7.3 Factors of safety and load

7.3.1 Factors of safety and Permissible deflections

Factors of safety of conductors and ground wires

The factor of safety (f.o.s) of a conductor (or ground wire) is the ratio of the ultimate strength of the conductor (or ground wire) to the load imposed under assumed loading condition. Rule 76 (1)(c) of the Indian Electricity Rules, 1956, stipulates as follows:

The minimum factor of safety for conductors shall be two, based on their ultimate tensile strength. In addition, the conductor tension at 32°C without external load shall not exceed the following percentages of the ultimate tensile strength of the conductor:

Initial unloaded tension 35 percent
Final unloaded tension 25 percent

The rule does not specify the loading conditions to which the minimum factor of safety should correspond. Generally, these loading conditions are taken as the minimum temperature and the maximum wind in the area concerned. However, meteorological data show that minimum temperature occurs during the winter when, in general, weather is not disturbed and gales and storms are rare. It therefore appears that the probability of the occurrence of maximum wind pressures, which are associated with gales and stormy winds and prevail for appreciable periods of hours at a time, simultaneously with the time of occurrence of the lowest minimum temperatures is small, with the result that the
conductors may be subjected rarely, if at all, to loading conditions of minimum temperature and the maximum wind.

However, no data are available for various combinations of temperatures and wind conditions, for the purpose of assessing the worst loading conditions in various parts of the country. The problem is also complicated by the fact that the combination of temperature and wind to produce the worst loading conditions varies with the size and material of the conductor. Furthermore, it is found that in a number of cases the governing conditions is the factor of safety required under ‘everyday’ condition (or the average condition of 32°C, with a little or no wind to which the conductor is subjected for most of the time) rather than the factor of safety under the worst loading conditions as illustrated in Table 7.9

**Table 7.9 Factors of safety under various conditions**

<table>
<thead>
<tr>
<th>Conductor size</th>
<th>Al 30/3.00+7/3.00 mm Panther</th>
<th>St 30/2.59+7/2.59 mm Wolf</th>
<th>St 6/4.72+7/1.57 mm Dog</th>
<th>St 30/3.71+7/3.71 mm Panther</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind pressure</td>
<td>75 kg/sqm</td>
<td>75 kg/sqm</td>
<td>75 kg/sqm</td>
<td>150 kg/sqm</td>
</tr>
<tr>
<td>Temp.range</td>
<td>5-60°C</td>
<td>5-60°C</td>
<td>5-60°C</td>
<td>5-60°C</td>
</tr>
<tr>
<td>Span</td>
<td>335m</td>
<td>335m</td>
<td>245m</td>
<td>300m</td>
</tr>
<tr>
<td>Factors of safety</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Under the worst loading condition</td>
<td>2.76</td>
<td>2.66</td>
<td>2.28</td>
<td>2.29</td>
</tr>
<tr>
<td>Under everyday condition</td>
<td>4.00</td>
<td>4.01</td>
<td>4.02</td>
<td>4.00</td>
</tr>
</tbody>
</table>

**Factors of safety to towers**

The factors of safety adopted in the designs have a great bearing on the cost of structures prove economical as well as safe and reliable.

Rule 76 (1)(a) of the Indian Electrical Rules, 1956, specifies the following factors of safety, to be adopted in the design of steel transmission line towers:
1. under normal conditions  2.0
2. under broken-wire conditions  1.5

It is interesting to compare this practice with that followed in the USSR. In the USSR, while for normal conditions the f.o.s. is 1.5, that for the broken-wire condition is 1.2 for suspension tower, and 1.33 for anchor towers. In the case of transmission lines at 500kV and above, in addition of these factors of safety, an impact condition is also imposed. When the conductor breaks, there is a sudden impact on the tower occurring for 0.4 to 0.6 second. The impact factor is assumed as 1.3 in the case of suspension tower with rigid clamps and 1.2 in the case of anchor tower, and the loads acting on the tower are increased correspondingly. Thus, the final force in the case of suspension towers is increased by a factor 1.3 x 1.1 and in the case of anchor towers by 1.2 x 1.2. The corresponding factor of safety assumed under the impact conditions is 1.1 and 1.2.

**Permissible deflections**

Sufficient data are not available with regard to the permissible limits of deflection of towers, as specified by the various authorities. However, one practice given below is followed in the USSR:

Assuming that there is no shifting of the foundation, the deflection of the top of the support in the longitudinal direction from the vertical should not exceed the following limits:

For dead-end heavy-angle structure  \((1/120)\) H  
For small angle and straight line structures with strain insulators  \((1/100)\) H
For supports with heights exceeding 160m and intended to be used at crossing locations \((1/140) H\)

Where \(H\) is the height of the tower.

The above limits of deflection are applicable to supports having a ratio of base width to height less than \(1/12\). For suspension supports with heights up to 60m, no limit of deflection of the tower top from the vertical is specified. As regards the cross-arms, the following limits of deflection in the vertical plane under normal working conditions are stipulated:

1. For dead-ends and supports with strain insulators and also for suspension supports at special crossings:
   
   a. for the position of the cross-arms lying beyond the tower legs \((1/70) A\)
   
   b. for the position of the cross-arms lying between a pair of legs \((1/200)L\)

![Figure 7.12 Limits of deflection (USSR practice)](image)

2. For the suspension supports which are not intended to be used at crossing locations:
a. for the position of the cross-arms lying beyond the tower legs \((1/50)A\)

b. for the position of the cross-arms lying between a pair of tower legs \((1/150)L\)

Where \(A\) = length of the cross-arm lying beyond the tower leg, and

\(L\) = length of the cross-arm lying between the two tower legs (Figure 7.12)

### 7.3.2 Loads

The various factors such as wind pressures, temperature variations and broken-wire conditions, on the basis of which the tower loadings are determined, are discussed in this section.

### A new approach

During the past two decades, extensive live load surveys have been carried out in a number of countries with a view to arriving at realistic live loads, based on actual determination of loadings in different occupancies. Also developments in the field of wind engineering have been significant.

A correct estimation of wind force on towers is important, as the stresses created due to this force decide the member sizes. The standardization of wind load on structure is a difficult task and generally involves three stages:

1. analysis of meteorological data
2. simulation of wind effects in wind tunnels
3. synthesis of meteorological and wind tunnel test results.
The overall load exerted by wind pressure, $p$, on structures can be expressed by the resultant vector of all aerodynamic forces acting on the exposed surfaces. The direction of this resultant can be different from the direction of wind. The resultant force acting on the structure is divided into three components as shown in Figure 7.13.

1. a horizontal component in the direction of wind called drag force $F_D$
2. a horizontal component normal to the direction of wind called horizontal lift force $F_{LH}$
3. a vertical component normal to the direction of wind called the vertical lift force $F_{LV}$

**Aerodynamic coefficient**

The aerodynamic coefficient $C$ is defined as the ratio of the pressure exerted by wind at a point of the structure to the wind dynamic pressure. The aerodynamic coefficient is influenced by the Reynolds number $R$, the roughness of surface and the type of finish applied on the structure. Thus both the structure
and nature of wind (which depends on topography and terrain) influence the aerodynamic coefficient, $C$.

For the three components of the wind overall force, there are corresponding aerodynamic coefficients, namely, a drag coefficient, a horizontal lift coefficient, and a vertical lift coefficient.

**Pressure and force coefficients**

There are two approaches to the practical assessment of wind forces, the first using pressure coefficients and the second using force coefficients. In the former case, the wind force is the resultant of the summation of the aerodynamic forces normal to the surface. Each of these aerodynamic forces is the product of wind pressure $p$ multiplied by the mean pressure coefficient for the respective surface $C_p$ times the surface area $A$. Thus

\[ F = (C_p p) A \]  \hspace{1cm} (7.1)

This method is adopted for structures like tall chimneys which are subjected to considerable variation in pressure.

*Figure 7.14(a) Frontal area of a structure*
In the second case, the wind force is the product of dynamic pressure \( q \) multiplied by the overall force coefficient \( C_f \) times the effective frontal area \( A_1 \) for structures. Thus

\[
F = (C_f q) A_1 \quad (7.2)
\]

The second approach shown in Figure 6.14(a) is considered practical for transmission line towers.

Although wing effects on trusses and towers are different, the force coefficient \( C_f \) are similar and are dependent on the same parameters.

**Trusses**

Force coefficients for trusses with flat-sided members or rounded members normal to the wind are dependent upon solidity ratio, \( \phi \).

Solidity ratio \( \phi \) is defined as the ratio of effective area of the frame normal to the wind direction \( S_T \) to the area enclosed by the projected frame boundary (Figure 7.14(b))

\[
\phi = \frac{S_T \times 2}{h(b_1 + b_2)} \quad (7.3)
\]

Where \( S_T \) is the shaded area

**Shielding effects**

In the case of trusses having two or more parallel identical trusses, the windward truss has a shielding effect upon the leeward truss. This effect is dependent upon the spacing ratio \( d/h \) (Figure 7.15). The shielding factor \( \psi \) reduces the force coefficient for the shielded truss and is generally given as a function of solidity ratio and spacing ratio in codes (Table 7.10).³
$S_T$: Total area of structural components of a panel projected normal to face (hatched area)

$\phi$: Solidity Ratio

$$\phi = \frac{2}{h(b_1 + b_2)}$$

Source: International conference on “Trends in transmission Line Technology” by the confederation of Engineering industry

**Figure 7.14(b) Calculation of solidity ratio**

**Towers**

The overall force coefficient for towers which consist of one windward truss and one leeward truss absorbs the coefficient of both solidity ratio $\phi$ and shielding factor $\psi$. Thus

- $C_f$ for windward truss = $C_f^1 \phi$ \hspace{1cm} (7.4)
- $C_f$ for leeward truss = $C_f^1 \psi \phi$ \hspace{1cm} (7.5)
- $C_f$ for tower = $C_f^1 \phi (1 + \psi)$ \hspace{1cm} (7.6)
Where $C_f^1$ is the force coefficient for individual truss and $C_f$ is the force coefficient for the overall tower.

Table 7.11 gives the overall force coefficients for square sections towers recommended in the French and the British codes.\(^3\)

![Figure 7.15 Spacing ratio for leeward truss](image)

**Figure 7.15 Spacing ratio for leeward truss**

The wind force $F$ on latticed towers is given by

$$F = C_f PA_e \quad (7.7)$$

Where $C_f$ is the overall force coefficient, $P$ dynamic wind pressure, and $A_e$ the surface area in m\(^2\) on which wind impinges.

**Wires and cables**

Table 7.12 gives the force coefficient as a function of diameter $d$, dynamic pressure and roughness for wires and cables for infinite length ($1/d>100$) according to the French and the British Practices.

Based on the considerations discussed above, the practice followed in the USSR in regard to wind load calculations for transmission line towers is summarized below.
Wind velocity forms the basis of the computations of pressure on conductors and supports. The wind pressure is calculated from the formula

\[ F = \alpha C_f A_e \frac{V^2}{16} \]  \hspace{1cm} (7.8)

Where \( F \) is the wind force in kg, \( V \) the velocity of wind in metres/second, \( A_e \), the projected area in the case of cylindrical surfaces, and the area of the face perpendicular to the direction of the wind in the case of lattice supports in square metres, \( C_f \), the aerodynamic coefficient, and \( \alpha \) a coefficient which takes into account the inequality of wind velocity along the span for conductors and ground wires. The values of aerodynamic coefficient \( C_f \), are specified as follows:

For conductors and ground-wires:
- For diameters of 20mm and above \hspace{1cm} 1.1
- For diameters less than 20mm \hspace{1cm} 1.2

For supports:
- For lattice metallic supports according to the Table 6.13.

Values of the coefficient \( \alpha \)

![Figure 7.16 Definition of aspect ratio a / b](image)
The values of $\alpha$ are assumed as given in Table 7.14. Wind velocity charts have been prepared according to 5-year bases. That is, the five-year chart gives the maximum wind velocities which have occurred every five years, the 10-year chart gives the maximum velocities which have occurred every ten years and so on. The five-year chart forms the basis of designs for lines up to 35 kV, the 10-year chart for 110 kV and 220 kV lines and the 15-year chart for lines at 400 kV and above. In other words, the more important the line, the greater is the return period taken into account for determining the maximum wind velocity to be assumed in the designs. Although there are regions with maximum wind velocities less than 25 m/sec. In all the three charts, e.g., the 10-year chart shows the regions of 17, 20, 24, 28, 32, 36 and greater than 40 m/sec., the minimum velocity assumed in the designs is 25 m/sec. For lines up to and including 220 kV, and 27 m/sec. For 330 kV and 500 kV lines.

The new approach applicable to transmission line tower designs in India is now discussed.

The India Meteorological Department has recently brought out a wind map giving the basic maximum wind speed in km/h replacing the earlier wind pressure maps. The map is applicable to 10m height above mean ground level.
### Table 7.10 Shielding factors for parallel trusses

<table>
<thead>
<tr>
<th>Country, Code</th>
<th>$\psi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>France, Regles NV65</td>
<td></td>
</tr>
<tr>
<td>Italy, CNR UNI 10012</td>
<td></td>
</tr>
<tr>
<td>$\phi \leq 0.6 \left{ \begin{array}{l} \frac{d}{h} \leq 2 \ 2 &lt; \frac{d}{h} \leq 5 \end{array} \right}$</td>
<td>$1-1.2\phi$</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$1-0.4\phi \left( 5 - \frac{d}{h} \right) \leq 1$</td>
</tr>
</tbody>
</table>

#### Soviet Union, SNIP II-A.11 - 62

<table>
<thead>
<tr>
<th>$d/h$</th>
<th>1</th>
<th>2</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>0.1</td>
<td>0.85</td>
<td>0.90</td>
<td>0.93</td>
<td>0.97</td>
</tr>
<tr>
<td>0.2</td>
<td>0.68</td>
<td>0.75</td>
<td>0.80</td>
<td>0.85</td>
</tr>
<tr>
<td>0.3</td>
<td>0.50</td>
<td>0.60</td>
<td>0.67</td>
<td>0.73</td>
</tr>
<tr>
<td>0.4</td>
<td>0.33</td>
<td>0.45</td>
<td>0.53</td>
<td>0.62</td>
</tr>
<tr>
<td>0.5</td>
<td>0.15</td>
<td>0.30</td>
<td>0.40</td>
<td>0.50</td>
</tr>
<tr>
<td>0.6</td>
<td>0.15</td>
<td>0.30</td>
<td>0.40</td>
<td>0.50</td>
</tr>
<tr>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 7.11 Overall force coefficients $C_f$ for square-section towers

<table>
<thead>
<tr>
<th>Country, Code</th>
<th>Flat-side members</th>
<th>Rounded members</th>
</tr>
</thead>
<tbody>
<tr>
<td>France, Regles NV65 0.08&lt;\phi&lt;0.35</td>
<td>3.2 - 2\phi</td>
<td>0.7 (3.2 - 2\phi)</td>
</tr>
<tr>
<td>Great Britain, CP3: Ch V:Part 2:1972</td>
<td>Subcritical flow dV&lt;6m²s⁻¹</td>
<td>Supercritical flow dV&gt;6m²s⁻¹</td>
</tr>
</tbody>
</table>

$$\phi = \begin{cases} 
0.1 & 3.8 \\
0.2 & 3.3 \\
0.3 & 2.8 \\
0.4 & 2.3 \\
0.5 & 2.1 \\
\end{cases} \begin{cases} 
2.2 & 1.9 \\
1.7 & 1.6 \\
1.4 & 1.4 \\
\end{cases} \begin{cases} 
1.2 & 1.3 \\
1.4 & 1.4 \\
\end{cases}$$
### Table 7.12 Force coefficients, $C_f$ for wires and cables

<table>
<thead>
<tr>
<th>Country, Code</th>
<th>Description</th>
<th>$C_f$</th>
</tr>
</thead>
</table>
| **France, Regles NV65** Smooth surface members of circular section | $d \geq 0.28$  
$0.5 < d \sqrt{p} < 1.5$  
$d \sqrt{p} \geq 1.5$ | +0.6  
+1.0  
+1.2 $-0.4d \sqrt{p}$  
+0.6 |
| Moderately smooth wires and rods | $d \sqrt{p} \leq 0.5$  
$0.5 < d \sqrt{p} < 1.5$  
$d \sqrt{p} \geq 1.5$ | +1.0  
$1.135 - 0.27d \sqrt{p}$  
+0.73 |
| Fine stranded cables | $d \sqrt{p} \leq 0.5$  
$0.5 < d \sqrt{p} < 1.5$  
$d \sqrt{p} \geq 1.5$ | +1.2  
$1.4 - 0.4d \sqrt{p}$  
+0.8 |
| **Great Britain, CP3: ChV:Part 2: 1972** Smooth surface wires, rods and pipes | $d \sqrt{p} < 1.5$  
$d \sqrt{p} > 1.5$ | +1.2  
+0.5 |
| Moderately smooth wires and rods | $d \sqrt{p} < 1.5$  
$d \sqrt{p} > 1.5$ | +1.2  
+0.7 |
| Fine stranded cables | $d \sqrt{p} < 1.5$  
$d \sqrt{p} > 1.5$ | +1.2  
+0.9 |
| Thick stranded cables | $d \sqrt{p} < 1.5$  
$d \sqrt{p} > 1.5$ | +1.3  
+1.1 |
Table 7.13 Aerodynamic coefficient for towers

<table>
<thead>
<tr>
<th>Aspect ratio = a/b</th>
<th>0.15</th>
<th>0.25</th>
<th>0.35</th>
<th>0.45</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5 - 0.7</td>
<td>3</td>
<td>2.6</td>
<td>2.2</td>
<td>1.8</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>2.7</td>
<td>2.3</td>
<td>2.0</td>
</tr>
<tr>
<td>1.5 - 2.0</td>
<td>3</td>
<td>3.0</td>
<td>2.6</td>
<td>2.2</td>
</tr>
</tbody>
</table>

Table 7.14 Space coefficient α for conductors and ground wires

<table>
<thead>
<tr>
<th>At wind velocities</th>
<th>Coefficient α</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 20m/sec</td>
<td>1.00</td>
</tr>
<tr>
<td>Up to 25m/sec</td>
<td>0.85</td>
</tr>
<tr>
<td>Up to 30m/sec</td>
<td>0.75</td>
</tr>
<tr>
<td>Up to 35m/sec and above</td>
<td>0.70</td>
</tr>
</tbody>
</table>

For supports: $\alpha = 1$

The basic wind speed in m/s $V_b$ is based on peak gust velocity averaged over a time interval of about three seconds and corresponding to 10m height above mean ground level in a flat open terrain. The basic wind speeds have been worked out for a 50-year return period and refer to terrain category 2 (discussed later). Return period is the number of years the reciprocal of which gives the probability of extreme wind exceeding a given wind speed in any one year.

Design wind speed

The basic wind speed is modified to include the effects of risk factor ($k_1$), terrain and height ($k_2$), local topography ($k_3$), to get the design wind speed ($V_Z$).

$$V_Z = V_b k_1 k_2 k_3 \quad (7.9)$$
Where \(k_1, k_2,\) and \(k_3\) represent multiplying factor to account for chosen probability of exceedence of extreme wind speed (for selected values of mean return period and life of structure), terrain category and height, local topography and size of gust respectively.

**Risk probability factor \((k_1)\)**

**Table 7.15 Risk coefficients for different classes of structures**

<table>
<thead>
<tr>
<th>Class of structure</th>
<th>Mean probable design life of structure in years</th>
<th>(k_1) for each basic wind speed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>33</td>
</tr>
<tr>
<td>1. All general buildings and structures</td>
<td>50</td>
<td>1.0</td>
</tr>
<tr>
<td>2. Temporary sheds, structures such as those used during construction operation (for example, form-work and falsework), structures in construction stages and boundary walls</td>
<td>5</td>
<td>0.82</td>
</tr>
<tr>
<td>3. Buildings and structures presenting a low degree of hazzard to life and property in event of failure, such as isolated towers in wooded areas, farm buildings except residential buildings</td>
<td>25</td>
<td>0.94</td>
</tr>
<tr>
<td>4. Important buildings &amp; structures like hospitals, communications buildings or towers, power plant structures.</td>
<td>100</td>
<td>1.05</td>
</tr>
</tbody>
</table>

In the design of structures a regional basic wind velocity having a mean return period of 50 years is used. The life period and the corresponding \(k_1\) factors for different classes of structures for the purpose of design are included in the Table 7.15.

The factor \(k_1\) is based on the statistical concepts which take account of the degree of reliability required period of time in years during which there will be exposure to wind, that is, life of structure. Whatever wind speed is adopted for
design purposes, there is always a probability (however small) that may be exceeded in a storm of exceptional violence; the greater the period of years over which there will be exposure to the wind, the greater is this probability. Higher return periods ranging from 100 years to 1,000 years in association with greater periods of exposure may have to be selected for exceptionally important structures such as natural draft cooling towers, very tall chimneys, television transmission towers, atomic reactors, etc.

**Terrain categories (k₂ factors)**

Selection of terrain categories is made with due regards to the effect of obstructions which constitute the ground surface roughness. Four categories are recognised as given in Table 7.16.

**Variation of basic wind speed with height in different terrains**

The variation of wind speed with height of different sizes of structures depends on the terrain category as well as the type of structure. For this purpose three classes of structures given in the note under Table 7.17 are recognised by the code.

Table 7.17 gives the multiplying factor by which the reference wind speed should be multiplied to obtain the wind speed at different heights, in each terrain category for different classes of structures.

The multiplying factors in Table 7.17 for heights well above the heights of the obstructions producing the surface roughness, but less than the gradient height, are based on the variation of gust velocities with height determined by the following formula based on the well known power formula explained earlier:
\[ V_z = V_{gz} \left( \frac{Z}{Z_g} \right)^k = 1.35 V_b \left( \frac{Z}{Z_g} \right)^k \quad (7.10) \]

Where \( V_z \) = gust velocity at height \( Z \),

\( V_{gz} \) = velocity at gradient height

\( = 1.35 V_b \) at gradient height,

\( k \) = the exponent for a short period gust given in Table 6.18,

\( Z_g \) = gradient height,

\( V_b \) = regional basic wind velocity, and

\( Z \) = height above the ground.

The velocity profile for a given terrain category does not develop to full height immediately with the commencement of the terrain category, but develop gradually to height \( (h_x) \), which increases with the fetch or upwind distance \( (x) \). The values governing the relation between the development height \( (h_x) \) and the fetch \( (x) \) for wind flow over each of the four terrain categories are given in the code.

**Topography (\( k_3 \) factors)**

The effect of topography will be significant at a site when the upwind slope \( (\theta) \) is greater than 3°, and below that, the value of \( k_3 \) may be taken to be equal to 1.0. The value of \( k_3 \) varies between 1.0 and 1.36 for slopes greater than 3°.

The influence of topographic feature is considered to extend 1.5 \( L_e \) upwind and 2.5\( L_e \) of summit or crest of the feature, where \( L_e \) is the effective horizontal length of the hill depending on the slope as indicated in Figure 7.8. The values of \( L_e \) for various slopes are given in Table 7.18a.
If the zone downwind from the crest of the feature is relatively flat ($\theta < 3^\circ$) for a distance exceeding $L_e$, then the feature should be treated as an escarpment. Otherwise, the feature should be treated as a hill or ridge.

**Table 7.16 Types of surface categorised according to aerodynamic roughness**

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
</tr>
</thead>
</table>
| 1        | Exposed open terrain with few or no obstructions  
- Open sea coasts and flat treeless plains |
| 2        | Open terrain with well scattered obstructions having heights generally ranging from 1.5 to 10m  
- Air fields, open parklands and undeveloped sparsely built-up outskirts of towns and suburbs. |
| 3        | Terrain with numerous closely spaced obstructions having the size of buildings or structures up to 10m in height.  
- Well-wooded areas and suburbs, towns and industrial areas fully or partially developed. |
| 4        | Terrain with numerous large high closely spaced obstructions  
- Large city centres and well-developed industrial complexes. |

Topography factor $k_3$ is given by the equation

$$k_3 = 1 + C_s \quad (7.11)$$

Where $C$ has the values appropriate to the height $H$ above mean ground level and the distance $x$ from the summit or crest relative to effective length $L_e$ as given in the Table 7.18b.

The factor’s’ is determined from Figure 7.18 for cliffs and escarpments and Figure 7.19 for ridges and hills.
Design wind pressure

The design wind pressure $p_z$ at any height above mean ground level is obtained by the following relationship between wind pressure and wind velocity:

$$p_z = 0.6 V_z^2 \quad (7.12)$$

Where $p_z =$ design wind pressure in $\text{N/m}^2$, and
\( V_z = \text{design wind velocity in m/s}. \)

The coefficient 0.6 in the above formula depends on a number of factors and mainly on the atmospheric pressure and air temperature. The value chosen corresponds to the average Indian atmospheric conditions in which the sea level temperature is higher and the sea level pressure slightly lower than in temperate zones.

Figure 7.18 Factors for cliff and escarpment
Table 7.17 Factors to obtain design wind speed variation with height in different terrains for different classes of buildings structures

<table>
<thead>
<tr>
<th>Height (m)</th>
<th>Terrain Category 1 class</th>
<th>Terrain Category 2 class</th>
<th>Terrain Category 3 class</th>
<th>Terrain Category 4 class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>A</td>
</tr>
<tr>
<td>10</td>
<td>1.05</td>
<td>1.03</td>
<td>0.99</td>
<td>1.00</td>
</tr>
<tr>
<td>15</td>
<td>1.09</td>
<td>1.07</td>
<td>1.03</td>
<td>1.05</td>
</tr>
<tr>
<td>20</td>
<td>1.12</td>
<td>1.10</td>
<td>1.06</td>
<td>1.07</td>
</tr>
<tr>
<td>30</td>
<td>1.15</td>
<td>1.13</td>
<td>1.09</td>
<td>1.12</td>
</tr>
<tr>
<td>50</td>
<td>1.20</td>
<td>1.18</td>
<td>1.14</td>
<td>1.17</td>
</tr>
<tr>
<td>100</td>
<td>1.26</td>
<td>1.24</td>
<td>1.20</td>
<td>1.24</td>
</tr>
<tr>
<td>150</td>
<td>1.30</td>
<td>1.28</td>
<td>1.24</td>
<td>1.28</td>
</tr>
<tr>
<td>200</td>
<td>1.32</td>
<td>1.30</td>
<td>1.26</td>
<td>1.30</td>
</tr>
<tr>
<td>250</td>
<td>1.34</td>
<td>1.32</td>
<td>1.28</td>
<td>1.32</td>
</tr>
<tr>
<td>300</td>
<td>1.35</td>
<td>1.34</td>
<td>1.30</td>
<td>1.34</td>
</tr>
<tr>
<td>350</td>
<td>1.37</td>
<td>1.35</td>
<td>1.31</td>
<td>1.36</td>
</tr>
<tr>
<td>400</td>
<td>1.38</td>
<td>1.36</td>
<td>1.32</td>
<td>1.37</td>
</tr>
<tr>
<td>450</td>
<td>1.39</td>
<td>1.37</td>
<td>1.33</td>
<td>1.39</td>
</tr>
<tr>
<td>500</td>
<td>1.40</td>
<td>1.38</td>
<td>1.34</td>
<td>1.39</td>
</tr>
</tbody>
</table>

Note:

Class A: Structures and claddings having maximum dimension less than 20m.

Class B: Structures and claddings having maximum dimension between 20m and 50m.

Class C: Structures and claddings having maximum dimension greater than 50m.

Table 7.18a Variation of effective horizontal length of hill and upwind slope \( \theta \)

<table>
<thead>
<tr>
<th>Slope ( \theta )</th>
<th>( L_e )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 3^\circ &lt; \theta \leq 17^\circ )</td>
<td>L</td>
</tr>
<tr>
<td>( \theta &gt; 17^\circ )</td>
<td>( Z/0.3 )</td>
</tr>
</tbody>
</table>

Note: \( L \) is the actual length of the upwind slope in the wind direction, and \( Z \) is the effective height of the feature.
Table 7.18b Variation of factor C with slope $\theta$

<table>
<thead>
<tr>
<th>Slope $\theta$</th>
<th>Factor C</th>
</tr>
</thead>
<tbody>
<tr>
<td>$3^\circ &lt; \theta \leq 17^\circ$</td>
<td>1.2 (Z/L)</td>
</tr>
<tr>
<td>$&gt;17^\circ$</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Example

Calculate the design wind speed for a tower 20m high situated in a well-wooded area (Category 3) and for 100-year probable life near an abrupt escarpment of height 35m (fig 7.17a). The tower is located around Madras. The crest of the escarpment is 10m effective distance from the plains. The tower is located on the downwind side 5m from the crest.

\[
tan \theta = \frac{10}{35} = 0.2857; \, \theta = 15.94^\circ
\]

\[
X = +5 \quad L = 10m \quad H = 20m
\]

\[
X/L = +5/10 = +0.5 \quad H/L = 20/10 = 2
\]

The basic wind speed for Madras = 50m/s

$k_1$ factor for 100-year probable life = 1.08

$k_2$ factor for 20m height for well-wooded area (terrain category 3) (class A)=1.01

$k_3$ factor for topography:

For $X/L = +0.5$ and $H/L = 2$, the $s$ factor from Figure 3.9 is found as $s=0.05$

From Table 3.12b, factor $C = 1.2Z/L = 1.2 \times 20/10 = 2.4$

Therefore, $k_3 = 1 + 0.05 \times 2.4 = 1.12$

Design wind speed $V_Z = V_b \times k_1 \times k_2 \times k_3 = 50 \times 1.08 \times 1.01 \times 1.12 = 61.08$

Note: Values of $k$ factor can be greater than, equal to or less than, one based on the conditions encountered.
Wind force on the structure

The force on a structure or portion of it is given by

\[ F = C_f A_e p_d \]  \hspace{1cm} (7.13)

Where \( C_f \) is the force coefficient,

\( A_e \) is the effective projected area, and

\( p_d \) is the pressure on the surface

The major portion of the wind force on the tower is due to the wind acting on the frames and the conductors and ground wires.

Wind force on single frame

Force coefficients for a single frame having either flat-sided members or circular members are given in Table 7.19 with the following notations:

- \( D \) - diameter
- \( V_d \) - design wind speed
- \( \phi \) - solidity ratio
Wind force on multiple frames

The wind force on the windward frame and any unshielded parts of the other frame is calculated using the coefficients given in Table 7.19. The wind load on parts of the sheltered frame is multiplied by a shielding factor $\psi$, which is dependent upon the solidity ratio of windward frame, the types of members and the spacing ratio. The values of shielding factors are given in Table 7.20.

The spacing ratio $d/h$ (same as aspect ratio $a/b$ for towers) has already been defined in Figure 7.16.

While using Table 7.20 for different types of members, the aerodynamic solidity ratio $\beta$ to be adopted is as follows:

$$\text{Aerodynamic solidity ratio } \beta = \text{solidity ratio } \phi \text{ for flat-sided members.} \quad (7.14)$$

Wind force on lattice towers

Force coefficients for lattice towers of square or equilateral triangle sections with flat-sided members for wind direction against any face are given in Table 7.21.

Force coefficients for lattice towers of square sections with circular members are given in the Table 7.22.
### Table 7.19 Force coefficients for single frames

<table>
<thead>
<tr>
<th>Solidity Ratio $\phi$</th>
<th>Force coefficient $C_f$ for Circular section</th>
<th>Subcritical flow $DV_d &lt; 6m^2/s$</th>
<th>Supercritical flow $DV_d \geq 6m^2/s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat-sided members</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>1.9</td>
<td>1.2</td>
<td>0.7</td>
</tr>
<tr>
<td>0.2</td>
<td>1.8</td>
<td>1.2</td>
<td>0.8</td>
</tr>
<tr>
<td>0.3</td>
<td>1.7</td>
<td>1.2</td>
<td>0.8</td>
</tr>
<tr>
<td>0.4</td>
<td>1.7</td>
<td>1.1</td>
<td>0.8</td>
</tr>
<tr>
<td>0.5</td>
<td>1.6</td>
<td>1.1</td>
<td>0.8</td>
</tr>
<tr>
<td>0.75</td>
<td>1.6</td>
<td>1.5</td>
<td>1.4</td>
</tr>
<tr>
<td>1.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

### Table 7.20 Shielding factors for multiple frames

<table>
<thead>
<tr>
<th>Effective solidity ratio $\beta$</th>
<th>Frame spacing ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>0</td>
<td>1.0</td>
</tr>
<tr>
<td>0.1</td>
<td>0.9</td>
</tr>
<tr>
<td>0.2</td>
<td>0.8</td>
</tr>
<tr>
<td>0.3</td>
<td>0.7</td>
</tr>
<tr>
<td>0.4</td>
<td>0.6</td>
</tr>
<tr>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>0.7</td>
<td>0.3</td>
</tr>
<tr>
<td>1.0</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Note $\beta = \phi$ for flat-sided members

Force coefficients for lattice towers of equilateral-triangular towers composed of circular members are given in the Table 7.23.
The wind load on a square tower can either be calculated using the overall force coefficient for the tower as a whole given in Tables 7.21 to 7.23, using the equation \( F = C_f A_e p_d \), or calculated using the cumulative effect of windward and leeward trusses from the equation

\[
F = C_f (1 + \psi)A_e p_d \quad (7.15)
\]

Tables 7.19 and 7.20 give the values of \( C_f \) and \( \psi \) respectively.

In the case of rectangular towers, the wind force can be calculated based on the cumulative effect of windward and leeward trusses using the equation \( F = C_f (1 + \psi)A_e p_d \), the value of \( C_f \) and \( \psi \) being adopted from 7.19 and 7.20 respectively.

While calculating the surface area of tower face, an increase of 10 percent is made to account for the gusset plates, etc.

**Wind force on conductors and ground wires**

**Table 7.21 Overall force coefficients for towers composed of flat-sided members**

<table>
<thead>
<tr>
<th>Solidity Ratio ( \phi )</th>
<th>Force coefficient for Square towers</th>
<th>Equilateral triangular towers</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>3.8</td>
<td>3.1</td>
</tr>
<tr>
<td>0.2</td>
<td>3.3</td>
<td>2.7</td>
</tr>
<tr>
<td>0.3</td>
<td>2.8</td>
<td>2.3</td>
</tr>
<tr>
<td>0.4</td>
<td>2.3</td>
<td>1.9</td>
</tr>
<tr>
<td>0.5</td>
<td>2.1</td>
<td>1.5</td>
</tr>
</tbody>
</table>
### Tables 7.22 Overall force coefficient for square towers composed of rounded members

<table>
<thead>
<tr>
<th>Solidity ratio of front face $\phi$</th>
<th>Force coefficient for</th>
<th>Subcritical flow $DV_d &lt; 6m^2/s$</th>
<th>Supercritical flow $DV_d \geq 6m^2/s$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>on to face</td>
<td>on to corner</td>
<td>on to face</td>
</tr>
<tr>
<td>0.05</td>
<td>2.4</td>
<td>2.5</td>
<td>1.1</td>
</tr>
<tr>
<td>0.1</td>
<td>2.2</td>
<td>2.3</td>
<td>1.2</td>
</tr>
<tr>
<td>0.2</td>
<td>1.9</td>
<td>2.1</td>
<td>1.3</td>
</tr>
<tr>
<td>0.3</td>
<td>1.7</td>
<td>1.9</td>
<td>1.4</td>
</tr>
<tr>
<td>0.4</td>
<td>1.6</td>
<td>1.9</td>
<td>1.4</td>
</tr>
<tr>
<td>0.5</td>
<td>1.4</td>
<td>1.9</td>
<td>1.4</td>
</tr>
</tbody>
</table>

### Table 7.23 Overall force coefficients for equilateral triangular towers composed of rounded members

<table>
<thead>
<tr>
<th>Solidity ratio of front face $\phi$</th>
<th>Force coefficient for</th>
<th>Subcritical flow $DV_d &lt; 6m^2/s$</th>
<th>Supercritical flow $DV_d \geq 6m^2/s$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.05</td>
<td>1.8</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>1.7</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>0.2</td>
<td>1.6</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>0.3</td>
<td>1.5</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>0.4</td>
<td>1.5</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>1.4</td>
<td>1.2</td>
<td></td>
</tr>
</tbody>
</table>
Table 7.24 Force coefficients for wires and cables (VD > 100)

<table>
<thead>
<tr>
<th>Flow regime</th>
<th>Force coefficient $C_f$ for Smooth surface wire</th>
<th>Force coefficient $C_f$ for Moderately smooth wire (galvanized or painted)</th>
<th>Force coefficient $C_f$ for Fine stranded cables</th>
<th>Force coefficient $C_f$ for Thick stranded cables</th>
</tr>
</thead>
<tbody>
<tr>
<td>$DV_d &lt; 0.6m^2/s$</td>
<td>-</td>
<td>-</td>
<td>1.2</td>
<td>1.3</td>
</tr>
<tr>
<td>$DV_d \geq 0.6m^2/s$</td>
<td>-</td>
<td>-</td>
<td>0.9</td>
<td>1.1</td>
</tr>
<tr>
<td>$DV_d &lt; 6m^2/s$</td>
<td>1.2</td>
<td>1.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$DV_d \geq 6m^2/s$</td>
<td>0.5</td>
<td>0.7</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Force coefficients for conductors and ground wires are given in Table 7.24 according to the diameter (D), the design wind speed ($V_D$), and the surface roughness, D being expressed in metres and $V_D$ in metres/second.

For conductors commonly used in power transmission, $DV_d$ is always less than 0.6$m^2/s$, so that the force coefficient applicable is 1.2 (from the table).

The wind force on the conductor is calculated from the expression $F = C_f A_e p_d$ with the usual notations.

In the case of long-span transmission line conductors, due to the large aspect ratio ($\lambda = L/D$), the average drag per unit length is reduced. In other words, when span are long, the wind pressure on the entire span is not uniform. Besides, the conductor itself is not rigid and swings in the direction of the gusts and therefore the relative velocity is less than the actual gust velocity. Further, under the effect of wind, there is a twisting effect on the conductor and a part of the wind energy is absorbed in the conductor in the process. All these
considerations can be accounted for in a single factor called the ‘space factor’, which varies from 0.7 to 0.85; this factor decreases with increase in wind velocity and span length.

The wind force $F$ on the conductor may not be calculated from the following expressions:

$$F = \alpha \cdot C_f \cdot A_e \cdot p_d$$

Where $\alpha$ is the space factor (0.7 to 0.85) and $C_f, A_e$ and $p_d$ have the usual notations.

**Maximum and minimum temperature charts**

Knowledge of the maximum and the minimum temperatures of the area traversed by a transmission line is necessary for calculating sags and tensions of conductors and ground wires under different loading conditions. The maximum and the minimum temperatures normally vary for different localities under different diurnal and seasonal conditions.

IS: 802 (Part 1)-1977 (Second Revision) gives the absolute maximum and minimum temperatures that are expected to prevail in different areas in the country. The maximum temperature isopleths range from 37.5° to 50.0°C in steps of 2.5° and the minimum temperature isopleths from -7.5° to 17.5° in steps of 2.5°.

The absolute maximum temperature values are increased by 17°C to allow for the sun's radiation, heating effect of current, etc., in the conductor. In
case the temperature-rise curves of conductors are readily available, the actual rise in temperature in the conductor due to the heating effect of current for a given line capacity is read from the curves and added to the absolute maximum-temperature values. To the values thus arrived at, is added the rise in temperature due to sun's radiation which is normally taken as 6° to 7°C for conductors at temperatures below 40°C and 2° to 3°C for conductors at higher temperatures.

**Seismic effects**

The force attracted by a structure during a seismic disturbance is a function of the ground acceleration and the properties of the structure. The seismic disturbance is essentially a dynamic phenomenon, and therefore assuming an equivalent lateral static seismic force to simulate the earth-quake effects is an oversimplification of the problem. However, allover the world, in regions affected by earthquakes, the structures designed based on the equivalent static approach have withstood the earthquake shocks satisfactorily, which justifies the use of this method. The equivalent static method can be derived from first principles from Newton's second law of motion thus:

\[
\text{Seismic lateral force } P = Ma \quad (7.16) a
\]

\[
= (W/g) a = W (a/g) \quad (7.16) b
\]

Where \( M \) = mass of the structure,

\( W \) = weight of the structure,

\( a \) = acceleration experienced by the structure due to earthquake, and

\( g \) = acceleration due to gravity.
This force is dependent on a number of factors, the more important among them being
- stiffness of the structure
- damping characteristics of the structure
- probability of a particular earthquake occurring at a particular site where the structure is located
- importance of the structure based on the consequences of failure
- foundation characteristics.

Incorporating the above variables in the form of coefficients, IS:1893-1975 gives the following formula for the calculation of horizontal equivalent seismic force:

\[ P = \alpha H W \text{ in which } \alpha_H = \beta I F_o \left( \frac{S_a}{g} \right) \]  \hspace{1cm} (7.16c)

\[ = \beta l \alpha_o \]  \hspace{1cm} (7.16d)

Where \( \beta \) = a coefficient depending on the soil- foundation system (Table of the Code),

\( I \) = coefficient depending on the importance of structure (for transmission towers this may be taken as 1.0),

\( F_o \) = seismic zone factor,

\( \left( \frac{S_a}{g} \right) \) = average acceleration coefficient which takes into account the period of vibration of the structure and damping characteristics to be read from Figure 7.20,

\[ \alpha_H = \text{Seismic coefficient, and} \]
\[ \alpha_0 = \text{ad hoc basic seismic coefficient.} \]

Seismic coefficients specified in IS: 1893-1975 are based on a number of simplifying assumptions with regard to the degree of desired safety and the cost of providing adequate earthquake resistance in structures. A maximum value of \( \alpha_0 = 0.08 \) has been adopted in the Code arbitrarily because the practice in Assam before the code was introduced was to design structures for this value, again fixed somewhat arbitrarily. The structures constructed with this seismic coefficient have performed well and withstood the 1950 Assam earthquake (Richter’s Scale Magnitude 8.3).

For transmission line towers, the weight \( W \) of the structure is low in comparison with buildings. The natural period is such that the \((Sa/g)\) value is quite low (See Figure 7.20). Because the mass of the tower is low and the \((Sa/g)\) value is also low, the resultant earthquake force will be quite small compared to the wind force normally considered for Indian conditions. Thus, earthquake seldom becomes a governing design criterion.

Full-scale dynamic tests have been conducted by the Central Research Institute of Electric Power Industry, Tokyo, on a transmission test line. In this study, the natural frequency, mode shape and damping coefficient were obtained separately for the foundation, the tower, and the tower-conductor coupled system. Detailed response calculation of the test line when subjected to a simulated El Centro N-S Wave (a typical earthquake) showed that the tower members could withstand severe earthquakes with instantaneous maximum stress below yield point.
No definite earthquake loads are specified for transmission line towers in the Design Standards on structures for Transmission in Japan, which is frequently subjected to severe earthquakes. The towers for the test line referred to above were designed to resist a lateral load caused by a wind velocity of 40m/sec. The towers of this test line have been found to perform satisfactorily when tested by the simulated earthquake mentioned above. A detailed study based on actual tests and computer analysis carried out in Japan indicates that, generally speaking, transmission towers designed for severe or moderate wind loads would be safe enough against severe earthquake loads.8.9 In exceptional cases, when the towers are designed for low wind velocities, the adequacy of the towers can be checked using the lateral seismic load given by equation (7.16d):

\[ \text{Example: } \]

Let the period of the tower be two seconds and damping five percent critical. Further, the soil-foundation system gives a factor of $\beta = 1.2$ (for isolated
footing) from Table 3 of 18:1893-1975. The importance factor for transmission tower I = 1.00 (as per the Japanese method).

Referring to Figure 7.20, the spectral acceleration coefficient \( (Sa/g) = 0.06 \). Assuming that the tower is located in Assam (Zone V - from Figure 1 of 18:1893-1975 - Seismic Zones of India), the horizontal seismic coefficient

\[
\alpha_H = \beta I F_0 (Sa/g) \\
= 1.2 \times 1 \times 0.4 \times 0.06 \\
= 0.0288
\]

Therefore, the horizontal seismic force for a tower weighing 5,000 kg is

\[
P = \alpha_H W \\
= 0.0288 \times 5,000 \\
= 144 \text{ kg (quite small)}
\]

**Broken-wire conditions**

It is obvious that the greater the number of broken wires for which a particular tower is designed, the more robust and heavier the tower is going to be. On the other hand, the tower designed for less stringent broken-wire conditions will be lighter and consequently more economical. It is clear therefore that a judicious choice of the broken-wire conditions should be made so as to achieve economy consistent with reliability.

The following broken-wire conditions are generally assumed in the design of towers in accordance with 18:802 (Part 1)-1977:
**For voltage. up to 220 kV**

**Single-circuit towers**

It is assumed that either anyone power conductor is broken or one ground wire is broken, whichever constitutes the more stringent condition for a particular member.

**Double-circuit towers**

1. **Tangent tower with suspension strings (0° to 2°):**

   It is assumed that either anyone power conductor is broken or one ground wire is broken, whichever constitutes the more stringent condition for a particular member.

2. **Small angle towers with tension strings (2° to 15°) and medium angle tower with tension strings (15° to 30°):**

   It is assumed that either any two of the power conductors are broken on the same side and on the same span or anyone of the power conductors and anyone ground wire are broken on the same span, whichever combination is more stringent for a particular member.

3. **Large angle (30° to 60°) and dead-end towers with tension strings:**

   It is assumed that either three power conductors are broken on the same side and on the same span or that any two of the power conductors and anyone ground wire are broken on the same span, whichever combination constitutes the most stringent condition for a particular member.
Cross-arms

In all types of towers, the power conductor supports and ground wire supports are designed for the broken-wire conditions.

For 400 kV line.

Single circuit towers (with two sub-conductors per phase)

1. Tangent towers with suspension strings (0° to 2°):
   It is assumed that any ground wire or one sub-conductor from any bundle conductor is broken, whichever is more stringent for a particular member.
   The unbalanced pull due to the sub-conductor being broken may be assumed as equal to 25 percent of the maximum working tension of all the sub-conductors in one bundle.

2. Small angle tension towers (2° to 15°):

3. Medium angle tension towers (15° to 30°):

4. Large angle tension (30° to 60°) and dead-end towers:
   It is assumed that any ground wire is broken or all sub-conductors in the bundle are broken, whichever is more stringent for a particular member.

Double-circuit towers (with two sub-conductors per phase)

1. Tangent towers with suspension strings (0° to 2°):
   It is assumed that all sub-conductors in the bundle are broken or any ground wire is broken, whichever is more stringent for a particular member.
2. Small-angle tension towers (2° to 15°):

3. Medium-angle tension towers (15° to 30°):

   It is assumed that either two phase conductors (each phase comprising two conductors) are broken on the same side and on the same span, or anyone phase and anyone ground wire is broken on the same span, whichever combination is more stringent for a particular member.

4. Large-angle tension (30° to 60°) and Dead-end towers:

   It is assumed that either all the three phases on the same side and on the same span are broken, or two phases and anyone ground wire on the same span is broken, whichever combination is more stringent for a particular member.

± 500 kV HVDC bipole

During the seventh plan period (1985-90), a ±500 kV HVDC bipole line with four subconductors has been planned for construction from Rihand to Delhi (910km). The following broken-wire conditions have been specified for this line:

1. Tangent towers (0°):

   This could take up to 2° with span reduction. It is assumed that either one pole or one ground wire is broken, whichever is more stringent for a particular member.
2. Small-angle towers (0° to 15°):

   It is assumed that either there is breakage of all the subconductors of the bundle in one pole or one ground wire, whichever is more stringent.

   When used as an anti-cascading tower (tension tower for uplift forces) with suspension insulators, all conductors and ground wires are assumed to be broken in one span.

3. Medium-angle towers (15° to 30°):

   It is assumed that either one phase or one ground wire is broken, whichever is more stringent.

4. Large-angle towers 30° to 60° and dead-end towers:

   It is assumed that all conductors and ground wires are broken on one side.

   It would be useful to review briefly the practices regarding the broken-wire conditions assumed in the USSR, where extensive transmission networks at various voltages, both A.C. and D.C., have been developed and considerable experience in the design, construction and operation of networks in widely varying climatic conditions has been acquired.

   For suspension supports, under the conductor broken conditions, the conductors of one phase are assumed to be broken, irrespective of the number of conductors on the support, producing the maximum stresses on the support; and under the ground wire broken condition, one ground wire is assumed to be broken, which produces the maximum stresses with the phase conductors intact.
For anchor supports, any two phase conductors are assumed to be broken (ground wire remaining intact) which produce the maximum stresses on the support, and the ground wire broken conditions (with the conductors intact) are the same as in the case of suspension supports.

The broken-wire conditions specified for a tower also take into account the type of conductor clamps used on the tower. For example, if 'slip' type clamps are used on the line, the towers are not designed for broken-wire conditions, even for 220 kV, 330 kV and 500 kV lines.

The designs of anchor supports are also checked for the erection condition corresponding to only' one circuit being strung in one span, irrespective of the number of circuits on the support, the ground wires being not strung, as well as for the erection condition corresponding to the ground wires being strung in one span of the support, the conductors being not strung. In checking the designs for erection conditions, the temporary strengthening of individual sections of supports and the installation of temporary guys are also taken into account.

In the case of cross-arms, in addition to the weight of man and tackle, the designs are checked up for the loadings corresponding to the method of erection and the additional loadings due to erection devices.

### 7.3.3 Loadings and load combinations

The loads on a transmission line tower consist of three mutually perpendicular systems of loads acting vertical, normal to the direction of the line, and parallel to the direction of the line.
It has been found convenient in practice to standardise the method of listing and dealing with loads as under:

Transverse load
Longitudinal load
Vertical load
Torsional shear
Weight of structure

Each of the above loads is dealt with separately below.

**Transverse load**

The transverse load consists of loads at the points of conductor and ground wire support in a direction parallel to the longitudinal axis of the cross-arms, plus a load distributed over the transverse face of the structure due to wind on the tower (Figure 7.21).

---

**Figure 7.21 Loadings on tower**
Transverse load due to wind on conductors and ground wire

The conductor and ground wire support point loads are made up of the following components:

1. Wind on the bare (or ice-covered) conductor/ground wire over the wind span and wind on insulator string.

2. Angular component of line tension due to an angle in the line (Figure 7.22).

The wind span is the sum of the two half spans adjacent to the support under consideration. The governing direction of wind on conductors for an angle condition is assumed to be parallel to the longitudinal axis of the cross-arms (Figure 7.23). Since the wind is blowing on reduced front, it could be argued that this reduced span should be used for the wind span. In practice, however, since the reduction in load would be relatively small, it is usual to employ the full span.

In so far as twin-conductor bundle in horizontal position (used for lines at 400 kV) is concerned, it has been found that the first sub-conductor in each phase does not provide any shielding to the second sub-conductor. Accordingly, the total wind load for bundled conductors is assumed as the sum total of wind load on each sub-conductor in the bundle.

Under broken-wire conditions, 50 percent of the nonnal span and 10 percent of the broken span is assumed as wind span.
Wind load on conductor

Wind load on conductors and ground wire along with their own weight produces a resultant force, which is calculated as follows. The calculation covers the general case of an ice-coated conductor:

Let \( d \) be the diameter of conductor in mm and \( t \) the thickness of ice coating in mm (Figure 7.24).

Then, weight of ice coating on 1-metre length of conductor,

\[
W_1 = \frac{\pi}{4} \left[ (d + 2t)^2 - d^2 \right] x \frac{1}{10^6} x 900 \text{kg} \quad (7.17a)
\]

(Ice is assumed to weigh 900 kg/m\(^3\))

Weight per metre length of ice-coated conductor

\( W = w + w_1 \)

Where \( w \) = weight of bare conductor per metre length, and

\( w_1 \) = weight of ice coating per metre length.

Horizontal wind load on ice-coated conductor per metre length,

\[
P = \frac{2}{3} x \frac{(d + 2t)}{1000} p \text{kg} \quad (7.17b)
\]

Where \( p \) = wind pressure in kg/m\(^2\) on two-thirds the projected area of conductor.
Figure 7.22 Transverse load on the cross-arm due to line deviation

Resultant force per metre length, $R$

$$R = \sqrt{W^2 + P^2} \quad (7.17c)$$

**Wind load on insulator string**

The wind load the insulator string is calculated by multiplying the wind pressure assumed on the towers and the effective area of the insulator string. It is usual to assume 50 percent of the projected area of the insulator string as the effective area for purposes of computing wind load on insulator strings. The projected area of insulator string is taken as the product of the diameter of the insulator disc and the length of the insulator string. The total wind load on the insulator strings used for 66 kV to 400 kV transmission lines worked out for a wind pressure of 100 kg/m² is given in Table 7.25. The Table also gives the approximate weight of the insulator string normally used.
Transverse load due to line deviation

The load due to an angle of deviation in the line is computed by finding the resultant force produced by the conductor tensions (Figure 7.22) in the two adjacent spans.

It is clear from the figure that the total transverse load = \( 2TS\sin\theta/2 \) where \( \theta \) is the angle of deviation and \( T \) is the conductor tension.

Any tower type designed for a given line angle has a certain amount of flexibility of application. The range of angles possible with their corresponding spans are shown on a Span-Angle Diagram, the construction of which is given later.

Wind on tower

To calculate the effect of wind on tower, the exact procedure would be to transfer the wind on tower to all the panel points. This would, however, involve a number of laborious and complicated calculations. An easier assumption would
be to transfer the equivalent loads on the conductor and ground wire supports that are already subjected to certain other vertical, transverse and longitudinal loads.

The wind load on towers is usually converted, for convenience in calculating and testing, into concentrated loads acting at the point of conductor and ground wire supports. This equivalent wind per point is added to the above component loads in arriving at the total load per support point.

Calculation of wind load on towers is made on the basis of an assumed outline diagram and lattice pattern prepared, from considerations of loadings and various other factors. Adjustments, if required, are carried out after the completion of preliminary designs and before arriving at final designs.

Table 7.25 Wind load on insulator strings (disc size: 255 x 146 mm)

<table>
<thead>
<tr>
<th>Voltage (kV)</th>
<th>Length of suspension insulator string (cm)</th>
<th>Diameter of each disc (cm)</th>
<th>Projected area of the string (Cot.2 x Cot.3) (sq.m)</th>
<th>Effective area for wind load (assumed 50% of Cot.4)(m²)</th>
<th>Computed wind load on insulator string in kg for wind pressure of 100kg/m²</th>
<th>Weight of insulator string (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>66</td>
<td>107</td>
<td>25.5</td>
<td>0.2718</td>
<td>0.1359</td>
<td>13.6</td>
<td>31.8</td>
</tr>
<tr>
<td>132</td>
<td>168</td>
<td>25.5</td>
<td>0.4267</td>
<td>0.2134</td>
<td>21.4</td>
<td>57.1</td>
</tr>
<tr>
<td>220</td>
<td>265</td>
<td>25.5</td>
<td>0.6731</td>
<td>0.3366</td>
<td>33.7</td>
<td>122.5</td>
</tr>
<tr>
<td>400</td>
<td>415</td>
<td>25.5</td>
<td>1.0541</td>
<td>0.5271</td>
<td>52.7</td>
<td>200.0</td>
</tr>
</tbody>
</table>
The wind load is assumed to be applied horizontally, acting in a direction normal to the transmission line.

The projected area is an unknown quantity until the actual sections are known. Therefore, it is necessary to make an assumption in order to arrive at the total wind load on the structure. Experience has shown that the net area of the tower lies between 15 and 25 percent of the gross area, depending on the spread and size of the structure. The gross area in turn is the area bounded by the outside perimeter of the tower face. For towers approximately 60m in height or higher, it will be found that the ratio of net area to gross area is much smaller at the bottom of the tower than at the top. This variation should be taken into consideration in calculating the wind load.

The projected area $A$ on which the wind acts is computed by considering one face only. For accounting the wind force on the leeward face, a factor of 1.5
is used in accordance with the relevant provision of the Indian Electricity Rules, 1956. The wind load on the tower, for the purpose of analysis, is assumed to act at selected points, generally at the cross-arm and also at the waist in the case of corset type towers. One of the following methods is adopted to determine the magnitude of loads applicable at the aforesaid selected points. Figure 7.25 gives the framework of a tower with reference to which the methods are explained.

**Method 1**

The wind loads are first calculated for various members or parts of the tower. Thereafter, the moments of all these loads taken about the tower base are added together. The total load moment so obtained is replaced by an equivalent moment assuming that equal loads are applied at the selected points.

**Method 2**

The loads applied on the bottom cross-arms are increased with corresponding reduction in the loads applied on the upper cross-arms.
**Method 3**

The tower is first divided into a number of parts corresponding to the ground wire and conductor support points. The wind load on each point is then calculated based on solidity ratio; the moment of this wind load about the base is divided by the corresponding height which gives the wind load on two points of the support in the double circuit tower shown.

**Method 4**

The equivalent loads are applied at a number of points or levels such as:

1. ground wire peak
2. all cross-arm points
3. waist level (also portal base level if desired) in the case of corset type towers.

The wind loads on different parts of the tower are determined by choosing an appropriate solidity ratio. Out of the load on each part, an equivalent part (that is, a part load which produces an equal moment at the base of that part) is transferred to the upper loading point and the remaining part to the base. This process is repeated for the various parts of the tower from the top downwards.

It can be seen that the load distribution in Method 4 is based on a logical approach in which importance is given not only to moment equivalence but also to shear equivalence at the base. Thus Method 4 is considered to be superior to others. A typical wind load calculation based on this method is given in Figure 7.26. Table 7.26 compares the wind loads arrived at by the four methods. Although the design wind load based on method 4 is higher than that in the other-three methods, it is still lower than the actual load (2,940kg).

A realistic approach is to apply the wind load at each node of the tower. This is practically impossible when calculations are done manually. However, while this could be handled quite satisfactorily in a computer analysis, the representation of wind load in prototype tower tests poses problems. Therefore, the current practice is to adopt Method 4 in computer analysis and design, which are also being validated by prototype tests. Further research is needed for satisfactory representation of forces due to wind on tower during tests if the actual wind load which can be accounted for in computer analysis is to be simulated.
### Table 7.26 Comparison of various methods of wind load computations

<table>
<thead>
<tr>
<th>Tower details</th>
<th>Method 1</th>
<th>Method 2</th>
<th>Method 3</th>
<th>Method 4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load in Kg</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P1</td>
<td>0</td>
<td>0</td>
<td>205</td>
<td>90</td>
</tr>
<tr>
<td>P2</td>
<td>253</td>
<td>200</td>
<td>175</td>
<td>108</td>
</tr>
<tr>
<td>P3</td>
<td>253</td>
<td>200</td>
<td>195</td>
<td>135</td>
</tr>
<tr>
<td>P4</td>
<td>253</td>
<td>400</td>
<td>305</td>
<td>626</td>
</tr>
<tr>
<td>Total</td>
<td>1,518</td>
<td>1,600</td>
<td>1,555</td>
<td>1,828</td>
</tr>
<tr>
<td>Actual wind load</td>
<td>2,940</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Longitudinal load

Longitudinal load acts on the tower in a direction parallel to the line (Figure 7.21b) and is caused by unequal conductor tensions acting on the tower. This unequal tension in the conductors may be due to dead-ending of the tower, broken conductors, unequal spans, etc., and its effect on the tower is to subject the tower to an overturning moment, torsion, or a combination of both. In the case of dead-end tower or a tower with tension strings with a broken wire, the full tension in the conductor will act as a longitudinal load, whereas in the case of a tower with suspension strings, the tension in the conductor is reduced to a certain extent under broken-wire condition as the string swings away from the broken span and this results in a reduced tension in the conductor and correspondingly a reduced longitudinal load on the tower.

The question then arises as to how much reduction in the longitudinal load should be allowed in the design of suspension towers to account for the swing of the insulator string towards the unbroken span under broken-wire conditions.
The general practice followed in India is to assume the unbalanced pull due to a broken conductor as equal to 50 percent of the maximum working tension of the conductor.

In this practice, as in the practices of other countries, the longitudinal load is somewhat arbitrarily fixed in the tower design. However, it is now possible through computer programs to calculate the actual longitudinal loads during the construction of the line, taking into account the effective span lengths of the section (between angle towers), the positioning of insulator strings, and the resulting deformations of supports, thus enabling a check on the proper choice of supports.

For the ground wire broken condition, 100 percent of the maximum working tension is considered for design purposes.

The unbalanced pull due to a broken conductor/ground wire in the case of tension strings is assumed equal to the component of the maximum working tension of the conductor or the ground wire, as the case may be, in the longitudinal direction along with its components in the transverse direction. This is taken for the maximum as well as the minimum angle of deviation for which the tower is designed and the condition, which is most stringent for a member, is adopted. The forces due to impact, which arises due to breaking, are assumed to be covered by the factor of safety allowed in the designs.

When there is a possibility of the tower being used with a longer span by reducing the angle of line deviation, the tower member should also be checked.
for longitudinal and transverse components arising out of the reduced angle of line deviation.

**Vertical load**

Vertical load is applied to the ends of the cross-arms and on the ground wire peak (Figure 7.21c) and consists of the following vertical downward components:

1. Weight of bare or ice-covered conductor, as specified, over the governing weight span.
2. Weight of insulators, hardware, etc., covered with ice, if applicable.
3. Arbitrary load to provide for the weight of a man with tools.

In addition to the above downward loads, any tower, which will be subjected to uplift, must have an upward load applied to the conductor support points. While the first two components can be evaluated quite accurately, a provision of 150 kg is generally made for the weight of a lineman with tools (80 kg for the weight of man and 70 kg for tools).

Another uncertain factor that arises is the extra load to be allowed in the design over and above the normal vertical load, to enable the tower to be used with weight spans larger than the normal spans for which it is designed (in other words, the choice of a suitable weight span for which the tower is to be designed). It is not possible for the designer to make an assumption regarding the weight span unless he has a fairly accurate knowledge of the terrain over which the line has to pass; and therefore the economic weight span will be different for different types of terrain.
An allowance of 50 percent over the normal vertical load is considered to be quite adequate to cover the eventuality of some of the towers being used with spans larger than the normal spans. This slight increase in the design vertical load will not affect the line economy to an appreciable extent, as the contribution of vertical loads towards the total load on the tower members is small. However, where the lines have to run through hilly and rugged terrain, a higher provision is made, depending on the nature of the terrain. The Canadian practice usually makes an allowance of 100 percent over the normal vertical load; this large allowance is probably due to the rugged and hilly terrain encountered in the country. It should be noted that, for the design of uplift foundations and calculation of tensile stress in corner legs and also in some members of the structure, the worst condition for the design is that corresponding to the minimum weight span.

**Weight of structure**

The weight of the structure, like the wind on the structure, is an unknown quantity until the actual design is complete. However, in the design of towers, an assumption has to be made regarding the dead weight of towers. The weight will no doubt depend on the bracing arrangement to be adopted, the strut formula to be used and the quality or qualities of steel used, whether the design is a composite one comprising both mild steel and high tensile steel or makes use of mild steel only. However, as a rough approximation, it is possible to estimate the probable tower weight from knowledge of the positions of conductors and ground wire above ground level and the overturning moments. Ryle has evolved an empirical formula giving the approximate weight of any tower in terms of its height and maximum working over-turning moment at the base. The tower weight is represented by
\[ W = KH \sqrt{M} \quad (7.18) \]

Where \( W \) = weight of tower above ground level in kg,

\( H \) = overall height of the tower above ground level in metres,

\( M \) = overturning moment at ground level, in kg m (working loads), and

\( K \) = a constant which varies within a range of 0.3970 to 0.8223.

The towers investigated covered ranges of about 16 to 1 in height, 3,000 to 1 in overturning moment, and 1,200 to 1 in tower weight.

A reliable average figure for tower weight may be taken as 0.4535 \( H \sqrt{M} \) kg, for nearly all the towers studied have weights between 0.3970 \( H \sqrt{M} \) and 0.5103 \( H \sqrt{M} \) tonnes. Ryle points out that any ordinary transmission line tower (with vertical or triangular configuration of conductors) giving a weight of less than, say, 0.3686 \( H \sqrt{M} \) may be considered inadequate in design and that any tower weighing more than, say, 0.567 \( H \sqrt{M} \) must be of uneconomic design.

In the case of towers with horizontal configuration of conductors, the coefficient \( K \) lies in the range of 0.5103 to 0.6748. Ryle recommends that the average weight of such towers may be represented by 0.6238\( H \sqrt{M} \) kg.

Values of \( K \) for heavy angle towers tend to be less than for straight-line towers. This is due to the fact that on a tower with a wider base-angle it is easier to direct the leg lines towards the load centre of gravity. Values of \( K \) tend to be higher the larger the proportion of the tower represented by cross-arms or ‘top hamper’.

The weight of a river-crossing suspension tower with normal cross-arms lies between 0.4820 \( H \sqrt{M} \) for towers of about 35 metres in height, and 0.7088
H \sqrt{M} \text{ for very tall towers of height 145 metres. Towers with special 'top-hamper' or long-span terminal-type towers may be 10-20 percent heavier.}

The tower weights given by these formulae are sufficiently accurate for preliminary estimates. The formulae are also found to be extremely useful in determining the economic span length and general line estimates including supply, transport and erection of the tower.

It is obvious that, when the height of the upper ground wire is raised, the conductors being kept at the same height, the weight of the tower does not increase in proportion to the height of the ground wire alone. Taking this factor into consideration, Ailleret has proposed the presentation of Ryle's formula in the form $PH$. While this is more logical, Ryle's formula is simpler, and for general estimating purposes, sufficiently accurate.

In Ryle's formula, safe external loads acting on a tower are used as against ultimate loads generally adopted for design as per IS:802(Part 1)- 1977. Since the factor of safety applied for normal conditions is 2.0, the Ryle's equation is not directly applicable if the tower weight is calculated based on loads determined as per the above Code. In this case the following formulae are applicable.

For suspension towers,

$$W = 0.1993 \ H \sqrt{M} + 495 \quad (7.19)$$

For angle towers

$$W = 0.2083 \ H \sqrt{M} + 400 \quad (7.20)$$
Since there is no appreciable difference in the above two equations, the common equation given below may be used for both the tower types:

\[ W = 0.205 \, H \sqrt{M} + 450 \quad (7.21) \]

A more detailed evaluation of tower weight due to Walter Bllckner is presented below. This is based on the principle of summing up the minimum weight of struts panel by panel.

The minimum weight of a single strut is given by

\[ w = Ah \gamma = \frac{P}{\sigma_K} l \gamma \quad (7.22) \]

Where \( A \) is the cross-section of strut,

\( l \) is the unsupported length,

\( \gamma \) is the density, and

\( P \) is the compression load on strut

For a given compression load \( P \) and unsupported length \( l \), the lightest angle section is that which permits the highest crippling stress \( \sigma_K \). For geometrically similar sections

\[ \sigma_K = \left( C \sqrt{P} \right) / l \quad (7.23) \]

Where \( C \) is a constant

Taking the factor \( \left( \sqrt{P} \right) / l \) as a reference, the crippling stresses of all geometrically similar sections for any compression loadings and strut lengths can be plotted as curves, which enable the characteristics of the various sections to be clearly visualised.
At the higher values of \(\sqrt{P}/l\), high-tensile steels (for example, St 52) are economical. This applies especially with staggered strutting, in which the maximum moment of inertia is utilised.

The theoretical minimum weight of the complete tower \(w_m\) is given by the sum of the weights of the members according to equation (7.22). The weight \(g_m\) per metre of tower height for one panel is

\[
g_m = \frac{\gamma}{I_E} \sum \frac{IP}{\sigma_K} + g_z \quad (7.24)
\]

Where \(n = \) number of members,

\(P = \) truss force,

\(g_z = \) additional weight of bolts, etc. per metre of tower height, and

\(I_E = \) equivalent height of panel.

If this expression is integrated from \(x = 0\) to \(x = h\), we get the weight of the tower body \(G_T\) with leg members of St 52 roughly in kilograms without cross-arms, etc:

\[
G_T = Q \left( \frac{h^2}{2b_0} + \frac{Q}{\Delta_b} \right) + 4.4 \sqrt{\frac{M_T}{\Delta_b}} \left( b_m^{3/2} - b_0^{3/2} \right) \quad (7.25)
\]

Where \(h = \) height of tower from the top crossarm to ground level in metres,

\(Q = M_{b_{\text{max}}}/h; M_b = \) maximum normal moment at ground level \((x = h)\) in tonne-metres,

\(b_0 = \) tower width at the top cross-arm \((x = 0)\) in metres,

\(b_m = \) tower width at ground level \((x = h)\) in metres,

\(\Delta_b = \) taper in metres/metre, and
\[ M_T = \text{torsion under abnormal loading in tonne-metres.} \]

**Weights of typical towers used in India**

The weights of various types of towers used on transmission lines, 66 kV to 400 kV, together with the spans and sizes of conductor and ground wire used on the lines, are given in Table 7.27. Assuming that 80 percent are tangent towers, 15 percent 30° towers and 5 percent 60° towers and dead-end towers, and allowing 15 percent extra for extensions and stubs, the weights of towers for a 10 km line are also given in the Table.

**Table 7.27 Weights of towers used on various voltage categories in India**

<table>
<thead>
<tr>
<th>Span (m)</th>
<th>400kV Single circuit</th>
<th>220kV Double circuit</th>
<th>220kV Single circuit</th>
<th>132kV Double circuit</th>
<th>132kV Single circuit</th>
<th>66kV Double circuit</th>
<th>66kV Single circuit</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>320</td>
<td>320</td>
<td>320</td>
<td>320</td>
<td>320</td>
<td>245</td>
<td>245</td>
</tr>
<tr>
<td>Conductor:</td>
<td>Moose 54/3.53mm Al. + 7/3.53mm St</td>
<td>Zebra 54/3.18mm Al + 7/3.8mm St</td>
<td>Zebra 54/3.18mm Al + 7/3.8mm St</td>
<td>Panther 30/3mm Al + 7/3mm St</td>
<td>Panther 30/3mm Al + 7/3mm St</td>
<td>Dog 6/4.72mm Al + 7/1.57mm St</td>
<td>Dog 6/4.72mm Al + 7/1.57mm St</td>
</tr>
<tr>
<td>Groundwire:</td>
<td>7/4mm 110kgf/mm² quality</td>
<td>7/3.15mm 110kgf/mm² quality</td>
<td>7/3.15mm 110kgf/mm² quality</td>
<td>7/3.15mm 110kgf/mm² quality</td>
<td>7/3.15mm 110kgf/mm² quality</td>
<td>7/2.5mm 110kgf/mm² quality</td>
<td>7/2.5mm 110kgf/mm² quality</td>
</tr>
<tr>
<td>Tangent Tower</td>
<td>7.7</td>
<td>4.5</td>
<td>3.0</td>
<td>2.8</td>
<td>1.7</td>
<td>1.2</td>
<td>0.8</td>
</tr>
<tr>
<td>30° Tower</td>
<td>15.8</td>
<td>9.3</td>
<td>6.2</td>
<td>5.9</td>
<td>3.5</td>
<td>2.3</td>
<td>1.5</td>
</tr>
<tr>
<td>60° and Dead-end Tower</td>
<td>23.16</td>
<td>13.4</td>
<td>9.2</td>
<td>8.3</td>
<td>4.9</td>
<td>3.2</td>
<td>2.0</td>
</tr>
<tr>
<td>Weight of towers for a 10-km line</td>
<td>279</td>
<td>202</td>
<td>135</td>
<td>126</td>
<td>76</td>
<td>72</td>
<td>48</td>
</tr>
</tbody>
</table>

Having arrived at an estimate of the total weight of the tower, the estimated tower weight is approximately distributed between the panels. Upon completion of the design and estimation of the tower weight, the assumed weight used in the load calculation should be reviewed. Particular attention should be
paid to the footing reactions, since an estimated weight, which is too high, will make the uplift footing reaction too low.

**Table 7.28 Various load combinations under the normal and broken-wire conditions for a typical 400kV line**

<table>
<thead>
<tr>
<th>Tower type</th>
<th>Longitudinal loads</th>
<th>Transverse loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Normal Condition</td>
<td>Broken-wire condition</td>
</tr>
<tr>
<td></td>
<td>Normal Condition</td>
<td>Broken-wire condition</td>
</tr>
<tr>
<td>A</td>
<td>0.0</td>
<td>0.5 x T x Cos $\phi/2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WC + WI + D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6WC + WI + 0.5DA</td>
</tr>
<tr>
<td>B</td>
<td>0.0</td>
<td>1.0 x MT x Cos $\phi/2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WC + WI + D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6WC + WI + 0.5D</td>
</tr>
<tr>
<td>B Section Tower</td>
<td>0.0</td>
<td>1.0 x MT</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WC + WI + 0.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6WC + WI + 0.0</td>
</tr>
<tr>
<td>C</td>
<td>0.0</td>
<td>1.0 x MT x Cos $\phi/2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WC + WI + D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6WC + WI + 0.0</td>
</tr>
<tr>
<td>C Section Tower</td>
<td>0.0</td>
<td>1.0 x MT</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WC + WI + 0.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6WC + WI + 0.0</td>
</tr>
<tr>
<td>D 60°</td>
<td>0.0</td>
<td>1.0 x MT x Cos $\phi/2$</td>
</tr>
<tr>
<td>Dead-end with slack span (slack span side broken)</td>
<td>0.7 MT</td>
<td>1.0 x MT</td>
</tr>
<tr>
<td>Dead-end slack span side broken</td>
<td>0.7 MT</td>
<td>0.3 MT x Cos 15°</td>
</tr>
<tr>
<td>Dead-end</td>
<td>MT</td>
<td>Nil</td>
</tr>
</tbody>
</table>

**Load combinations**

The various loads coming on the tower under the normal and broken-wire conditions (BWC) have been discussed. An appropriate combination of the various loads under the two conditions should be considered for design purposes. Table 7.28 gives a summary of the various load combinations under the two conditions for a typical 400 kV transmission line using a twin-conductor bundle. The following notations have been used in the Table.

Tension at 32°C without wind = $T$

Maximum tension = $MT$
Wind on conductor = WC
Wind on insulator = WI
Angle of deviation = $\phi$

Load due to deviation of 'A' type tower under BWC = $DA = 2 \times T \times \sin\left(\frac{\phi}{2}\right)$
Load due to deviation for others = $D = 2 \times MT \times \sin\left(\frac{\phi}{2}\right)$

The vertical loads due to conductors and ground wire are based on the appropriate weight spans; these are in addition to the dead weight of the structure, insulators and fittings.

**Example:**

Calculation of tower loading for a typical 132 kV double circuit line.

**Basic data**

1. Type of tower: Tangent tower with 2 degrees line deviation
2. Normal span: 335 m
2. Wind pressure
   
   a. Tower (on 1 1/2 times the exposed area of one face): 200 kg/m²
   b. Conductors and ground wire (on fully projected area): 45 kg/m²

**Characteristics of conductor**

1. Size conforming to : 30/3.00mmAl+7/3.00mm
   IS:398-1961    St ACSR
2. Overall diameter of the conductor : 21 mm

3. Area of the complete conductor : 26.2 mm²

4. Ultimate tensile strength : 9,127 kg

5. Weight : 976 kg/m

6. Maximum working tension : 3,800 kg (say)

**Characteristics of ground wire**

1. Size conforming to : 7/3.15 mm galvanised IS: 2141-1968 Stranded steel wire of 110 kgf/mm² quality

2. Diameter : 9.45 mm

3. Area of complete ground wire : 54.5 mm²

4. Ultimate tensile strength : 5,710 kg
5. Weight : 428 kg/km

6. Maximum working tension : 2,500 kg (say)

**Tower loadings:**

1. Transverse load

For the purpose of calculating the wind load on conductor and ground wire, the wind span has been assumed as normal span.

   a. Wind load on conductor (Normal condition) = \(\frac{335 \times 45 \times 21}{1,000}\) = 317 kg

   Wind load on conductor (broken-wire condition) = 0.6 \times 317 = 190 kg

   b. Wind load on ground wire (Normal condition) = \(\frac{335 \times 9.45 \times 45}{1,000}\) = 142 kg

   Wind load on ground wire (Broken-wire condition) = 0.6 \times 142 = 85 kg

   c. Wind load on tower

   The details in regard to the method of calculating the equivalent wind load on tower (We) are given in Figure 6.26.

   d. Wind load on insulator string.

   Diameter of the insulator skirt = 254 mm

   Length of the insulator string with arcing horns = 2,000 mm
Projected area of the cylinder with diameter equal to that of the insulator skirt
= 2,000 x 254 sq. mm
= 0.508 sq. m.

Net effective projected area of the insulator string exposed to wind = 50 percent of 0.508 sq.m.
= 0.254 sq.m.

Wind load on insulator string = 200 x 0.254
= 50.8 kg
Say 50 kg

e. Transverse component of the maximum working tension (deviation load)

(1) For power conductor = 2 x sin 10 x 3,800 kg
= 133 kg
(2) For ground wire = 2 x sin 10 x 2,500 kg
= 87 kg

f. Deviation loads under the broken-wire condition

(1) Conductor point = 3,800 x sin 1° x 0.5
= 33 kg
(2) Ground wire point = 2,500 x sin 1°
= 44 kg
2. Longitudinal load

Longitudinal load under broken-conductor condition = \(3,800 \times \cos 1^\circ \times 0.5\)

\[= 1,900 \text{ kg}\]

3. Vertical load

For the purpose of calculating vertical loads, the weight span has been considered equal to 1 1/2 times the normal span.
At conductor point
Weight of conductor per weight span = 335 x 1.5 x 0.976 = 490 kg
Weight of insulator string including hardware = 60 kg
Weight of a lineman with tools = 150 kg
Total vertical load at one conductor point = 490 + 60 + 150 = 700 kg

At ground wire point
Weight of ground wire per weight span = 335 x 1.5 x 0.428 = 215 kg
Weight of ground wire attachment = 20 kg
Weight of a lineman with tools = 150 kg
Total vertical load at ground wire point = 215 + 20 + 150
= 385 kg
Say 390 kg

Vertical loads under broken-wire conditions
Vertical load under conductor-broken condition = (0.6 x 490) + 60 + 150
= 504 kg
Say 500 kg

Vertical load under ground wire broken condition = (0.6 x 215) + 20 + 150
= 299 kg

4. Torsional shear

Torsional shear per face at the top conductor position = 1,900 x 3.5 / 2 x 1.75 = 1,900 kg
where 3.5m is the distance between the conductor point of suspension and the centre line of the structure and 1.75 m is the width of the tower at top conductor level.
Table 7.29 Tower Loading (kg) per conductor/ground wire point

<table>
<thead>
<tr>
<th>Details</th>
<th>Conductor Normal condition</th>
<th>Broken-wire condition</th>
<th>Ground wire Normal condition</th>
<th>Broken-wire condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. For tower design</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Due to wind on conductors</td>
<td>317</td>
<td>190</td>
<td>142</td>
<td>85</td>
</tr>
<tr>
<td>Due to deviation</td>
<td>133</td>
<td>33</td>
<td>87</td>
<td>44</td>
</tr>
<tr>
<td>Equivalent wind on tower</td>
<td>We</td>
<td>We</td>
<td>We</td>
<td>We</td>
</tr>
<tr>
<td>Wind load on insulator string</td>
<td>50</td>
<td>50</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Transverse load (Total)</td>
<td>500 + We</td>
<td>273 + We</td>
<td>229 + We</td>
<td>129 + We</td>
</tr>
<tr>
<td>Transverse load (rounded)</td>
<td>500 + We</td>
<td>280 + We</td>
<td>230 + We</td>
<td>130 + We</td>
</tr>
<tr>
<td>Vertical load</td>
<td>700</td>
<td>500</td>
<td>390</td>
<td>299</td>
</tr>
<tr>
<td>Longitudinal load</td>
<td>-</td>
<td>1,900</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2. For cross-arm design</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse load</td>
<td>Same as for tower design</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical load</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal load</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: The loads indicated in Figure3.18 are half the above loads as they represent loads on one face of the tower.

1) Loads shown are in kg, per conductor/ground wire point per face of the tower.
2) Similar loading diagrams are to be drawn for other broken-wire conditions also.
5. Dead weight of the structure = \(W_s\) up to the point where stresses are being computed is considered.

The tower loading per conductor/ground wire point is summarised in Table 7.27. The loading diagram is given in Figure 7.27.

**Span-angle diagram**

The load imposed on a tower by one conductor can be considered in terms of the two components

\[ P_1 = T \sin \frac{\theta}{2} \quad \text{and} \quad P_2 = T \cos \frac{\theta}{2} \quad (7.26) \]

Where, as in Figure 7.16, \(T\) is the conductor tension and \(\theta\) is the line angle. If \(\theta\) increases, \(P_2\) decreases and correspondingly decreases its effect on the tower, but \(P_1\) and its effect increase. Thus, to allow for an increase in the line angle, the effect of the increase in \(P_1\) must be offset. As \(P_1\) combines with the load due to wind on the conductor, it is logical, to reduce the wind load by reducing the span. Therefore, a tower designed for a normal span \(L_n\), line angle \(\theta_n\), unit wind load per unit length of conductor \(W_h\) and conductor tension \(T\), can be used with a new span length \(L\) and line angle \(\theta\), if

\[ W_h L_n + 2T \sin \frac{\theta_n}{2} = W_h L + 2T \sin \frac{\theta}{2} \quad (7.27) \]

This equation relating \(L\) and \(\theta\), being of the first order, represents a straight line and therefore is readily plotted as a Span-angle diagram as illustrated below.
Example

Construct a span-angle diagram for the single circuit suspension tower.

Normal span - 300 metres with 3° angle Conductor:

Tension = 4,080 kg
Unit wind load = 1.83 kg/metre

Ground wire:

Tension = 2,495 kg
Unit wind load = 1.364 kg/metre

The equation (3.38) relating \( L \) and \( \theta \) for conductor

\[
1.83L + 2 \times 4,080 \sin \frac{\theta}{2} = 1.83 \times 300 + 2 \times 4,080 \times 0.02618
\]

\[= 762.63\]

For \( \theta = 0 \), \( L = \) maximum tangent span

\[= 762.63 / 1.83 = 416.7 \text{ say, 420 metres.}\]

If the minimum span length required = 150 metres,

\[
8,160 \sin \frac{\theta}{2} = 762.63 - 1.83 \times 150
\]

\[\sin \frac{\theta}{2} = \frac{488.13}{8,160} = 0.05982\]

\[\theta = 6^\circ 52'\]

Similarly for the ground wire:

With \( \theta = 0 \), \( L = 396 \) metres, say, 400 metres.

If the minimum span length required is 150 metres, as before, \( \theta = 7^\circ 42'\)

As the load imposed on the tower by the conductors is greater than that by the ground wire, and the above results are similar, the values derived for the conductor are assumed in drawing the span-angle diagram of Figure 7.19.
Figure 7.28 Span angle diagram