the compressive force acting on the foundation, and \( \mu \) is the coefficient of friction, which varies between 0.35 and 0.55 depending on the type of soil. In the program, the conservative value of 0.35 has been assumed. However, when

![Fig 4 Stability of tower foundation against sliding](image-url)
tension and side thrust are acting (which is the critical condition), the above equation is rewritten as

\[
\text{Factor of safety} = \frac{T}{\text{Side thrust}}
\]

If the factor of safety is less than the specified value, the chimney width is increased.

**Stability against overturning**

Stability of the foundation against overturning may be checked by the following criteria, Fig 5.:

(i) the foundation tilts about a point in its base at a distance of 1/6th of its width from the toe

(ii) the weight of the footing acts at the centre of the base

(iii) mainly that part of the cone which stands over the heel, causes the stabilising moment.

However, for design purposes, this may be taken equal to half the weight of the cone of earth acting on the base. It is assumed to act at the tip of the heel, Fig 5.

**Design for downward load**

The maximum soil pressure below the base of the foundation (toe pressure) will depend on the vertical thrust on the footing and the moment at the base level due to the horizontal and other eccentric loadings.

When the vertical load acts eccentrically or the horizontal shear at the top of the pedestal is transferred to the soil below the footing, the soil pressure at this level will not be uniform. The unit maximum toe pressure \( P \) on the soil can be determined from the equation:

\[
P = \frac{W(1 + \varepsilon')}{A_B}
\]

where \( A \) and \( B \) are the base dimensions of the footing and \( \varepsilon' = \frac{M}{W} \), in which \( M \) = the maximum moment of the loads taken at the level and mid point of the base, and \( W \) = total vertical thrust including that of the footing. Equation (5) is applicable when the result lies within the middle third. When the footing is under biaxial moment, the maximum pressure at the critical corner should be worked out accordingly.

The maximum pressure on the soil so obtained should not exceed the safe bearing capacity of the soil. If it exceeds, the size of the footing is to be increased. The safe bearing pressure may, however, be increased by 25 percent if the loading considered includes dead load and wind or earthquake loads as per IS code. However, since the governing load is the wind load, this increase is not allowed in the program.

**Uprooting of stub**

Normally, the stub angle is taken inside the pad portion and anchored by cleat angle and keying rods. In this case, the
chimney, with the stub angle inside, works as a composite member.

Assuming that the stub angle is anchored in the footing as shown in Fig 6, the failure is assumed to be a cone surface plus a surface within the concrete having the same diameter as the anchor bolt head (in the case of headed anchors) or the anchor bolt body (in the case of headless anchors). Thus, the horizontal projected area of the potential failure cone is given by

\[ A_p = 0.75 \times d_f^2 \]

where, \( d_f \) = diameter of the anchor plate or bolt head. Incidentally, it is desirable to keep the effective size of anchor head as small as possible to reduce embedment requirements.

Neglecting the effect of anchor plate,

\[ A_p = A_r \]

Then, the following condition has to be satisfied, if the uprooting of stub should not take place

\[ A_r \times f_t \leq T \]

where, \( f_t \) = nominal direct tension stress of concrete.

As per IS 456, the value of tensile stress is given as

\[ 0.7 \times V_{ck} \]

where \( V_{ck} \) = characteristic strength of concrete in N/mm².

However, the principal tensile stress in the concrete along the potential pull-out failure plane is assumed to vary from a maximum at the mechanical anchor on the end of the steel embedment to zero at the surface of the concrete. The average resistance provided by the concrete can be taken as \( 4 \times V_{ck} \) acting on the projected tensile stress area. This value has been suggested by the American Code and is in British units. Converting it to metric units, this may be taken as 1.06 \( V_{ck} \), where \( V_{ck} \) is in kg/cm².

To take into account the effect of other factors, ACI :318-77 has specified two \( c \) factors. When the anchor head is between the far face reinforcement and the near face concrete, the pull-out strength of the concrete is dependent primarily on the tensile strength of the concrete and the \( c \) factor is to be assumed as 0.65. When the anchor heads are beyond the far face reinforcement, the entire depth is involved in the failure and hence a \( c \) factor of 0.85 has been suggested by the ACI Code. Based on the above discussions, the following formula has been used in the program.

\[ f_t = 1.06 \times \sqrt{V_{ck}} \]

where, \( c = 0.65 \) or 0.85.

If equation (8) is not satisfied, the value of \( y \) is increased.

### Rock foundation

**Anchor type foundation for hard rock**

Uplift resistance to anchored footing is provided by the bond between the grouted steel and rock through the grouting materials which is usually decided by experiments. This bond will increase when deformed bars or bars with indentation are used instead of plain rods. Use of eye-bolts, fox-bolts or threaded rods can also increase the uplift capacity.

The program assumes that 2000-mm long, 40-mm diameter rods are used in the rock foundation for anchorage. Then, assuming a bond stress between the rod and rock as 3 kg/cm², anchor capacity is calculated as

\[ A_r = 3 \times 4 \times 10 \times n \]

where, \( n \) = number of anchor rods, which is given as input.

Then,

\[ \text{factor of safety against uplift } = A_r / (T + W_f) \]

where, \( W_f \) = weight of footing. If this factor of safety is less than 1, the length of anchor rods is increased. The program also checks for the failure of rock mass.

The base width of foundation and depth of concrete are found out by

\[ B = \sqrt{A_r} \times \text{UBC} \]

and

\[ D = f_t \]

where, \( f_t \) = development length of anchor rods, and UBC = ultimate bearing capacity of soil.

### Benching of hard rock

If benching is used for rock foundation, as shown in Fig 1 (d), then the breadth of excavation is found out by

\[ B = \sqrt{\text{A_r} / \text{UBC} + 2D} \]

\( D \) is initially assumed as 1.0 m in the program. The horizontal projected area of the potential failure cone is given by

\[ A_p = D \times (D + d_h) \]

where, \( d_h \) = diameter of the anchor plate. It has to be noted that \( B \) should be greater than \( (2D + d_h) \). Neglecting the effect of anchor plate,

\[ A_p = D^2 \]

Then, factor of safety against uplift is

\[ F_u = \frac{A_r}{T} \]

where, \( T \) = direct tensile stress of concrete as discussed earlier. Incidentally, this factor of safety is equal to the factor of safety against uprooting of stub. If equation (17) is not satisfied, the value of \( D \) is increased.

### Reinforced concrete foundation

When the forces acting on the foundation are high, reinforced concrete foundation as shown in Fig 1 (b) is adopted. In this type of foundation, the slender chimney is not likely to act as a rigid body due to the heavy shear force. The comparatively slender chimney shall rather act as a cantilever beam embedded in elastic soil and fixed at the base. Analysis of such a foundation and design of the chimney/shaft for combined bending and direct pull/thrust are, therefore, very important for structural safety of the foundation. However, rigorous analysis of these footings, which involves thorough understanding of soil-structure interaction and the coefficient of sub-grade reaction, is complicated and tedious for routine design works. Moreover, in view of the large number of
assumptions inherent in such a solution, the results will always be of questionable nature for practical designs. Simplified solutions are, therefore, preferred.

The following procedure, as recommended by National Thermal Power Corporation (NTPC) is adopted in the program for the design of the chimney for combined bending moment and pull.

For the foundation shown in Fig 6.

Equivalent concrete Area, \( A_{eq} = B_1 + m \)

(Cross sectional area of stub) \( \ldots (18) \)

\[ m = \text{modular ratio, which may be taken as 16 for M15 concrete.} \]

\[ I_x = \frac{B_1^2}{12} \times \text{Iec of stub angle} \ldots (19) \]

\[ z = I_x / B_1 \ldots (20) \]

Position of maximum bending moment in chimney

\[ I = \left( \frac{\sqrt{25}}{12} \right) \left( 1.1 \times \sin \gamma / (1 + \sin \gamma) \right) \ldots (21) \]

where, \( \gamma = \text{weight of soil. If } I > H_2 \text{ then } I = H_2 \)

Now the following check is made:

\[ \sigma_c^+ \leq 1.33 \]

\[ \sigma_{bc}^+ \leq \sigma_{bc} \]

\( \sigma_c^+ = \left( \frac{2}{3} + \frac{M}{I} \right) \)

\( \sigma_{bc}^+ = \frac{N}{A} \ldots (24) \)

If biaxial bending is there, the bending moment in the other axis is simply added in the expression for \( \sigma_{bc}^+ \). When equation (22) is not satisfied, the size of stub is increased or extra reinforcement is provided.

As shown above, if the stub angle is embedded in the chimney to its full depth and anchored to the base-slab, the chimney is treated as a composite member with the stub angle inside the chimney working as rigid reinforcement. When the leg of the tower is fixed at the top of the shaft by anchor bolts, as shown in Fig 1(b), the shaft is designed for and reinforced against tension/thrust plus the bending stresses from the moments — uniaxial or biaxial — as the case may be.

The base slab is designed as per simple bending theory, i.e., the footing is assumed to behave as a flexural member cantilevered from the chimney portion. Hence, formulae commonly used in the design of reinforced concrete flexural members are made use of in this program.\(^{10}\)

The macro flow chart of this program is shown in Fig 7.

To check the validity of this program, several examples were worked out by hand and checked with the computer output. A typical output for normal dry soil (unreinforced foundation) is shown in Appendix I. Details of the foundations for other types of soils for the same compression, tension and shear are shown in Table 1.
### TABLE 1 Details of foundation for different types of soil

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>$B$ (mm)</th>
<th>$H_1$ (mm)</th>
<th>$H_2$ (mm)</th>
<th>$H_3$ (mm)</th>
<th>Concrete volume $m^3$</th>
<th>Volume of excavation $m^3$</th>
<th>$Y_f$ (kg/m$^3$)</th>
<th>$c^*$ (degree)</th>
<th>UBC kg/cm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Dry</td>
<td>1450</td>
<td>50</td>
<td>450</td>
<td>2500</td>
<td>5.639</td>
<td>37.979</td>
<td>1440</td>
<td>30</td>
<td>2.6800</td>
</tr>
<tr>
<td>2. Wet</td>
<td>2050</td>
<td>50</td>
<td>820</td>
<td>2130</td>
<td>9.848</td>
<td>68.479</td>
<td>1440</td>
<td>15</td>
<td>1.3675</td>
</tr>
<tr>
<td>3. Partially submerged</td>
<td>2180</td>
<td>50</td>
<td>1000</td>
<td>1950</td>
<td>14.036</td>
<td>93.775</td>
<td>1440</td>
<td>15</td>
<td>1.3675</td>
</tr>
<tr>
<td>4. Fully submerged</td>
<td>2150</td>
<td>50</td>
<td>820</td>
<td>2130</td>
<td>10.313</td>
<td>74.431</td>
<td>1440</td>
<td>15</td>
<td>1.3675</td>
</tr>
<tr>
<td>5. Wet black cotton</td>
<td>3080</td>
<td>50</td>
<td>1350</td>
<td>1600</td>
<td>24.775</td>
<td>141.663</td>
<td>1440</td>
<td>15</td>
<td>1.3675</td>
</tr>
<tr>
<td>7. Soft rock</td>
<td>1480</td>
<td>50</td>
<td>540</td>
<td>2410</td>
<td>6.007</td>
<td>39.288</td>
<td>1440</td>
<td>20</td>
<td>6.2500</td>
</tr>
</tbody>
</table>

### Conclusions

The procedures involved in the design of tower foundations are given. The features of the computer program that has been developed based on these concepts are explained. This program takes into account reinforced as well as unreinforced foundations. Similarly, foundations on rocks could also be designed by this program. For resisting uplift, the program gives a choice between resistance with and without passive pressure. Using some iterations, the optimum foundation details could be arrived at. The program also gives printouts of the quantities of earthwork excavation, concrete, reinforcement, etc. Thus, it reduces the tedious task of tower foundation design, which may involve only a few seconds work on a personal computer.

### Acknowledgement

The authors would like to thank Mr Venkataraman of Indian Telephone Industries and Mr M. Sundara Raj for the fruitful discussions.

### References


### Appendix 1 Typical output of tower foundation design for normal dry soil (unreinforced foundation)

<table>
<thead>
<tr>
<th>TOWER DETAILS</th>
<th>MAX. THRUST</th>
<th>ULT. UPLIFT</th>
<th>SHEAR</th>
<th>BASE WIDTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>TANGENT</td>
<td>48400 KG</td>
<td>31900</td>
<td>6248</td>
<td>85D CM</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SOIL DETAILS</th>
<th>NORMAL DRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>SOIL TYPE</td>
<td>38 DIG</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DETAILS OF FOUNDATION</th>
<th>BASE - 1450 MM.</th>
</tr>
</thead>
<tbody>
<tr>
<td>UPPER 1500 MM NORMAL DRY SOIL</td>
<td>LOWER STRATA SUBMERGED SOIL</td>
</tr>
<tr>
<td>PART-SUBM. TYPE UPPER 750 MM NORMAL DRY SOIL</td>
<td>LOWER STRATA SUBMERGED SOIL</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>BASE - 1450 MM.</th>
</tr>
</thead>
<tbody>
<tr>
<td>CHIMNEY WIDTH: 550 MM</td>
</tr>
<tr>
<td>CHIMNEY HT. 2725 MM</td>
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</tbody>
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<table>
<thead>
<tr>
<th>BASE - 1450 MM.</th>
</tr>
</thead>
<tbody>
<tr>
<td>PUMPING BX 50 MM</td>
</tr>
<tr>
<td>PUMPING WIDTH: 550 MM</td>
</tr>
<tr>
<td>CHIMNEY HT. 2725 MM</td>
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