Course 22. Design of Steel Structures II (Web Course)

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1. BEAM – COLUMN CONNECTIONS

1.1 Introduction:

Beam-to-column connections are neither ideally pinned nor ideally fixed and posses a finite non-zero stiffness. However they are classified as simple (pinned), semi-rigid and rigid (fixed) depending on the connection stiffness (Fig. 1.1). Such a classification helps in simplifying the analysis of frames. A connection having a small stiffness can be assumed as pinned while a connection having a large stiffness can be assumed as fixed. In the former case, the actual mid-span bending moments will be less than what is designed for while in the latter case the mid-span deflection will be more than what is calculated. Traditionally, certain configurations are idealized as pinned and certain other configurations are idealized as fixed but with a variety of new configurations being used it is important to have guideline indicating the range of stiffness for which the idealization can be used without serious discrepancy between analysis and actual behaviour. This is done by means of connection classification.

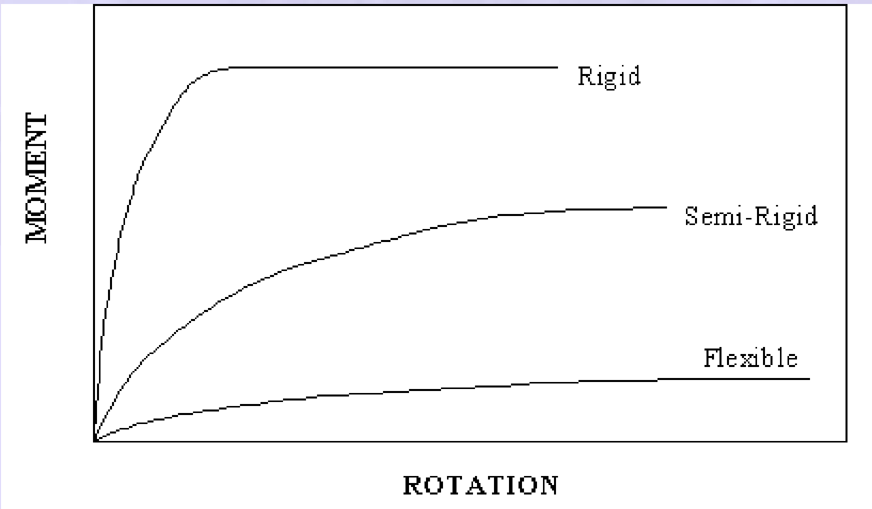


Fig. 1.1 Moment-rotation relationships for connections

1.1.1 Connection classification:

The Classification proposed by Bjorhovde et al. (1990) is recommended by the IS 800 code and is explained here. Connections are classified according to their ultimate strength or in terms of their initial elastic stiffness. The classification is based on the non-dimensional moment parameter ($m^1 = M_u / M_{pb}$) and the non-dimensional rotation ($q^1 = q_r / q_p$) parameter, where q_p is the plastic rotation. The Bjorhovde's classification is based on a reference length of the beam equal to 5 times the depth of the beam. The limits used for connection classification are shown in Table.1.1 and are graphically represented in Fig .1.1

Table.1.1 Connection classification limits: In terms of strength

Nature of the connection	In terms of strength	In terms of Stiffness
Rigid connection	$m^1 \geq 0.7$	$m^1 \geq 2.5\theta^1$
Semi-Rigid connection	$0.7 > m^1 > 0.2$	$2.5\theta^1 > m^1 > 0.5\theta^1$
Flexible connection	$m^1 \leq 0.2$	$m^1 \leq 0.5\theta^1$

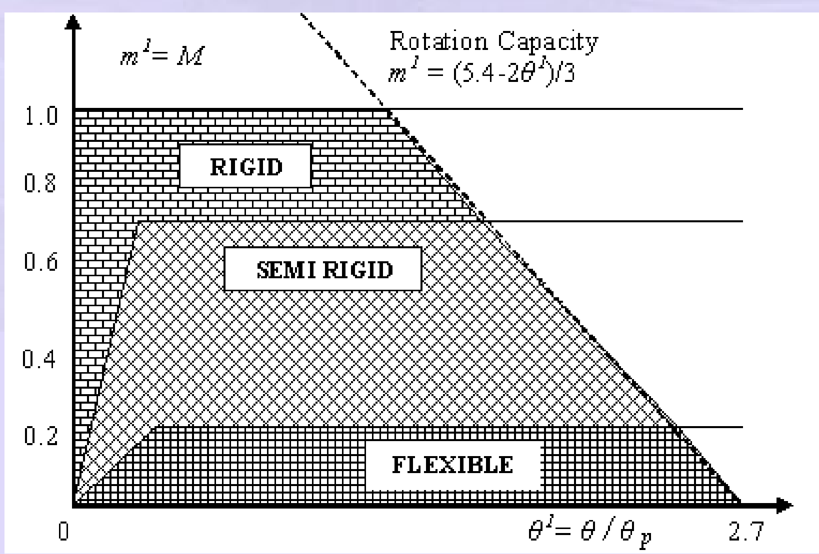


Fig. 1.2 Classification of Connections according to Bjorhovde (1990)

1.2 Connection configurations:

1.2.1 Simple connections:

Simple connections are assumed to transfer shear only shear at some nominal eccentricity. Therefore such connections can be used only in non-sway frames where the lateral loads are resisted by some alternative arrangement such as bracings or shear walls. Simple connections are typically used in frames up to about five storey in height, where strength rather than stiffness govern the design. Some typical details adopted for simple connections are shown in Fig. 1.3.

The clip and seating angle connection [Fig.1.3 (a)] is economical when automatic saw and drill lines are available. An important point in design is to check end bearing for possible adverse combination of tolerances. In the case of unstiffened seating angles, the bolts connecting it to the column may be designed for shear only assuming the seating angle to be relatively flexible. If the angle is stiff or if it is stiffened in some way then the bolted connection should be designed for the moment arising due to the eccentricity between the centre of the bearing length and the column face in addition to shear. The clip angle does not contribute to the shear resistance because it is flexible and opens out but it is required to stabilise the beam against torsional instability by providing lateral support to compression flange.

The connection using a pair of web cleats, referred to as framing angles, [Fig.1.3 (b)] is also commonly employed to transfer shear from the beam to the column. Here again, if the depth of the web cleat is less than about 0.6 times that of the beam web, then the bolts need to be designed only for the shear force. Otherwise by assuming pure shear transfer at the column face, the bolts connecting the cleats to the beam web should be designed for the moment due to eccentricity.

The end plate connection [Fig. 1.3(c)] eliminates the need to drill holes in the beam. A deep end plate would prevent beam end rotation and thereby end up transferring significant moment to the column. Therefore the depth of the end plate should be limited to that required for shear transfer. However adequate welding should be provided between end plate and beam web. To ensure significant deformation of the end plate before bolt fracture, the thickness of the end plate should be less than one-half of the bolt diameters for Grade 8.8 bolts and one-third of the bolt diameter for Grade 4.6 bolts.

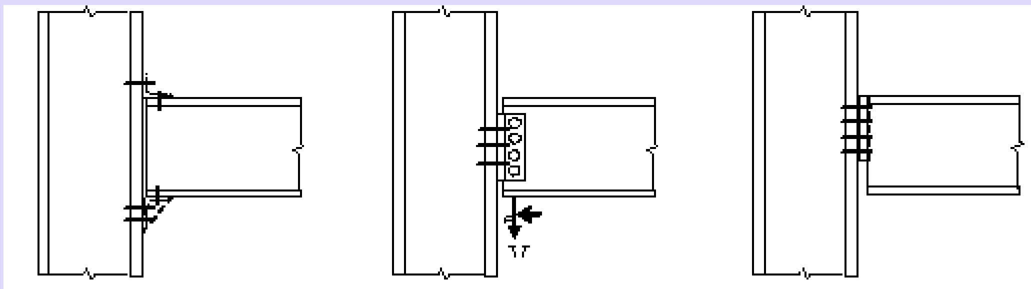


Fig. 1.3 Simple beam-to-column connections (a) Clip and seating angle (b) Web cleats (c) Curtailed end plate

1.2.2 Rigid connections:

Rigid connections transfer significant moments to the columns and are assumed to undergo negligible deformations. Rigid connections are necessary in sway frames for stability and also contribute in resisting lateral loads. In high-rise and slender structures, stiffness requirements may warrant the use of rigid connections. Examples of rigid connections are shown in Fig. 1.4.

Using angles or T-sections to connect beam flanges to the column is not economical due to the large number of bolts required. Further, these connections require HSFG bolts for rigidity. Therefore extended end-plate connections have become the popular method for rigid connections. It is fairly easy to transfer about 0.7 to 0.8 times the yield moment capacity of the beam using these connections. Column web stiffening will normally be required and the bolts at the bottom are for preventing the springing action. These bolts can however be used for shear transfer. In the case of deep beams connected to relatively slender columns a haunched connection as shown in Fig. 1.4c may be adopted. Additional column web stiffeners may also be required in the form of diagonal stiffeners [Fig. 1.4(b)] or web plates [Fig. 1.4(c)].

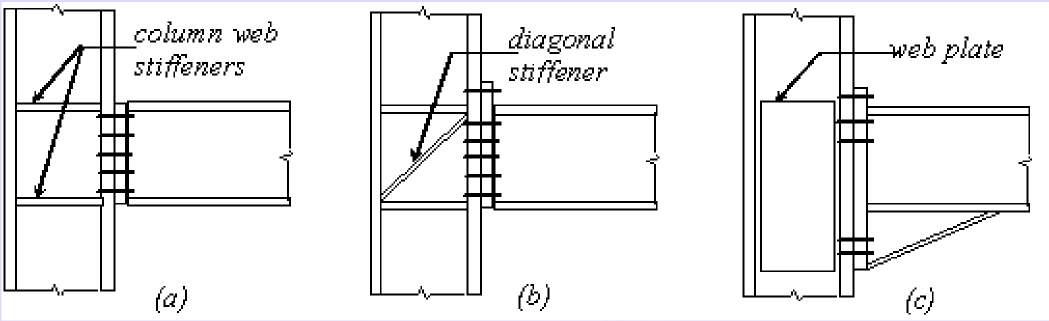


Fig. 1.4 Rigid beam-to-column connections (a) Short end plate (b) Extended end plate (c) Haunched

1.2.3 Semi rigid connections:

Semi-rigid connections are those fall between simple and rigid connections. The fact that most simple connections do have some degree of rotational rigidity was recognised and efforts to utilise it led to the development of the semi-rigid connections. Similarly rigid connections do experience some degree of joint deformation and this can be utilised to reduce the joint design moments. They are used in conjunction with other lateral load resisting systems for increased safety and performance. Use of semi-rigid connections makes the analysis somewhat difficult but leads to economy in member designs. The analysis of semi-rigid connections is usually done by assuming linear rotational springs at the supports or by advanced analysis methods, which account for non-linear moment-rotation characteristics. Examples of semi-rigid connections are shown in Fig. 1.5.

The moment-rotation characteristics will have to be determined based on experiments conducted for the specific design. These test results are then made available as data bases. Simple models are proposed in the form of equations with empirical constants derived based on test results. Depending on the degree of accuracy required, the moment-rotation characteristics may be idealized as linear, bilinear or non-linear curves.

For obtaining the moment rotation relationship the Frye-Morris polynomial model is recommended by IS 800. The model has the form shown in the following equation

$$\theta_r = C_1\,(KM)^1 + C_2\,(KM)^3 + C_3\,(KM)^5$$

Where, K = a standardization parameter dependent upon the connection type and geometry and C_1, C_2, C_3 = curve fitting constants.

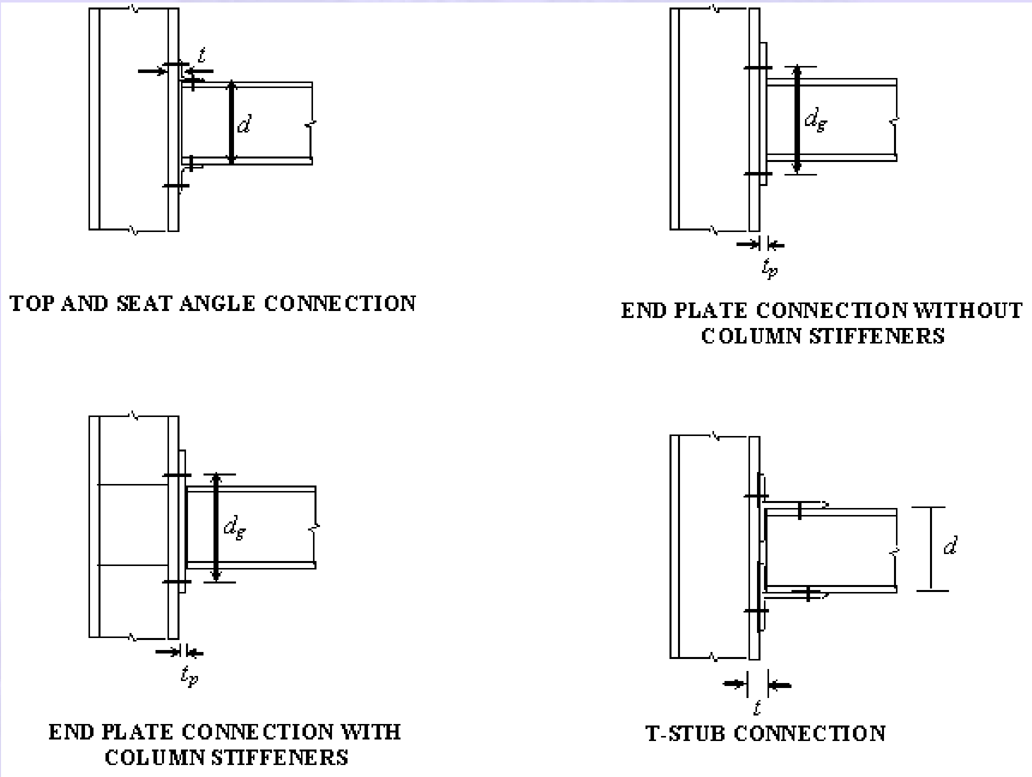


Fig. 1.5 Semi-rigid beam-to-column connections

Table.1.2. shows the curve fitting constants and standardization constants for Frye-Morris Model. (All size parameters are in mm) Depending on the type of connection, the stiffnesses given in Table.1.3 may be assumed either for preliminary analysis or when using a linear moment curvature relationship. The values are based on the secant stiffenesses at a rotation of 0.01 radian and typical dimension of connecting angle and other components as given in the Table 1.3.

The major advantage of semi-rigid connections is that they are cheaper than rigid connections and allow the optimum utilization of the beam member. To understand the second point, consider a beam with simple supports over a span L, subjected to a concentrated load W at mid-span. The mid-span bending moment will be WL/4. On the other hand, if the beam is provided with rigid supports, the maximum moment is WL/8 and occurs at the mid span as well as the support. The moment at the support gets transferred to the column and so may not be desirable. By using a semi-rigid connection we can control the mid span and support moments to the desired value.

Table 1.2. ***Connection constants in frye –morris model***

Connection type	Curve-fitting constants	Standardization constants
Top and seat angle connection	$C_1 = 8.46 \times 10^{-4}$ $C_2 = 1.01 \times 10^{-4}$ $C_3 = 1.24 \times 10^{-8}$	$K = 1.28 \times 10^{-6} d^{-1.5}\, t^{-0.5}\, l_a^{-0.7}\, d_b^{-1.5}$
End plate connection without column stiffeners	$C_1 = 1.83 \times 10^{-3}$ $C_2 = -1.04 \times 10^{-4}$ $C_3 = 6.38 \times 10^{-6}$	$K = 9.10 \times 10^{-7}\, d_g^{-2.4}\, t_p^{-0.4}\, d_b^{-1.5}$
End plate connection with column stiffeners	$C_1 = 1.79 \times 10^{-3}$ $C_2 = 1.76 \times 10^{-4}$ $C_3 = 2.04 \times 10^{-4}$	$K = 6.10 \times 10^{-5} d_g^{-2.4}\, t_p^{-0.6}$
T-stub connection	$C_1 = 2.1 \times 10^{-4}$ $C_2 = 6.2 \times 10^{-6}$ $C_3 = -7.6 \times 10^{-9}$	$K = 4.6 \times 10^{-6} d^{-1.5}\, t^{-0.5}\, l_t^{-0.7}\, d_b^{-1.1}$

Where

d_a = depth of the angle in mm t_a= thickness of the top angle in mm

l_a= length of the angle in mm d_b= diameter of the bolt in mm

d_g= center to center of the outermost bolt of the end plate connection in mm

t_p= thickness of ends- plate in mm

t= thickness of column flange and stub connector in mm

d= depth of the beam in mm l_t= length of the top angle in mm

Table 1.3 Secant stiffnesses

Sl No	**Type of Connection**	**Dimension in mm**	**Secant Stiffeness kNm/radian**
1.	Single Web Connection Angle	d_a=250 t_a=10 g=35	1150
2.	Double Web –Angle Connection	d_a=250 t_a=10 g=77.5	4450
3	Top and seat angle connection without double web angle connection	d_a=300 t_a=10 l_a=140 d_b=20	2730
4	Header Plate	d_p=175 t_p=10 g=75 t_w=7.5	2300

1.3 Summary

The types of connections between beam and column were described. The connection configurations were illustrated and the advantages of semi-rigid connections were outlined. The method of modeling the non linear moment rotation relationships was illustrated.

1.4 References

1) IS: 800 (Daft 2005) Code of Practice for Use of Structural Steel in General Building Construction, Bureau of Indian Standards. New Delhi.

2) Chen, W.F. and Toma. S. Advanced analysis of steel frames. . Boca Raton (FL): CRC Press, 1994

3) Mazzolani, F.M. and Piluso, V (1996) Theory and Design of Seismic Resistant Steel Frames, E & F Spon Press, UK.

4) Owens G W and Cheal B D (1988) Structural Steelwork Connections, Butterworths, London.

Examples

1. Design a bolted end plate connection between an ISMB 400 beam and an ISHB 200 @ 40 kg/m column so as to transfer a hogging factored bending moment of 150 KN-m and a vertical factored shear of 150 KN. Use HSFG bolts of diameter 22 mm.

Assume 6 HSFG 8.8 grade bolts of 22mm dia and 180 × 600-mm end plate as shown in figure.

1) Bolt forces

Taking moment about the center of the bottom flange and neglecting the contribution of bottom bolts and denoting the force in the top bolts by F

$4F \times 384 = 150 \times 10^3$

$F = 97.6$ kN

Tension Resistance of the bolt $T_f = T_{nf} / \gamma_{mb}$

$T_{nf} = 0.9 \times f_{ub} \times A_n \leq f_{yb} \times A_{sb} \times \gamma_{m1} \times \gamma_{m0}$

$A_{sb} = \pi / 4\ \Xi\ 22^2 = 380.13\ mm^2$

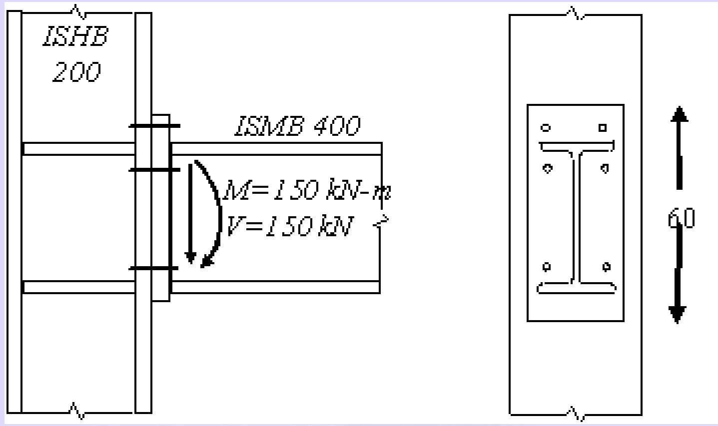
$A_n = 0.8 \times A_{sb} = 304.1\ mm^2$

$T_{nf} = 0.9 \times 800 \times 304.11 = 218.96$ KN < 276.458 KN ($f_{yb} \times A_{sb} \times \gamma_{m1} / \gamma_{m0}$)

$T_f = 218.96 / 1.25 = 175.168$ KN

Design tension capacity of bolt = 175.168 kN

Allowable prying force Q = 175.168 - 97.6 = 77.568 kN



2) Thickness of end plate assuming 10 mm fillet weld to connect the beam with end plate, distance from center line of bolt to toe of fillet weld b = 60-10 = 50 mm; end plate width be = 180 mm effective width of end plate per bolt w = be/2 = 180/2 = 90 mm

$M_p = F \times b/2 = 97.6 \times 10^3 \times 50/2 = 2440$ N-m

$t_{min} = \sqrt{(1.15 \times 4 \times M_p / p_y \times w)} = 22.33$ mm

provide (T) 30 mm thick end plate

3) Design for prying action distance from the centre line of bolt to prying force n is the minimum of edge distance or $1.1\, T \sqrt{\beta Po/Py} = 1.1 \times 30 \sqrt{(2 \times 512/250)} = 55.66$ mm

so, n = 40 mm moment at the toe of the weld = Fb - Qn = $97.6 \times 50 - 77.568 \times 40$ = 2412 N-m moment capacity = $(py/1.15) \times (wT^2/4)$

$= (250/1.15)(90 \times 30^2/4) = 4402$ N-m > 2412 N-m Safe !

Prying force $Q = \frac{b}{2n}\left[F - \frac{\beta\gamma P_0 w T^4}{27nb^2}\right]$

β=2 (non-preloaded)

γ =1.5 (for factored load)

$$Q = \frac{50}{2 \text{ x } 40}\left[97.6 - \frac{2 \text{ x } 1.5 \text{ x } 0.560 \text{ x } 90 \text{ x } 30^4}{27 \text{ x } 40 \text{ x } 50^2}\right]$$

= 32.65 KN < allowable prying force

Hence Safe!

4) Check for combined shear and tension

Shear capacity of 22 dia HSFG bolt V_{sdf} = 68.2 KN

Shear per bolt V = 150/6 = 25 KN

Applied tensile load on bolt = 97.6 + 32.65 = 130.25 KN

Design tension capacity = 175.168 KN

$(V/V_{sdf})^2 + (T_e/T_{ndf})^2 = (25.0 / 68.2)^2 + (130.25 /175.168)^2 = 0.687 < 1.0$

Hence Safe!

2 INDUSTRIAL BUILDINGS

2.1 Introduction

Any building structure used by the industry to store raw materials or for manufacturing products of the industry is known as an industrial building. Industrial buildings may be categorized as Normal type industrial buildings and Special type industrial buildings. Normal types of industrial building are shed type buildings with simple roof structures on open frames. These buildings are used for workshop, warehouses etc. These building require large and clear areas unobstructed by the columns. The large floor area provides sufficient flexibility and facility for later change in the production layout without major building alterations. The industrial buildings are constructed with adequate headroom for the use of an overhead traveling crane. Special types of industrial buildings are steel mill buildings used for manufacture of heavy machines, production of power etc. The function of the industrial building dictates the degree of sophistication.

2.1.1 Building configuration

Typically the bays in industrial buildings have frames spanning the width direction. Several such frames are arranged at suitable spacing to get the required length (Fig. 2.1). Depending upon the requirement, several bays may be constructed adjoining each other. The choice of structural configuration depends upon the span between the rows of columns, the head room or clearance required the nature of roofing material and type of lighting. If span is less, portal frames such as steel bents (Fig. 2.2a) or gable frames (Fig. 2.2b) can be used but if span is large then buildings with trusses (Fig. 2.2 c & d) are used.

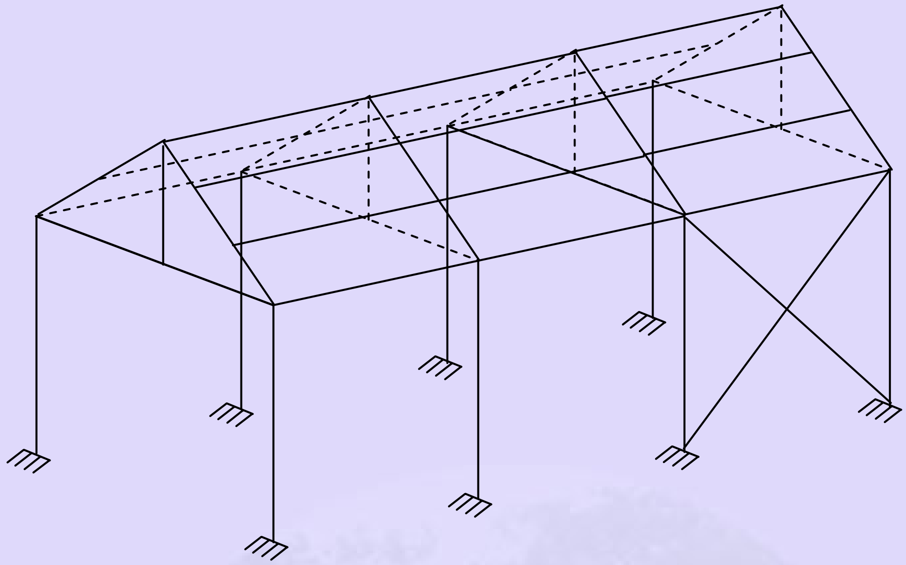
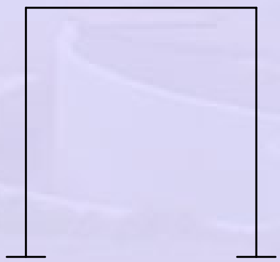


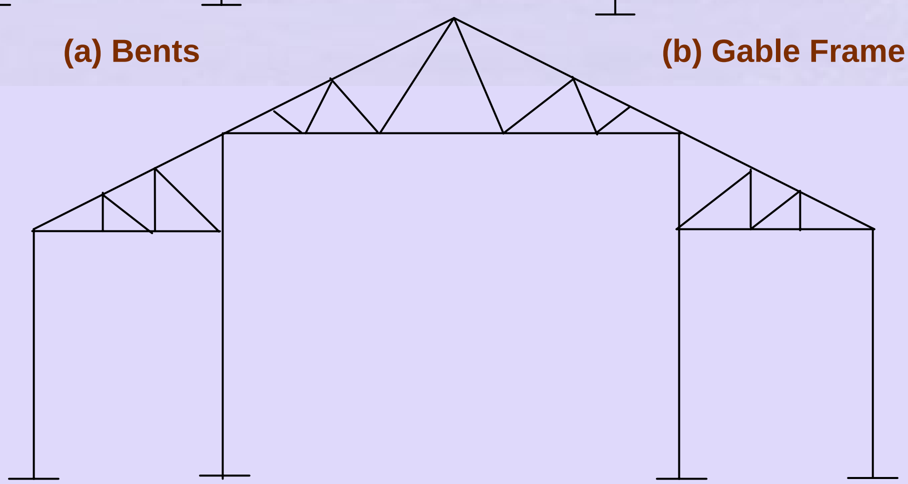
Fig. 2.1 Typical structural layout of an industrial

The horizontal and vertical bracings, employed in single and multi-storey buildings, are also trusses used primarily to resist wind and other lateral loads. These bracings minimize the differential deflection between the different frames due to crane surge in industrial buildings. They also provide lateral support to columns in small and tall buildings, thus increasing the buckling strength.

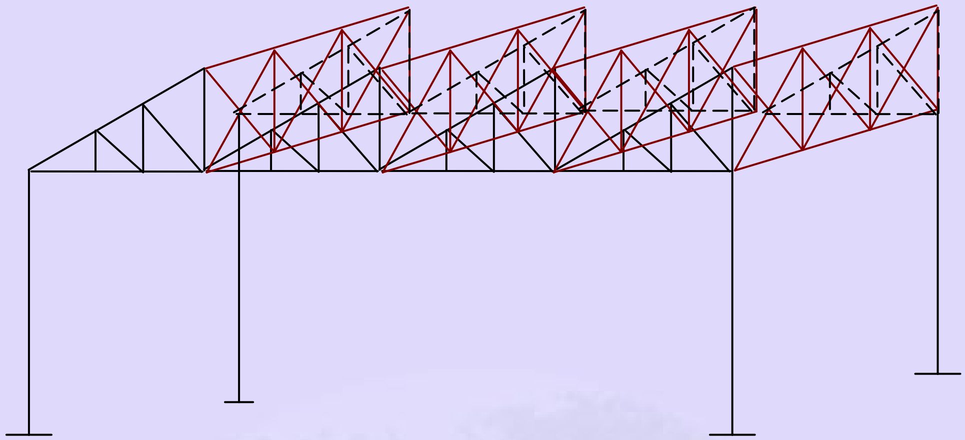


(a) Bents

(b) Gable Frame



(c) Industrial Building with Side Spans



(d) Industrial building with North light trusses
Fig. 2.2 Typical frame types used in industrial buildings

Floors

Different types of floor are required in any factory from their use consideration such as production, workshop, stores, amenities, and administration. The service condition will vary widely in these areas, so different floors types are required. Industrial floors shall have sufficient resistance to abrasion, impact, acid action and temperatures depending on the type of activity carried out. High strength and high performance concretes can satisfy most of these requirements economically and is the most common material used.

Foundation for vibrating machinery (such as reciprocating and high speed rotating machinery) should be placed upon rock or firm ground and it should be separated from adjacent floor to avoid vibrations.

Roof System

While planning a roof, designer should look for following quality lightness, strength, water proofness, insulation, fire resistance, cost, durability and low maintenance charges.

Sheeting, purlin and supporting roof trusses supported on column provide common structural roof system for industrial buildings. The type of roof covering, its insulating value, acoustical properties, the appearance from inner side, the weight and the maintenance are the various factors, which are given consideration while designing the roof system. Brittle sheeting such as asbestos, corrugated and trafford cement sheets or ductile sheeting such as galvanized iron corrugated or profiled sheets are used as the roof covering material. The deflection limits for purlins and truss depend on the type of sheeting. For brittle sheeting small deflection values are prescribed in the code.

Lighting

Industrial operations can be carried on most efficiently when adequate illumination is provided. The requirements of good lighting are its intensity and uniformity. Since natural light is free, it is economical and wise to use daylight most satisfactory for illumination in industrial plants whenever practicable.

Side windows are of much value in lighting the interiors of small buildings but they are not much effective in case of large buildings. In case of large buildings monitors are useful (Fig. 2.3).

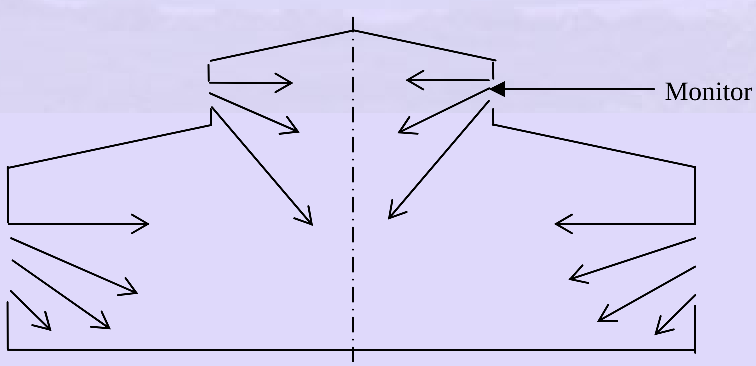


Fig. 2.3 Side windows and Monitors for natural light

Ventilation

Ventilation of industrial buildings is also important. Ventilation will be used for removal of heat, elimination of dust, used air and its replacement by clean fresh air. It can be done by means of natural forces such as aeration or by mechanical equipment such as fans. The large height of the roof may be used advantageously by providing low level inlets and high level outlets for air.

2.2 Loads

Dead load

Dead load on the roof trusses in single storey industrial buildings consists of dead load of claddings and dead load of purlins, self weight of the trusses in addition to the weight of bracings etc. Further, additional special dead loads such as truss supported hoist dead loads; special ducting and ventilator weight etc. could contribute to roof truss dead loads. As the clear span length (column free span length) increases, the self weight of the moment resisting gable frames (Fig. 2.2b) increases drastically. In such cases roof trusses are more economical. Dead loads of floor slabs can be considerably reduced by adopting composite slabs with profiled steel sheets as described later in this chapter.

Live load

The live load on roof trusses consist of the gravitational load due to erection and servicing as well as dust load etc. and the intensity is taken as per IS:875-1975. Additional special live loads such as snow loads in very cold climates, crane live loads in trusses supporting monorails may have to be considered.

Wind load

Wind load on the roof trusses, unless the roof slope is too high, would be usually uplift force perpendicular to the roof, due to suction effect of the wind blowing over the roof. Hence the wind load on roof truss usually acts opposite to the gravity load, and its magnitude can be larger than gravity loads, causing reversal of forces in truss members. The calculation of wind load and its effect on roof trusses is explained later in this chapter.

Earthquake load

Since earthquake load on a building depends on the mass of the building, earthquake loads usually do not govern the design of light industrial steel buildings. Wind loads usually govern. However, in the case of industrial buildings with a large mass located at the roof or upper floors, the earthquake load may govern the design. These loads are calculated as per IS: 1893-2002. The calculation of earthquake load and its effect on roof trusses is explained later in this chapter.

2.3 Industrial floors

The industrial buildings are usually one-story structures but some industrial building may consist of two or more storey. Reinforced concrete or steel-concrete composites slabs are used as a floor system. The rolled steel joists or trusses or plate girders support these slabs. The design of reinforced concrete slabs shall be done as per IS 456-2000. Steel-concrete composite slabs are explained in more detail below.

2.3.1 Steel-concrete composite floors

The principal merit of steel-concrete composite construction lies in the utilisation of the compressive strength of concrete in conjunction with steel sheets or beams, in order to enhance the strength and stiffness.

Composite floors with profiled decking consist of the following structural elements in addition to in-situ concrete and steel beams:

- Profiled decking
- Shear connectors
- Reinforcement for shrinkage and temperature stresses

Composite floors using profiled sheet decking have are particularly competitive where the concrete floor has to be completed quickly and where medium level of fire protection to steel work is sufficient. However, composite slabs with profiled decking are unsuitable when there is heavy concentrated loading or dynamic loading in structures such as bridges. The alternative composite floor in such cases consists of reinforced or pre-stressed slab over steel beams connected together using shear connectors to act monolithically (Fig. 2.4).

A typical composite floor system using profiled sheets is shown in Fig. 2.5. There is presently no Indian standard covering the design of composite floor systems using profiled sheeting. The structural behaviour of Composite floors using profiled decks is similar to a reinforced concrete slab, with the steel sheeting acting as the tension reinforcement. The main structural and other benefits of using composite floors with profiled steel decking are:

- Savings in steel weight are typically 30% to 50% over non-composite construction
- Greater stiffness of composite beams results in shallower depths for the same span. Hence lower storey heights are adequate resulting in savings in cladding costs, reduction in wind loading and savings in foundation costs.
- Faster rate of construction.

The steel deck is normally rolled into the desired profile from 0.9 mm to 1.5 mm galvanised sheets. It is profiled such that the profile heights are usually in the range of 38-75 mm and the pitch of corrugations is between 150 mm and 350 mm. Generally, spans of the order of 2.5 m to 3.5 m between the beams are chosen and the beams are designed to span between 6 m to 12 m. Trapezoidal profile with web indentations is commonly used.

The steel decking performs a number of roles, such as:

- It supports loads during construction and acts as a working platform
- It develops adequate composite action with concrete to resist the imposed loading

- It transfers in-plane loading by diaphragm action to vertical bracing or shear walls
- It stabilizes the compression flanges of the beams against lateral buckling, until concrete hardens.
- It reduces the volume of concrete in tension zone
- It distributes shrinkage strains, thus preventing serious cracking of concrete.

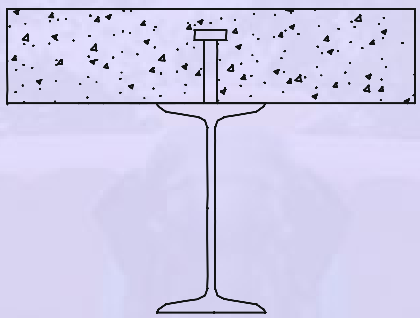


Fig. 2.4 Steel beam bonded to concrete slab with shear

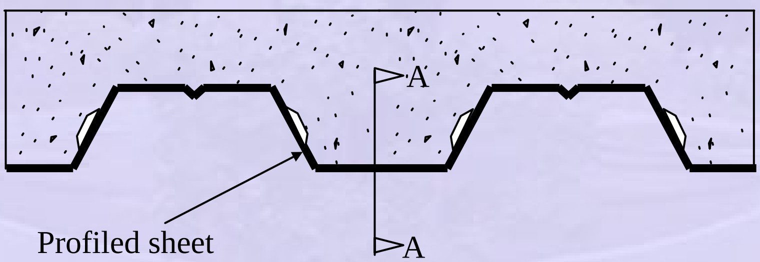


Fig. 2.5 Composite floor system using profiled sheets

Profiled sheeting as permanent form work

Construction stage: During construction, the profiled steel deck acts alone to carry the weight of wet concrete, self weight, workmen and equipments. It must be strong enough to carry this load and stiff enough to be serviceable under the weight of wet concrete only. In addition to structural adequacy, the finished slab must be capable of satisfying the requirements of fire resistance.

Design should make appropriate allowances for construction loads, which include the weight of operatives, concreting plant and any impact or vibration that may occur during construction. These loads should be arranged in such a way that they cause maximum bending moment and shear. In any area of 3 m by 3 m (or the span length, if less), in addition to weight of wet concrete, construction loads and weight of surplus concrete should be provided for by assuming a load of 1.5 kN/m^2. Over the remaining area a load of 0.75 kN/m^2 should be added to the weight of wet concrete.

Composite Beam Stage: The composite beam formed by employing the profiled steel sheeting is different from the one with a normal solid slab, as the profiling would influence its strength and stiffness. This is termed 'composite beam stage'. In this case, the profiled deck, which is fixed transverse to the beam, results in voids within the depth of the associated slab. Thus, the area of concrete used in calculating the section properties can only be that depth of slab above the top flange of the profile. In addition, any stud connector welded through the sheeting must lie within the area of concrete in the trough of the profiling. Consequently, if the trough is narrow, a reduction in strength must be made because of the reduction in area of constraining concrete. In current design methods, the steel sheeting is ignored when calculating shear resistance; this is probably too conservative.

Composite Slab Stage: The structural behaviour of the composite slab is similar to that of a reinforced concrete beam with no shear reinforcement. The steel sheeting provides adequate tensile capacity in order to act with the concrete in bending. However, the shear between the steel and concrete must be carried by friction and bond between the two materials. The mechanical keying action of the indents is important. This is especially so in open trapezoidal profiles, where the indents must also provide resistance to vertical separation. The predominant failure mode is one of shear bond rupture that results in slip between the concrete and steel.

Design method

As there is no Indian standard covering profiled decking, we refer to Eurocode 4 (EC4) for guidance. The design method defined in EC4 requires that the slab be checked first for bending capacity, assuming full bond between concrete and steel, then for shear bond capacity and, finally, for vertical shear. The analysis of the bending capacity of the slab may be carried out as though the slab was of reinforced concrete with the steel deck acting as reinforcement. However, no satisfactory analytical method has been developed so far for estimating the value of shear bond capacity. The loads at the construction stage often govern the allowable span rather than at the composite slab stage.

The width of the slab 'b' shown in Fig. 2.6(a) is one typical wavelength of profiled sheeting. But, for calculation purpose the width considered is 1.0 m. The overall thickness is h_t and the depth of concrete above main flat surface h_c. Normally, h_t is not less than 80 mm and h_c is not less than 40 mm from sound and fire insulation considerations.

The neutral axis normally lies in the concrete in case of full shear connection. For sheeting in tension, the width of indents should be neglected.

Therefore, the effective area 'A_p' per meter and height of centre of area above bottom 'e' are usually based on tests. The plastic neutral axis e_p is generally larger than e.

The simple plastic theory of flexure is used for analysis of these floors for checking the design at Limit State of collapse load. IS 456: 1978 assumes the equivalent ultimate stress of concrete in compression as 0.36 (f_{ck}) where (f_{ck}) is characteristic cube strength of concrete.

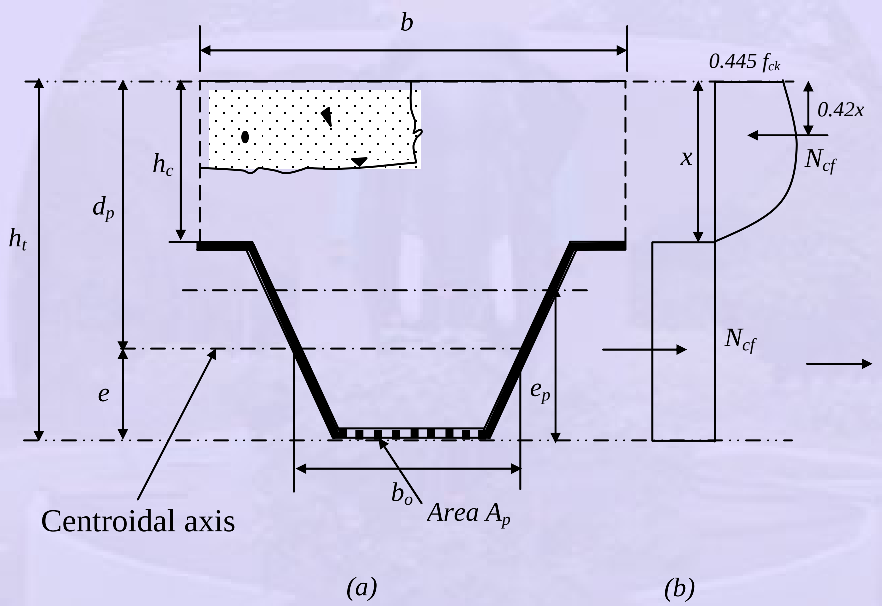


Fig.2.6 Resistance of composite slab to sagging bending

Full shear connection is assumed. Hence, compressive force N_{cf} in concrete is equal to steel yield force N_{pa}.

$$N_{cf} = N_{pq} = \frac{A_p f_{yp}}{\gamma_{ap}} \qquad \textbf{(2.1)}$$

$N_{cf} = 0.36\, f_{ck}.b.x$

where A_p = Effective area per meter width

f_{yp} = Yield strength of steel

γ_{ap} = Partial safety factor (1.15)

The neutral axis depth x is given by

$$x = \frac{N_{cf}}{b\left(0.36\, f_{ck}\right)} \qquad (2.2)$$

This is valid when $x \leq h_c$, i.e. when the neutral axis lies above steel decking.

$M_{p.Rd}$ is the design resistance to sagging bending moment and is given by:

$$_{p.Rd} = N_{cf}\,(d_p - 0.42\, x) \qquad (2.3)$$

Note that centroid of concrete force lies at 0.42 x from free concrete surface.

The shear resistance of composite slab largely depends on connection between profiled deck and concrete. The following three types of mechanisms are mobilised:

(i) Natural bond between concrete and steel due to adhesion

(ii) Mechanical interlock provided by dimples on sheet and shear connectors

(iii) Provision of end anchorage by shot fired pins or by welding studs (Fig. 2.7) when sheeting is made to rest on steel beams.

Natural bond is difficult to quantify and unreliable, unless separation at the interface between the sheeting and concrete is prevented. Dimples or ribs are incorporated in the sheets to ensure satisfactory mechanical interlock. These are effective only if the embossments are sufficiently deep. Very strict control during manufacture is needed to ensure that the depths of embossments are consistently maintained at an acceptable level. End anchorage is provided by means of shot-fired pins, when the ends of a sheet rest on a steel beam, or by welding studs through the sheeting to the steel flange.

Quite obviously the longitudinal shear resistance is provided by the combined effect of frictional interlock, mechanical interlock and end anchorage. No mathematical model could be employed to evaluate these and the effectiveness of the shear connection is studied by means of load tests on simply supported composite slabs as described in the next section.

Serviceability criteria

The composite slab is checked for the following serviceability criteria: Cracking, Deflection and Fire endurance. The crack width is calculated for the top surface in the negative moment region using standard methods prescribed for reinforced concrete. The method is detailed in the next chapter. Normally crack width should not exceed 3 mm. IS 456: 2000 gives a formula to calculate the width of crack. Provision of 0.4 % steel will normally avoid cracking problems in propped construction and provision 0.2 % of steel is normally sufficient in un-propped construction. If environment is corrosive it is advisable to design the slab as continuous and take advantage of steel provided for negative bending moment for resisting cracking during service loads.

The IS 456: 2000 gives a stringent deflection limitation of $\ell/350$ which may be un- realistic for un-propped construction. The Euro code gives limitations of $\ell/180$ or 20 mm which ever is less. It may be worth while to limit span to depth ratio in the range of 25 to 35 for the composite condition, the former being adopted for simply supported slabs and the later for continuous slabs. The deflection of the composite slabs is influenced by the slip-taking place between sheeting and concrete. Tests seem to be the best method to estimate the actual deflection for the conditions adopted.

The fire endurance is assumed based on the following two criteria:

- Thermal insulation criterion concerned with limiting the transmission of heat by conduction
- Integrity criterion concerned with preventing the flames and hot gases to nearby compartments.

It is met by specifying adequate thickness of insulation to protect combustible materials. R (time in minutes) denotes the fire resistance class of a member or component. For instance, R60 means that failure time is more than 60 minutes. It is generally assumed that fire rating is R60 for normal buildings.

2.3.2 Vibration

Floor with longer spans of lighter construction and less inherent damping are vulnerable to vibrations under normal human activity. Natural frequency of the floor system corresponding to the lowest mode of vibration and damping characteristics are important characteristics in floor vibration. Open web steel joists (trusses) or steel beams on the concrete deck may experience walking vibration problem.

Generally, human response to vibration is taken as the yardstick to limit the amplitude and frequency of a vibrating floor. The present discussion is mainly aimed at design of a floor against vibration perceived by humans. To design a floor structure, only the source of vibration near or on the floor need be considered. Other sources such as machines, lift or cranes should be isolated from the building. In most buildings, following two cases are considered-

i) People walking across a floor with a pace frequency between 1.4 Hz and 2.5 Hz.

ii) An impulse such as the effect of the fall of a heavy object.

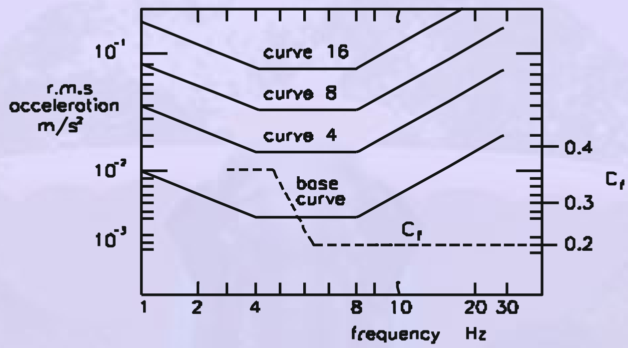


Fig. 2.7 Curves of constant human response to vibration, and Fourier component factor

BS 6472 present models of human response to vibration in the form of a base curve as in Fig. (2.19). Here root mean square acceleration of the floor is plotted against its natural frequency f_0 for acceptable level *R* based on human response for different situations such as, hospitals, offices etc. The human response *R=1* corresponds to a *"minimal level of adverse comments from occupants"* of sensitive locations such as hospital, operating theatre and precision laboratories. Curves of higher response *(R)* values are also shown in the Fig.2.7. The recommended values of *R* for other situations are

R = 4 for offices

R = 8 for workshops

These values correspond to continuous vibration and some relaxation is allowed in case the vibration is intermittent.

Natural frequency of beam and slab

The most important parameter associated with vibration is the natural frequency of floor. For free elastic vibration of a beam or one way slab of uniform

$$f_0 = K\left(\frac{EI}{mL^4}\right)^{\frac{1}{2}} \qquad \textbf{\textit{(2.4)}}$$

section the fundamental natural frequency is,

Where,

K = π/2 for simple support; and

K = 3.56 for both ends fixed.

EI = Flexural rigidity (per unit width for slabs)

L = span

m = vibrating mass per unit length (beam) or unit area (slab).

According to Appendix D of the Code (IS 800), the fundamental natural frequency can be estimated by assuming full composite action, even in non-composite construction. This frequency, f_1, for a simply supported one way system is given by

$$f_1 = 156\sqrt{EI_r / WL^4}$$

Where

E = modulus of elasticity of steel, (MPa)

I_T = transformed moment of inertia of the one way system (in term of equivalent steel) assuming the concrete flange of width equal to the spacing of the beam to be effective (mm^4)

L = span length (mm)

W = dead load of the one way joist (N/mm)

The effect of damping, being negligible has been ignored.

Un-cracked concrete section and dynamic modulus of elasticity should be used for concrete. Generally these effects are taken into account by increasing

$$\delta_m = \frac{5mgL^4}{384\,EI} \qquad \textbf{(2.5)}$$

the value of *I* by *10%* for variable loading. In absence of an accurate estimate of mass (*m*), it is taken as the mass of the characteristic permanent load plus *10%* of characteristic variable load. The value of f_0 for a single beam and slab can be evaluated in the following manner.

The mid-span deflection for simply supported member is,

Substituting the value of *'m'* from Eqn. (2.5) in Eqn. (2.4) we get,

Where, δ_m is in millimetres.

$$f_0 = \frac{17.8}{\sqrt{\delta_m}} \qquad \textbf{(2.6)}$$

However, to take into account the continuity of slab over the beams, total deflection δ in considered to evaluate f_0, so that,

$$f_0 = \frac{17.8}{\sqrt{\delta}} \qquad \textbf{(2.7)}$$

Where, $\delta = \delta_b + \delta_s$

δ_s – deflection of slab relative to beam

δ_b- deflection of beam.

From Equation. (2.6) and (2.7)

$$\frac{1}{f_0^2} = \frac{1}{f_{0s}^2} + \frac{1}{f_{0b}^2} \qquad \textbf{(2.8)}$$

Where f_{os} and f_{ob} are the frequencies for slab and beam each considered alone.

$$f_{0b} = \frac{\pi}{2}\left(\frac{EI_b}{msL^4}\right)^{1/2} \qquad \textbf{(2.9)}$$

$$f_{0s} = 3.56\left(\frac{EI_s}{ms^4}\right)^{1/2} \qquad \textbf{(2.10)}$$

From Eqn. (2.8) we get,

Where, *s* is the spacing of the beams.

In the frequency range of 2 to 8 Hz in which people are most sensitive to vibration, the threshold level corresponds approximately to 0.5% *g*, where *g* is the acceleration due to gravity. Continuous vibration is generally more annoying then decaying vibration due to damping. Floor systems with the natural frequency less than 8 Hz in the case of floors supporting machinery and 5 Hz in the case of floors supporting normal human activity should be avoided.

Response factor

Reactions on floors from people walking have been analyzed by Fourier Series. It shows that the basic fundamental component has amplitude of about *240N*. To avoid resonance with the first harmonics it is assumed that the floor has natural frequency $f_0 > 3$, whereas the excitation force due to a person walking

$$\overline{F} = 240\,C_f \qquad \textbf{(2.11)}$$

has a frequency *1.4* Hz to *2* Hz. The effective force amplitude is,

where C_f is the Fourier component factor. It takes into account the differences between the frequency of the pedestrians' paces and the natural frequency of the floor. This is given in the form of a function of f_0 in Fig. (2.19).

$$y = \frac{\overline{F}}{2k_e S}\sin 2\pi f_0 t \qquad \textbf{(2.12)}$$

The vertical displacement *y* for steady state vibration of the floor is given approximately by,

$$\text{Where } \frac{\overline{F}}{k_e} = \text{Static deflection floor}$$

$$\frac{1}{2\zeta} = \text{magnification factor at resonance}$$

$$= 0.03 \text{ for open plan offices with composite floor}$$

$$f_0 = \text{steady state vibration frequency of the floor}$$

RMS value of acceleration

The effective stiffness k_e depends on the vibrating area of floor, $L \times S$. The width *S* is computed in terms of the relevant flexural rigidities per unit width of floor which are I_s for slab and I_b/s for beam.

$$a_{rm.s} = 4\pi^2 f_0^{\,2}\,\frac{\overline{F}}{2\sqrt{2}k_e\zeta} \qquad \textbf{(2.13)}$$

$$S = 4.5\left(\frac{EI_s}{mf_0^{\,2}}\right)^{1/4} \qquad \textbf{(2.14)}$$

As f_{0b} is much greater than f_{0s}, the value of f_{0b} can be approximated as f_0. So, replacing mf_0^2 from Eqn. (2.9) in Eqn. (2.12), we get,

$$\frac{S}{L} = 3.6\left(\frac{I_s S}{I_0}\right)^{1/4} \qquad \textbf{(2.15)}$$

Eqn. (2.15) shows that the ratio of equivalent width to span increases with increase in ratio of the stiffness of the slab and the beam.

The fundamental frequency of a spring-mass system,

$$f_0 = \frac{1}{2\pi}\left(\frac{k_e}{M_e}\right)^{1/2} \qquad \textbf{(2.16)}$$

Where, M_e is the effective mass = mSL/4 (approximately)

From Eqn. (2.18),

$$k_e = \pi^2 f^2{}_0 \, mSL \qquad \textbf{(2.17)}$$

Substituting the value of k_e from Eqn. (50) and F from Eqn. (2.11) into Eqn. (2.13)

$$a_{rms} = 340\frac{C_f}{msL\zeta} \qquad \textbf{(2.18)}$$

From definition, Response factor,

Therefore, from Equation (52),

$$a_{rms} = 5\,x10^{-3}\,R \;\; m/s^2 \qquad \textbf{(2.19)}$$

To check the susceptibility of the floor to vibration the value of R should be compared with the target response curve as in Fig. (2.19).

$$R = 68000\frac{C_f}{msL\zeta} \qquad \text{in MKS units} \qquad \textbf{(2.20)}$$

2.4 Roof systems

Trusses are triangular frame works, consisting of essentially axially loaded members which are more efficient in resisting external loads since the cross section is nearly uniformly stressed. They are extensively used, especially to span large gaps. Trusses are used in roofs of single storey industrial buildings, long span floors and roofs of multistory buildings, to resist gravity loads. Trusses are also used in walls and horizontal planes of industrial buildings to resist lateral loads and give lateral stability.

2.4.1 Analysis of trusses

Generally truss members are assumed to be joined together so as to transfer only the axial forces and not moments and shears from one member to the adjacent members (they are regarded as being pinned joints). The loads are assumed to be acting only at the nodes of the trusses. The trusses may be provided over a single span, simply supported over the two end supports, in which case they are usually statically determinate. Such trusses can be analysed manually by the method of joints or by the method of sections. Computer programs are also available for the analysis of trusses.

From the analysis based on pinned joint assumption, one obtains only the axial forces in the different members of the trusses. However, in actual design, the members of the trusses are joined together by more than one bolt or by welding, either directly or through larger size end gussets. Further, some of the members, particularly chord members, may be continuous over many nodes. Generally such joints enforce not only compatibility of translation but also compatibility of rotation of members meeting at the joint. As a result, the members of the trusses experience bending moment in addition to axial force. This may not be negligible, particularly at the eaves points of pitched roof trusses, where the depth is small and in trusses with members having a smaller slenderness ratio (i.e. stocky members). Further, the loads may be applied in between the nodes of the trusses, causing bending of the members. Such stresses are referred to as secondary stresses. The secondary bending stresses can be caused also by the eccentric connection of members at the joints. The analysis of trusses for the secondary moments and hence the secondary stresses can be carried out by an indeterminate structural analysis, usually using computer software.

The magnitude of the secondary stresses due to joint rigidity depends upon the stiffness of the joint and the stiffness of the members meeting at the joint. Normally the secondary stresses in roof trusses may be disregarded, if the slenderness ratio of the chord members is greater than 50 and that of the web members is greater than 100. The secondary stresses cannot be neglected when they are induced due to application of loads on members in between nodes and when the members are joined eccentrically. Further the secondary stresses due to the rigidity of the joints cannot be disregarded in the case of bridge trusses due to the higher stiffness of the members and the effect of secondary stresses on fatigue strength of members. In bridge trusses, often misfit is designed into the fabrication of the joints to create prestress during fabrication opposite in nature to the secondary stresses and thus help improve the fatigue performance of the truss members at their joints.

2.4.2 Configuration of trusses

Pitched roof trusses

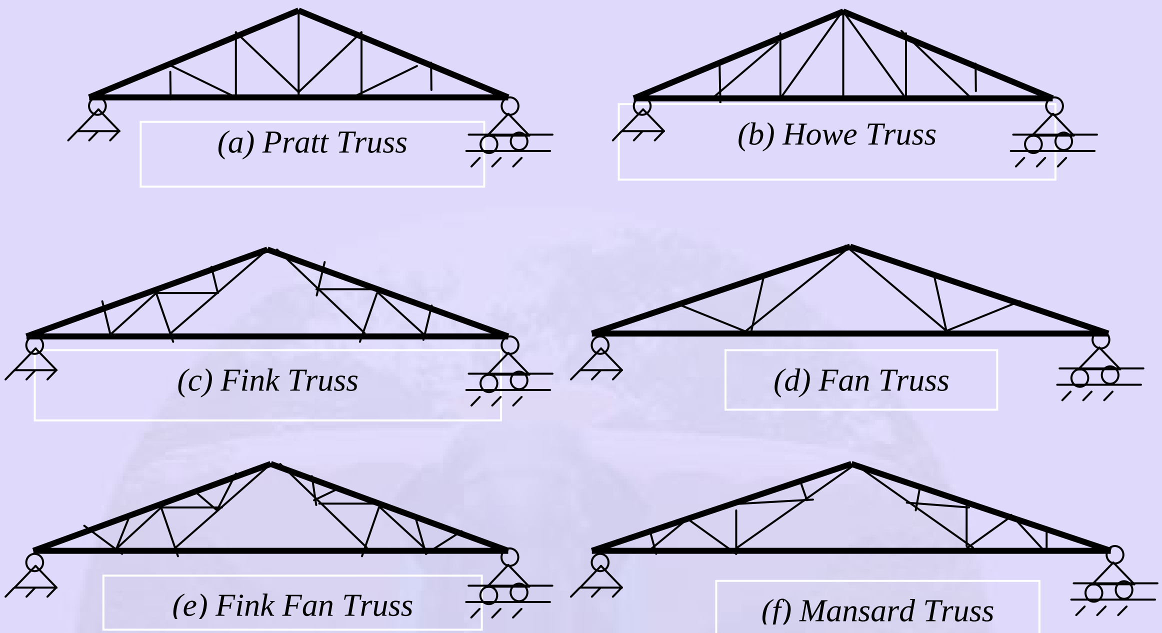


Fig. 2.9 Pitched roof trusses

Most common types of roof trusses are pitched roof trusses wherein the top chord is provided with a slope in order to facilitate natural drainage of rainwater and clearance of dust/snow accumulation. These trusses have a greater depth at the mid-span. Due to this even though the overall bending effect is larger at mid-span, the chord member and web member stresses are smaller closer to the mid-span and larger closer to the supports. The typical span to maximum depth ratios of pitched roof trusses are in the range of 4 to 8, the larger ratio being economical in longer spans. Pitched roof trusses may have different configurations. In Pratt trusses [Fig. 2.9(*a*)] web members are arranged in such a way that under gravity load the longer diagonal members are under tension and the shorter vertical members experience compression. This allows for efficient design, since the short members are under compression. However, the wind uplift may cause reversal of stresses in these members and nullify this benefit. The converse of the Pratt is the Howe truss [Fig. 2.9(*b*)]. This is commonly used in light roofing so that the longer diagonals experience tension under reversal of stresses due to wind load.

Fink trusses [Fig. 2.9(*c*)] are used for longer spans having high pitch roof, since the web members in such truss are sub-divided to obtain shorter members.

Fan trusses [Fig. 2.9(*d*)] are used when the rafter members of the roof trusses have to be sub-divided into odd number of panels. A combination of fink and fan [Fig. 2.9(*e*)] can also be used to some advantage in some specific situations requiring appropriate number of panels.

Mansard trusses [Fig. 2.9(*f*)] are variation of fink trusses, which have shorter leading diagonals even in very long span trusses, unlike the fink and fan type trusses.

The economical span lengths of the pitched roof trusses, excluding the Mansard trusses, range from 6 m to 12 m. The Mansard trusses can be used in the span ranges of 12 m to 30 m.

Parallel chord trusses

The parallel chord trusses are used to support North Light roof trusses in industrial buildings as well as in intermediate span bridges. Parallel chord trusses are also used as pre-fabricated floor joists, beams and girders in multi-storey buildings [Fig. 2.10(*a*)]. Warren configuration is frequently used [Figs.

2.10(*b*)] in the case of parallel chord trusses. The advantage of parallel chord trusses is that they use webs of the same lengths and thus reduce fabrication costs for very long spans. Modified Warren is used with additional verticals, introduced in order to reduce the unsupported length of compression chord members. The saw tooth north light roofing systems use parallel chord lattice girders [Fig. 2.10(c)] to support the north light trusses and transfer the load to the end columns.

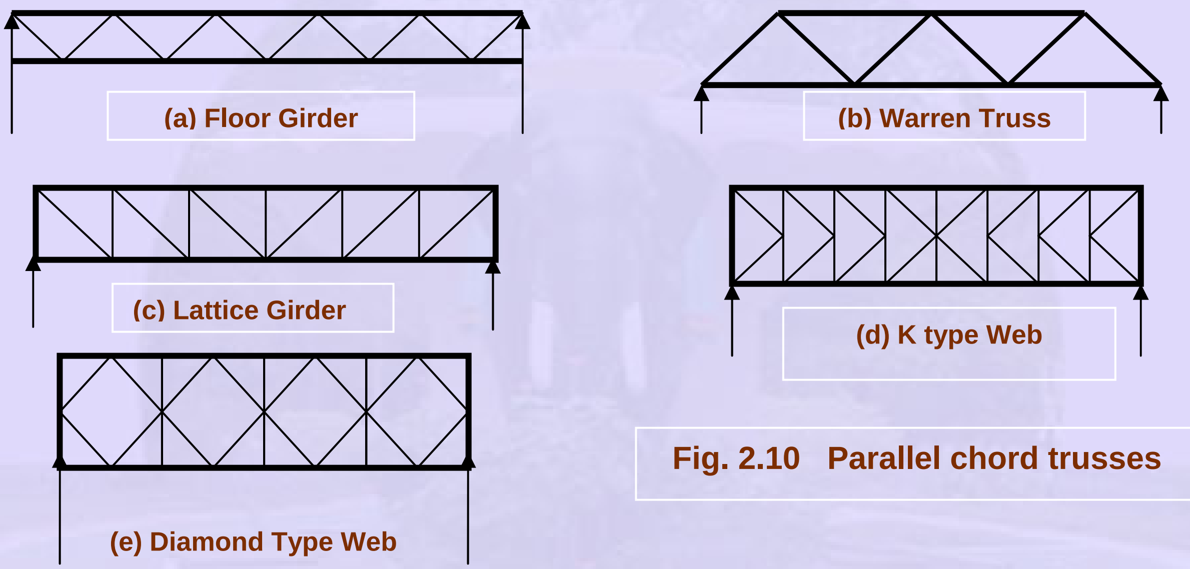


Fig. 2.10 Parallel chord trusses

The economical span to depth ratio of the parallel chord trusses is in the range of 12 to 24. The total span is subdivided into a number of panels such that the individual panel lengths are appropriate (6m to 9 m) for the stringer beams, transferring the carriage way load to the nodes of the trusses and the inclination of the web members are around 45 degrees. In the case of very deep and very shallow trusses it may become necessary to use K and diamond patterns for web members to achieve appropriate inclination of the web members. [Figs. 2.10(*d*), 2.10(*e*)]

Trapezoidal trusses

In case of very long span length pitched roof, trusses having trapezoidal configuration, with depth at the ends are used [Fig. 2.11(*a*)]. This configuration reduces the axial forces in the chord members adjacent to the supports. The secondary bending effects in these members are also reduced. The trapezoidal configurations [Fig. 2.11(*b*)] having the sloping bottom chord can be economical in very long span trusses (spans > 30 m), since they tend to reduce the web member length and the chord members tend to have nearly constant forces over the span length. It has been found that bottom chord slope equal to nearly half as much as the rafter slope tends to give close to optimum design.

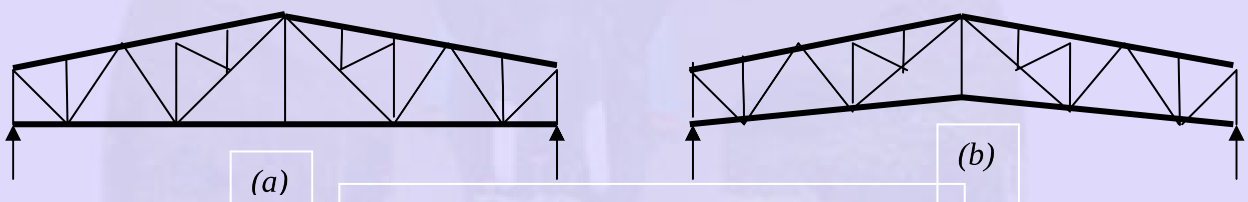
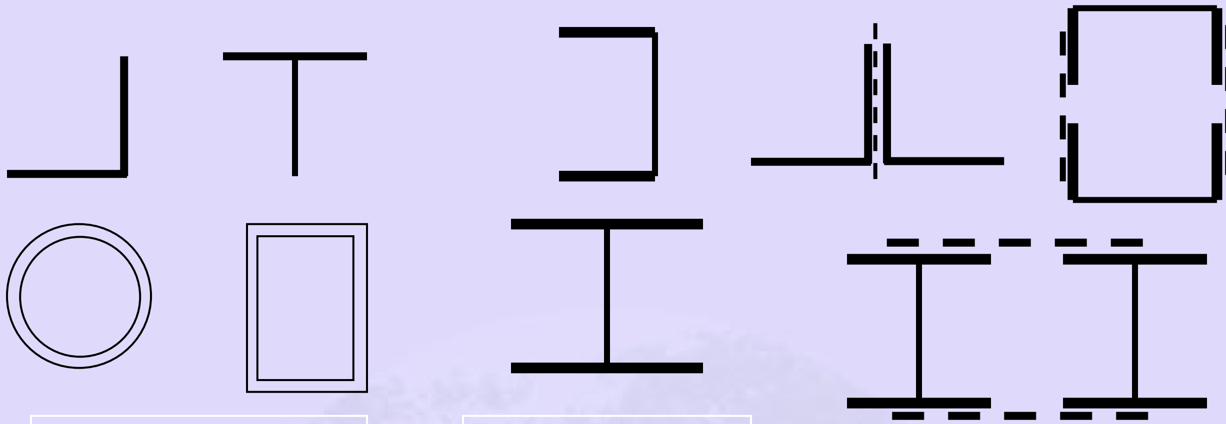


Fig 2.11 Trapezoidal trusses

2.4.3 Truss members

The members of trusses are made of either rolled steel sections or built-up sections depending upon the span length, intensity of loading, etc. Rolled steel angles, tee sections, hollow circular and rectangular structural tubes are used in the case of roof trusses in industrial buildings [Fig. 2.12(*a*)]. In long span roof trusses and short span bridges heavier rolled steel sections, such as channels, I sections are used [Fig. 2.12(*b*)]. Members built-up using I sections, channels, angles and plates are used in the case of long span bridge trusses [Fig. 2.12(*c*)]



(a) Light Section

(b) Heavy Sections

(c) Built-up Sections

Fig. 2.12 Cross sections of truss members

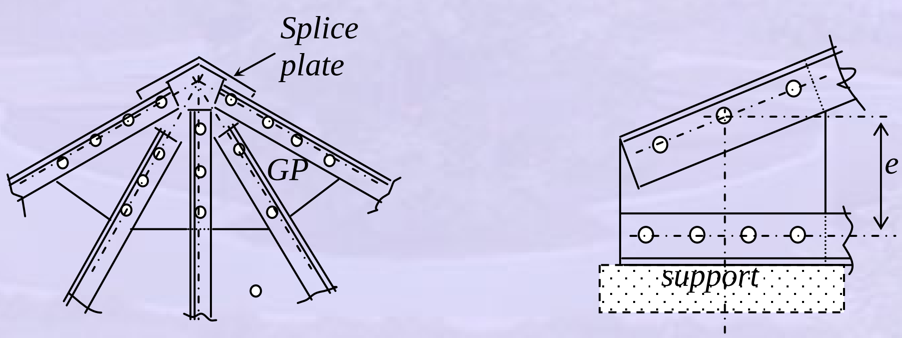
Accesses to surface, for inspection, cleaning and repainting during service, are important considerations in the choice of the built-up member configuration. Surfaces exposed to the environments, but not accessible for maintenance are vulnerable to severe corrosion during life, thus reducing the durability of the structure. In highly corrosive environments fully closed welded box sections, and circular hollow sections are used to reduce the maintenance cost and improve the durability of the structure.

2.4.4 Truss connections

Members of trusses can be joined by riveting, bolting or welding. Due to involved procedure and highly skilled labour requirement, riveting is not common these days. High strength friction grip (HSFG) bolting and welding have become more common. Shorter span trusses are usually fabricated in shops and can be completely welded and transported to site as one unit. Longer span trusses can be prefabricated in segments by welding in shop. These segments can be assembled by bolting or welding at site. This results in a much better quality of the fabricated structure.

Truss connections form a high proportion of the total truss cost. Therefore it may not always be economical to select member sections, which are efficient but cannot be connected economically. Trusses may be single plane trusses in which the members are connected on the same side of the gusset plates or double plane trusses in which the members are connected on both sides of the gusset plates.

It may not always be possible to design connection in which the centroidal axes of the member sections are coincident [Fig. 2.13(*a*)]. Small eccentricities may be unavoidable and the gusset plates should be strong enough to resist or transmit forces arising in such cases without buckling (Fig. 2.13b). The bolts should also be designed to resist moments arising due to in-plane eccentricities. If out-of-plane instability is foreseen, use splice plates for continuity of out-of-plane stiffness (Fig. 2.13a).



(a) Apex Connection **(b) Support connection**

Fig. 2.13 Truss connections

If the rafter and tie members are T sections, angle diagonals can be directly connected to the web of T by welding or bolting. Frequently, the connections between the members of the truss cannot be made directly, due to inadequate space to accommodate the joint length. In such cases, gusset plates are used to accomplish such connections. The size, shape and the thickness of the gusset plate depend upon the size of the member being joined, number and size of bolt or length of weld required, and the force to be transmitted. The thickness of the gusset is in the range of 8 mm to 12 mm in the case of roof trusses and it can be as high as 22 mm in the case of bridge trusses. The design of gussets is usually by rule of thumb. In short span (8 – 12 m) roof trusses, the member forces are smaller, hence the thickness of gussets are lesser (6 or 8 mm) and for longer span lengths (> 30 m) the thickness of gussets are larger (12 mm). The designs of gusset connections are discussed in a chapter on connections.

2.4.5 Design of trusses

Factors that affect the design of members and the connections in trusses are discussed in this section.

Instability considerations

While trusses are stiff in their plane they are very weak out of plane. In order to stabilize the trusses against out- of- plane buckling and to carry any accidental out of plane load, as well as lateral loads such as wind/earthquake loads, the trusses are to be properly braced out -of -plane. The instability of compression members, such as compression chord, which have a long unsupported length out- of-plane of the truss, may also require lateral bracing.

Compression members of the trusses have to be checked for their buckling strength about the critical axis of the member. This buckling may be in plane or out-of-plane of the truss or about an oblique axis as in the case of single angle sections. All the members of a roof truss usually do not reach their limit states of collapse simultaneously. Further, the connections between the members usually have certain rigidity. Depending on the restraint to the members under compression by the adjacent members and the rigidity of the joint, the effective length of the member for calculating the buckling strength may be less than the centre-to-centre length of the joints. The design codes suggest an effective length factor between 0.7 and 1.0 for the in-plane buckling of the member depending upon this restraint and 1.0 for the out of plane buckling.

In the case of roof trusses, a member normally under tension due to gravity loads (dead and live loads) may experience stress reversal into compression due to dead load and wind load combination. Similarly the web members of the bridge truss may undergo stress reversal during the passage of the moving loads on the deck. Such stress reversals and the instability due to the stress reversal should be considered in design. The design standard (IS: 800) imposes restrictions on the maximum slenderness ratio, *(ℓ/r).*

2.4.6 Economy of trusses

As already discussed trusses consume a lot less material compared to beams to span the same length and transfer moderate to heavy loads. However, the labour requirement for fabrication and erection of trusses is higher and hence the relative economy is dictated by different factors. In India these considerations are likely to favour the trusses even more because of the lower labour cost. In order to fully utilize the economy of the trusses the designers should ascertain the following:

- Method of fabrication and erection to be followed, facility for shop fabrication available, transportation restrictions, field assembly facilities.

- Preferred practices and past experience.
- Availability of materials and sections to be used in fabrication.
- Erection technique to be followed and erection stresses.
- Method of connection preferred by the contractor and client (bolting, welding or riveting).
- Choice of as rolled or fabricated sections.
- Simple design with maximum repetition and minimum inventory of material.

2.5 Plastic analysis

In plastic analysis and design of a structure, the ultimate load of the structure as a whole is regarded as the design criterion. The term *plastic* has occurred due to the fact that the ultimate load is found from the strength of steel in the plastic range. This method is rapid and provides a rational approach for the analysis of the structure. It also provides striking economy as regards the weight of steel since the sections required by this method are smaller in size than those required by the method of elastic analysis. Plastic analysis and design has its main application in the analysis and design of statically indeterminate framed structures.

2.5.1 Basics of plastic analysis

Plastic analysis is based on the idealization of the stress-strain curve as elastic-perfectly-plastic. It is further assumed that the width-thickness ratio of plate elements is small so that local buckling does not occur- in other words the sections will classify as plastic. With these assumptions, it can be said that the section will reach its plastic moment capacity and then undergo considerable rotation at this moment. With these assumptions, we will now look at the behaviour of a beam up to collapse.

Consider a simply supported beam subjected to a point load W at mid-span. as shown in Fig. 2.14(*a*). The elastic bending moment at the ends is $w\ell^2/12$ and at mid-span is $w\ell^2/24$, where ℓ is the span. The stress distribution across any cross section is linear [Fig. 2.15(*a*)]. As W is increased gradually, the bending moment at every section increases and the stresses also increase. At a section close to the support where the bending moment is maximum, the stresses in the extreme fibers reach the yield stress. The moment corresponding to this state is called the ***first yield moment*** M_y, of the cross section. But this does not imply failure as the beam can continue to take additional load. As the load continues to increase, more and more fibers reach the yield stress and the stress distribution is as shown in Fig 2.15(*b*). Eventually the whole of the cross section reaches the yield stress and the corresponding stress distribution is as shown in Fig. 2.15(*c*). The moment corresponding to this state is known as the ***plastic moment*** of the cross section and is denoted by M_p. In order to find out the fully plastic moment of a yielded section of a beam, we employ the force equilibrium equation, namely the total force in compression and the total force in tension over that section are equal.

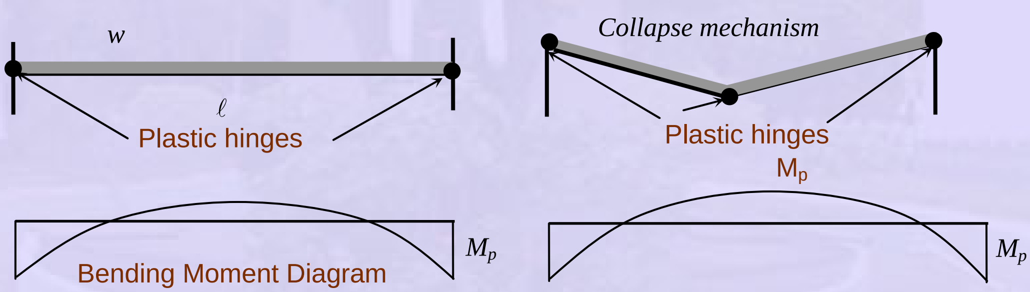


Fig. 2.14 Formation of a collapse mechanism in a fixed beam

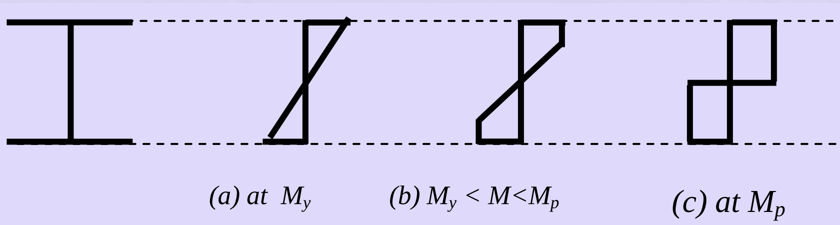


Fig. 2.15 Plastification of cross-section under

The ratio of the plastic moment to the yield moment is known as the ***shape factor*** since it depends on the shape of the cross section. The cross section is not capable of resisting any additional moment but may maintain this moment for some amount of rotation in which case it acts like a ***plastic hinge***. If this is so, then for further loading, the beam, acts as if it is simply supported with two additional moments M_p on either side, and continues to carry additional loads until a third plastic hinge forms at mid-span when the bending moment at that section reaches M_p. The beam is then said to have developed a ***collapse mechanism*** and will collapse as shown in Fig 2.14(*b*). If the section is thin-walled, due to local buckling, it may not be able to sustain the moment for additional rotations and may collapse either before or soon after attaining the plastic moment. It may be noted that formation of a single plastic hinge gives a collapse mechanism for a simply supported beam. The ratio of the ultimate rotation to the yield rotation is called the ***rotation capacity*** of the section. The yield and the plastic moments together with the rotation capacity of the cross-section are used to classify the sections.

Shape factor

As described previously there will be two stress blocks, one in tension, the other in compression, both of which will be at yield stress. For equilibrium of the cross section, the areas in compression and tension must be equal. For a rectangular cross section, the elastic moment is given by,

$$M = \frac{bd^2}{6} f_y \qquad \textbf{\textit{(2.21a)}}$$

The plastic moment is obtained from,

$$M_p = 2.b.\frac{d}{2}.\frac{d}{4}.f_y \qquad = \frac{bd^2}{4}f_y \qquad \textbf{\textit{(2.21b)}}$$

Thus, for a rectangular section the plastic moment M_p is about 1.5 times greater than the elastic moment capacity. For an I-section the value of shape factor is about 1.12.

Theoretically, the plastic hinges are assumed to form at points at which plastic rotations occur. Thus the length of a plastic hinge is considered as zero. However, the values of moment, at the adjacent section of the yield zone are more than the yield moment upto a certain length ΔL, of the structural member. This length ΔL, is known as the hinged length. The hinged length depends upon the type of loading and the geometry of the cross-section of the structural member. The region of hinged length is known as ***region of yield*** or ***plasticity.***

Rigid plastic analysis

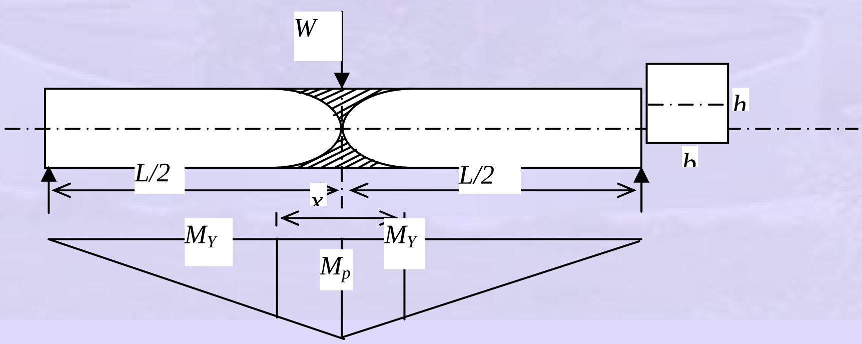


Fig. 2.16

In a simply supported beam (Fig. 2.16) with central concentrated load, the maximum bending moment occurs at the centre of the beam. As the load is increased gradually, this moment reaches the fully plastic moment of the section M_p and a plastic hinge is formed at the centre.

Let x (= ΔL) be the length of plasticity zone.

From the bending moment diagram shown in Fig. 2.16

$$(L-x)M_p=LM_y$$

$$x=L/3 \qquad \textbf{(2.22)}$$

Therefore the hinged length of the plasticity zone is equal to one-third of the span in this case.

$$M_p = \frac{Wl}{4}$$

$$= f_y \cdot \frac{bh^2}{4} \left(\because Z_p = \frac{bh^2}{4} \right)$$

$$M_y = f_y \cdot \frac{bh^2}{6} = \left(f_y \cdot \frac{bh^2}{4} \right) \frac{2}{3}$$

$$M_y = \frac{2}{3} M_p$$

2.5.2 Principles of plastic analysis

Fundamental conditions for plastic analysis

(i) **Mechanism condition:** *The ultimate or collapse load is reached when a mechanism is formed. The number of plastic hinges developed should be just sufficient to form a mechanism.*

(ii) **Equilibrium condition** : $\Sigma F_x = 0$, $\Sigma F_y = 0$, $\Sigma M_{xy} = 0$

(iii) **Plastic moment condition:** *The bending moment at any section of the structure should not be more than the fully plastic moment of the section.*

Collapse mechanisms

When a system of loads is applied to an elastic body, it will deform and will show a resistance against deformation. Such a body is known as a *structure*. On the other hand if no resistance is set up against deformation in the body, then it is known as a *mechanism.*

Various types of independent mechanisms are identified to enable prediction of possible failure modes of a structure.

(i) Beam mechanism

Fig. 2.17 shows a simply supported and a fixed beam and the corresponding mechanisms.

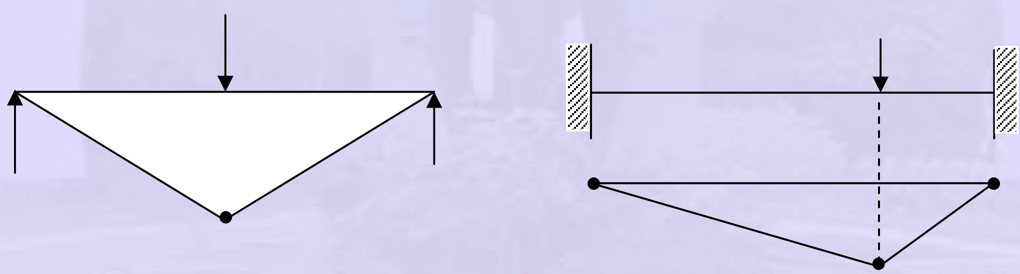
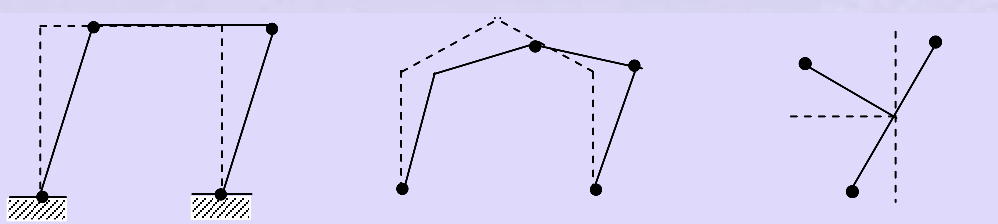


Fig. 2.17

(ii) Panel or Sway mechanism

Fig. 2.18 (A) shows a panel or sway mechanism for a portal frame fixed at both ends.



(A) Panel mechanism **(B) Gable mechanism** **(C) Joint mechanism**

Fig. 2.18

(iii) Gable mechanism

Fig. 2.18(B) shows the gable mechanism for a gable structure fixed at both the supports.

(iv) Joint mechanism

Fig. 2.18(C) shows a joint mechanism. It occurs at a joint where more than two structural members meet.

Combined mechanism

Various combinations of independent mechanisms can be made depending upon whether the frame is made of strong beam and weak column combination or strong column and weak beam combination. The one shown in Fig. 2.19 is a combination of a beam and sway mechanism. Failure is triggered by formation of hinges at the bases of the columns and the weak beam developing two hinges. This is illustrated by the right hinge being shown on the beam, in a position slightly away from the joint.

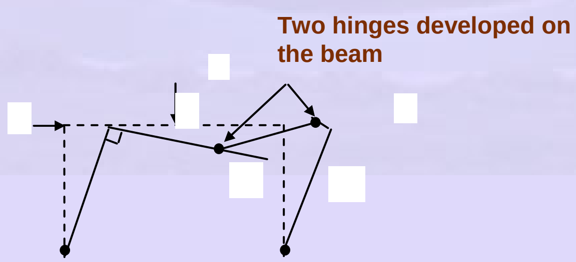


Fig. 2.19 Combined mechanism

From the above examples, it is seen that the number of hinges needed to form a mechanism equals the statical redundancy of the structure plus one.

Plastic load factor and theorems of plastic collapse

The plastic load factor at rigid plastic collapse *(λ_p)* is defined as the lowest multiple of the design loads which will cause the whole structure, or any part of it to become a mechanism.

In a limit state approach, the designer is seeking to ensure that at the appropriate factored loads the structure will not fail. Thus the rigid plastic load factor λ_p must not be less than unity.

The number of independent mechanisms *(n)* is related to the number of possible plastic hinge locations *(h)* and the number of degree of redundancy *(r)* of the frame by the equation.

$$n = h - r \qquad \textbf{(2.23)}$$

The three theorems of plastic collapse are given below.

Lower bound or Static theorem

A load factor (λ_s) computed on the basis of an arbitrarily assumed bending moment diagram which is in equilibrium with the applied loads and where the fully plastic moment of resistance is nowhere exceeded will always be less than or at best equal to the load factor at rigid plastic collapse, (λ_p). In other words, λ_p is the highest value of λ_s which can be found.

Upper bound or Kinematic theorem

A load factor (λ_k) computed on the basis of an arbitrarily assumed mechanism will always be greater than, or at best equal to the load factor at rigid *plastic collapse (λ_p). In other words, λ_p is the lowest value of λ_k which can be found.*

Uniqueness theorem

If both the above criteria are satisfied, then the resulting load factor corresponds to its value at rigid plastic collapse (λ_p).

Mechanism method

In the mechanism or kinematics method of plastic analysis, various plastic failure mechanisms are evaluated. The plastic collapse loads corresponding to various failure mechanisms are obtained by equating the internal work at the plastic hinges to the external work by loads during the virtual displacement. This requires evaluation of displacements and plastic hinge rotations.

As the plastic deformations at collapse are considerably larger than elastic ones, it is assumed that the frame remains rigid between supports and hinge positions i.e. all plastic rotation occurs at the plastic hinges.

Considering a simply supported beam subjected to a point load at midspan, the maximum strain will take place at the centre of the span where a plastic hinge will be formed at yield of full section. The remainder of the beam will remain straight, thus the entire energy will be absorbed by the rotation of the plastic hinge.

Considering a centrally loaded simply supported beam at the instant of plastic collapse (see Fig. 2.17)

Workdone at the plastic hinge = $M_p 2\theta$ **(2.24a)**

Workdone by the displacement of the load = $W\left(\frac{L}{2}.\theta\right)$ **(2.24b)**

At collapse, these two must be equal

$$2Mp.\theta = W\left(\frac{L}{2}.\theta\right)$$

$$M_p = \frac{WL}{4} \qquad \textbf{(2.25)}$$

The moment at collapse of an encastre beam with a uniform load is similarly worked out from Fig. 2.20. It should be noted that three hinges are required to be formed at *A, B* and *C* just before collapse.

Workdone at the three plastic hinges $= M_p(\theta + 2\theta + \theta) = 4M_p\theta$ **(2.26.a)**

Workdone by the displacement of the load $= W/L\ .\ L/2\ .\ L/2\ .\ \theta$ **(2.26.b)**

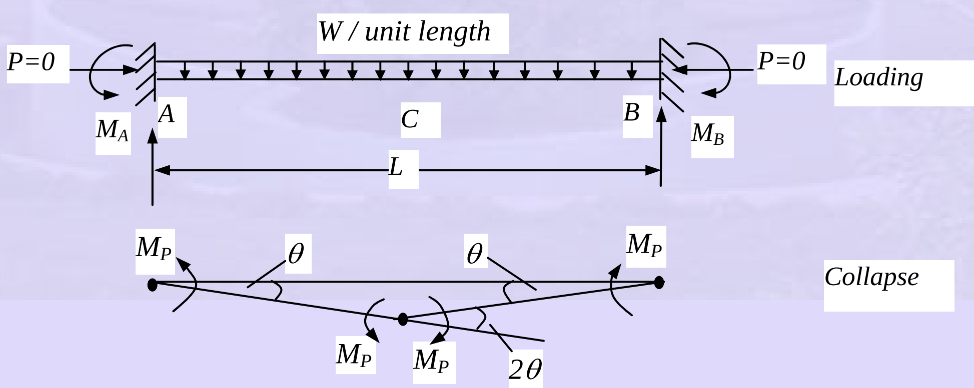


Fig. 2.20 Encastre beam

$$\frac{WL}{4}\theta = 4M_p\theta \qquad \textbf{(2.27)}$$

$$WL \quad = 16\,M_p$$

$$M_p = \frac{WL}{16} \qquad \textbf{\textit{(2.28)}}$$

In other words the load causing plastic collapse of a section of known value of M_p is given by eqn. (2.28).

Rectangular portal framework and interaction diagrams

The same principle is applicable to frames as indicated in Fig. 2.21(a) where a portal frame with constant plastic moment of resistance M_p throughout is subjected to two independent loads H and V.

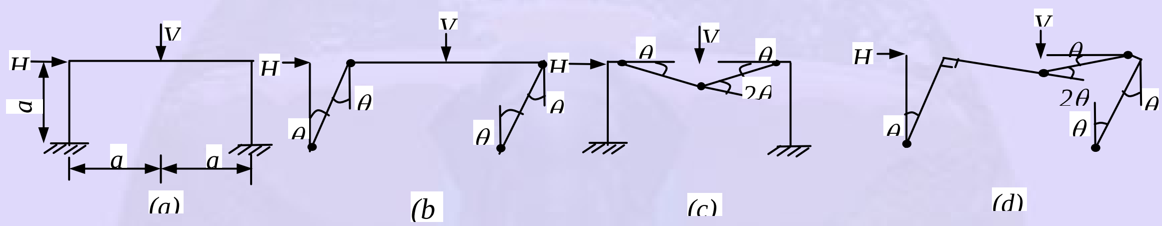


Fig.2.21 Possible failure mechanisms

This frame may distort in more than one mode. There are basic independent modes for the portal frame, the pure sway of Fig. 2.21 (b) and a beam collapse as indicated in Fig. 2.21 (c). There is now however the possibility of the modes combining as shown in Fig. 2.21(d).

From Fig. 2.21(b)

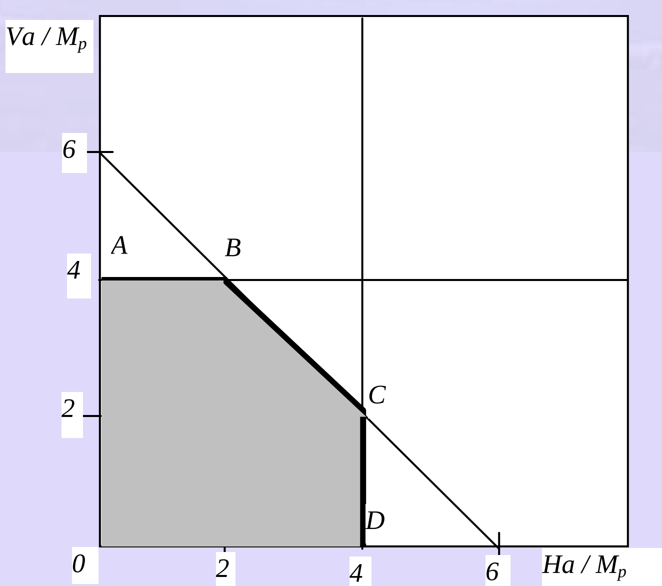


Fig. 2.22

Work done in hinges = $4 M_p\theta$

Work done by loads = $Ha\theta$

At incipient collapse $Ha / M_p = 4$ (2.29)

From Fig. 2.21 (c)

Work done in hinges = $4 M_p\theta$

Work done by loads = $Va\theta$

At incipient collapse = $Va / M_p = 4$ (2.30)

From Fig. 2.21(d)

Work done in hinges = $6 M_p\theta$

Work done by loads = $Ha\theta + Va\theta$

At incipient collapse $Ha / M_p + Va / M_p = 6$ (2.31)

The resulting equations, which represent the collapse criteria, are plotted on the interaction diagram of Fig. 2.22. Since any line radiating from the origin represents proportional loading, the first mechanism line intersected represents failure. The failure condition is therefore the line *ABCD* and any load condition within the area *OABCD* is therefore safe.

Stability

For plastically designed frames three stability criteria have to be considered for ensuring the safety of the frame. These are

1. General Frame Stability.
2. Local Buckling Criterion.
3. Restraints.

Effect of axial load and shear

If a member is subjected to the combined action of bending moment and axial force, the plastic moment capacity will be reduced.

The presence of an axial load implies that the sum of the tension and compression forces in the section is not zero (Fig. 2.23). This means that the neutral axis moves away from the equal area axis providing an additional area in tension or compression depending on the type of axial load.

the interaction equation can be obtained:

$$M_x/M_p = 1 - P^2/P_y \qquad \textbf{(2.32)}$$

The presence of shear forces will also reduce the moment capacity.

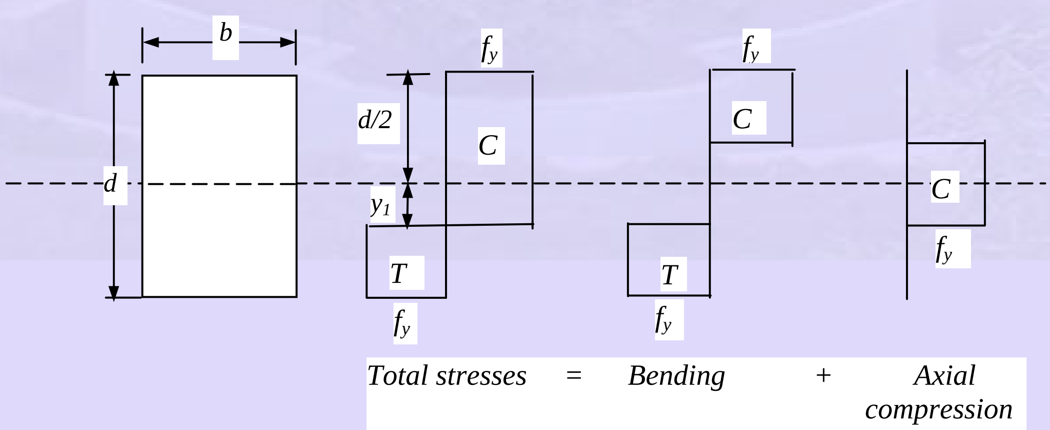


Fig. 2.23 Effect of axial force on plastic moment capacity

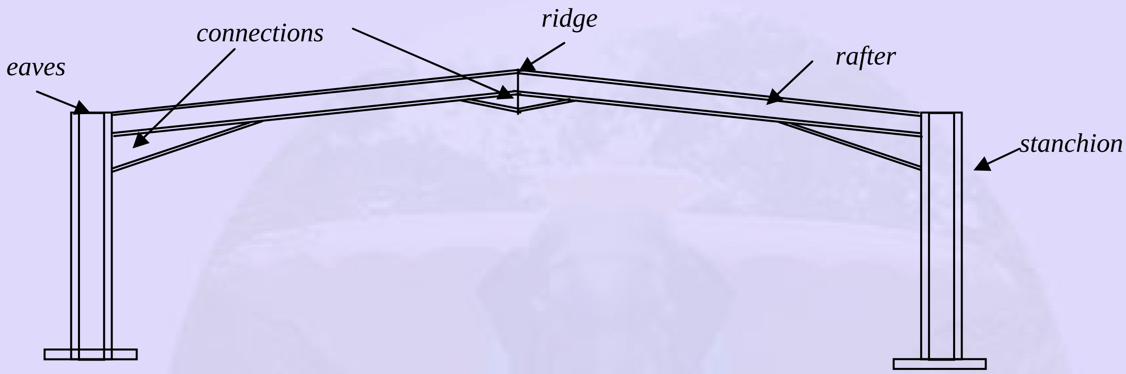
Plastic analysis for more than one condition of loading

When more than one condition of loading can be applied to a beam or structure, it may not always be obvious which is critical. It is necessary then to perform separate calculations, one for each loading condition, the section being determined by the solution requiring the largest plastic moment.

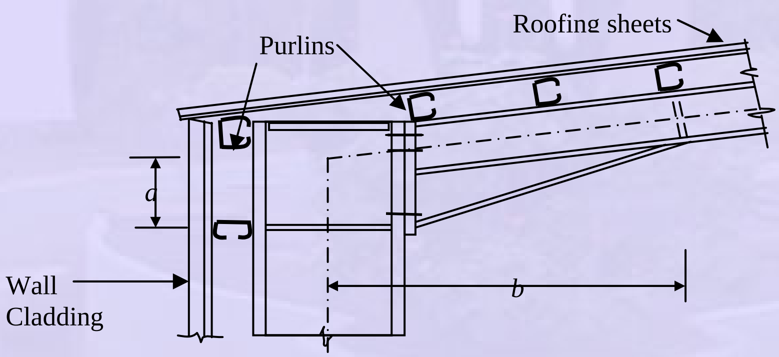
Unlike the elastic method of design in which moments produced by different loading systems can be added together, plastic moments obtained by different loading systems cannot be combined, i.e. the plastic moment calculated for a given set of loads is only valid for that loading condition. This is because the 'Principle of Superposition' becomes invalid when parts of the structure have yielded.

2.6 Portal frames

Portal frames are the most commonly used structural forms for single-storey industrial structures. They are constructed mainly using hot-rolled sections, supporting the roofing and side cladding via cold-formed purlins and sheeting rails. They may also be composed of tapered stanchions and rafters fabricated from plate elements. Portal frames of lattice members made of angles or tubes are also common, especially in the case of longer spans.



(a) Elevation



(b) Eaves detail

Fig. 2.24 Haunched gable portal frame

The slopes of rafters in the gable portal frames (Fig.2.24) vary in the range of *1* in *10* to *1* in *3.* Generally, the centre-to-centre distance between frames is of the order *6 to 7.5 m,* with eaves height ranging from *6 -15 m*. Normally, larger spacing of frames is used in the case of taller buildings, from the point of economy. Moment-resisting connections are to be provided at the eaves and crown to resist lateral and gravity loadings. The stanchion bases may behave as either pinned or fixed, depending upon rotational restraint provided by the foundation and the connection detail between the stanchion and foundations. The foundation restraint depends on the type of foundation and modulus of the sub-grade. Frames with pinned bases are heavier than those having fixity at the bases. However, frames with fixed base may require a more expensive foundation.

For the design of portal frames, plastic methods of analysis are mainly used, which allows the engineer to analyse frames easily and design it economically. The basis of the plastic analysis method is the need to determine the load that can be applied to the frame so that the failure of the frame occurs as a mechanism by the formation of a number of plastic hinges within the frame. The various methods of plastic analysis are discussed later.

The most common form of portal frame used in the construction industry is the pinned-base frame with different rafter and column member size and with haunches at both the eaves and apex connections (Fig.2.24).

Due to transportation requirements, field joints are introduced at suitable positions. As a result, connections are usually located at positions of high moment, i.e. at the interface of the column and rafter members (at the eaves) and also between the rafter members at the apex (ridge) (See Fig.2.24). It is very difficult to develop sufficient moment capacity at these connections by providing 'tension' bolts located solely within the small depth of the rafter section. Therefore the lever arm of the bolt group is usually increased by haunching the rafter members at the joints. This addition increases the section strength.

2.6.1 General design procedure

The steps in the plastic design of portals, according to SP: 6(6) – 1972, are given below:

a) Determine possible loading conditions.

b) Compute the factored design load combination(s).

c) Estimate the plastic moment ratios of frame members.

d) Analyse the frame for each loading condition and calculate the maximum required plastic moment capacity, M_p

e) Select the section, and

f) Check the design for other secondary modes of failure (IS: 800-1984).

The design commences with determination of possible loading conditions, in which decisions such as, whether to treat the distributed loads as such or to consider them as equivalent concentrated loads, are to be made. It is often convenient to deal with equivalent concentrated loads in computer aided and plastic analysis methods.

In step (b), the loads determined in (a) are multiplied by the appropriate load factors to assure the needed margin of safety. This load factor is selected in such a way that the real factor of safety for any structure is at least as great as that decided upon by the designer. The load factors to be used for various load combinations are presented in an earlier chapter on Limit states method.

The step (c) is to make an assumption regarding the ratio of the plastic moment capacities of the column and rafter, the frame members. Optimum plastic design methods present a direct way of arriving at these ratios, so as to obtain an optimum value of this ratio. The following simpler procedure may be adopted for arriving at the ratio.

(i) Determine the absolute plastic moment value for separate loading conditions.

(Assume that all joints are fixed against rotation, but the frame is free to sway). For beams, solve the beam mechanism equation and for columns, solve the panel (sway) mechanism equation. These are done for all loading combinations. The moments thus obtained are the absolute minimum plastic moment values. The actual section moment will be greater than or at least equal to these values.

(ii) Now select plastic moment ratios using the following guidelines.

- At joints establish equilibrium.
- For beams use the ratio determined in step (i)
- For columns use the corner connection moments M_p *(Col)* = M_p *(beam)*

In the step (d) each loading condition is analysed by a plastic analysis method for arriving at the minimum required M_p. Based on this moment, select the appropriate sections in step (e). The step (f) is to check the design according to secondary design considerations discussed in the following sections (IS: 800-1984).

2.6.2 Secondary design considerations

The 'simple plastic theory' neglects the effects of axial force, shear and buckling on the member strength. So checks must be carried out for the following factors.

a) Reductions in the plastic moment due to the effect of axial force and shear force.
b) Instability due to local buckling, lateral buckling and column buckling.
c) Brittle fracture.
d) Deflection at service loads.

In addition, proper design of connections is needed in order that the plastic moments can be developed at the plastic hinge locations.

Connections

In a portal frame, points of maximum moments usually occur at connections. Further, at corners the connections must accomplish the direction of forces change. Therefore, the design of connections must assure that they are capable of developing and maintaining the required moment until the frame fails by forming a mechanism.

There are four principal requirements, in design of a connection

a) *Strength* - The connection should be designed in such a way that the plastic moment (M_p) of the members (or the weaker of the two members) will be developed. For straight connections the critical or 'hinge' section is assumed at point *H* in Fig. 6 (*a*). For haunched connections, the critical sections are assumed at R_1 and R_2, [Fig. 6 (b)].

b) *Stiffness* - Average unit rotation of the connecting region should not exceed that of an equivalent length of the beam being joined. The equivalent length is the length of the connection or haunch measured along the frame line. Thus in Fig. 6(a).

$$\Delta L = r_1 + r_2$$

This requirement reduces to the following

$$\theta_h \leq \frac{M_p}{EI}.\Delta L \qquad \textbf{(2.33)}$$

Where θ_h is the joint rotation.

Eq. 12 states that the change in angle between sections R_1 and R_2 as computed shall not be greater than the curvature (rotation per unit of length) times the equivalent length of the knee.

c) *Rotation Capacity* – The plastic rotation capacity at the connection hinge is adequate to assure that all necessary plastic hinges will form in the structure to enable failure mechanism and hence all connections should be proportioned to develop adequate rotation at plastic hinges.

d) *Economy* - Extra connecting materials and labour required to achieve the connection should be kept to a minimum.

If the knee web is deficient in resisting the shear force, a diagonal stiffener may be used. (Fig. 2.25)

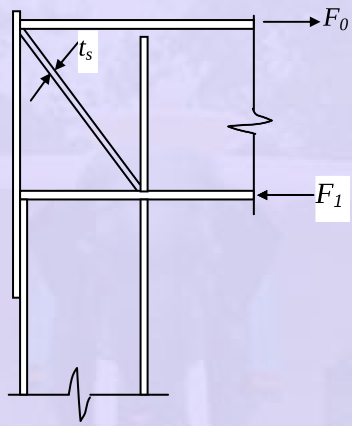


Fig. 2.25 Stiffened corner joint

Haunched connections

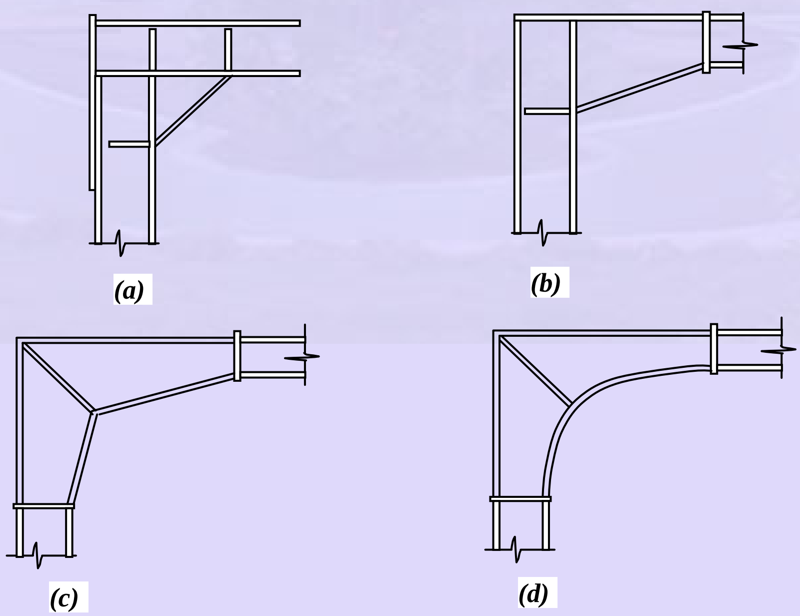


Fig. 2.26 Haunched connection types

Some of the typical haunched connections are shown in Fig. 2.26. Haunched connections are to be proportioned to develop plastic moment at the junction between the rolled steel section and the haunch. In order to force formation of hinge at the end of a tapered haunch (Fig. 2.26), make the flange thickness in the haunch, to be *50* percent greater than that of section joined. Check the shear resistance of the web to ensure M_p governs the strength.

2.7 Crane gantry girders

The function of the crane girders is to support the rails on which the traveling cranes move. These are subjected to vertical loads from crane, horizontal lateral loads due to surge of the crane, that is, the effect of acceleration and braking of the loaded crab and swinging of the suspended load in the transverse direction, and longitudinal force due to acceleration and braking of the crane as a whole. In addition to the weight of the crane, impact and horizontal surge must be considered. According to IS: 875, the values given in Table 13-9 may be taken for the design of crane gantry girders and columns. Both the horizontal forces, lateral and longitudinal, are assumed not to act together with the vertical loads simultaneously. Only one of them is to be considered acting with the vertical load at a time. Vertical load, of course, includes the additional load due to impact.

Table 2.1

Impact and surge of cranes

Type of Load	Additional Loads
Vertical – electrical operated hand operated	 25% of max. static wheel load 10% of max. static wheel load
Horizontal, lateral to rails electrically operated hand operated	5% of weight of crab plus weight lifted per rail 2 ½% of weight of crab plus weight lifted per rail
Horizontal, along rails	5% f max. static wheel load

The crane girder spans from column to column, usually having no lateral support at intermediate points excepting when a walkway is formed at the top level of the girder which restrains the girder from lateral bending. Thus under normal circumstances, the crane girder must be designed as laterally unsupported beam carrying vertical and horizontal load at the level of the top flange. Apparently a girder with heavier and wider compression flange is required. Figure 2.27 shows some typical sections adopted for crane girders

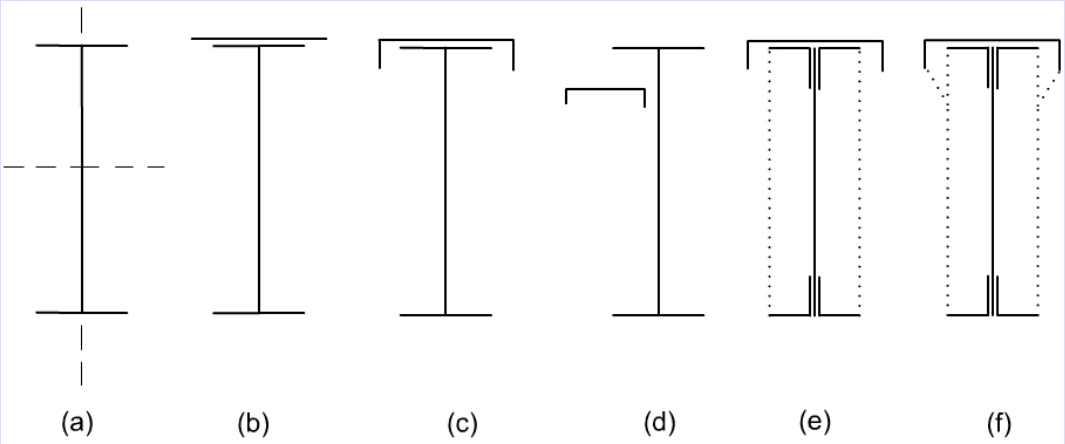


Fig. 2.27 Crane girders (Typical sections)

Fig. 2.27(a) shows a wide flange beam with out any reinforcement and may be used for short spans and very light crane loads. In Fig.2.27 (b), a cover plate is used on the compression face which improves the lateral buckling strength of the beam and provides larger moment of inertia about the vertical axis against the lateral loads. In Fig. 2.27(c), a channel has been used instead of the cover plate to further increase I_{vv}. In Fig. 2.27(d), the channel is used just below the compression flange of the wide flange beam and is supported by brackets to increase the torsional stiffness of the girder. Figure 2.27(e) and (f) show plate girder sections used for longer spans and heavier crane loads.

The fibre stresses in the gantry crane girders should rationally be computed considering bi-axial bending combined with torsion. The torsion is produced by the lateral force being applied at the top flange. To simplify analysis, it is assumed that the lateral moment is resisted only by the top flange bending horizontally without any assistance from the bottom flange. Of course, it is assumed to be restrained in the vertical plane. The design bending stress therefore will be full value f_y/γ_{mo}. For the moment in the vertical plane produced by vertical crane reaction and self weight of the girder, the full section of the girder is taken effective but with laterally unsupported compression flange. The design stress for vertical bending will be determined according to the rules for unrestrained compression flanges given in Cl.8.2.2. The two stresses should satisfy the relation give in Cl.9.3.1 and Cl.9.3.2.

The crane girders are supported either on brackets connected to columns of uniform section or on stepped columns. Brackets are used for lighter crane loads and the stepped columns for heavy crane loads and taller columns. Some arrangements are shown in Fig 2.27.

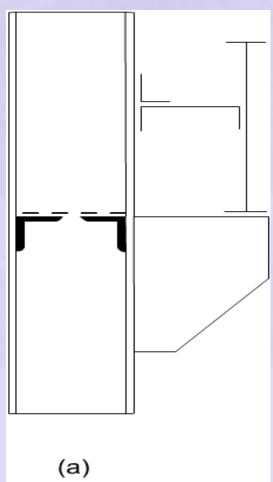


Fig. 2.28 Bracket support for gantry girders

The girder is supported on a suitably formed seat and it is also connected to the column near the top flange in each case in order to restrain it from lateral bending and twisting at the support point. In Fig. 2.28(a), a channel has been used for this purpose. To permit longitudinal movement, due to temperature and deflection, slotted holes are used to connect the channel with the column. In Fig. 2.28 (b) and (c) vertical plates have been employed for providing the restraint. Since such plates are flexible for horizontal bending no slotted holes are necessary. Where roof leg and crane leg are provided separately in a built-up column as shown at (c), the two legs are properly braced together to act as one piece. The bracing is designed to take 2 ½% of column load plus shear due to bending under crane loan and wind. The load of roof leg and crane are eccentric to the combined column axis and should be considered as such.

The crane columns must be properly braced in the longitudinal direction of the crane girders to be able to take the longitudinal forces due to crane. Such brancing may be provided every fourth or fifth bay. In other bays, struts must be used to transmit the longitudinal force to the bracing frame.

2.8 Design for wind action

The wind pressure on a structure depends on the location of the structure, height of structure above the ground level and also on the shape of the structure. The code gives the basic wind pressure for the structures in various parts of the country. Both the wind pressures viz. including wind of short duration and excluding wind of short duration, have been given. All structures should be designed for the short duration wind.

For buildings upto 10m in height, the intensity of wind pressure, as specified in the code, may be reduced by 25% for stability calculations and for the design of framework as well as cladding. For buildings over 10m and upto 30 m height, this reduction can be made for stability calculations and for design of columns only.

The total pressure on the walls or roof of an industrial building will depend on the external wind pressure and also on internal wind pressure. The internal wind pressure depends on the permeability of the buildings. For buildings having a small degree of permeability, the internal air pressure may be neglected. In the case of buildings with normal permeability the internal pressure can be $\pm$ 0.2p. Here '+' indicates pressure and '-' suction, 'p' is the basic wind pressure. If a building has openings larger than 20% of the wind pressure. If a building has openings larger than 20% of the wall area, the internal air pressure will be $\pm$0.5 p.

(a) Wind pressure on walls

The wind pressure per unit area 'p' on the wall is taken as 0.5p pressure on windward surface and 0.5p suction on leeward surface. When the walls form an enclosure, the windward wall will be subjected to a pressure of 0.5p and leeward wall to a suction of 0.5p. The total pressure on the walls will depend on the internal air pressure also.

For buildings with small permeability, design pressure on wall = 0.5p

For buildings with normal permeability, design pressure on wall = 0.7p

For buildings with large openings, design pressure on wall = p

(b) Wind loads on roofs

TABLE 2.2

Wind pressure on roofs (Wind normal to eaves) Sums of external and internal pressure

Roof of pitch	Zero Permeability		Normal Permeability				Large openings			
	External Pressure		$p_1^* = +0.2p$		$p_1^* = -0.2p$		$p_1^* = +0.5p$		$p_1^* = -0.5p$	
	Windward	Leeward	Windward	Leeward	Windward	Leeward	Windward	Leeward	Windward	Leeward
1	2	3	4	5	6	7	8	9	10	11
0	-1.00	-0.50	-1.2	-0.70	-0.8	-0.30	-1.5	-1.00	-0.5	0.00
10	-0.70	-0.50	-0.9	-0.70	-0.5	-0.30	-1.2	-1.00	-0.2	0.00
20	-0.40	-0.50	-0.6	-0.70	-0.2	-0.30	-0.9	-1.00	+0.1	0.00
30	-0.10	-0.50	-0.3	-0.70	+0.1	-0.30	-0.6	-1.00	+0.4	0.00
40	+0.10	-0.50	-0.1	-0.70	+0.3	-0.30	-0.4	-1.00	+0.6	0.00
50	+0.30	-0.50	+0.1	-0.70	+0.5	-0.30	-0.2	-1.00	+0.8	0.00
60	+0.40	-0.50	+0.2	-0.70	+0.6	-0.30	-0.1	-1.00	+0.9	0.00
70	+0.50	-0.50	+0.3	-0.70	+0.7	-0.30	0	-1.00	+1.00	0.00
80	+0.50	-0.50	+0.3	-0.70	+0.7	-0.30	0	-1.00	+1.00	0.00
90	+0.50	-0.50	+0.3	-0.70	+0.7	-0.30	0	-1.00	+1.00	0.00

p_1^* => internal pressure

The pressure normal to the slope of the roof is obtained by multiplying the basic pressure p by the factors given in Table 13-3. The table also shows the effect of internal pressure produced due to the permeability of the cladding or opening in walls and roof.

If the wind blows parallel to the ridge of the roof, the average external wind pressure of the roof may be taken as -0.6p on both slopes of the roof over a length from the gable end equal to the mean height of the roof above the surrounding ground level and as-0.4p over the remaining length of the roof on both slopes.

When the wind blows parallel to a surface, a wind force acts on the surface in the direction of the wind. This force is called the 'Wind Drag'. In the case of industrial buildings, when the wind blows normal to the ridges, the wind drag is equal to 0.05p measured on plan area of roof and when the direction of wind parallel to the ridge, wind drag is equal to 0.025p measured on plan area of roof.

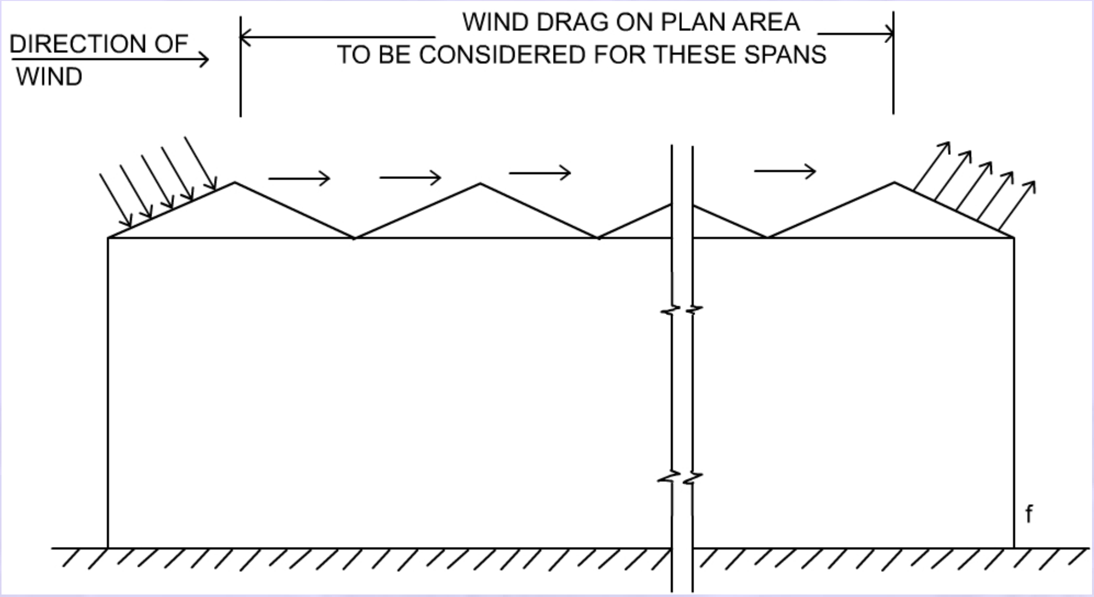


Fig. 2.29 Wind drag

In the multispan roofs with spans, heights and slopes nearly equal, the wind-ward truss gives shelter to the other trusses. For general stability calculations and for the design columns, the windward slope of wind-ward span and leeward slope of leeward span are subjected to the full normal pressure of suction as given in table 2.2 and on all other roof slopes, only wind drag is considered (see Fig. 2.29). For the design of roof trusses, however, full normal pressure or suction is considered on both faces, presuming that there was only one span.

The wind pressures given above are the average pressures on a roof slope. For designing the roof sheeting or the fastenings of roof sheeting, we may take a larger wind pressure because these pressures may considerably exceed the average value on small areas. For designing roof sheeting and its fastenings, the values given in Table 2.2 may be increased numerically by 0.3p. In a distance equal to 15% of the length of the roof from the gable ends, fastenings should be capable of resisting a section of 2.0p on the area of the roof sheeting them support.

2.9 Design for earthquake action

Single storey industrial buildings are usually governed by wind loads rather than earthquake loads. This is because their roofs and walls are light in weight and often pitched or sloping and also because the buildings are permeable to wind which results in uplift of the roof. However, it is always safe to check any building for both wind and earthquakes.

Earthquake loading is different from wind loading in several respects and so earthquake design is also quite different from design for wind and other gravity loads. Severe earthquakes impose very high loads and so the usual practice is to ensure elastic behaviour under moderate earthquake and provide ductility to cater for severe earthquakes. Steel is inherently ductile and so only the calculation of loads due to moderate earthquake is considered. This can be done as per the IS 1893 code. According to this code, a horizontal seismic coefficient times the weight of the structure should be applied as equivalent static earthquake load and the structure should be checked for safety under this load in combination with other loads as specified in IS 800. The combinations are as follows:

1. 1.5 (*DL* + *IL*)
2. 1.2 (*DL* + *IL* $\pm$ *EL*)
3. 1.5 (*DL* + *EL*)
4. 0.9 *DL* $\pm$ 1.5 *EL*

The horizontal seismic coefficient A_h takes into account the location of the structure by means of a zone factor Z, the importance of the structure by means of a factor I and the ductility by means of a factor R. It also considers the flexibility of the structure-foundation system by means of an acceleration ratio Sa/g , which is a function of the natural time period T. This last ratio is given in the form of a graph known as the response spectrum. The horizontal seismic coefficient A_h is given by

$$A_n = \frac{ZIS_a}{2R_g}$$

Where Z = Zone factor corresponding to the seismic zone obtained from a map (Table 2.3); *I* = Importance factor; *R* = Response reduction factor.

Table 2.3 Zone factor, Z

Seismic Zone	**II**	**III**	**IV**	**V**
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

For industries using hazardous materials and fragile products the importance factor may be taken as 1.5 but for most industries it may be taken as 1.0. The Response reduction factor R may be taken as 4 for buildings where special detailing as per section 12 of IS 800 has not been followed.

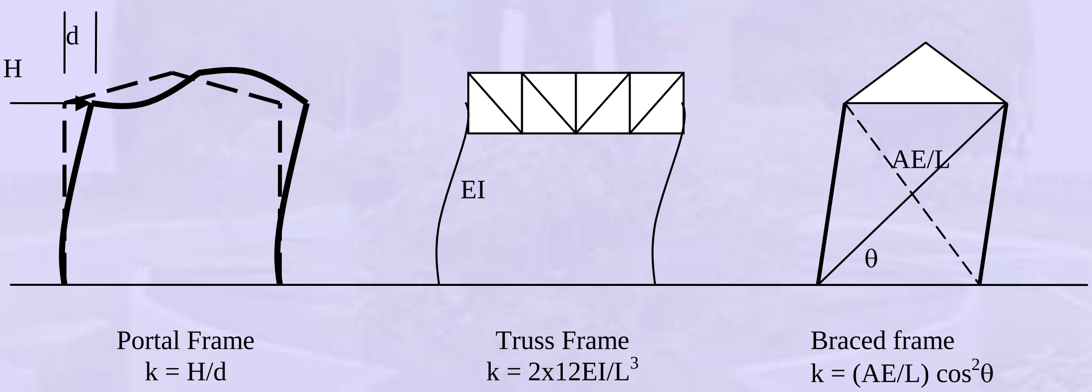


Fig. 2.30 Lateral stiffness for various

The natural time period T is very important and should be calculated correctly. For single storey structures, it may be taken as T = $2\pi\sqrt{(k/m)}$ where k is the lateral (horizontal) stiffness of the supporting structure and m is the mass of the roof usually taken as the sum of the roof dead load plus 50% of the live load divided by the acceleration due to gravity g. Guidelines for calculating k in some simple cases are given in Fig. 2.30.

Finally, the acceleration ratio Sa/g can be obtained from the graph corresponding to the soil type as shown in Fig. 2.31. In this figure, medium soil corresponds to stiff clay or sand and soft soil corresponds to loose clay and loamy soils.

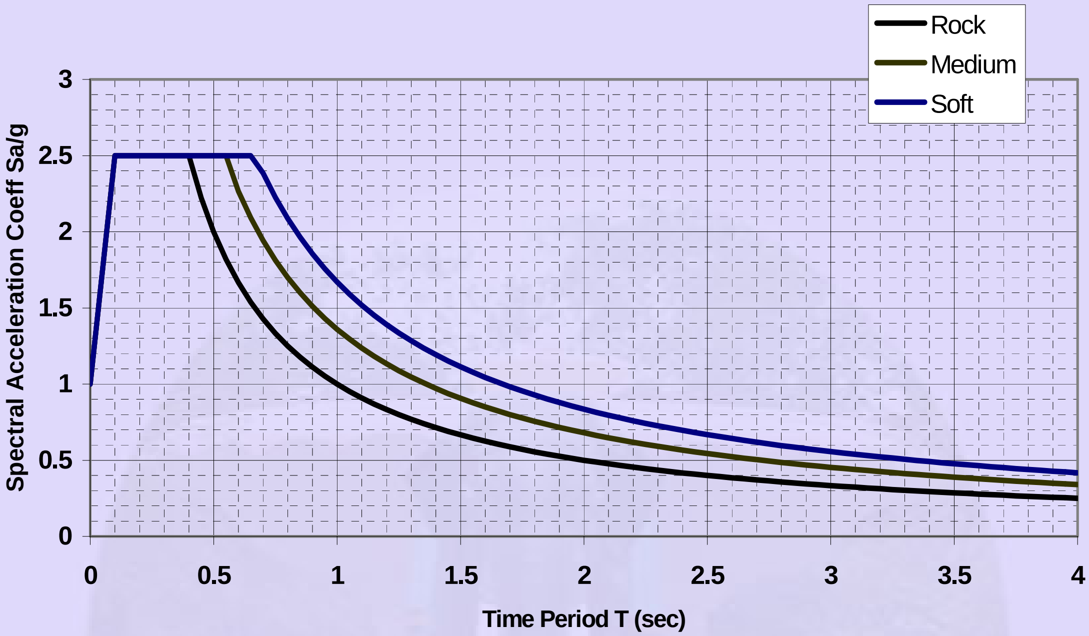


Fig. 2.31 response spectrum for 5% damping

2.10 Summary

In this chapter, initially the structural and functional requirements of industrial buildings were described. The loads to be considered were mentioned and then the floor and roof systems were explained. In particular, the steel-concrete composite slab using profiled deck sheets was explained along with the design methods. The topic of roof trusses was covered in detail.

The basic concepts of Plastic Analysis were discussed and the methods of computation of ultimate load causing plastic collapse were outlined. Theorems of plastic collapse and alternative patterns of hinge formation triggering plastic collapse have been discussed. The use of plastic analysis and design of steel portal frames for single storey industrial buildings was discussed.

The calculation of wind and earthquake loads was also described.

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Example Problem

An Industrial building of plan 15m×30m is to be constructed as shown in Fig.E1. Using plastic analysis, analyse and design the single span portal frame with gabled roof. The frame has a span of 15 m, the column height is 6m and the rafter rise is 3 m and the frames are spaced at 5 m centre-to-centre. Purlins are provided over the frames at 2.7 m c/c and support AC sheets. The dead load of the roof system including sheets, purlins and fixtures is 0.4 kN/m^2 and the live load is 0.52 kN/m^2. The portal frames support a gantry girder at 3.25 m height, over which an electric overhead travelling (EOT) crane is to be operated. The crane capacity is to be 300 kN and the crane girder weighs 300 kN while the crab (trolley) weight is 60 kN.

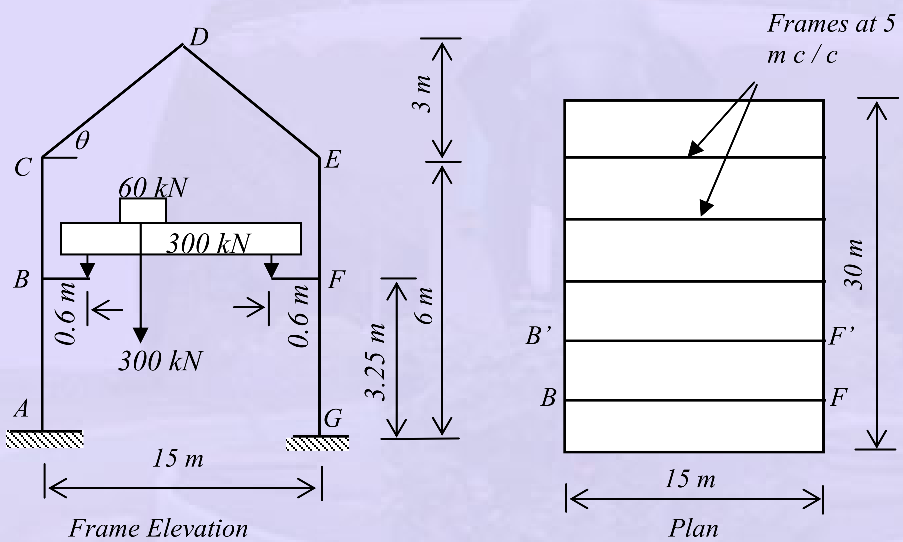


Fig. E1 Details of an Industrial Building

1.0 Load Calculations

1.1 Dead Load of roof given as 0.4 kN/m^2

Dead load/m run on rafter = 0.4 * 5 ≈ 2.0 kN/m

1.2 Live Load given as 0.52 kN/m^2

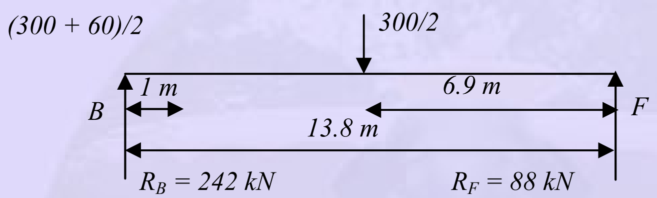
Live load/m run on rafter = 0.52 * 5 ≈ 2.6 kN/m

1.3 Crane Load

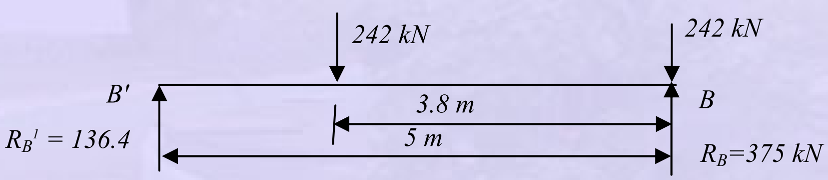
The extreme position of crane hook is assumed as 1 m from the centre line of rail. The span of crane is approximately taken as 13.8 m. And the wheel base along the gantry girder has been taken as 3.8 m

1.3.1 Vertical load on gantry

The weight of the crane is shared by two portal frames At the extreme position of crab, the reaction on wheel due to the lifted weight and the crab can be obtained by taking moments about the centreline of wheels (point B).



To get maximum wheel load on a frame from gantry girder BB', taking the gantry girder as simply supported.



Centre to centre distance between frames is 5 m c/c.

Assuming impact factor of 25%

Maximum wheel Load @ B = 1.25 (242 (1 + (5-3.8)/5)

= 375 kN.

Minimum wheel Load @ B = (88 /242)*375

=136.4 kN

1.3.2 Transverse Load (Surge):

Lateral load per wheel = 5% (300 + 60)/2 = 9 kN

(i.e. Lateral load is assumed as 5% of the lifted load and the weight of the crab acting on each rail).

Lateral load on each column $= \frac{9}{242} * 375$ = 13.9 kN

(By proportion)

1.4 Wind Load

Design wind speed, $V_z = k_1\ k_2\ k_3\ V_b$

From Table 1; IS: 875 (part 3) – 1987

k_1 = 1.0 (risk coefficient assuming 50 years of design life)

From Table 2; IS: 875 (part 3) – 1987

k_2 = 0.8 (assuming terrain category 4)

k_3 = 1.0 (topography factor)

Assuming the building is situated in Chennai, the basic wind speed is 50 m /sec

Design wind speed, $V_z = k_1\ k_2\ k_3\ V_b$

$V_z = 1 * 0.8 * 1 * 50$

$V_z = 40$ m/sec

Design wind pressure, $P_d = 0.6 * V_z^2$

$= 0.6 * (40)^2$

$= 0.96$ kN/m^2

1.4.1. Wind Load on individual surfaces

The wind load, W_L acting normal to the individual surfaces is given by

$$W_L = (C_{pe} - C_{pi})\, A^*P_d$$

(a) Internal pressure coefficient

Assuming buildings with low degree of permeability

$$C_{pi} = \pm 0.2$$

(b) External pressure coefficient

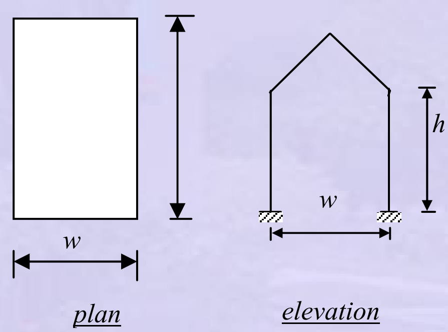
External pressure coefficient for walls and roofs are tabulated in Table 1 (a) and Table 1(b)

1.4.2 Calculation of total wind load

(a) For walls

h/w = 6/15 = 0.4

L/w = 30/15 = 2.0



Exposed area of wall per frame @ 5 m c/c is A = 5 * 6 = 30 m^2

Wind load on wall / frame, A p_d = 30 * 0.96 = 28.8 kN

Table 1 (a): Total wind load for wall

Wind Angle θ	C_{pe}		C_{pi}	$C_{pe} - C_{pi}$		Total wind(kN) $(C_{pe}-C_{pi})Ap_d$	
	Wind-ward	Lee-ward		Wind ward	Lee ward	Wind ward	Lee ward
0^0	0.7	-0.25	0.2	0.5	-0.45	14.4	-12.9
			-0.2	0.9	-0.05	25.9	-1.4
90^0	-0.5	-0.5	0.2	-0.7	-0.7	-20.2	-20.2
			-0.2	-0.3	-0.3	-8.6	-8.6

(b) For roofs

Exposed area of each slope of roof, per frame (5m length) is

$$A = 5 * \sqrt{(3.0)^2 + (7.5)^2} = 40.4\, m^2$$

For roof, Ap_d = 38.7 kN

Table 1 (b): Total wind load for roof

Wind angle	Pressure Coefficient			$C_{pe} - C_{pi}$		Total Wind Load(kN) $(C_{pe} - C_{pi})\,Ap_d$	
	C_{pe} Wind	C_{pe} Lee	C_{pi}	Wind ward	Lee ward	Wind ward Int.	Lee ward Int.
0^0	-0.328	-0.4	0.2	-0.528	-0.6	-20.4	-23.2
	-0.328	-0.4	-0.2	-0.128	-0.2	-4.8	-7.8
90^0	-0.7	-0.7	0.2	-0.9	-0.9	-34.8	-34.8
	-0.7	-0.7	-0.2	-0.5	-0.5	-19.4	-19.4

2.1 Dead Load

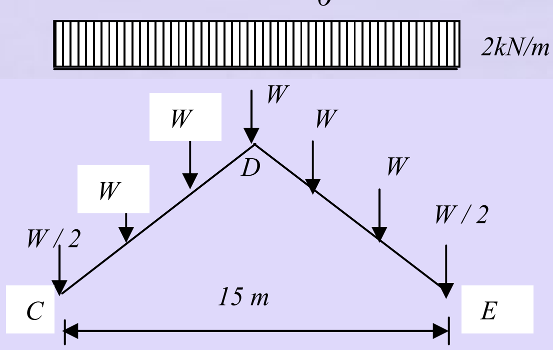
Replacing the distributed dead load of 2kN/m on rafter by equivalent concentrated loads at two intermediate points corresponding to purlin locations on each rafter,

$$W_D = \frac{2.0 * 15}{6} = 5kN$$

2.2 Superimposed Load

Superimposed Load = 2.57 kN/m

Concentrated load , $W_L = \dfrac{2.57 * 15}{6} = 6.4\, kN$



2.3 Crane Load

Maximum Vertical Load on columns = 375 kN (acting at an eccentricity of 600 mm from column centreline)

Moment on column = 375 *0.6 = 225 kNm.

Minimum Vertical Load on Column = 136.4 kN (acting at an eccentricity of 600 mm)

Maximum moment = 136.4 * 0.6 = 82 kNm

3.0 Partial Safety Factors

3.1 Load Factors

For dead load, γ_f = 1.5

For leading live load, γ_f = 1.5

For accompanying live load, γ_f = 1.05

3.2 Material Safety factor

$$\gamma_m = 1.10$$

4.0 Analysis

In this example, the following load combinations is considered, as it is found to be critical. Similar steps can be followed for plastic analysis under other load combinations.

(i) 1.5D.L + 1.5 C .L + 1.05 W.L

4.1. 1.5 D.L + 1.5 C.L+ 1.05 W.L

4.1.1Dead Load and Wind Load along the ridge (wind angle = 0 ^{o})

(a) Vertical Load

w @ intermediate points on windward side

w = 1.5 * 5.0 – 1.05 *(4.8/3) cos21.8

= 6 kN.

$$\frac{w}{2} @ \; eaves = \frac{6}{2} = 3.0\,kN$$

w @ intermediate points on leeward side

w = 1.5 * 5.0 – 1.05 * 7.8/3 cos21.8
= 5.0 kN

$$\frac{w}{2} @ \, eaves = \frac{5.0}{2} = 2.5\, kN$$

Total vertical load @ the ridge = 3.0 + 2.5 = 5.5 kN

b) Horizontal Load

H @ intermediate points on windward side

H = 1.05 * 4.8/3 sin 21.8

= 0.62 kN

H/2 @ eaves points = 0.62/2
= 0.31 kN

H @ intermediate purlin points on leeward side
= 1.05 * 7.8 /3 sin 21.8
= 1 kN

H/2 @ eaves = 0.5 kN

Total horizontal load @ the ridge = 0.5 - 0.31 = 0.19 kN

Table 3: Loads acting on rafter points

	Vertical Load (kN)		Horizontal Load (kN)	
	Windward	Leeward	Windward	Leeward
Intermediate Points	5.2	4.2	0.62	1.0
Eaves	2.6	2.1	0.31	0.5
Ridge	4.7		0.19	

4.1.2 Crane Loading

Moment @ B = 1.5 * 225 = 337.5 kNm

Moment @ F = 1.5 * 82 = 123 kNm

Horizontal load @ B & @ F = 1.5 * 13.9 = 20.8 kN

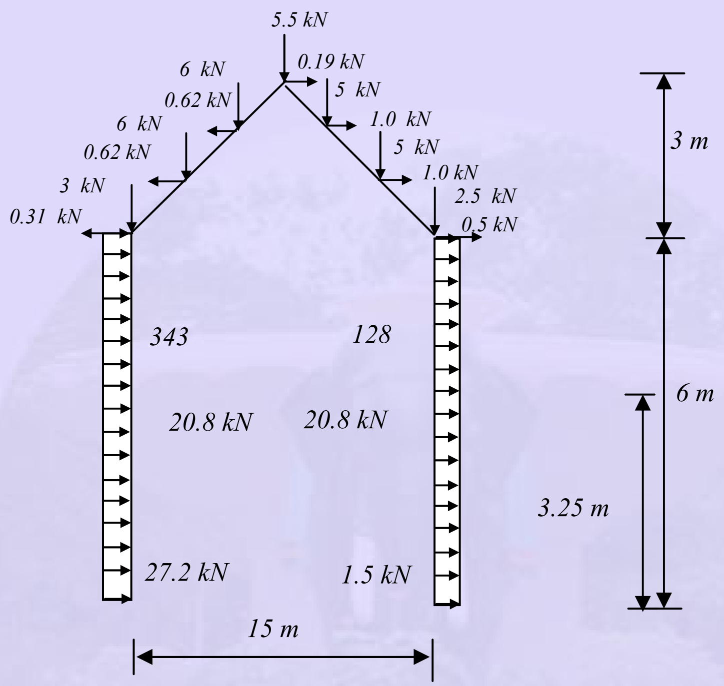
Note: To find the total moment @ B and F we have to consider the moment due to the dead load from the weight of the rail and the gantry girder. Let us assume the weight of rail as 0.3 kN/m and weight of gantry girder as 2.0 kN/m

Dead load on the column = $\left(\dfrac{2+0.3}{2}\right) * 5 = 5.75\ kN$ acting at e=0.6m

Factored moment @ B & F = 1.5 * 5.75 * 0.6 = 5.2 kNm

Total moment @B = 337.5 + 5.2 = 342 kNm

@ F = 123 + 5.2 = 128 kNm



Factored Load (1. 5D.L+1.5 C.L +1.05 W.L)

4.2 1.5 D.L + 1.5 C.L + 1.05 L.L

4.2.1 Dead Load and Live Load

@ intermediate points on windward side = 1.5 * 5.0 + 1.05 * 6.4
= 14.2 kN

@ ridge = 14.2 kN

@ eaves = 14.2 / 2 $\approx$ 7.1 kN.

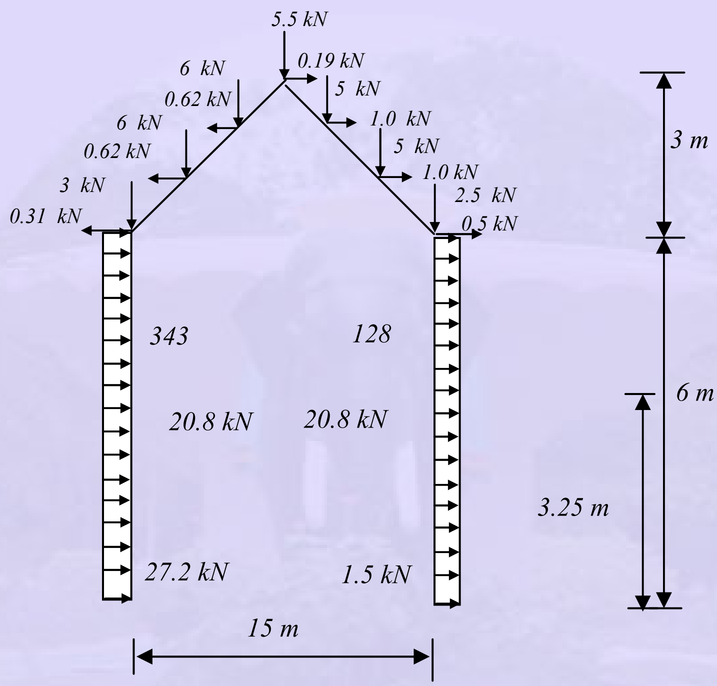
4.2.2 Crane Load

Moment @ B = 342 kNm

Horizontal load @ B = 20.8 kN

Moment @ F = 128 kNm

Horizontal load @ F = 20.8 kN



Factored Load (1. 5D.L+1.5 C.L +1.05 W.L)

4.3 Mechanisms

We will consider the following mechanisms, namely

(i) Beam mechanism

(ii) Sway mechanism

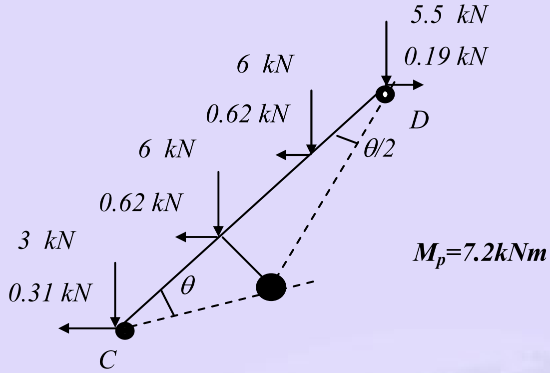
(iii) Gable mechanism and

(iv) Combined mechanism

(v) Beam Mechanism

(1) Member CD

Case 1: 1.5 D.L + 1.5 C.L + 1.05 W.L



Internal Work done, Wi = $M_p\theta$ + M_p (θ/2) + M_p (θ + θ/2)

= $M_p(3\theta)$

External Work done, W_e = 6 * 2.5θ - 0.62 * 1 * θ + 6 * 2.5 * θ/2 – 0.62 * 1 * θ/2

= 21.6θ

Equating internal work done to external work done

W_i = W_e

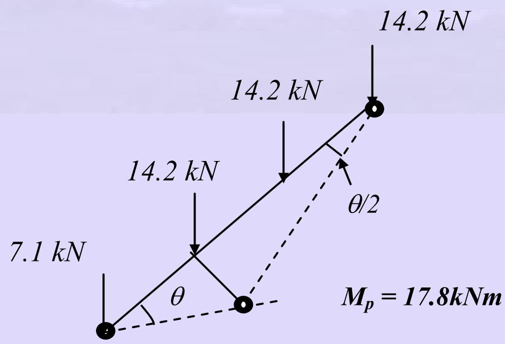
M_p (3θ) = 21.6θ

M_p = 7.2 kNm

Case 2: 1.5 D.L + 1.5 C.L + 1.05 L.L

Internal Work done,

W_i = M_p 3θ (as in case 1)



External work done, W_e = 14.2 * 2.5 θ + 14.2 *2.5θ / 2

= 53.3θ

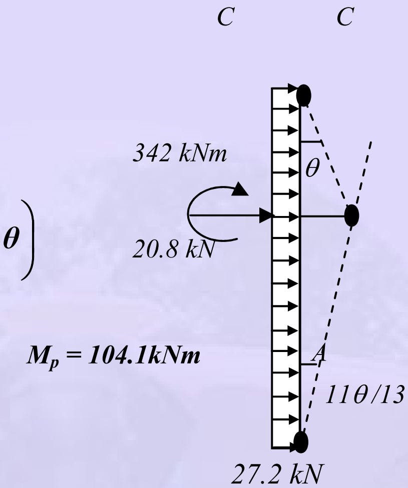
Equating $W_i = W_e$,

$$M_p (3\theta) = 53.3\,\theta$$

$$M_p = 17.8 \text{ kNm}$$

Note: Member DE beam mechanism will not govern.

(2) Member AC



Internal Work done,

$$W_i = M_p\theta + M_p\left(\theta + \frac{11}{13}\theta\right) + M_p\left(\frac{11}{13}\theta\right)$$

$$= 3.69\ M_p\theta$$

External Work done,

$$W_e = 20.8*3.25*\frac{11}{13}\theta + 342*\frac{11}{13}\theta + \frac{1}{2}*27.2*3.25\left(\frac{11}{13}\theta\right)$$

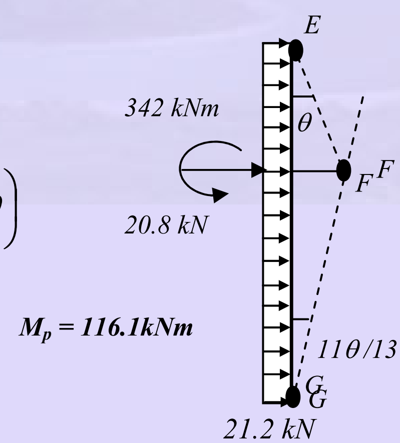
$$= 383.9\theta$$

Equating $W_i = W_e$, we get

$$3.69\ M_p\theta = 383.9\,\theta$$

$$M_p = 104.1 \text{ kNm.}$$

(3) Member EG



Internal Work done,

$$W_i = M_p\theta + M_p\left(\theta + \frac{11}{13}\theta\right) + M_p\left(\frac{11}{13}\theta\right)$$

$$= 3.69\ M_p\theta$$

External Work done,

$$W_e = 20.8*3.25*\frac{11}{13}\theta + 342*\theta + \frac{1}{2}(21.2)*3.25\left(\frac{11}{13}\theta\right)$$

$$= 428.3\theta$$

Equating $W_i = W_e$, we get

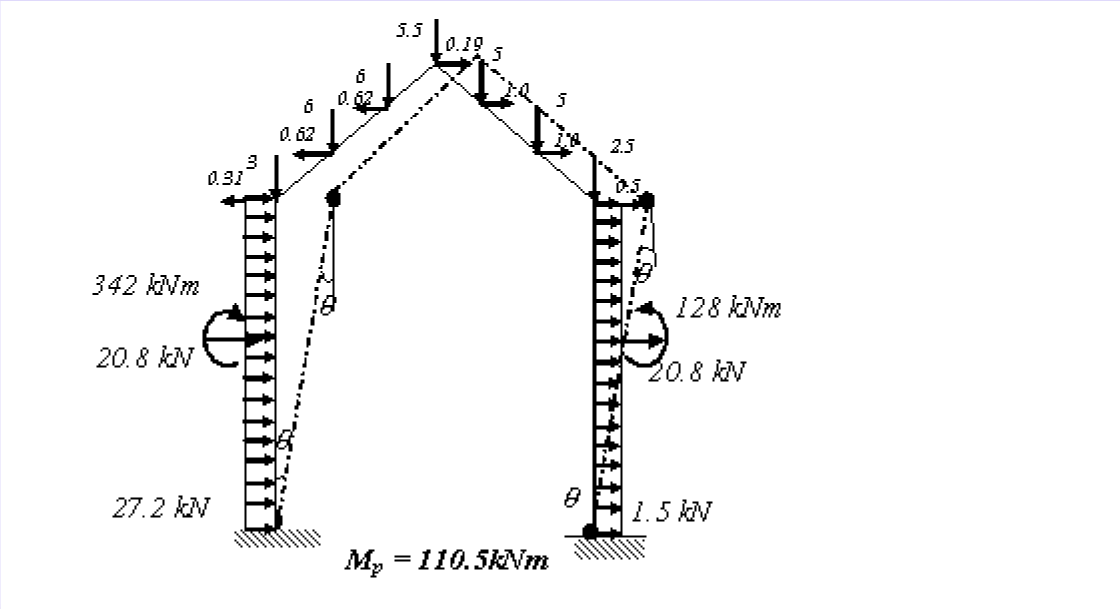
$3.69\ M_p\theta\ = 428.3\theta$

$M_p\ = 116.1$ kNm

For members AC & EG, the 1st load combination will govern the failure mechanism.

4.3.1 Panel Mechanism

Case 1: 1.5 D.L + 1.5 C.L + 1.05 W.L



Internal Work done, $W_i\ = M_p\ (\theta) + M_p\ (\theta) + M_p\ (\theta) + M_p\ (\theta)$

$= 4M_p\theta$

External Work done, W_e

$W_e =$ 1/2 (27.2) * 6θ + 20.8 * 3.25θ + 342θ - 0.31 * 6θ - 0.62 * 6θ - 0.62 (6θ)+ 0.19 * 6θ + 1.0 *6θ + 1.0 * 6θ + 0.5 * 6θ+1/2 (1.5) * 6θ + 20.8 * 3.25θ - 128 * θ

= 442.14θ

Equating $W_i = W_c$, we get

$4M_p\theta = 442.14\theta$

$M_p = 110.5$ kNm

The second load combination will not govern.

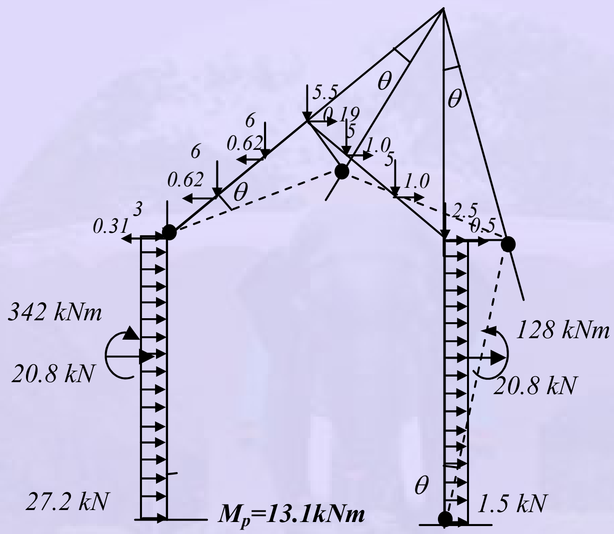
4.3.3 Gable Mechanism

Case 1: 1.5 D.L + 1.05 W.L + 1.5 C.L

Internal Work done = $M_p\theta + M_p2\theta + M_p(2\theta) + M_p\theta = 6M_p\theta$

External Work done, W_e =
$-0.62 * 1 * \theta - 0.62 * 2 * \theta + 0.19 * 3 * \theta + 1.0 * 4 * \theta + 1.0 * 5 * \theta + 0.5 * 6 * \theta + 6 * 2.5 * \theta + 6 * 5 * \theta + 5.5 * 7.5 * \theta + 5 * 5 * \theta + 5 * 2.5 * \theta + ½ * 1.5 * 6\theta + 20.8 * 3.25 * \theta - 128*\theta$

$W_e = 78.56\theta$



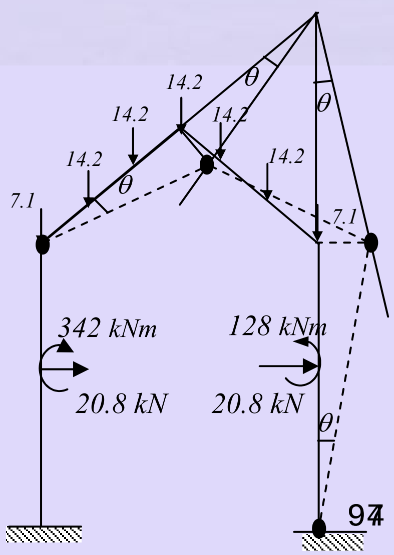
Equating $W_i = W_e$, we get

$6M_p = 78.56\theta$

$M_p = 13.1$ kNm.

Case 2: 1.5 D.L + 1.05L.L + 1.5 C.L

M_p=37.3kNm



Internal Work done, $W_i = M_p\theta + M_p (2\theta) + M_p (2\theta) + M_p\theta = 6M_p\theta$

External Work done, W_e

$= 14.2 * 2.5*\theta + 14.2 * 5 * \theta + 14.2 * 7.5\theta + 14.2 * 5 * \theta + 14.2 * 2.5\theta - 128 * \theta + 20.8 * 3.25\theta$
$= 223.6\theta$

Equating $W_i = W_e$, we get

$6M_p\theta = 223.6\theta$

$M_p = 37.3$ kNm

4.3.4 Combined Mechanism

Case1: 1.5 D.L + 1.05 W.L + 1.5 C.L

(i)

Internal Work done, $W_i = M_p (\theta) + M_p (\theta + \theta/2) + M_p (\theta/2 + \theta/2) + M_p (\theta/2)$

$= M_p (\theta + \theta + \theta/2 + \theta/2 + \theta/2 + \theta/2 + \theta/2)$

$= 4 M_p\theta$

$M_p = 100.7$

External Work done, W_e=
$1/2 * 27.2 * 6\theta + 20.8 * 3.25* \theta + 342\theta - 0.31 * 12 * \theta/2 - 0.62 * 11 * \theta/2 - 0.62 * 10 *\theta/2 + 0.19 * 9 * \theta/2 + 1.0 * 8 * \theta /2 + 1.0 * 7 * \theta /2 + 0.5 * 6* \theta/2 + 1/2 (1.5) * 6\theta/2 + 20.8 * 3.25 * \theta/2 - 128 * \theta/2 – 6 * 2.5 * \theta/2 – 6 * 5.0 * \theta/2 – 5.5 * 7.5 * \theta/2 – 5 * 5 * \theta/2 – 5 * 2.5 * \theta/2$
$= 402.86\theta$

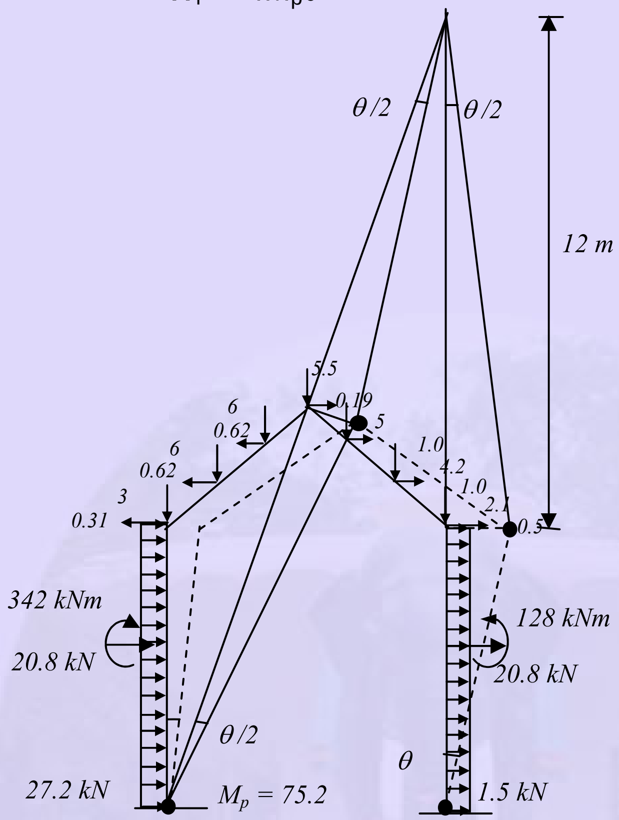
Equating $W_i = W_e$

$4M_p\theta = 402.86\theta$

$M_p = 100.7$ kNm

(ii) Internal work done, $W_i = M_p\,\theta/2 + M_p(\theta/2 + \theta/2) + M_p(\theta/2 + \theta) + M_p\theta$

$W_i = 4M_p\theta$



External Work done,

$$W_e = 20.8*3.25*\frac{\theta}{2} + 342*\frac{\theta}{2} + \frac{1}{2}*27.2*6\left(\frac{\theta}{2}\right) - 0.31*6*\frac{\theta}{2} - 0.62*7*\frac{\theta}{2}$$

$$-0.62*8*\frac{\theta}{2} + 0.19*9*\frac{\theta}{2} + 6*2.5*\frac{\theta}{2} + 6*5.0*\frac{\theta}{2} + 5.5*7.5*\frac{\theta}{2} + 1.0*10*\frac{\theta}{2}$$

$$+1.0*11*\frac{\theta}{2} + 0.5*12*\frac{\theta}{2} + 5*5.0*\frac{\theta}{2} + 5*2.5*\frac{\theta}{2} + 20.8*3.25\theta - 128*\theta$$

$$+\frac{1}{2}*1.5*6\theta$$

$= 300.85\theta$

Equating $W_i = W_e$, we get

$4M_p\theta = 300.85\theta$

$M_p = 75.2$ kNm

Similarly analysis can be performed for hinges occurring at purlin locations also but they have been found to be not critical in this example case

From all the above analysis, the largest value of M_p required was for member EG under

1.5 DL + 1.5 CL + 1.05 WL

Therefore the Design Plastic Moment = 116.1 kNm.

5.0 DESIGN

For the design it is assumed that the frame is adequately laterally braced so that it fails by forming mechanism. Both the column and rafter are analysed assuming equal plastic moment capacity. Other ratios may be adopted to arrive at an optimum design solution.

5.1 Selection of section

Plastic Moment capacity required= 116 kNm

Required section modulus, $Z_p = M_p / f_{yd}$

$$= \frac{\left(116*10^6\right)}{250/1.10}$$

$$= 510.4*10^3 \ mm^3$$

ISMB 300 @ 0.46 kN/ m provides

Z_p = 683 * 10^{-3} mm^3

b = 140 mm

T_i = 13.1 mm

A = 5.87 * 10^{3} mm^2

t_w =7.7 mm

r_{xx} =124 mm

r_{yy} =28.6 mm

5.2 Secondary Design Considerations

5.2.1 Check for Local buckling of flanges and webs

Flanges

$$\frac{b_f}{T_1} = \frac{136}{\sqrt{f_y}}$$

b_f = 140/2 = 70 mm

T_1 = 13.1 mm

t = 7.7 mm

$$\frac{b_f}{T_1} = \frac{70}{13.1} = 5.34 < 8.6$$

Web

$$\frac{d_1}{t} \leq \left[\frac{1120}{\sqrt{f_y}} - \frac{1600}{\sqrt{f_y}}\left(\frac{P}{P_y}\right)\right]$$

$$\frac{300}{7.7} \leq \left[\frac{1120}{\sqrt{250_y}} - \frac{1600}{\sqrt{250_y}}(0.27)\right]$$

$$38.9 \leq 68, Hence\ O.K$$

5.2.2 Effect of axial force

Maximum axial force in column, P = 40.5 kN

Axial load causing yielding, $P_y = f_{yd} * A$

$$= \frac{250}{1.10} x5.87*10^3$$

$$= 1334\, kN$$

$$\frac{P}{P_y} = \frac{40.5}{1334} = 0.03 < 0.15$$

Therefore the effect of axial force can be neglected.

5.2.3 Check for the effect of shear force

Shear force at the end of the girder = P- w/2

= 40.5 -6.8 kN

= 33.7 kN

Maximum shear capacity V_{ym}, of a beam under shear and moment is given by

V_{ym} = 0.55 A_w* f_{yd} / 1.10

= 0.55 * 300* 7.7* 250/1.10

=289 kN>> 33.7 kN

Hence O.K.

3. MULTI – STOREY BUILDINGS

3.1 Introduction

In developed countries a very large percentage of multi-storeyed buildings are built with steel where as steel is not so commonly used in construction of multi-storeyed frames in India even though it is a better material than reinforced concrete. The use of steel in multi-storey building construction results in many advantages for the builder and the user. The advantages of using steel frames in the construction of multi-storey buildings are listed below:

• Steel, by virtue of its high strength to weight ratio enables large spans and light weight construction.

• Steel structures can have a variety of structural forms like braced frames and moment resistant frames suitable to meet the specific requirements.

• Steel frames are faster to erect compared with reinforced concrete frames resulting in economy.

• The elements of framework are usually prefabricated in the factory under effective quality control thus enabling a better product.

• Subsequent alterations or strengthening of floors are relatively easy in steel frames compared with concrete frames.

• The steel frame construction is more suitable to withstand lateral loads caused by wind or earthquake.

3.1.1 Structural configurations

The structural components in a typical multi-storey building, consists of a floor system which transfers the floor loads to a set of plane frames in one or both directions. The floor system also acts as a diaphragm to transfer lateral loads from wind or earthquakes. The frames consist of beams and columns and in some cases braces or even reinforced concrete shear walls. As the height of the building increases beyond ten stories (tall building), it becomes necessary to reduce the weight of the structure for both functionality and economy. For example a 5% reduction in the floor and wall weight can lead to a 50% reduction in the weight at the ground storey. This means that the columns in the lower storeys will become smaller leading to more availability of space and further reduction in the foundation design.

Floor systems

Since concrete floors are functionally more suitable, have less vibration and more abrasion and fire resistance, the usual tendency is to make them act either with profiled steel decks and/or with steel beams to give a light weight floor system. Similarly masonry walls may be replaced with glazing and curtains or blinds to reduce the weight. The different types of floors used in steel-framed buildings are as follows:

a) Concrete slabs supported by open-web joists

b) One-way and two-way reinforced concrete slabs supported on steel beams

c) Concrete slab and steel beam composite floors

d) Profiled decking floors

e) Precast concrete slab floors.

Concrete slabs supported with open-web joists

Steel forms or decks are usually attached to the joists by welding and concrete slabs are poured on top. This is one of the lightest types of concrete floors. For structures with light loading, this type is economical. A sketch of an open-web joist floor is shown in Fig.3.1.

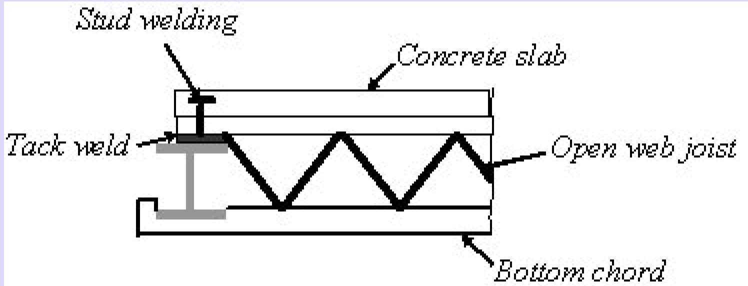


Fig.3.1 Open- web joists

One-way and two-way reinforced concrete slabs.

These are much heavier than most of the newer light weight floor systems and they take more time to construct, thus negating the advantage of speed inherent in steel construction. This floor system is adopted for heavy loads. One way slabs are used when the longitudinal span is two or more times the short span. In one-way slabs, the short span direction is the direction in which loads get transferred from slab to the beams. Hence the main reinforcing bars are provided along this direction. However, temperature, shrinkage and distribution steel is provided along the longer direction.

The two-way concrete slab is used when aspect ratio of the slab i.e. longitudinal span/transverse span is less than 2 and the slab is supported along all four edges. The main reinforcement runs in both the directions. A typical cross-section of a one-way slab floor with supporting steel beams is shown in Fig.3.2. Also shown is the case when the steel beam is encased in concrete for fire protection.

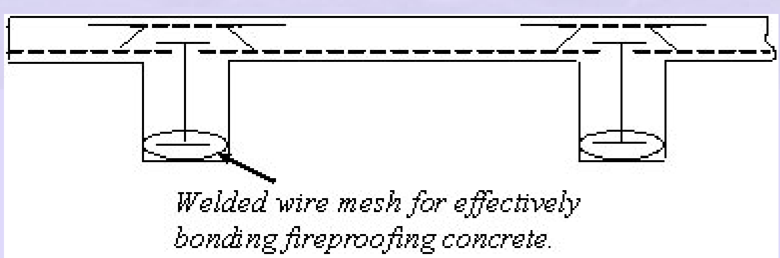
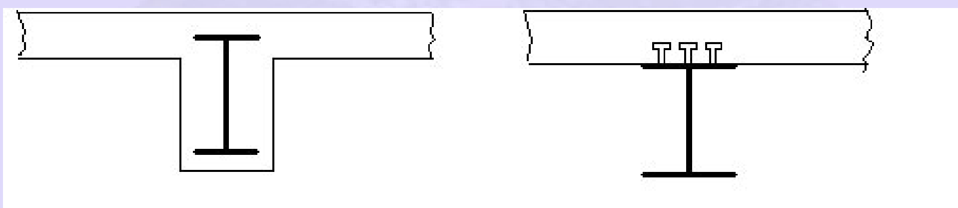


Fig.3.2 Cross section of one-way slab floor

Composite floors with a reinforced concrete slab and steel beams

Composite floors have steel beams bonded with concrete slab in such a way that both of them act as a unit in resisting the total loads. The sizes of steel beams are significantly smaller in composite floors, because the slab acts as an integral part of the beam in compression. The composite floors require less steel tonnage in the structure and also result in reduction of total floor depth. These advantages are achieved by utilising the compressive strength of concrete by keeping all or nearly all of the concrete in compression and at the same time utilises a large percentage of the steel in tension. The types of composite floor systems normally employed are shown in Fig. 3.3.



(a) Steel beam encased in concrete (Rarely used nowadays)

(b) Steel beam acting composite with concrete slab using shear connectors

Fig.3.3 Composite floors

Profiled steel decking floors

Composite floor construction consisting of profiled and formed steel decking with a concrete topping is also popular for office and apartment buildings where the loads are not very heavy. The advantages of steel-decking floors are given below:

(i) They do not need form work

(ii) The lightweight concrete is used resulting in reduced dead weight

(iii) The decking distributes shrinkage strains, thus prevents serious cracking

(iv) The decking stabilises the beam against lateral buckling, until the concrete ardens

(v) The cells in decking are convenient for locating services.

More details of composite construction using profiled decking floors are provided in the chapter on Industrial Buildings.

Precast concrete floors

Precast concrete floors offer speedy erection and require only minimal formwork. Light-weight aggregates are generally used in the concrete, making the elements light and easy to handle. Typical precast concrete floor slab sections are shown in Fig.4.4. It is necessary to use cast in place mortar topping of 25 to 50 mm before installing other floor coverings. Larger capacity cranes are required for this type of construction when compared with those required for profiled decking. Usually prestressing of the precast elements is also done.

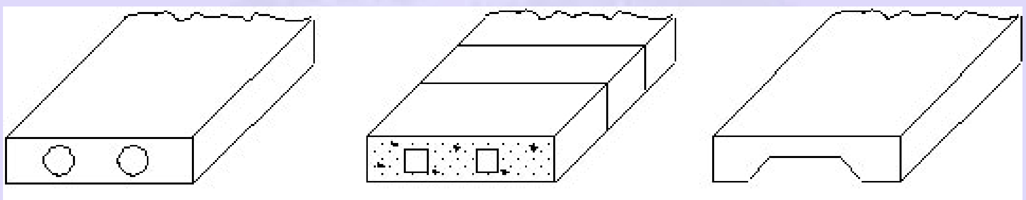


Fig.4.4 Precast concrete floor slabs

1.1.2 Lateral load resisting systems

The lateral loads from wind and earthquakes are resisted by a set of steel frames in orthogonal directions or by reinforced concrete shear walls. Steel frames are broadly classified as braced-frames and moment-resisting frames depending on the type of configuration and beam-to-column connection provided.

Moment resisting frames

Moment resisting frames (Fig. 3.5a) rely on the ability of the frame itself to act as a partially (semi-) or fully rigid jointed frame while resisting the lateral loads. Due to their flexibility, moment resisting frames experience a large horizontal deflection called drift (Fig. 3.5), especially in tall buildings but can be used for medium rise buildings having up to ten stories. The rigid connection types discussed in the chapter on beam-to-column connections can be used in such frames.

Braced frames

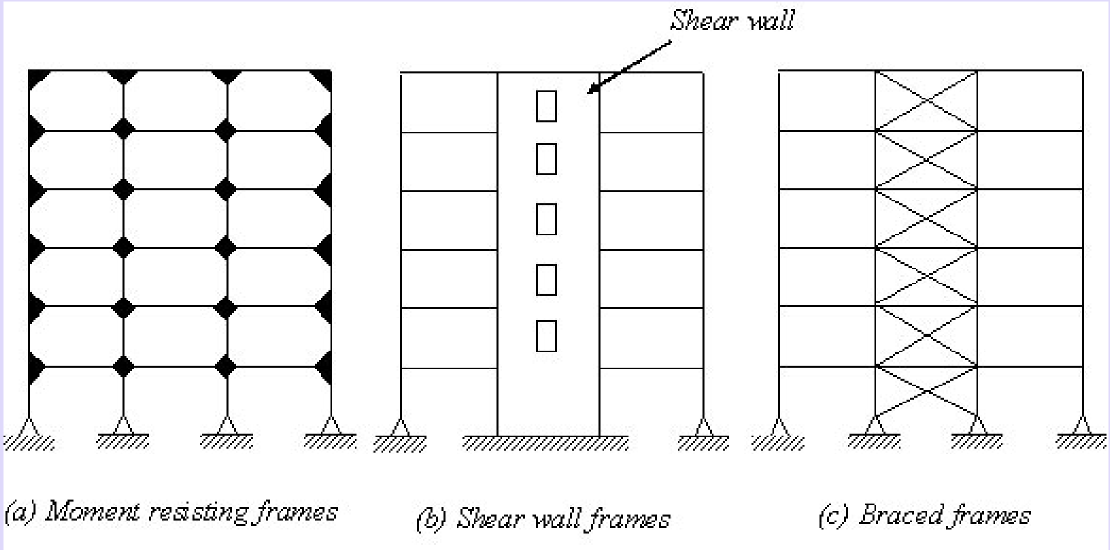


Fig. 3.5 Lateral load resisting systems

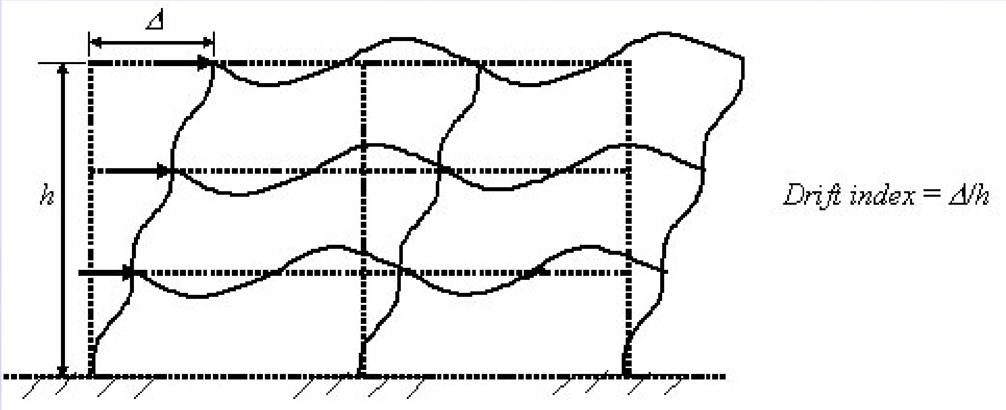


Fig. 3.6 Lateral drift

Braced Frames (Fig. 3.5c) are usually designed with simple beam-to-column connections where only shear transfer takes place but may occasionally be combined with moment resisting frames. In braced frames, the beam and column system takes the gravity load such as dead and live loads. Lateral loads such as wind and earthquake loads are taken by a system of braces. Usually bracings are provided sloping in all four directions because they are effective only in tension and buckle easily in compression. Therefore in the analysis, only the tension brace is considered effective. Braced frames are quite stiff and have been used in very tall buildings.

3.2 Loading

Loading on tall buildings is different from low-rise buildings in many ways such as large accumulation of gravity loads on the floors from top to bottom, increased significance of wind loading and greater importance of dynamic effects. Thus, multi-storeyed structures need correct assessment of loads for safe and economical design. Except dead loads, the assessment of loads can not be done accurately. Live loads can be anticipated approximately from a combination of experience and the previous field observations. Wind and earthquake loads are random in nature and it is difficult to predict them. They are estimated based on a probabilistic approach. The following discussion describes some of the most common kinds of loads on multi-storeyed structures.

3.2.1 Gravity loads

Dead loads due the weight of every element within the structure as well as live loads that are acting on the structure when in service constitute gravity loads. The dead loads are calculated from the member sizes and estimated material densities. Live loads prescribed by codes are empirical and conservative based on experience and accepted practice. The equivalent minimum loads for office and residential buildings as per IS 875 are as specified in Table -3.1

Table – 3.1 Live load magnitudes [IS: 875 - 1987 Part -II]

Occupancy classification	**Uniformly distributed load (kN/m^2)**	**Concentrated load (kN)**
Office buildings		
• Offices and Staff rooms	2.5	2.7
• Class rooms	3.0	2.7
• Corridors, Store rooms and Reading rooms	4.0	4.5
Residential buildings		
• Apartments	2.0	1.8
• Restaurants	4.0	2.7
• Corridors	3.0	4.5

A floor should be designed for the most adverse effect of uniformly distributed load and concentrated load over 0.3 m by 0.3 m as specified in Table-3.1, but they should not be considered to act simultaneously. All other structural elements such as beams and columns are designed for the corresponding uniformly distributed loads on floors.

Reduction in imposed (live) load may be made in designing columns, load bearing walls etc., if there is no specific load like plant or machinery on the floor. This is allowed to account for reduced probability of full loading being applied over larger areas. The supporting members of the roof of the multi-storeyed building is designed for 100% of uniformly distributed load; further reductions of 10% for each successive floor down to a minimum of 50% of uniformly distributed load is done. The live load at a floor level can be reduced in the design of beams, girders or trusses by 5% for each 50m^2 area supported, subject to a maximum reduction of 25%. In cases where the reduced load of a lower floor is less than the reduced load of an upper floor, then the reduced load of the upper floor should be adopted in the lower floor also.

3.2.2 Wind load

The wind loading is the most important factor that determines the design of tall buildings over 10 storeys, where storey height approximately lies between 2.7 – 3.0 m. Buildings of up to 10 storeys, designed for gravity loading can accommodate wind loading without any additional steel for lateral system. Usually, buildings taller than 10 storeys would generally require additional steel for lateral system. This is due to the fact that wind loading on a tall building acts over a very large building surface, with greater intensity at greater heights and with a larger moment arm about the base. So, the additional steel required for wind resistance increases non-linearly with height as shown in Fig. 3.7.

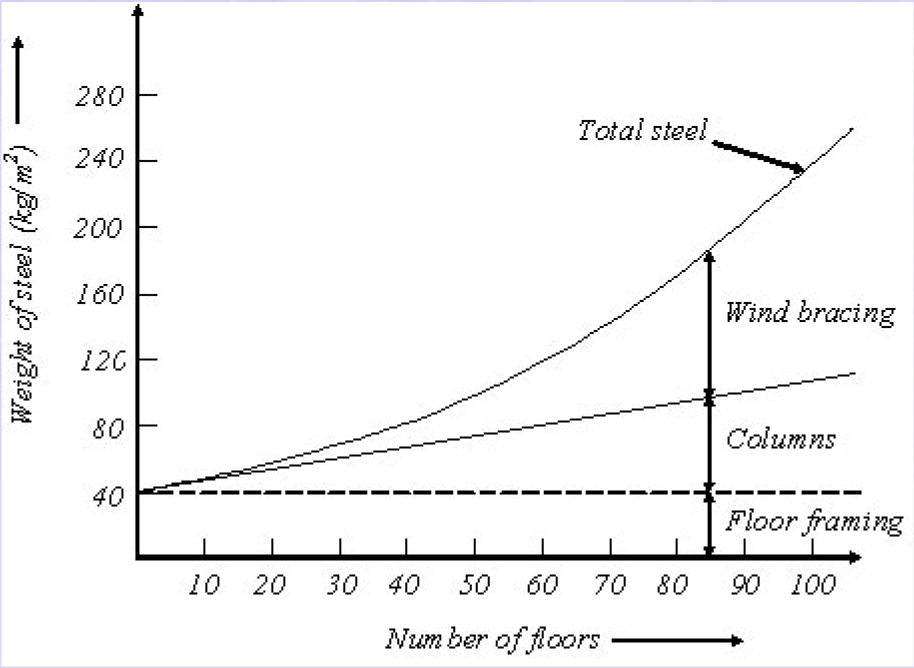


Fig.3.7 Weight of steel in multi-storeyed buildings

As shown in Fig.3.7 the lateral stiffness of the building is a more important consideration than its strength for tall multi-storeyed structures. Wind has become a major load for the designer of multi-storeyed buildings. Prediction of wind loading in precise scientific terms may not be possible, as it is influenced by many factors such as the form of terrain, the shape, slenderness, the solidity ratio of building and the arrangement of adjacent buildings. The appropriate design wind loads are estimated based on two approaches. Static approach is one, which assumes the building to be a fixed rigid body in the wind. This method is suitable for buildings of normal height, slenderness, or susceptible to vibration in the wind. The other approach is the dynamic approach. This is adopted for exceptionally tall, slender, or vibration prone buildings. Sometimes wind sensitive tall buildings will have to be designed for interference effects caused by the environment in which the building stands. The loading due to these interference effects is best ascertained using wind tunnel modeled structures in the laboratory.

However, in the Indian context, where the tallest multi-storeyed building is only about 35 storeys high, multi-storeyed buildings do not suffer wind-induced oscillation and generally do not require to be examined for the dynamic effects. For detailed information on evaluating wind load, the reader is referred to IS: 875-1987 (Part-III).

3.2.3 Earthquake load

Seismic motion consists of horizontal and vertical ground motions, with the vertical motion usually having a much smaller magnitude. Further, factor of safety provided against gravity loads usually can accommodate additional forces due to vertical acceleration due to earthquakes. So, the horizontal motion of the ground causes the most significant effect on the structure by shaking the foundation back and forth. The mass of building resists this motion by setting up inertia forces throughout the structure.

The magnitude of the horizontal shear force F depends on the mass of the building M, the acceleration of the ground a, and the nature of the structure. If a building and the foundation were rigid, it would have the same acceleration as the ground as given by Newton's second law of motion, i.e. F = Ma. However, in practice all buildings are flexible to some degree. For a structure that deforms slightly, thereby absorbing some energy, the force will be less than the product of mass and acceleration. But, a very flexible structure will be subject to a much larger force under repetitive ground motion. This shows the magnitude of the lateral force on a building is not only dependent on acceleration of the ground but it will also depend on the type of the structure. As an inertia problem, the dynamic response of the building plays a large part in influencing and in estimating the effective loading on the structure. The earthquake load is estimated by Seismic co-efficient method or Response spectrum method. The later takes account of dynamic characteristics of structure along with ground motion. For detailed information on evaluating earthquake load, reader is referred to IS: 1893 - 2002 and the chapter on Industrial Buildings.

3.3 Analysis for gravity loads

Simple framing

If a simple framing is used, the analysis is quite simple because they can be considered as simply supported. In such cases, shears and moments can be determined by statics. The gravity loads applied to the columns are relatively easy to estimate, but the column moments may be a little more difficult to find out. The column moments occur due to uneven distribution and unequal magnitude of live load. If the beam reactions are equal on each side of interior column, then there will be no column moment. If the reactions are unequal, the moment produced in the column will be equal to the difference between reactions multiplied by eccentricity of the beam reaction with respect to column centre line.

Semi rigid framing

The analysis of semi-rigid building frames is complex. The semi-rigid frames are analysed and designed by using special techniques developed based on experimental evidence on the behaviour of the connections. For more details the reference quoted in the chapter on beam-to-column connections may be consulted.

Rigid framing

Rigid frame buildings are analysed by one of the approximate methods to make an estimate of member sizes before going to exact methods such as slope-deflection or moment-distribution method. If the ends of each girder are assumed to be completely fixed, the bending moments due to uniform loads are as shown in full lines of Fig. 3.8(a). If the ends of beam are connected by simple connection, then the moment diagram for uniformly distributed load is shown in

Fig. 3.8(b). In reality, a moment somewhere between the two extremes will occur which is represented by dotted line in Fig.3.8(a). A reasonable procedure is to assume fixed end moment in the range of $wl^2/10$, where l is clear span and w is magnitude of uniformly distributed load.

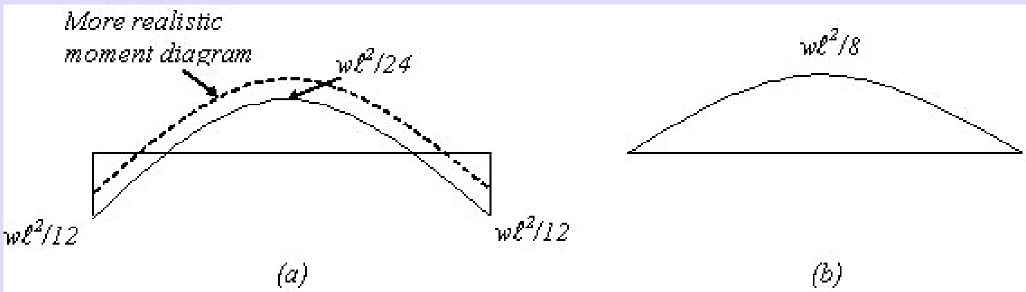


Fig. 3.8 (a) Fixed beam

(b) Simply - supported beam bending moment diagrams

The following assumptions are made for arrangement of live load in the analysis of frames:

a) Consideration is limited to combination of:

1. Design dead load on all spans with full design live load on two adjacent spans and

2. Design dead load on all spans with full design live load on alternate spans.

b) When design live load does not exceed three-fourths of the design dead load, the load arrangement of design dead load and design live load on all the spans can be used.

Unless more exact estimates are made, for beams of uniform cross-section which support substantially uniformly distributed loads over three or more spans which do not differ by more than 15% of the longest, the bending moments and shear forces used for design is obtained using the coefficients given in Table 3.2 and Table 3.3 respectively. For moments at supports where two unequal spans meet or in cases where the spans are not equally loaded, the average of the two values for the hogging moment at the support may be used for design. Where coefficients given in Table 3.2 are used for calculation of bending moments, redistribution of moments is not permitted.

Substitute frame method

Rigid frame high-rise buildings are highly redundant structures. The analysis of such frames by conventional methods such as moment distribution method or Kane's method is very lengthy and time consuming. Thus, approximate methods (such as two cycled moment distribution method) are adopted for the analysis of rigid frames under gravity loading, one of such methods is Substitute Frame Method.

Substitute frame method is a short version of moment distribution method. Only two cycles are carried out in the analysis and also only a part of frame is considered for analysing the moments and shears in the beams and columns. The assumptions for this method are given below:

1) Moments transferred from one floor to another floor are small. Hence, the moments for each floor are separately calculated.

2) Each floor will be taken as connected to columns above and below with their far ends fixed.

If the columns are very stiff, no rotation will occur at both ends of a beam and the point of contraflexure will be at about 0.2 l. The actual beam can be replaced by a simply – supported beam of span 0.6 l as shown in Fig. 3.9(a). If, the columns are flexible, then all the beams can be considered as simply supported of span l as the beam – column joint will rotate like a hinge, an approximate model for middle floor beam is shown in Fig. 3.9(b).

Table 3.2: Bending moment coefficients

TYPE OF LOAD	SPAN MOMENTS		SUPPORT MOMENTS	
	Near middle span	At middle of interior span	At support next to the end support	At other interior supports
Dead load + Imposed load (fixed)	+ 1/12	+1/24	- 1/10	- 1/12
Imposed load (not fixed)	+1/10	+1/12	- 1/9	- 1/9
For obtaining the bending moment, the coefficient is multiplied by the total design load and effective span.				

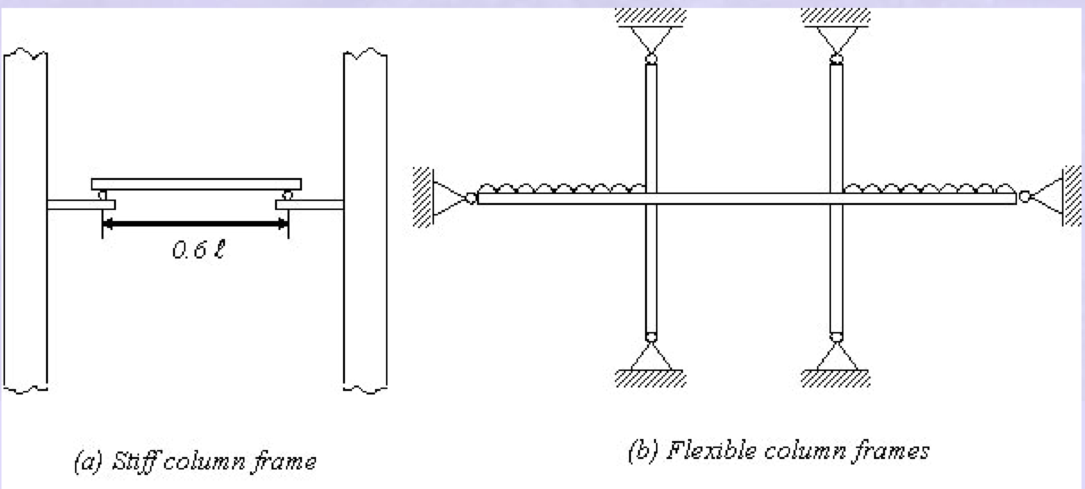


Fig.3.9 Substitute models for analysis of frames

Table 3.3: Shear force coefficients

TYPE OF LOAD	At end support	At support next to the end support		At all other interior supports
		Outer side	Inner side	
Dead load + Imposed load(fixed)	0.40	0.60	0.55	0.50
Imposed load(not fixed)	0.45	0.60	0.60	0.60
For obtaining the shear force, the coefficient is multiplied by the total design load				

Drift in Rigid Frames

The lateral displacement of rigid frames subjected to horizontal loads is due to the following three modes:

- Girder Flexure
- Column Flexure
- Axial deformation of columns

The sum of the storey drifts from the base upward gives the drift at any level and the storey drifts can be calculated from summing up the contributions of all the three modes discussed earlier in that particular storey. If the total drift or storey drift exceeds the limiting value then member sizes should be increased to avoid excessive drift.

3.4 Analysis for lateral loads

3.4.1 Braced frames

In this section, simple hand methods for the analysis of statically determinate or certain low-redundant braced structures is reviewed.

Member Force Analysis

Analysis of the forces in a statically determinate triangulated braced frame can be made by the method of sections. For instance, consider a typical diagonal braced pin-jointed bay as shown in Fig. 3.10. When this bay is subjected to an external shear Q_i in *i*-th storey and external moments M_i and M_{i-1} at floors *i* and *i-1*, respectively, the force in the brace can be found by considering the horizontal equilibrium of the free body above section *XX*, thus,

$$F_{BC} \, Cos\theta = Q_i$$

Hence,

$$F_{BC} = Q_i / Cos\theta$$

The axial force F_{BD} in the column *BD* is found by considering moment equilibrium of the upper free body about *C*, thus

$$F_{BD} * \ell = M_{i-1}$$

Hence,

$$F_{BD} = M_{i-1} / \ell$$

Similarly the force F_{AC} in column *AC* is obtained from the moment equilibrium of the upper free body about *B*. It is given by

$$F_{AC} = M_i / \ell$$

This procedure can be repeated for the members in each storey of the frame. The member forces in more complex braced frames such as knee-braced, X-braced and K-braced frames can also be obtained by taking horizontal sections.

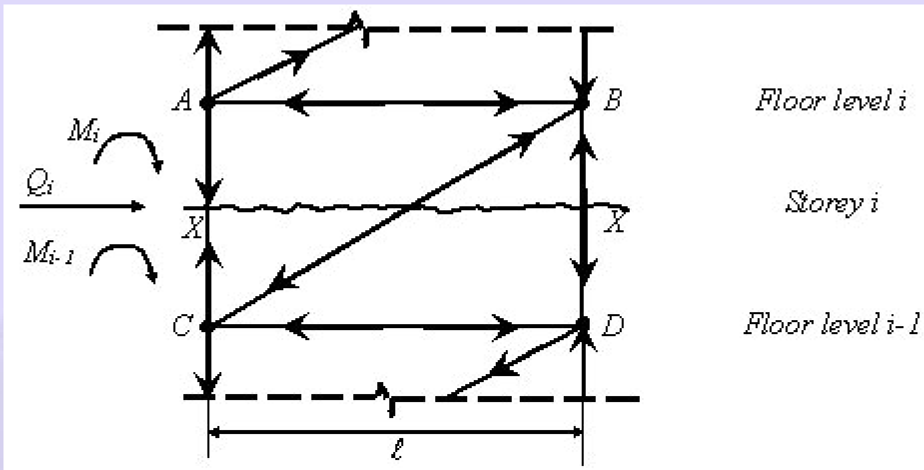


Fig. 3.10 Single diagonal braced panel

Drift Analysis

Drift in building frames is a result of flexural and shear mode contributions, due to the column axial deformations and to the diagonal and girder deformations, respectively. In low rise braced structures, the shear mode displacements are the most significant and, will largely determine the lateral stiffness of the structure. In medium to high rise structures, the higher axial forces and deformations in the columns, and the accumulation of their effects over a greater height, cause the flexural component of displacement to become dominant.

The storey drift in a braced frame reaches a maximum value at or close to the top of the structure and is strongly influenced by the flexural component of deflection. This is because the inclination of the structure caused by the flexural component accumulates up the structure, while the storey shear component diminishes toward the top.

Hand analysis for drift allows the drift contributions of the individual frame members to be seen, thereby providing guidance as to which members should be increased in size to effectively reduce an excessive total drift or storey drift. The following section explains a method for hand evaluation of drift.

3.4.2 Moment-resisting frames

Multi-storey building frames subjected to lateral loads are statically indeterminate and exact analysis by hand calculation takes much time and effort. Using simplifying assumptions, approximate analyses of these frames yield good estimate of member forces in the frame, which can be used for checking the member sizes. The following methods can be employed for lateral load analysis of rigidly jointed frames.

- The Portal method.
- The Cantilever method
- The Factor method

The portal method and the cantilever method yield good results only when the height of a building is approximately more than five times its least lateral dimension. Either classical techniques such as slope deflection or moment distribution methods or computer methods using stiffness or flexibility matrices can be used if a more exact result is desired.

The portal method

This method is satisfactory for buildings up to 25 stories, hence is the most commonly used approximate method for analysing tall buildings. The following are the simplifying assumptions made in the portal method:

1. A point of contraflexure occurs at the centre of each beam.
2. A point of contraflexure occurs at the centre of each column.
3. The total horizontal shear at each storey is distributed between the columns of that storey in such a way that each interior column carries twice the shear carried by each exterior column.

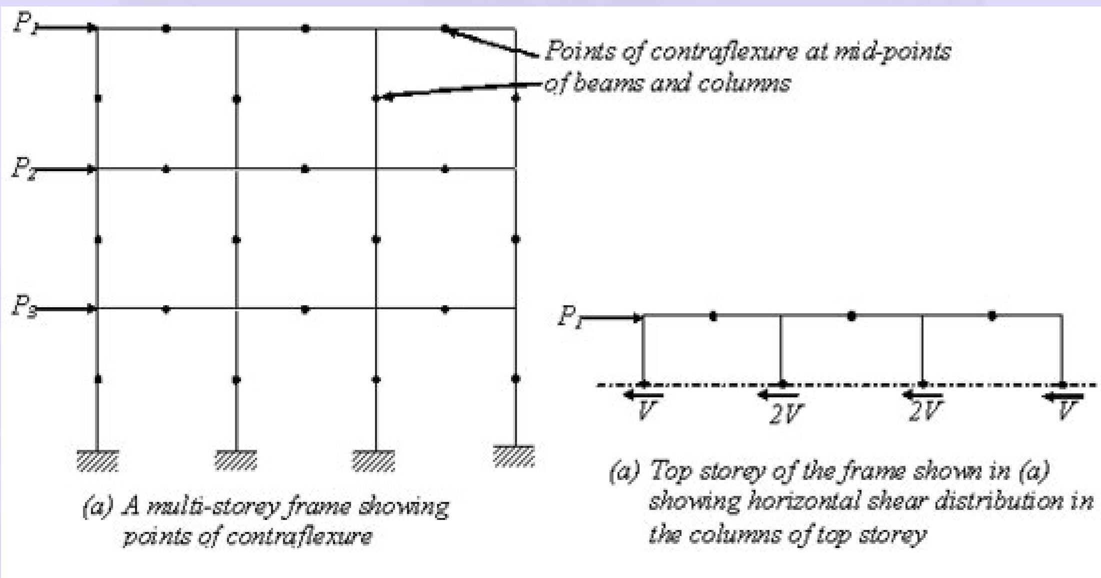


Fig.3.11 Portal method of analysis

The above assumptions convert the indeterminate multi-storey frame to a determinate structure. The steps involved in the analysis of the frame are detailed below:

1. The horizontal shears on each level are distributed between the columns of that floor according to assumption (3).

2. The moment in each column is equal to the column shear multiplied by half the column height according to assumption (2).

3. The girder moments are determined by applying moment equilibrium equation to the joints: by noting that the sum of the girder moments at any joint equals the sum of the column moments at that joint. These calculations are easily made by starting at the upper left joint and working joint by joint across to the right end.

4. The shear in each girder is equal to its moment divided by half the girder length. This is according to assumption (1).

5. Finally, the column axial forces are determined by summing up the beam shears and other axial forces at each joint. These calculations again are easily made by working from left to right and from the top floor down.

Assumptions of the Portal method of analysis are diagrammatically shown in Fig.3.11.

The cantilever method

This method gives good results for high-narrow buildings compared to those from the Portal method and it may be used satisfactorily for buildings of 25 to 35 storeys tall. It is not as popular as the portal method.

The simplifying assumptions made in the cantilever method are:

1. A point of contraflexure occurs at the centre of each beam

2. A point of contraflexure occurs at the centre of each column.

3. The axial force in each column of a storey is proportional to the horizontal distance of the column from the centre of gravity of all the columns of the storey under consideration.

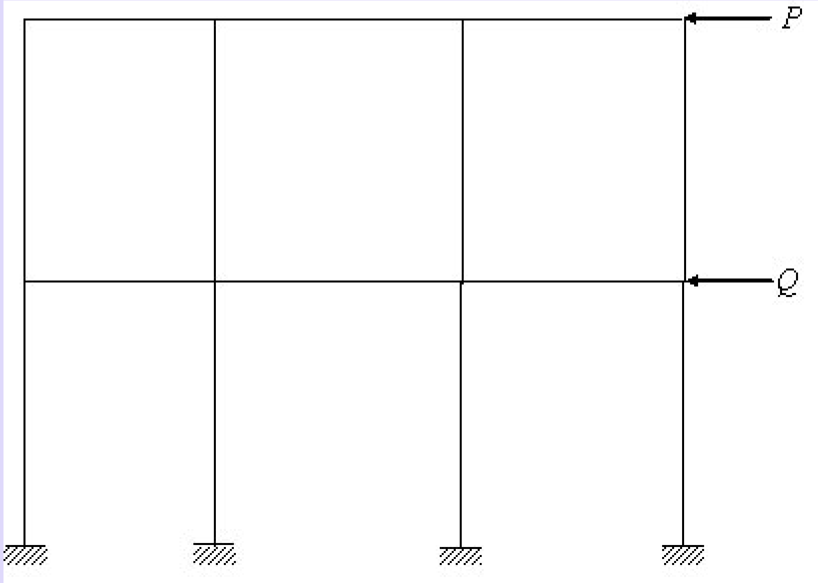


Fig. 3.12(a) Typical frame

The steps involved in the application of this method are:

1. The centre of gravity of columns is located by taking moment of areas of all the columns and dividing by sum of the areas of columns.

2. A lateral force P acting at the top storey of building frame is shown in Fig. 3.12(a). The axial forces in the columns are represented by F_1, F_2, F_3 and F_4 and the columns are at a distance of x_1, x_2 , x_3 and x_4 from the centroidal axis respectively as shown in Fig. 3.12(b).

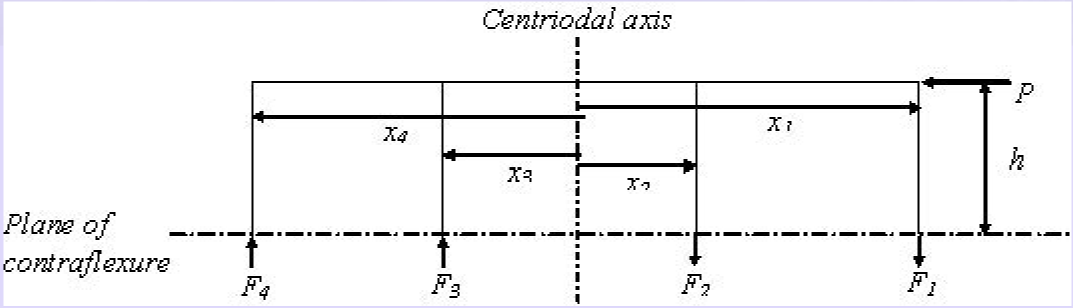


Fig. 3.12(b) Top storey of the frame above plane of contraflexure

By taking the moments about the centre of gravity of columns of the storey,

$$P\,h - F_1x_1 - F_2x_2 - F_3x_3 - F_4x_4 = 0$$

The axial force in one column may be assumed as F and the axial forces of remaining columns can be expressed in terms of F using assumption (3).

3. The beam shears are determined joint by joint from the column axial forces.

4. The beam moments are determined by multiplying the shear in the beam by half the span of beam according to assumption (1).

5. The column moments are found joint by joint from the beam moments.

The column shears are obtained by dividing the column moments by the half-column heights using assumption (2)

The factor method

The factor method is more accurate than either the portal method or the cantilever method. The portal method and cantilever method depend on assumed location of hinges and column shears whereas the factor method is based on assumptions regarding the elastic action of the structure. For the application of Factor method, the relative stiffness (k = I/l), for each beam and column should be known or assumed, where, I is the moment of inertia of cross section and l is the length of the member.

The application of the factor method involves the following steps:

1. The girder factor g, is determined for each joint from the following expression.

$$g = \frac{\sum k_c}{\sum k}$$

Where, Σ kc - Sum of relative stiffnesses of the column members meeting at that joint.

Σ k - Sum of relative stiffnesses of all the members meeting at that joint.

Each value of girder factor is written at the near end of the girder meeting at the joint.

2. The column factor c, is found for each joint from the following expression

c = 1-g

Each value of column factor c is written at the near end of each column meeting at the joint. The column factor for the column fixed at the base is one.

At each end of every member, there will be factors from step 1 or step 2. To these factors, half the values of those at the other end of the same member are added.

3. The sum obtained as per step 2 is multiplied by the relative stiffness of the respective members. This product is termed as column moment factor C, for the columns and the girder moment factor G, for girders.

4. Calculation of column end moments for a typical member ij - The column moment factors [C values] give approximate relative values of column end moments. The sum of column end moments is equal to horizontal shear of the

storey multiplied by storey height. Column end moments are evaluated by using the following equation,

$M_{ij} = C_{ij}\,A$

where, M_{ij} - moment at end i of the ij column

C_{ij} - column moment factor at end i of column ij

A - storey constant given by

$$A = \left(\frac{\text{Total horizantal Shear of Storey x Height of the Sotey}}{\text{Sum of the column end memory factors of the storey}}\right)$$

5. Calculation of beam end moments - The girder moment factors [G values] give the approximate relative beam end moments. The sum of beam end moments at a joint is equal to the sum of column end moments at that joint. Beam end moments can be worked out by using following equation,

$M_{ij} = G_{ij}\,B$

Where, M_{ij} - moment at end i of the ij beam

G_{ij} - girder moment factor at end i of beam ij

$$B = \left(\frac{\text{Sum of column moments at the jo int}}{\text{Sum of the girder end memory factors of that joi nt}}\right)$$

B - joint constant given by

Illustration of calculation of G values:

Consider the joints B and C in the frame shown in Fig. 3.13.

Joint B: $g_B = k_1 /(k_1 + k_2 + k_3)$

$c_B = 1 - g_B$

Joint C: $g_C = k_4 /(k_2 + k_4 + k_5)$

$c_C = 1 - g_C$

As shown in Fig. 3.13, we should obtain values like x and y at each end of the beam and column. Thereafter we multiply them with respective k values to get the column or girder moment factors. Here, $G_{BC} = x\ k_2$ and $G_{CB} = y\ k_2$. Similarly we calculate all other moment factors.

3.5 Computer analysis of rigid frames

Although the approximate methods described earlier have served structural engineers well for decades, they have now been superseded by computer analysis packages. Computer analysis is more accurate, and better able to analyse complex structures. A typical model of the rigid frame consists of an assembly of beam-type elements to represent both the beams and columns of the frame. The columns are assigned their principal inertia and sectional areas. The beams are assigned with their horizontal axis inertia and their sectional areas are also assigned to make them effectively rigid. Torsional stiffnesses and shear deformations of the columns and beams are neglected.

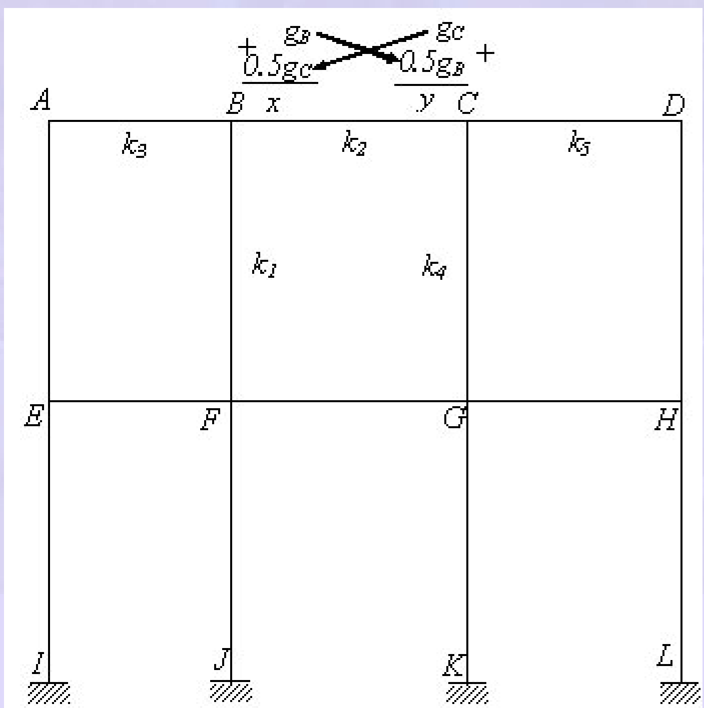


Fig.3.13 Typical frame

Some analysis programs include the option of considering the slab to be rigid in plane, and some have the option of including P-Delta effects. If a rigid slab option is not available, the effect can be simulated by interconnecting all vertical elements by a horizontal frame at each floor, adding fictitious beams where necessary, assuming the beams to be effectively rigid axially and in flexure in the horizontal plane.

Modern design offices are generally equipped with a wide variety of structural analysis software programs, invariably based on the stiffness matrix method. These Finite Element Analysis packages such as MSC/NASTRAN, SAP - 90, STAAD etc., give more accurate results compared with approximate methods, but they involve significant computational effort and therefore cost. They are generally preferred for complex structures. The importance of approximate hand methods for the analysis of forces and deflections in multi-storeyed frames can not be ignored; they have served the Structural Engineer well for many decades and are still useful for preliminary analysis and checking.

3.6 Advanced structural forms

The bracing systems discussed so far are not efficient for buildings taller than 60 stories. This section introduces more advanced types of structural forms that are adopted in steel-framed multi-storeyed buildings taller than 60 storeys.

Framed -tube structures

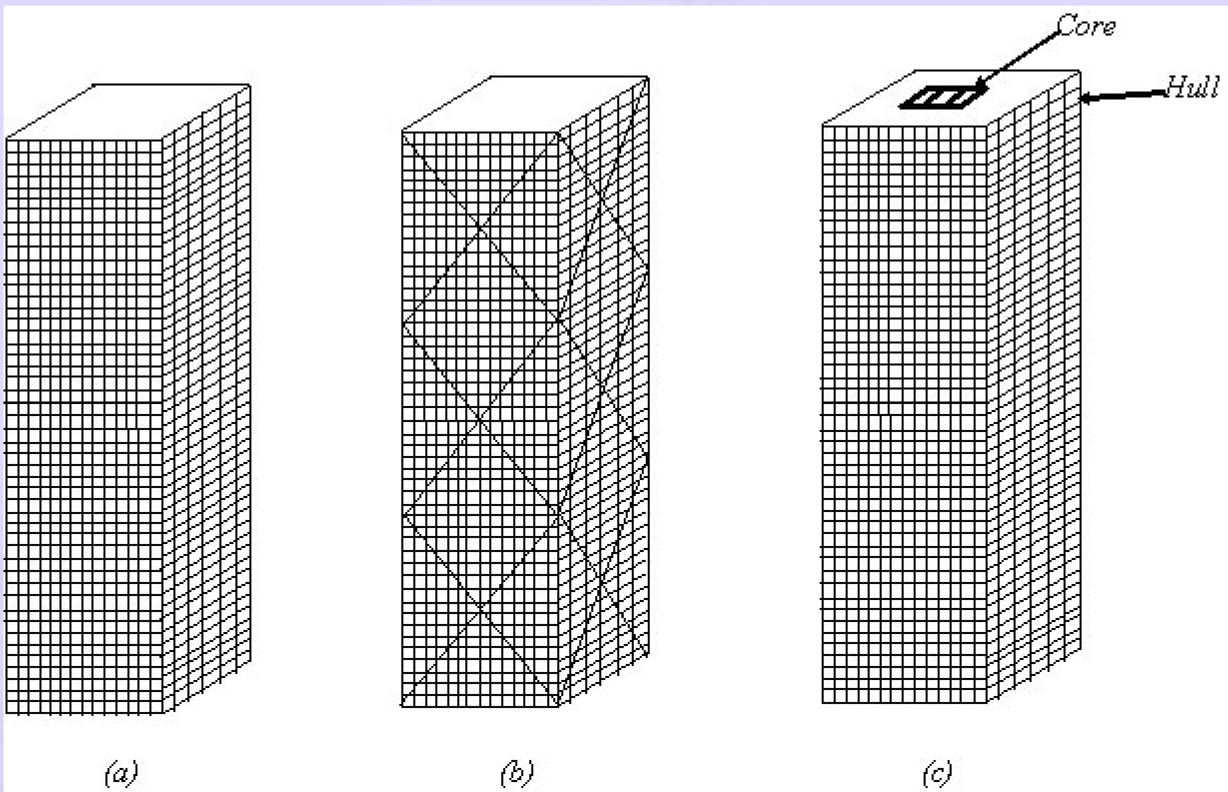


Fig. 3.14(a) Framed tube (b) Braced framed tube (c)Tube-in-Tube frame

The framed tube is one of the most significant modern developments in high-rise structural form. The frames consist of closely spaced columns, 2 - 4 m between centres, joined by deep girders. The idea is to create a tube that will act like a continuous perforated chimney or stack. The lateral resistance of framed tube structures is provided by very stiff moment resisting frames that form a tube around the perimeter of the building. The gravity loading is shared between the tube and interior columns. This structural form offers an efficient, easily constructed structure appropriate for buildings having 40 to100 storeys.

When lateral loads act, the perimeter frames aligned in the direction of loads act as the webs of the massive tube cantilever and those normal to the direction of the loading act as the flanges. Even though framed tube is a structurally efficient form, flange frames tend to suffer from shear lag. This results in the mid face flange columns being less stressed than the corner columns and therefore not contributing to their full potential lateral strength. Aesthetically, the tube looks like the grid-like façade as small windowed and is repetitious and hence use of prefabrication in steel makes the construction faster. A typical framed tube is shown in Fig.3.14 (a).

Braced tube structures

Further improvements of the tubular system can be made by cross bracing the frame with X-bracing over many stories, as illustrated in Fig. 3.14(b). This arrangement was first used in Chicago's John Hancock Building in 1969.

As the diagonals of a braced tube are connected to the columns at each intersection, they virtually eliminate the effects of shear lag in both the flange and web frames. As a result the structure behaves under lateral loads more like a braced frame reducing bending in the members of the frames. Hence, the spacing of the columns can be increased and the depth of the girders will be less, thereby allowing large size windows than in the conventional framed tube structures.

In the braced tube structure, the braces transfer axial load from the more highly stressed columns to the less highly stressed columns and eliminates differences between load stresses in the columns.

Tube-in-Tube structures

This is a type of framed tube consisting of an outer-framed tube together with an internal elevator and service core. The inner tube may consist of braced frames. The outer and inner tubes act jointly in resisting both gravity and lateral loading in steel-framed buildings. However, the outer tube usually plays a dominant role because of its much greater structural depth. This type of structures is also called as Hull (Outer tube) and Core (Inner tube) structures. A typical Tube-in-Tube structure is shown in Fig. 3.14c.

Bundled tube

The bundled tube system can be visualised as an assemblage of individual tubes resulting in multiple cell tube. The increase in stiffness is apparent. The system allows for the greatest height and the most floor area. This structural form was used in the Sears Tower in Chicago. In this system, introduction of the internal webs greatly reduces the shear lag in the flanges. Hence, their columns are more evenly stressed than in the single tube structure and their contribution to the lateral stiffness is greater.

3.7 Summary

Approximate analyses of simple braced frame as well as for frames with moment resisting joints are described. Simplified analyses of building frames with gravity loads as well as frames with lateral loads have been discussed. More accurate methods making use of flexibility or stiffness matrices are generally incorporated in sophisticated software in many design offices. Advanced structural forms used in tall buildings were also described.

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4. SPACE FRAMES

4.1 Introduction

4.1.1. General

Tensile structures are economical and efficient solution for large span structures. Although such structures are very common in all the developed countries, they are yet to make great strides in India.

The structural and material science engineers are already addressing themselves to the task of development of new construction material, and techniques and computer based tools for analysis and design to meet the challenge of providing long span structures.

Currently INDIA is poised for a major initiative in infrastructure.

Development of tensile structures should naturally play their rightful role in this initiative. In view of the tremendous oppurtunity for their use. There is need to increase the general level of the competence in the analysis and design of tensile structure in India.

Development of new techniques is space structures to reduce the deflection and the effective use of materials like steel are the great advantages in ensuring cost effectiveness.

In the space structures and the tensile structure systems steel is widely used for effective construction and to establish the use of steel in our country, we need a new technique like prestressing of steel and effective use of steel members by reducing the compression force.

4.1.2. Introduction to space frames

A space frames is structural system with three dimensional assembly of linear elements, so arranged that the loads are transferred in a three dimensional manner. These structures are commonly used for large span structures, which are more advantageous and economical for providing roofs for large span building.

4.2. Types of space frames

They are classified broadly in three categories

i. Skeleton (braced) frame work

e.g. domes, barrel vaults, double and multiplier grids, braced plates. They are more popular. They are innumerable combinations and variation possible and follow regular geometric forms.

ii. Stressed skin systems

e.g. Stressed skin folded plates, stressed skin domes and barrel vaults, pneumatic structures.

iii. Suspended (cable or membrane) structures

e.g. Cable roofs.

4.3. Space truss

Skeleton, three dimensional frame works consisting of pin connected bars are called space trusses. They are characterised by hinged joints with no moments or torsional resistance. All members carry only axial compression or tension.

4.3.1 Space grids

A grid may be defined as two or more sets of parallel beams intersecting each other at any angle and loaded by an external loading normal to the plane. They are characterised as two way or three way depending upon whether the members intersecting at a node run in two or three directions.

4.3.2. Double layer grid

A space truss can be formed by two or three layers of grids. A double layer grid consist of two plane grids forming the top and bottom layers, parallel to each other and interconnected by vertical and diagonal members. A space truss is a combination of prefabricated tetrahedral, octahedral or skeleton pyramids or inverted pyramids having triangular, square or hexagonal basis with top and bottom members normally not lying in the same vertical plane.

Double layer flat grid truss, having greater rigidity allow greater flexibility in layout and permit changes in the positioning of columns. Its high rigidity ensures that the deflections of the structures are within limits. They are usually built from simple prefabricated units of standard shape. Due to its high indeterminacy, buckling of any member under any concentrated load may not lead to the collapse of the entire structure.

Various types of double layer grids are shown in fig 4.1. They can be developed by varying the direction of top and bottom layers with respect to each other, by different positioning of the top layer nodal points and also by changing the size of the top layer grid with respect to bottom layer grid. Some examples of double layer flat grid truss constructed in India and abroad are given in Table 4.1.

Table 4.1: Some examples of Double layer grids

SI.No	Name and location	Types of frame work	Year
1	Al Wahda sports hall at Abudhabi	Square over diagonal grid (set orthogonally) covering an area of 54 x 43.4m tuball spherical connectors	1989
2	Tennis court at Deira city center, UAE	Square on square offset set diagonally 50.4 x 58.8m Depth:2.1m plate connectors.	1995
3	Indian Oil Corporation Ltd., LPG Bottling Plant, Cuddapah India.	Diagonal (size 2.8m) over square topology, consists of several largest having a size of 47.6 x 39.6m Depth:2m for larger shed 1.4m smaller shed, schkul spherical node.	1998
4	Indian Oil Corporation Ltd., LPG Bottling Plant, Ennore, Chennai	Similar configuration (vide fig)	1999

4.3.3. Advantages of space truss

1. They are light, structurally efficient and use materials optimally. It can be designed in such a way that the total weight comes between 15 to 20kg/m^2

2. It can be built up from simple, prefabricated units of standard size and shape. Hence they can be mass-produced in the factory, can be easily and rapidly assembled at site using semi-skilled labour.

3. The small size components simplify the handling, transportation and erection.

4. They are an elegant and economical means of covering large column free spaces.

5. They allow great flexibility in designing layout and positioning of end supports.

6. Services such as lighting, air conditioning etc., can be integrated with space structures.

7. The use of complicated and expensive temporary supports during erection are eliminated.

8. They posses great rigidity and stiffness for a given span/depth ratio and hence are able to resist large concentrated and unsymmetrical loading. Local overloading can be taken care by built-in reserve strength. They do not collapse locally.

4.3.4. Advantages of steel pipes

The tubular sections having lot of advantages compared to the other sections. The advantages are:

1. The load carrying capacity increases because of increase in moment of inertia.

2. Circular section may have as much as 30 to 40% less surface area than that of an equivalent rolled shape and thus reduces the cost of maintenance, cost of painting.

3. There is no better section than the tabular one for torsional resistance.

4. Tubes are of special interest to architect from an aesthetics viewpoint.

5. The external surface of the tube does not permit the collection of moisture and dust thus reducing the possibility of corrosion.

6. Under dynamic loading the tube has a higher frequency of vibration than any other cross section including a solid round bar.

4.3.5. Components of space truss

1. Axial members which are preferably tubes.
2. Connectors which join the members together
3. Bolts connecting members with nodes.

Depending upon the connecting system space truss systems are classified as Nodular and Modular systems.

Nodular systems

They consist of members and nodes.

Mero connector

It is an abbreviation for Dr. Merigenhausen, a German, inventor of the connector. With his invention in 1942, he commercialized the space frames successfully due to factory mass production of standard components and easy field assembly. It can accept as many as 18 members (Fig 4.2 shows a K-K system mero).

Tuball

It was developed by Dr. Eekhout, Neitherlands in 1984. It consists of 14 of hollow sphere as cap and 3/4 as cup. It is made of steroidal graphite. The ends of members are fitted with treated solid props by welding. It is lighter, less expensive. A typical cross section of the tuball system illustrated in Fig 4.3. Each end of a member has a cast end piece with a threaded boring to receive a bolt.

There are also other type connectors such as triodetic, nodus, schkul etc. The search for an ideal and simple connector is going on in India and abroad. The connectors explained above are all patented nodes and royalty to be paid to make use of them.

Octatube

Developed Prof.Dr. Ir.Mick Eekhout of netherlands. It is a plate connector and developed in 1973. It can be fabricated at any well equipped workshop. The joint consist of three plates an octogonal base plate and two half octagonal plates. Each member end is pressed to form a flat shape. A member is connected to a joint by two bolts. The plates are welded together to form the shape as shown in Fig 4.4.

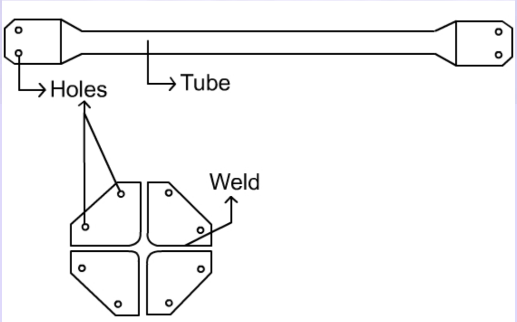


Fig 4.4 Octatube Connector

Plate connector

A new type of connector is being developed in India by Dr. A.R. Santhakumar, Dean, Civil engineering, Anna university to suit Indian requirements and conditions (vide Fig 4.5). It is successfully used in Gymnasium in Shenoy Nagar, Chennai. It can be easily fabricated in any local workshop and it can take 13 members.

It consists of a 9" x 9" square M.S base plate. Two rectangular plates with chamfered top corners are welded to the base plate perpendicular to each other across the diagonal. A solid piece with a slit is welded to the pipe ends. Web members are connected to the vertical plates while chord members are connected to the base plates

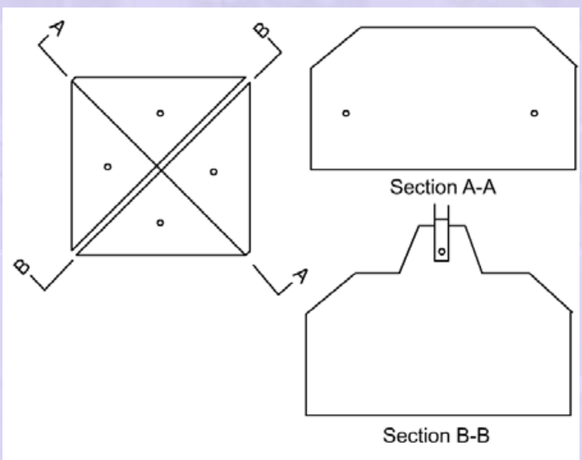


Fig 4.5 Plate Connector

4.4. Optimisation

Many design are possible to satisfy the functional requirements and a trial and error procedure may be employed to choose the optimal design. Selection of the best geometry of a structure or the member sizes are examples of optimal design procedures. The computer is best suited for finding the optimal solutions. Optimisation then becomes an automated design procedure, providing the optimal values for certain design quantities while considering the design criteria and constraints.

Computer-aided design involving user machine interaction and automated optimal design, characterised by pre-programmed logical decisions, based upon internally stored information, are not mutually exclusive, but complement each other. As the techniques of interactive computer-aided design develop, the need to employ standard routines for automated design of structural subsystems will become increasingly relevant.

The numerical methods of structural optimisation, with application of computers automatically generate a near optimal design in an interactive manner. A finite number of variables has to be established, together with the constraints relating to these variables. An initial guess-solution is used as the starting point for a systematic search for better designs and the process of search is terminated when certain criteria are satisfied.

Those quantities defining a structural system that are fixed during the automated design are called pre-assigned parameters or simply parameters and those quantities that are not pre-assigned are called design variables. The design variables cover the material properties, the topology of the structure, its geometry and the member sizes. The assignment of the parameters as well as the definition of their values are made by the designer, based on his experience.

Any set of values for the design variables constitutes a design of the structure. Some designs may be feasible while others are not. The restrictions that must be satisfied in in order to produce a feasible design are called constraints. There are two types of constraints: design constraints and behaviour constraints. Examples of design constraints are minimum thickness of a member, maximum height of a structure, etc. Limitations on the maximum stresses, displacement or buckling strength are typical examples of behaviour constraints. These constraints are expressed mathematically as a set of inequalities:

$$g_j(\{X\}) < 0 \qquad j = 1,2,....,m$$

Where {X} is the design vector, and m is the number of inequality constraints.

In addition, we have also to consider equality constraints of the form

$$h_j(\{X\}) = 0 \qquad j = 1,2,....,k$$

Where k is the number of equality constraints.

Example

The three bar truss example first solved by Schmitt is shown in Fig 4.6. The applied loadings and the displacement directions are also shown in this figure.

1. Design constraints: The conditions that the area of members cannot be less than zero can be expressed as

$$g_1 \equiv -X_1 \leq 0$$
$$g_2 \equiv -X_2 \leq 0$$

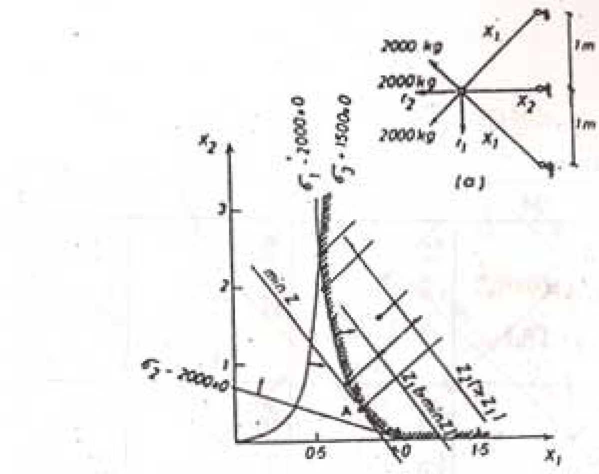
2. Behaviour constriants: The three members of the truss should be safe, that is the stresses in them should be less than the allowable stresses in tension (2,000kg/cm^2) and compression (1,500kg/cm^2). This is expressed as

$$
\begin{aligned}
& g_3 \equiv \sigma_1 - 2000 \le 0 \text{ Tensile stress limit in member 1} \\
& g_4 \equiv -\sigma_1 - 1500 \le 0 \text{ compressive stress limitaion in member 2 and so on} \\
& g_5 \equiv \sigma_2 - 2500 \le 0 \\
& g_6 \equiv -\sigma_2 - 1500 \le 0 \\
& g_7 \equiv \sigma_3 - 2000 \le 0 \\
& g_8 \equiv -\sigma_3 - 2000 \le 0
\end{aligned}
$$

3. Stress-force relationships: Using the stress-strain relationship s = [E]{Δ} and the force-displacement relationship F=[K]{Δ}, the stress-force relationship is obtained as {σ}=[E][K]$^{-1}${F}which can be shown as

$$
\begin{aligned}
\sigma_1 &= 2000\left(\frac{X_2 + \sqrt{2}X_1}{2X_1X_2 + \sqrt{2}X_1{}^2}\right) \\
\sigma_2 &= 2000\left(\frac{\sqrt{2}X_1}{2X_1X_2 + \sqrt{2}X_1{}^2}\right) \\
\sigma_3 &= 2000\left(\frac{X_2}{2X_1X_2 + \sqrt{2}X_1{}^2}\right)
\end{aligned}
$$

4. Constraint design inequalities: Only constraints g_3, g_5, g_8 will affect the design. Since these constraints can now be expressed interms of design variable X_1 and X_2 using the stress-force relationship derived above, they can be represented as the area on one side of the straight line in the two-dimensional plot. (Fig 4.6)



Example of a 3 bar truss.

Fig 4.6

4.5. Design space

Each design variable X_1, X_2 ...is viewed as one dimension in a design space a particular set of variable as a point in this space. In the general case of n variable, we have an n-dimensioned space. In the example where we have only two variables, the space reduces to a plane figure shown in (Fig 4.6(b)). The arrows indicate the inequality representation and the shaded zone shows the feasible region. A design falling in the feasible region is an unconstrained design and the one falling on boundary is a constrained design.

An infinite number of feasible design is possible. In order to find the best one, it es necessary of form a function of the variables to use for comparision of feasible design alternatives. The objective (merit) function is a function whose least value is sought in an optimisation procedure. In other words, the optimisation problem consists in the determination of the vector of variables X that will minimise a certain given objective functions.

$$Z = F(\{X\})$$

In the example chosen, assuming the volume of material as the objective function, we get

$$Z = 2(141\ X_1) + 100\ X_2$$

The locus of all points satisfying F ({X}) = constant, form a straight line in a two -dimensional space. (In this general case of n-dimensional space, it will form a surface). For each value of constraint, a different straight line is obtained. Fig 4.6(b) shows the objective function contours. Every design on a particular contour has the same volume or weight. It can be seen that the minimum value of F ({X}) in the feasible region occurs at point A.

There are different approaches to this problem which constitute the various methods of optimization. The traditional approach searches the solution by pre-selecting a set of critical constraints and reducing the problem to a set of equations in fewer variables. Successive reanalysis of the structure for improved sets of constraints will tend towards the solution. Different re-analysis methods can be used, the iterative methods being the most attractive in the case of space structures.

4.6. Optimality criteria

An interesting approach in optimization is a process known as optimality criteria. The approach to the optimum is based on the assumption that some characteristic will be attained at such optimum. The well-known example is the fully stressed design where it is assumed that, in an optimal structure, each member is subjected to its limiting stress under at least one loading condition.

The optimality criteria procedures are useful for space structures because they constitute an adequate compromise to obtain practical and efficient solutions. In many studies, it has been found that the shape of the objective function around the optimum is flat, which means that an experienced designer can reach solutions which are close to the theoretical optimum.

4.7. Mathematical programming

It is difficult to anticipate which of the constraints will be critical at the optimum. Therefore, the use of inequality constraints is essential for a proper formulation of the optimal design problem.

The mathematical programming (MP) methods are included to solve the general optimisation problem by numerical search algorithms while being general regarding the objective function and constraints. On the other hand, approximations are often required tube efficient on large practical problems such as space structures.

Optimal design processes involves the minimization of weight subject to certain constraints. Mathematical programming methods and structural theorems are available to achieve such a design goal.

Of the various mathematical programming methods available for optimisation, the linear programming methods is widely adopted in structural engineering practice because of its simplicity. The objective function, which is the minimisation of weight, is linear and a set of constraints, which can be expressed by linear equations involving the unknowns (area, moment of inertia, etc., of the members), are used for solving the problems. This can be mathematically expressed as follows:

Suppose it is required to find a specified number of design variables x_1, $x_2 \ldots x_n$ such that the objective function.

$$Z = C_1x_1 + C_2x_2 + \ldots C_nx_n$$

is minimised, satisfying the constraints.

$$
\begin{aligned}
&a_{11}x_1 = a_{12}x_2 + ...a_{1n}x_n \le b_1 \\
&a_{21}x_1 = a_{22}x_2 + ...a_{2n}x_n \le b_2 \\
&. \\
&. \\
&a_{m1}x_1 = a_{m2}x_2 + ...a_{mn}x_n \le b_m
\end{aligned}
$$

The simplex algorithm is a versatile procedure for solving linear programming (LP) problems with a large number of variables and constraints.

The simplex algorithm is now available in the form of standard computer software package which uses the matrix representation of the variables and constraints, especially when their number is very large.

The above set of equations is expressed in the matrix form as follows:

Find $X = \begin{bmatrix} x_1 \\ x_2 \\ . \\ . \\ x_n \end{bmatrix}$ Which minimised

The objective function $f(x) = \sum_{n-1}^{n} C_i x_i$

subject to the constraints

$$\sum_{k-1}^{n} a_{jk} x_k \le b_j \,, \; j = 1, 2, ...m$$

$$\text{and } x_i \ge 0, \; i = 1, 2, ...n$$

Where C_i, a_{jk} and b_j are constants

The stiffness method of analysis is adopted and the optimisation is achieved by mathematical programming

The structure is divided into a number of groups and the analysis is carried out groupwise. Then the member forces are determined. The critical members are found out from each group. From the initial design, the objective function and the constraints are framed. Then, by adopting the fully stressed design (optimality criteria) method, the linear programming problem is solved and the optimal solution found out. In each group every member is designed for the fully stressed condition and the maximum size required is assigned for all the members in that group. After completion of the design, one more analysis and design routine for the structure as a whole is completed for alternative cross-section.

4.8. Geometry as variable

In method 1, only the member sizes were treated as variables whereas the geometry was assumed as fixed. Method 2 treats the geometry also as a variable and gets the most preferred geometry. The geometry developed by the computer results in the minimum weight of space frame for any practically acceptable configuration. For solutions, since a iterative procedure is adopted for the optimum structural design, it is obvious that the use of a computer is essential.

The algorithm used for optimum structural design is similar to that given by Samuel L. Lipson which presumes that an initial feasible configuration is available for the structure. The structure is divided into a number of groups and the externally applied loadings are obtained. For the given configuration, the upper limits and the lower limits on the design variables, namely the joint coordinates are fixed. Then (k-1) new configurations are generated randomly as

$$X_{ij} = 1_i + r_{ij} (u_i - 1_i)$$

$i = 1,2,...n$

$j = 1,2,...k$

where k is the total number of configurations in the complex, usually larger than (n+1) where n is the number of design variables and r_{ij} is the random number for the i^{th} coordinates of the j^{th} points, the random numbers having a uniform distribution over the interval 10 to 1 u_i is the upper limit and 1_i is the lower limit of the i^{th} independent variable.

Thus, the complex containing k number of feasible solutions is generated and all these configurations will satisfy the explicit constraints, namely, the upper and lower bounds on the design variables. Next, for all these k configurations, analysis and fully stressed designs are carried out and their corresponding total weights determined. Since the fully stressed design concept is an economical and practical design, it is used for steel area optimization. Every area optimisation problem is associated with more than one analysis and design. For the analysis of the truss, the matrix method has been used. Therefore, the entire generated configuration also satisfy the implicity constraints, namely, the allowable stress constraints.

From the value of he objective function (total weight of the structure) of k configurations, the vector which yield the maximum weight is search and discarded, and the centrid c of each joint of the k-1 configurations is determined from.

$$X_{ie} = \frac{1}{K-1}\left\{ \underset{j-1}{K} \sum \left(x_{ij} \right) - x_{iw} \right\}$$

i = 1, 2, 3.........n

In which x_{ie} and x_{iw} are the i_{th} coordinates of the centroid c and the discarded point ω.

Then a new point is generated by reflecting the worst point through the centroid, x_{ie}.

That is, $x_{iw} = x_{ie} + a (x_{ie} - x_{iw})$

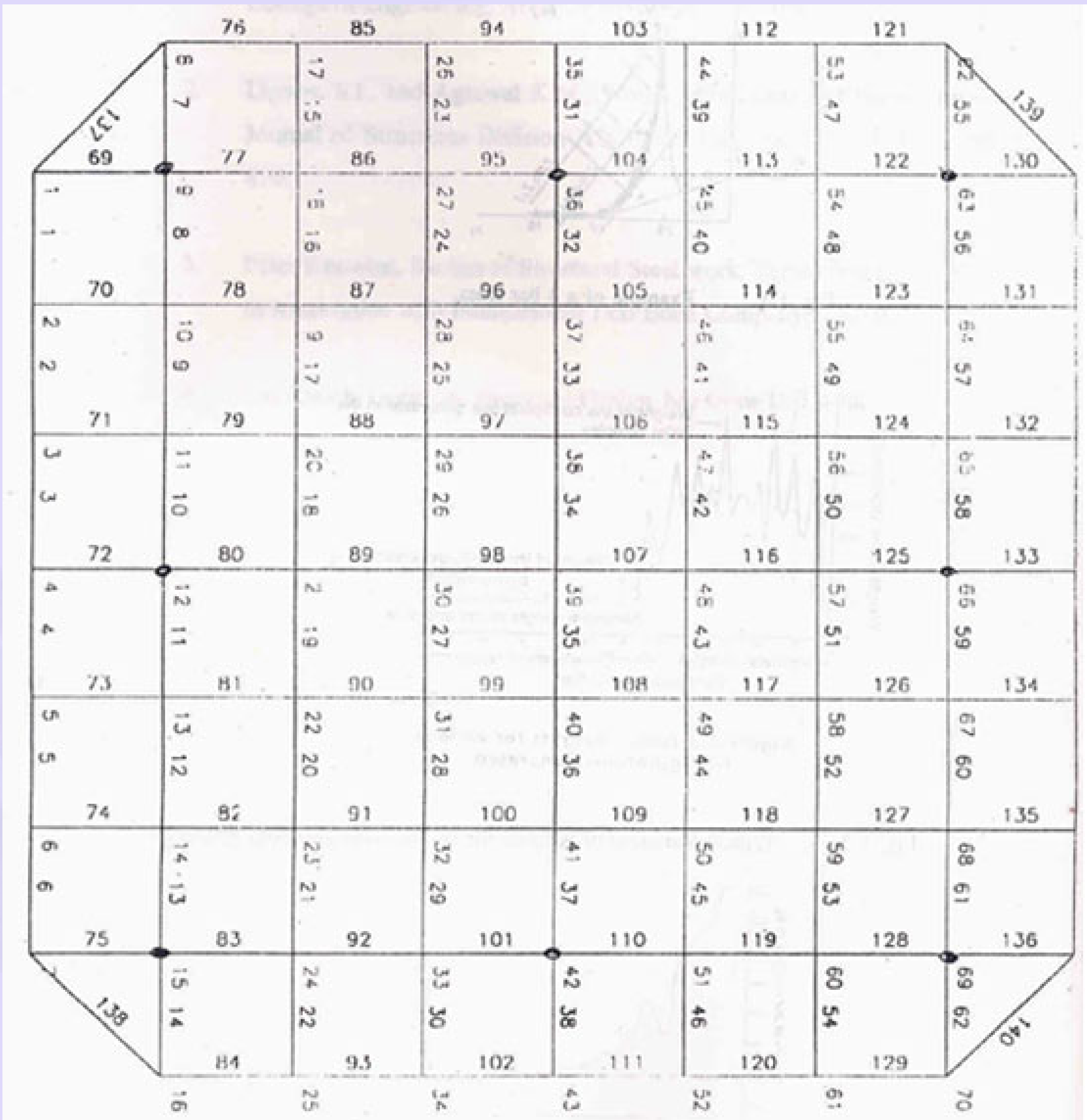
i = 1,2,.......n

where α is a constant.

This new point is first examined to satisfy the explicity constraints. If it exceeds the upper or lower bound value, then the value is taken as the corresponding limiting value, namely, the upper or lower bound. Now the area optimisation is carried out for the newly generated configuration and if the functional value is better than the second worst, the point is accepted as an improvement and the process of developing the new configuration is repeated as mentioned earlier. Otherwise, the newly generated point is moved halfway toward the centroid of the remaining points and the area optimisation is repeated for the new configuration,. This process is repeated over a fixed number of iterations and the end of every iteration, the weight and the corresponding configuration are printed out, which will shown the minimum weight achievable within the limits (u_i and 1_i) of the configuration.

4.9. Case studies

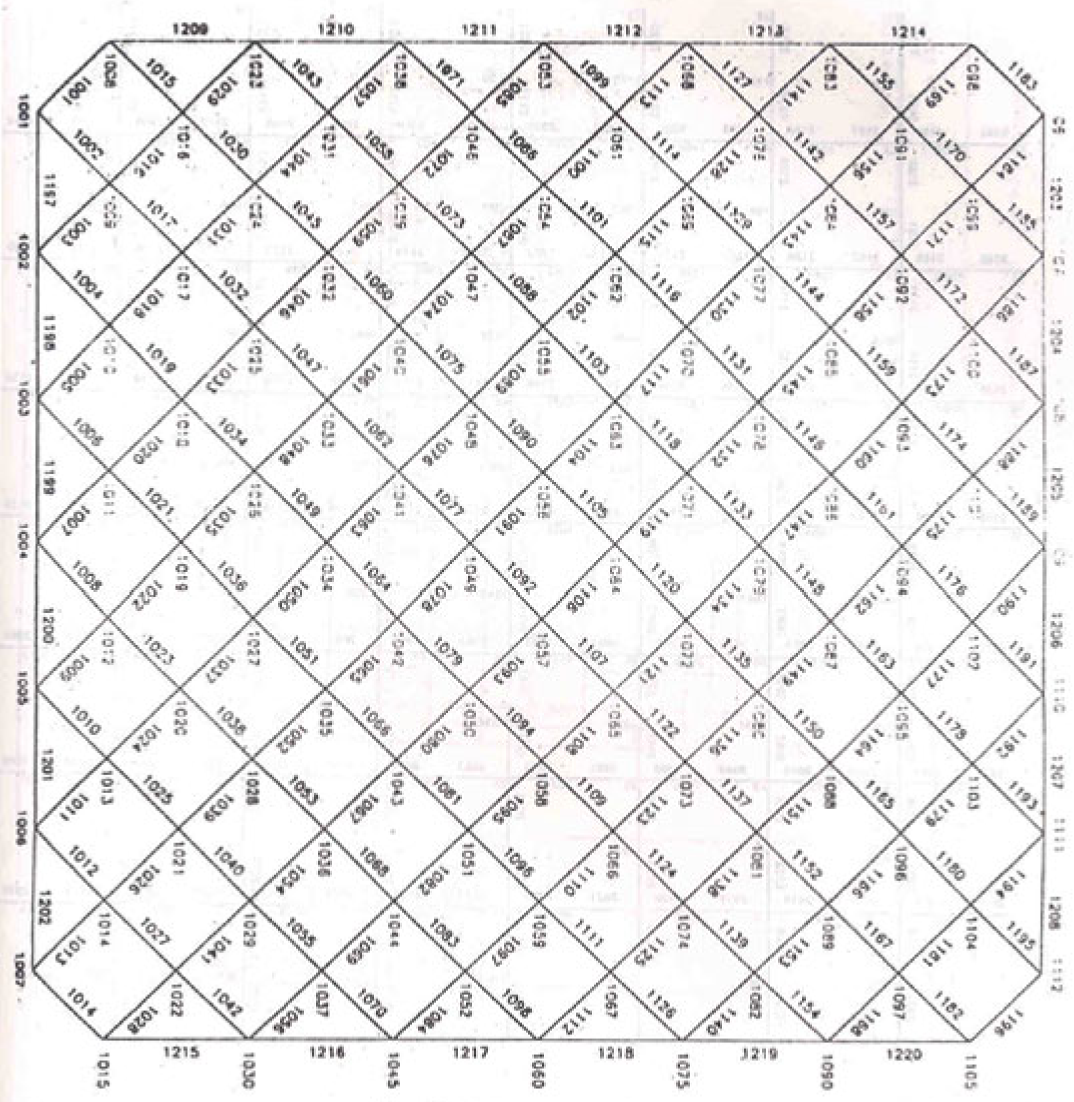
The example chosen for optimum design is a flat space structure 16.8m x 16.8m having columns only on the periphery. The plan of bottom chord, top chord and bracing members are shown in Fig 4.7 a, b, c respectively.



Plan of bottom chrod of the space truss.

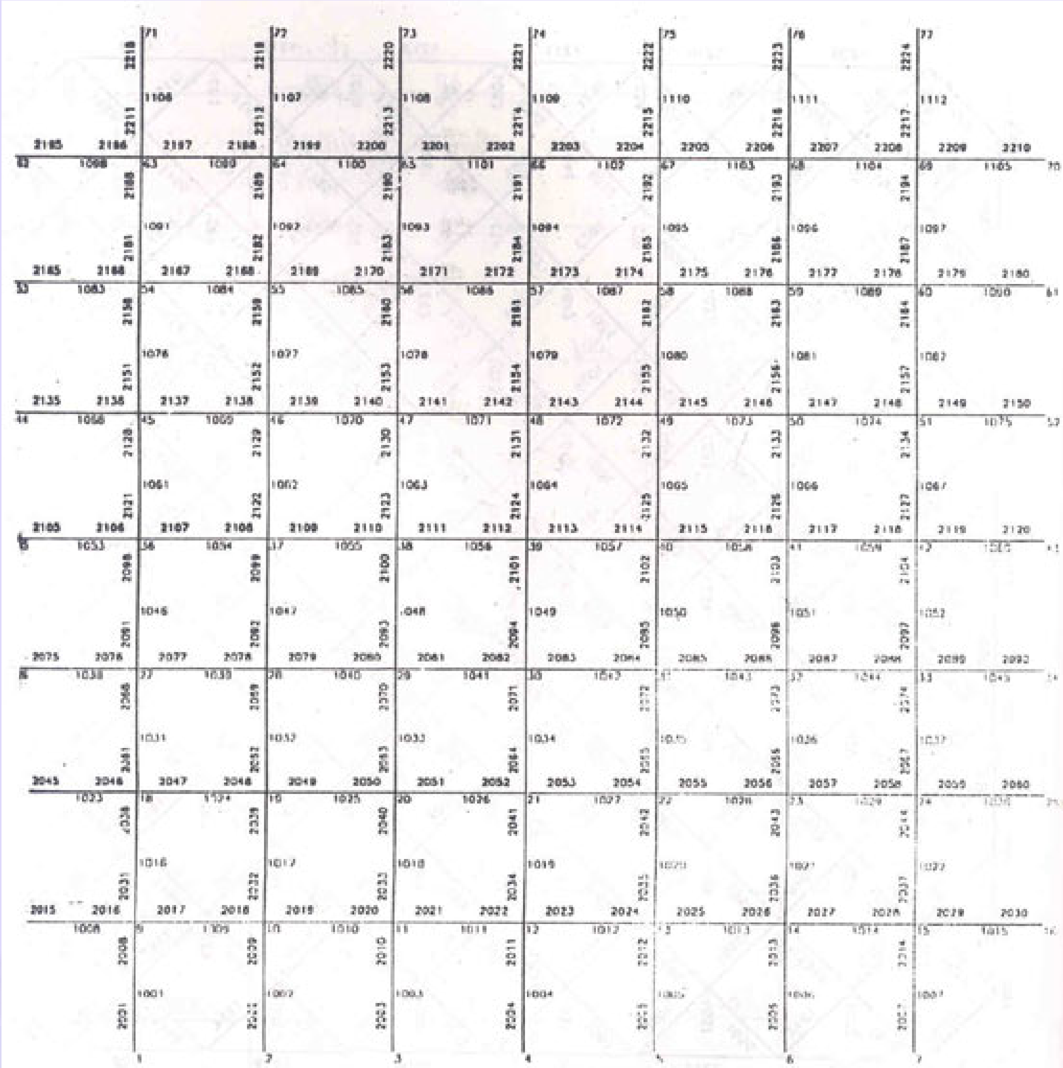
The loading conditions chosen are dead load, live load and wind load. The initial feasible configuration has eight panels in the X and Y direction for the bottom panel.

The number of design variable chosen is 5.



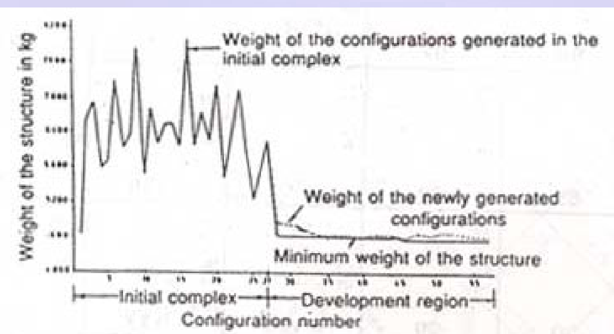
Plan of top chord of the space truss.

In the initial complex 27 configuration are generated, including the initial feasible configuration. Random numbers required for generation of these configurations are fed into the computer as input. The variables involved one type of arrangement, height of the truss, number of panels is x direction, number of panels in y direction and location of columns. The number of iterations for development during optimisation procedure is pictorially represented in Fig 4.8. The final configuration selected is based in the least weight of 15 Kg/Sq mt for the initial feasible of weight with respect to height of space frame.



Plan of bracings of the space truss.

Fig 4.7 a, b, c



Typical Variation of weights for various configurations generated.

Fig 4.8

The depth of space frame as it varied with iteration for various weights of space frame is shown in Fig. 4.9

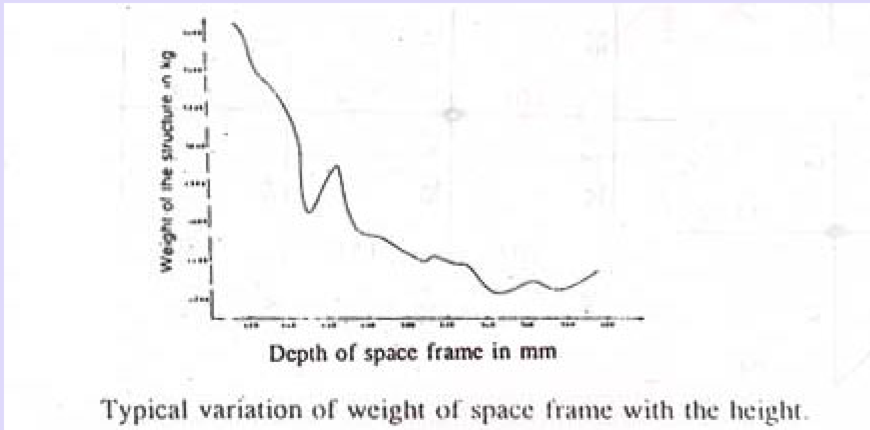


Fig 4.9

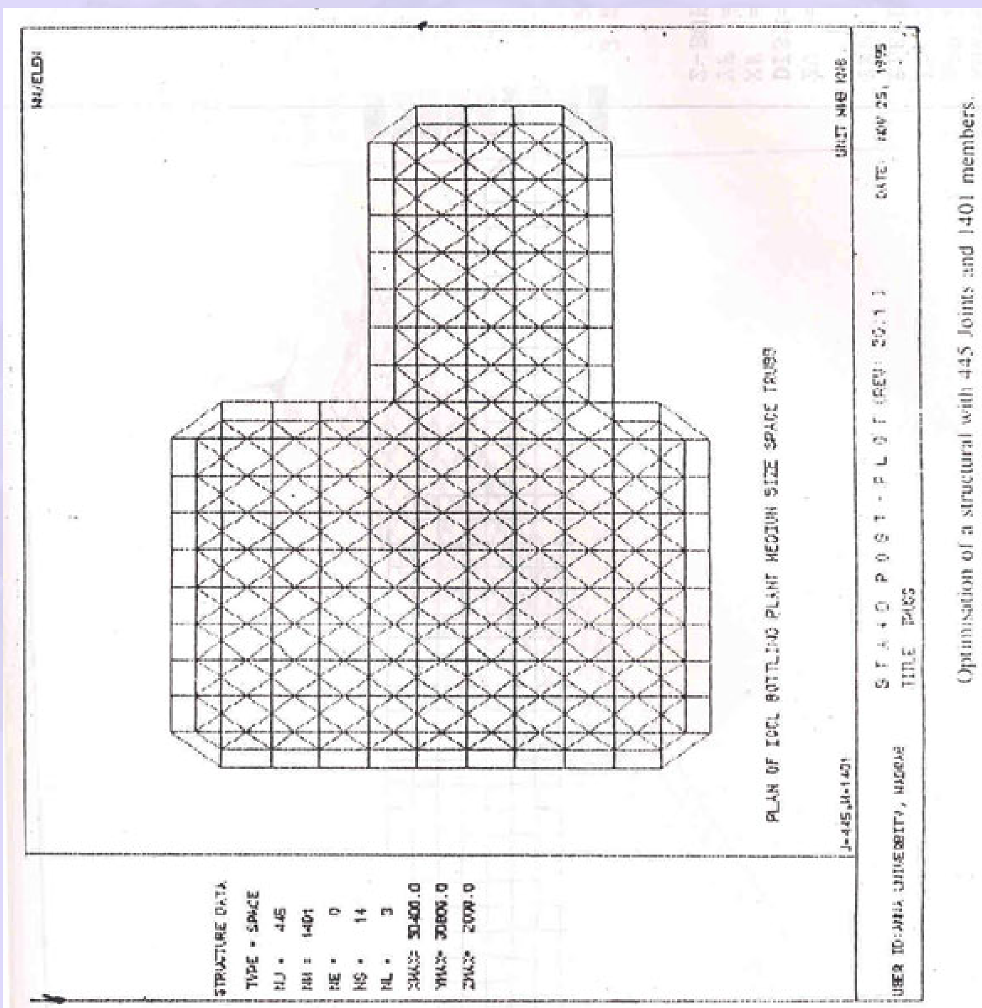
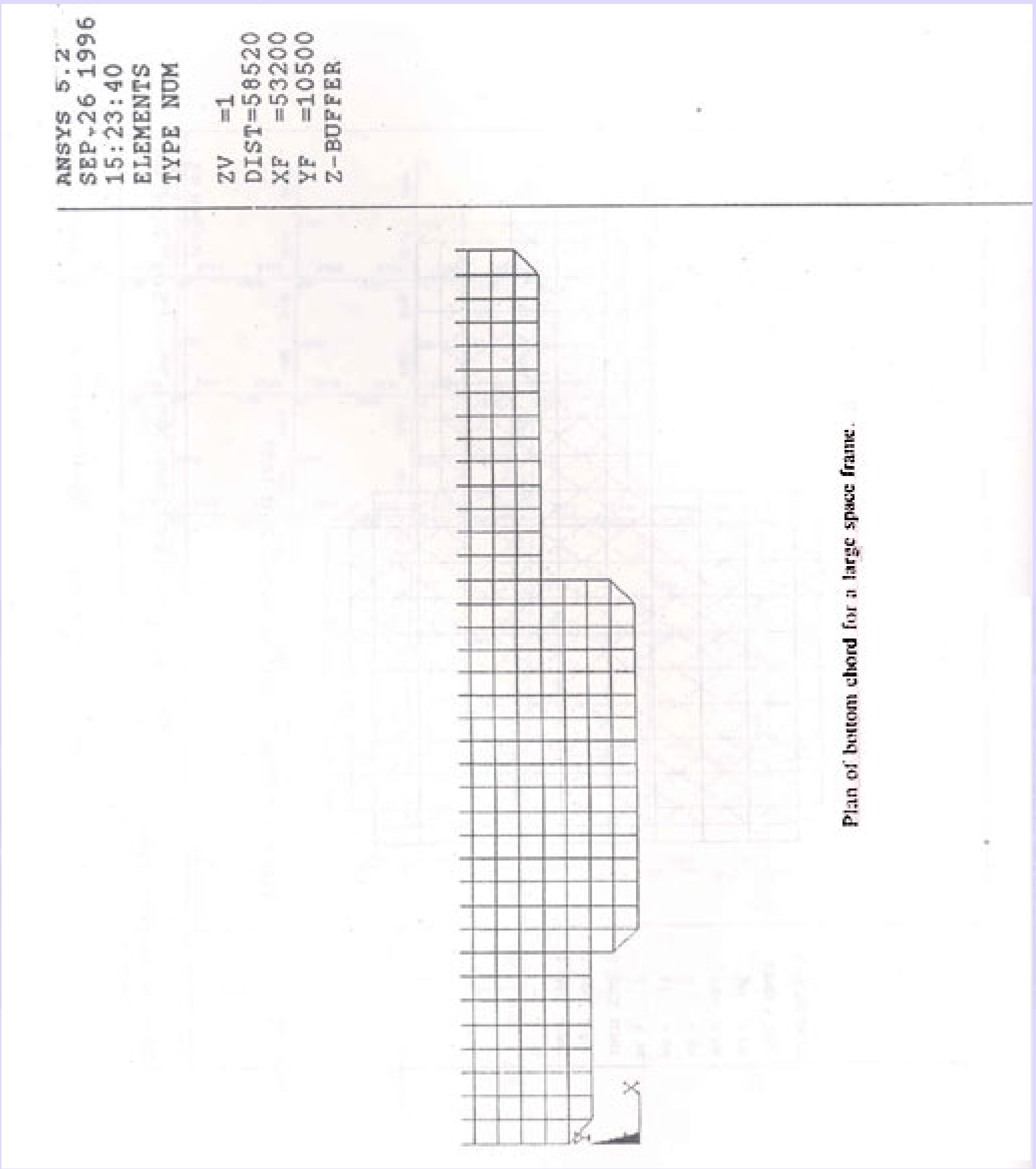


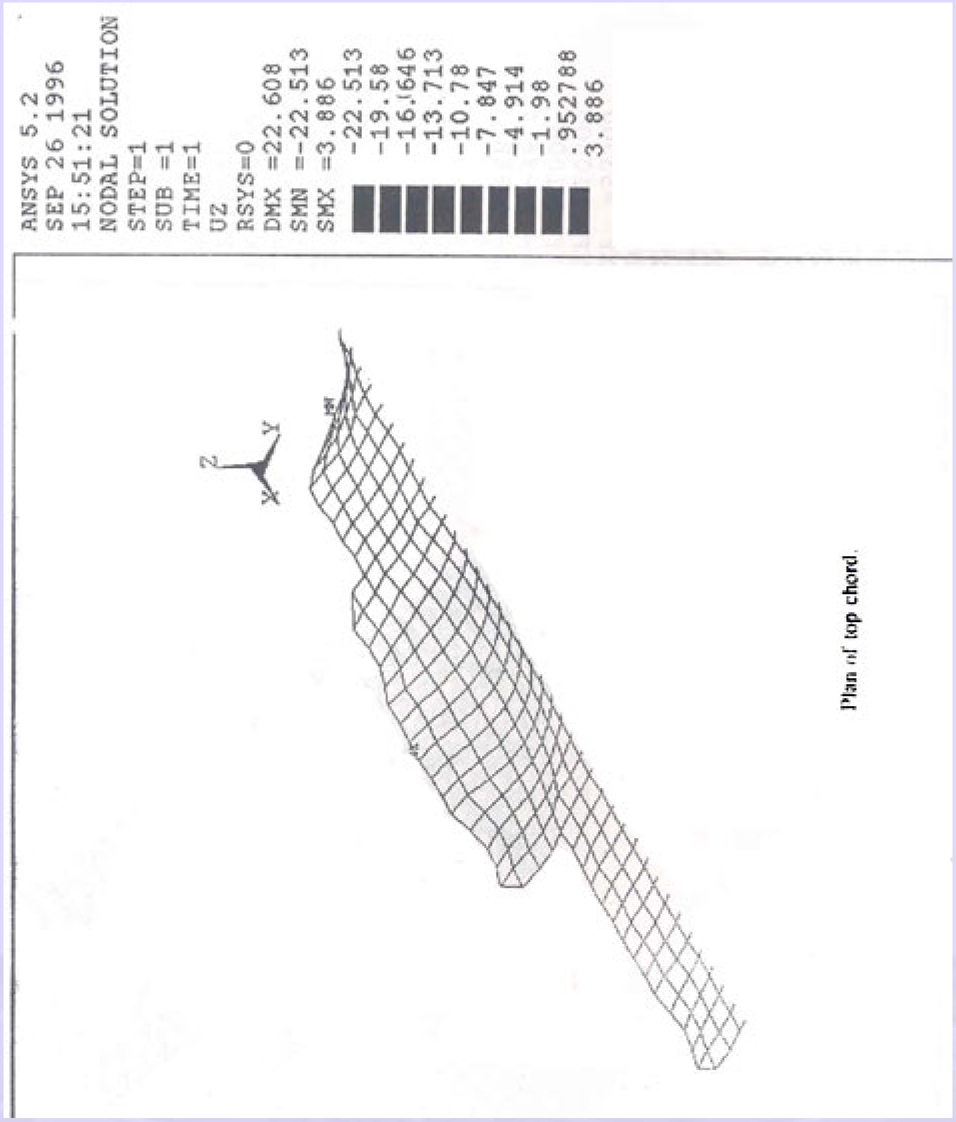
Fig 4.10

Fig.4.10 shows the final optimised structure with 445 joints and 1402 members.

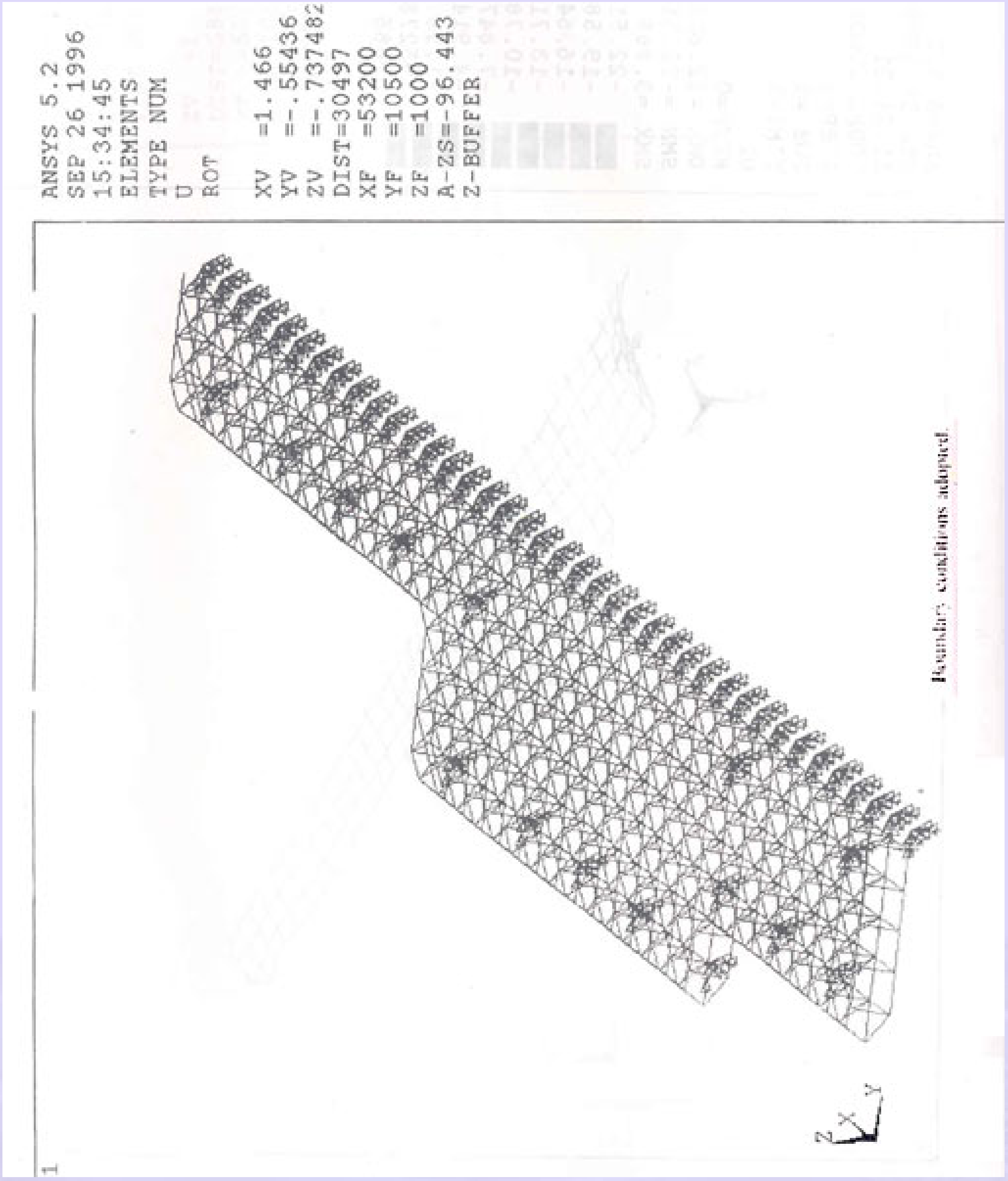
Fig 4.11 (a, b, c, d, e) show various views of the space frame as it defermes during loading for the boundary conditions adopted.



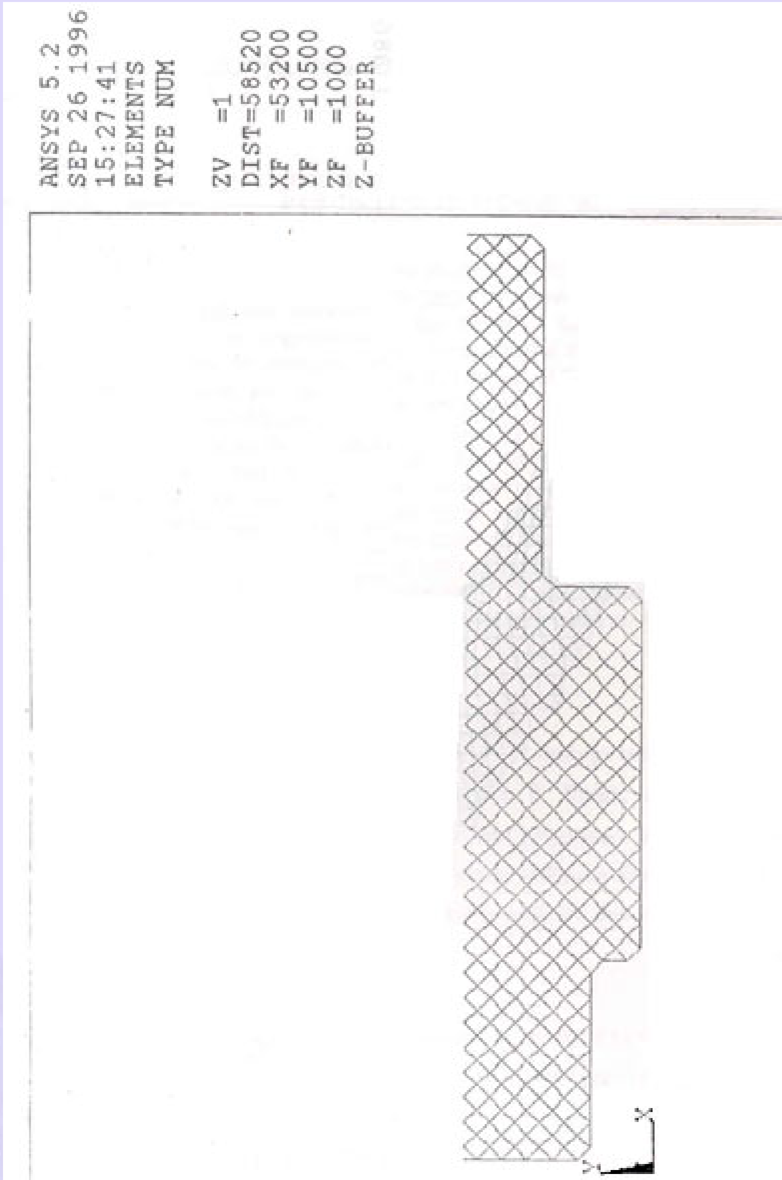
Plan of bottom chord for a large space frame.



Plan of top chord.



Boundary conditions adopted.



Deformation profile at top chord level.

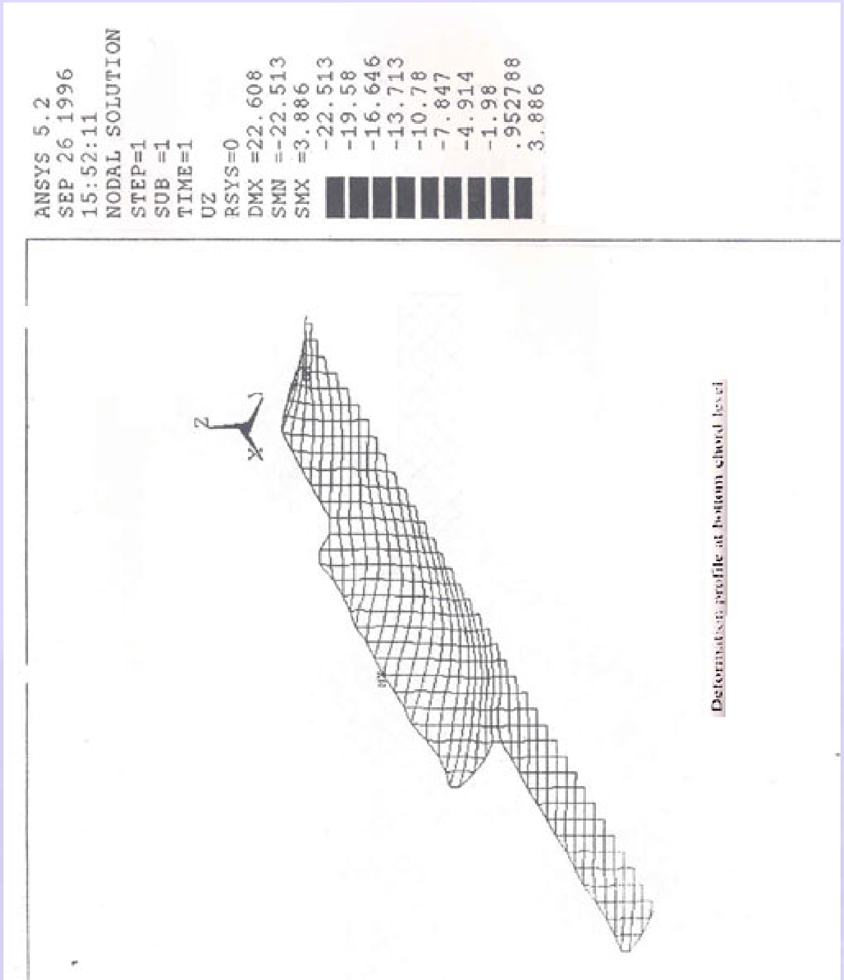


Fig 4.11 a, b, c, d, e

4.10 Conclusion

The case study shows the behaviour of an actual space frame constructed for a typical industrial structure. Such frames can be provided for factories, sports complexes and airports.

4.11. References

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5. COLD FORM STEEL

5.1 Introduction

Thin sheet steel products are extensively used in building industry, and range from purlins to roof sheeting and floor decking. Generally these are available for use as basic building elements for assembly at site or as prefabricated frames or panels. These thin steel sections are ***cold-formed***, i.e. their manufacturing process involves forming steel sections in a cold state (i.e. without application of heat) from steel sheets of ***uniform*** thickness. These are given the generic title ***Cold Formed Steel Sections***. Sometimes they are also called ***Light Gauge Steel Sections*** or ***Cold Rolled Steel Sections***. The thickness of steel sheet used in cold formed construction is usually 1 to 3 mm. Much thicker material up to 8 mm can be formed if pre-galvanised material is not required for the particular application. The method of manufacturing is important as it differentiates these products from ***hot rolled steel*** sections. Normally, the yield strength of steel sheets used in cold-formed sections is at least 280 N/mm^2, although there is a trend to use steels of higher strengths, and sometimes as low as 230 N/mm^2.

Manufacturers of cold formed steel sections purchase steel coils of 1.0 to 1.25 m width, slit them longitudinally to the correct width appropriate to the section required and then feed them into a series of roll forms. These rolls, containing male and female dies, are arranged in pairs, moving in opposite direction so that as the sheet is fed through them its shape is gradually altered to the required profile. The number of pairs of rolls (called **stages**) depends on the complexity of the cross sectional shape and varies from 5 to 15. At the end of the rolling stage a flying shearing machine cuts the member into the desired lengths.

An alternative method of forming is by press - braking which is limited to short lengths of around 6 m and for relatively simple shapes. In this process short lengths of strip are pressed between a male and a female die to fabricate one fold at a time and obtain the final required shape of the section. Cold rolling is used when large volume of long products is required and press breaking is used when small volumes of short length products are produced.

Galvanizing (or zinc coating) of the preformed coil provides very satisfactory protection against corrosion in internal environments. A coating of 275 g/m^2 (total for both faces) is the usual standard for internal environments. This corresponds to zinc coating of 0.04 mm. Thicker coatings are essential when moisture is present for long periods of time. Other than galvanising, different methods of pre-rolling and post-rolling corrosion protection measures are also used.

Although the cold rolled products were developed during the First World War, their extensive use worldwide has grown only during the last 20 years because of their versatility and suitability for a range of lighter load bearing applications. Thus the wide range of available products has extended their use to primary beams, floor units, roof trusses and building frames. Indeed it is difficult to think of any industry in which Cold Rolled Steel products do not exist in one form or the other. Besides building industry, they are employed in motor vehicles, railways, aircrafts, ships, agricultural machinery, electrical equipment, storage racks, house hold appliances and so on. In recent years, with the evolution of attractive coatings and the distinctive profiles that can be manufactured, cold formed steel construction has been used for highly pleasing designs in practically every sector of building construction.

In this chapter, the background theory governing the design of cold formed steel elements is presented in a summary form. Designs of cold formed steel sections are dealt with in IS: 801-1975 which is currently due under revision. In the absence of a suitable Limit State Code in India, the Code of Practice for Cold Formed Sections in use in the U.K. (BS 5950, Part 5) is employed for illustrating the concepts with suitable modifications appropriate to Indian conditions.

In the last chapter, the special features and attractions of cold formed steel sections for many industrial applications were presented and discussed. In view of the use of very thin steel sheet sections, (generally in the 1 mm - 3 mm range), particular attention has to be paid to buckling of these elements. Stiffened and unstiffened elements were compared and the concept of effective width to deal with the rapid design of compression elements together with suitable design simplifications, outlined. Finally, the methods adopted for the design of laterally restrained beams and unrestrained beams were discussed. The techniques of eliminating lateral buckling in practice, by providing lateral braces or by attachment to floors etc were described so that the compression flanges would not buckle laterally.

In this chapter the design of columns for axial compression, compression combined with bending as well as for torsional-flexural buckling will be discussed. The diversity of cold formed steel shapes and the multiplicity of purposes to which they are put to, makes it a difficult task to provide general solution procedures covering all potential uses. Some design aspects are nevertheless included to provide a general appreciation of this versatile product. It is not unusual to design some cold formed steel sections on the basis of prototype tests or by employing empirical rules. These are also discussed in a summary form herein.

5.2 Advantages of cold formed sections

Cold forming has the effect of increasing the yield strength of steel, the increase being the consequence of cold working well into the strain-hardening range. These increases are predominant in zones where the metal is bent by folding. The effect of cold working is thus to enhance the mean yield stress by 15% - 30%. For purposes of design, the yield stress may be regarded as having been enhanced by a minimum of 15%.

Some of the main advantages of cold rolled sections, as compared with their hot-rolled counterparts are as follows:

• Cross sectional shapes are formed to close tolerances and these can be consistently repeated for as long as required.

• Cold rolling can be employed to produce almost any desired shape to any desired length.

• Pre-galvanised or pre-coated metals can be formed, so that high resistance to corrosion, besides an attractive surface finish, can be achieved.

• All conventional jointing methods, (i.e. riveting, bolting, welding and adhesives) can be employed.

• High strength to weight ratio is achieved in cold-rolled products.

• They are usually light making it easy to transport and erect.

It is possible to displace the material far away from the neutral axis in order to enhance the load carrying capacity (particularly in beams).

There is almost no limit to the type of cross section that can be formed. Some typical cold formed section profiles are sketched in Fig.5.1.

In Table 1 hot rolled and cold formed channel section properties having the same area of cross section are shown. From Table 5.1, it is obvious that thinner the section walls, the larger will be the corresponding moment of inertia values (I_{xx} and I_{yy}) and hence capable of resisting greater bending moments. The consequent reduction in the weight of steel in general applications produces economies both in steel costs as well as in the costs of handling transportation and erection. This, indeed, is one of the main reasons for the popularity and the consequent growth in the use of cold rolled steel. Also cold form steel is protected against corrosion by proper galvanising or powder coating in the factory itself. Usually a thickness limitation is also imposed, for components like lipped channels.

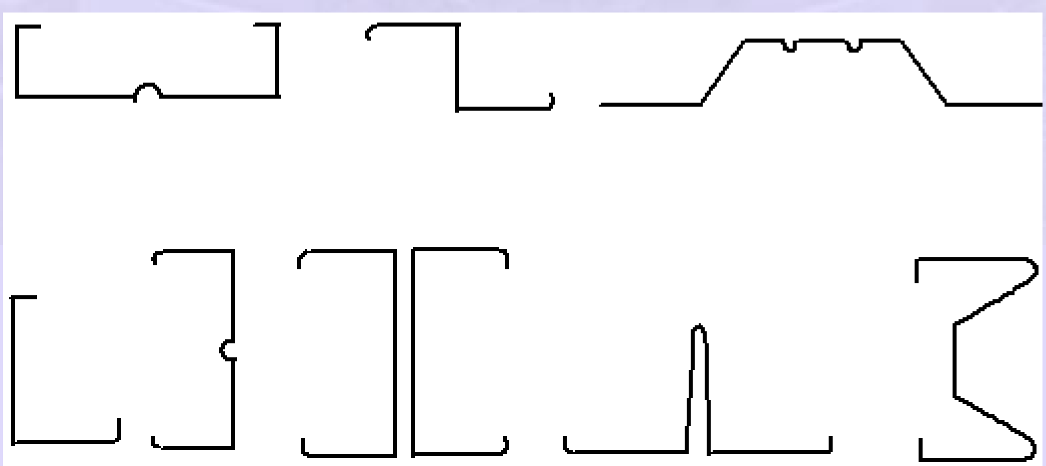
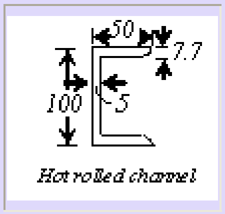
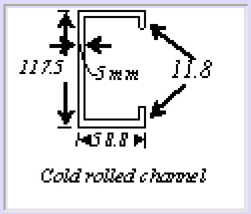
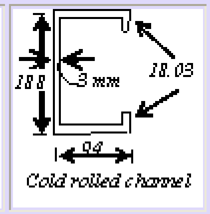
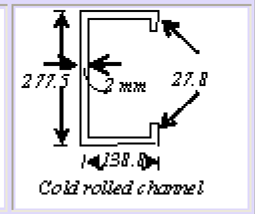


Fig. 5.1 Typical cold formed steel profiles

Table -5.1 Comparison of hot rolled and cold rolled sections

				
A	1193 mm^2	1193 mm^2	1193 mm^2	1193 mm^2
I_{xx}	1.9×10^6 mm^4	2.55×10^6 mm^4	6.99×10^6 mm^4	15.53×10^6 mm^4
Z_{xx}	38×10^3 mm^3	43.4×10^3 mm^3	74.3×10^3 mm^3	112×10^3 mm^3
I_{yy}	0.299×10^6 mm^4	0.47×10^6 mm^4	1.39×10^6 mm^4	3.16×10^6 mm^4
Z_{yy}	9.1×10^3 mm^3	11.9×10^3 mm^3	22×10^3 mm^3	33.4×10^3 mm^3

While the strength to weight ratios obtained by using thinner material are significantly higher, particular care must be taken to make appropriate design provisions to account for the inevitable buckling problems.

5.2.1 Types of stiffened and unstiffened elements

As pointed out before, cold formed steel elements are either ***stiffened*** or ***unstiffened***. An element which is supported by webs along both its longitudinal edges is called a ***stiffened*** element. An ***unstiffened*** element is one, which is supported along one longitudinal edge only with the other parallel edge being free to displace. Stiffened and unstiffened elements are shown in Fig. 5.2.

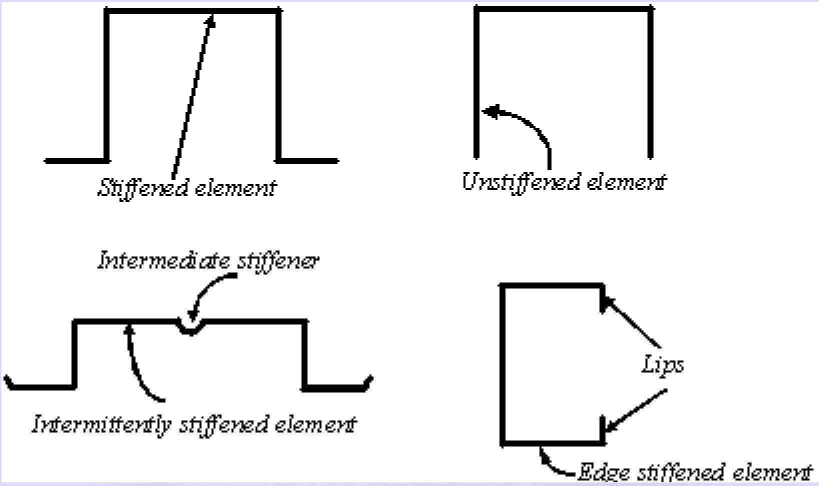


Fig.5.2 Stiffened and unstiffened elements

An ***intermittently stiffened element*** is made of a very wide thin element which has been divided into two or more narrow sub elements by the introduction of intermediate stiffeners, formed during rolling.

In order that a flat compression element be considered as a ***stiffened element***, it should be supported along one longitudinal edge by the web and along the other by a web or lip or other edge stiffener, (eg. a bend) which has sufficient flexural rigidity to maintain straightness of the edge, when the element buckles on loading. A rule of thumb is that the depth of simple “lips” or right angled bends should be at least one-fifth of the adjacent plate width. More exact formulae to assess the adequacy of the stiffeners are provided in Codes of Practice. If the stiffener is adequate, then the edge stiffened element may be treated as having a local buckling coefficient (K) value of 4.0. If the edge stiffener is inadequate (or only partially adequate) its effectiveness is disregarded and the element will be regarded as ***unstiffened***, for purposes of design calculations.

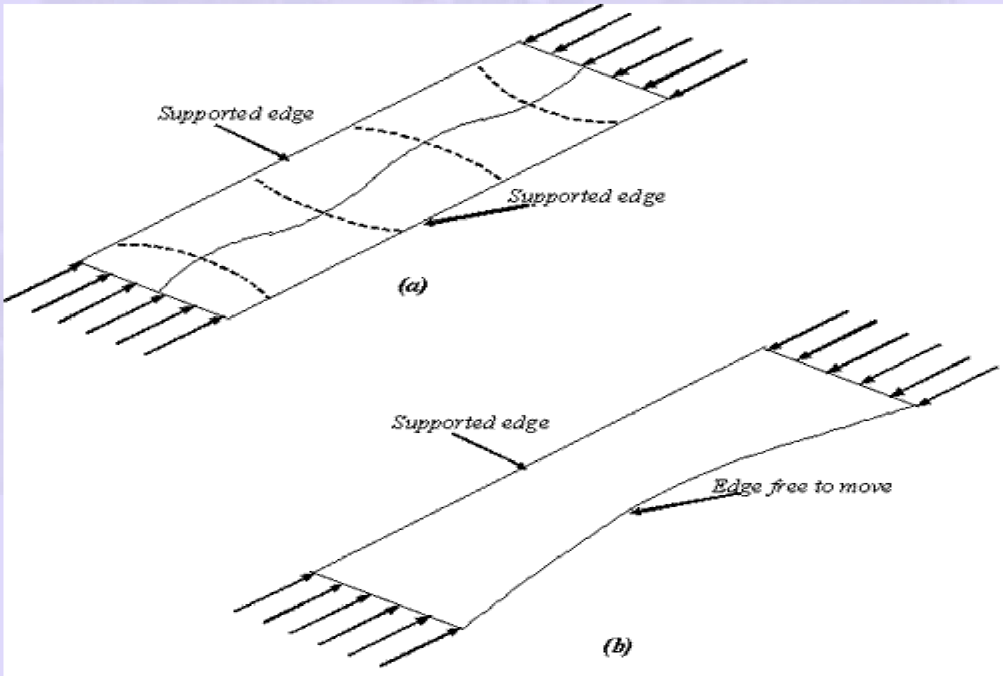
5.3 Local buckling

Local buckling is an extremely important facet of cold formed steel sections on account of the fact that the very thin elements used will invariably buckle before yielding. Thinner the plate, the lower will be the load at which the buckles will form.

5.3.1 Elastic buckling of thin plates

It has been shown in the chapter on “Introduction to Plate Buckling” that a flat plate simply supported on all edges and loaded in compression (as shown in Fig. 5.3(a)) will buckle at an elastic critical stress given by

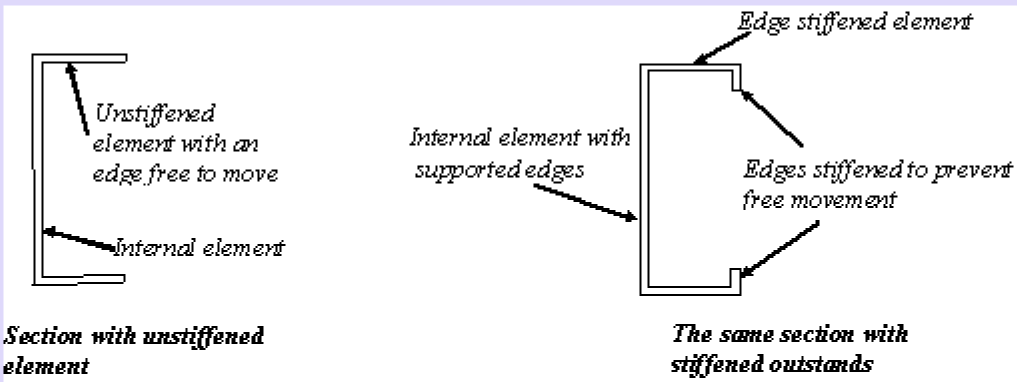
$$p_{cr} = \frac{K\pi^2 E}{12\left(1-v^2\right)}\left(\frac{t}{b}\right)^2 \qquad \textbf{(5.1)}$$



5.3 (a) Axially compressed plate simply supported on all edges

5.3 (b) Axially compressed plate with one edge supported and the other edge free to move

Substituting the values for π, v = 0.3 and E = 205 kN/mm^2, we obtain the value of p_{cr} as $p_{cr} \approx 185 \times 10^3 \times K \left(\frac{t}{b}\right)^2$ with units of N/ mm^2 ***(5.1a)***



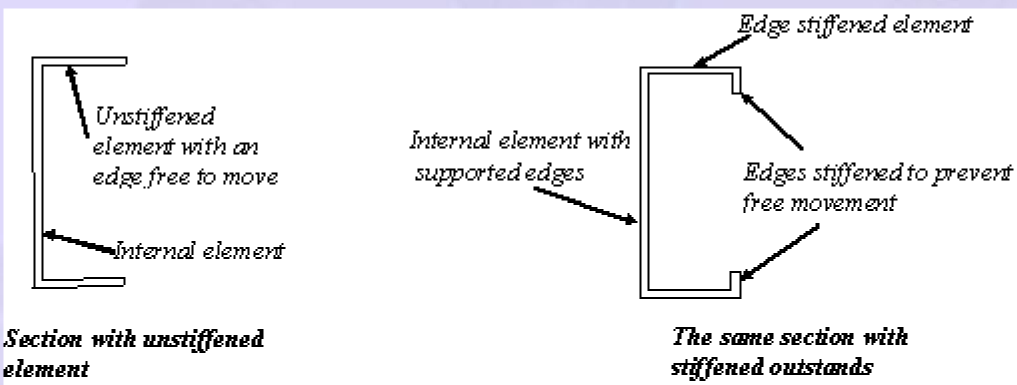


Fig.5.4 The technique of stiffening the element

The value of K is dependent on support conditions. When all the edges are simply supported K has a value of 4.0.

When one of the edges is free to move and the opposite edge is supported, (as shown in Fig. 5.3b), the plate buckles at a significantly lower load, as K reduces dramatically to 0.425. This shows that plates with free edges do not perform well under local buckling. To counter this difficulty when using cold formed sections, the free edges are provided with a lip so that they will be constrained to remain straight and will not be free to move. This concept of stiffening the elements is illustrated in Fig. 5.4.

5.3.2 Post - critical behaviour

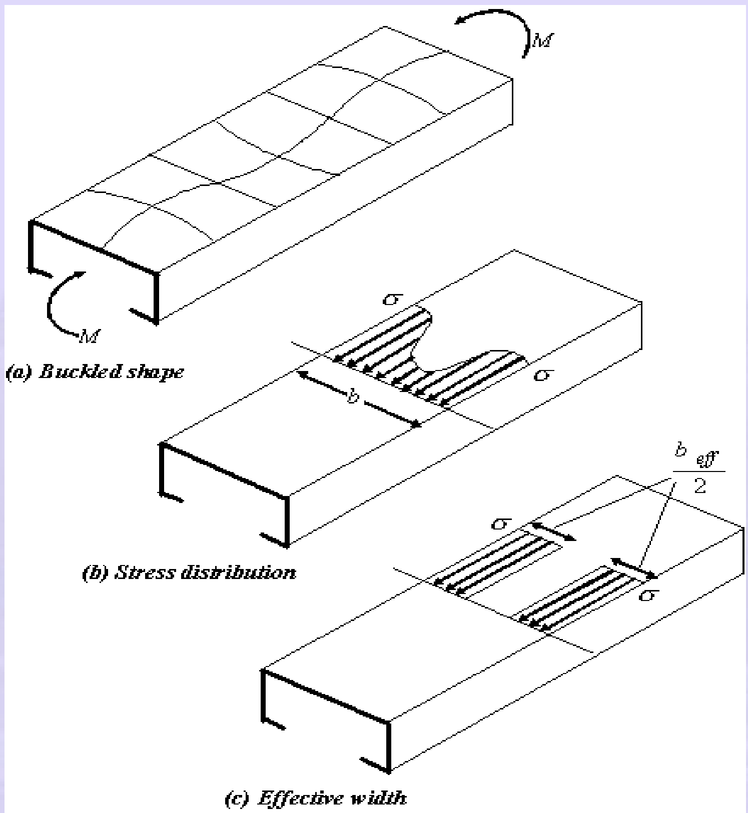


Fig. 5.5 Local buckling effects

Let us consider the channel subjected to a uniform bending by the application of moments at the ends. The thin plate at the top is under flexural compression and will buckle as shown in Fig. 5.5 (a). This type of buckling is characterised by ripples along the length of the element. The top plate is supported along the edges and its central portion, which is far from the supports, will deflect and shed the load to the stiffer edges. The regions near the edges are prevented from deflecting to the same extent. The stresses are non uniform across the section as shown in Fig.5.5 (b). It is obvious that the applied moment is largely resisted by regions near the edges (i.e. elements which carry increased stresses) while the regions near the centre are only lightly stressed and so are less effective in resisting the applied moment.

From a theoretical stand point, flat plates would buckle instantaneously at the elastic critical load. Under incremental loading, plate elements which are not perfectly flat will begin to deform out of plane from the beginning rather than instantaneously at the onset of buckling and fail at a lower load. This means that a non-uniform state of stress exists throughout the loading regime. The variation of mean stress with lateral deflection for flat plates and plates with initial imperfection, under loading are shown in Fig. 5.6.

This tendency is predominant in plates having b/t (breadth/thickness) ratios of 30-60. For plates having a b/t value in excess of 60, the in-plane tensile stresses or the "membrane stresses" (generated by the stretching of the plates) resist further buckling and cause an increase in the load-carrying capacity of wide plates.

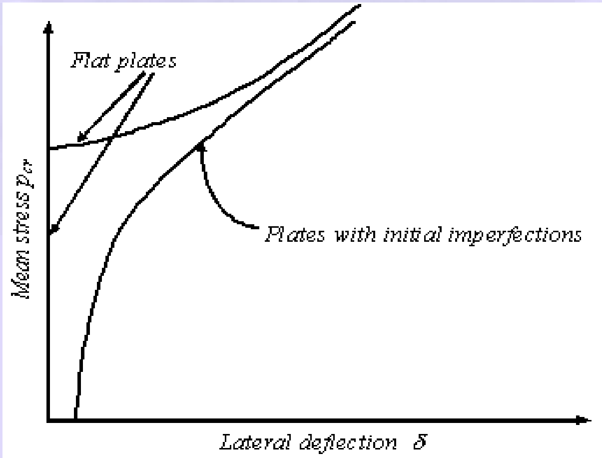


Fig. 5.6 Mean stress Vs lateral deflection relation

5.3.3 Effective width concept

The effects of local buckling can be evaluated by using the concept of ***effective width***. Lightly stressed regions at centre are ignored, as these are least effective in resisting the applied stresses. Regions near the supports are far more effective and are taken to be fully effective. The section behaviour is modeled on the basis of the effective width (b_{eff}) sketched in Fig. 5.5(c).

The effective width, (b_{eff}) multiplied by the edge stress (σ) is the same as the mean stress across the section multiplied by the total width (b) of the compression member.

The ***effective width*** of an element under compression is dependent on the magnitude of the applied stress f_c, the width/thickness ratio of the element and the edge support conditions.

5.3.4 Code provisions on "Local buckling of compressed plates"

The effective width concept is usually modified to take into account the effects of yielding and imperfection. For example, BS5950: Part 5 provides a semi-empirical formula for basic effective width, b_{eff}, to conform to extensive experimental data.

When $f_c > 0.123\ p_{cr}$, then

$$\frac{b_{eff}}{b} = \left[1 + 14\left\{\left[\frac{f_c}{p_{cr}}\right]^{0.5} - 0.35\right\}^4\right]^{-0.2} \qquad \textbf{\textit{(5.2a)}}$$

When $f_c < 0.123\ p_{cr}$, then $b_{eff} = b$ ***(5.2b)***

Where

f_c = compressive stress on the effective element, N/ mm^2

p_{cr} = local buckling stress given by

p_{cr} = 185,000 K (t / b)^2 N/ mm^2

K = load buckling coefficient which depends on the element type, section geometry etc.

t = thickness of the element, in mm

b = width of the element, in mm

The relationship given by eqn. 5.2(a) is plotted in Fig.5.7

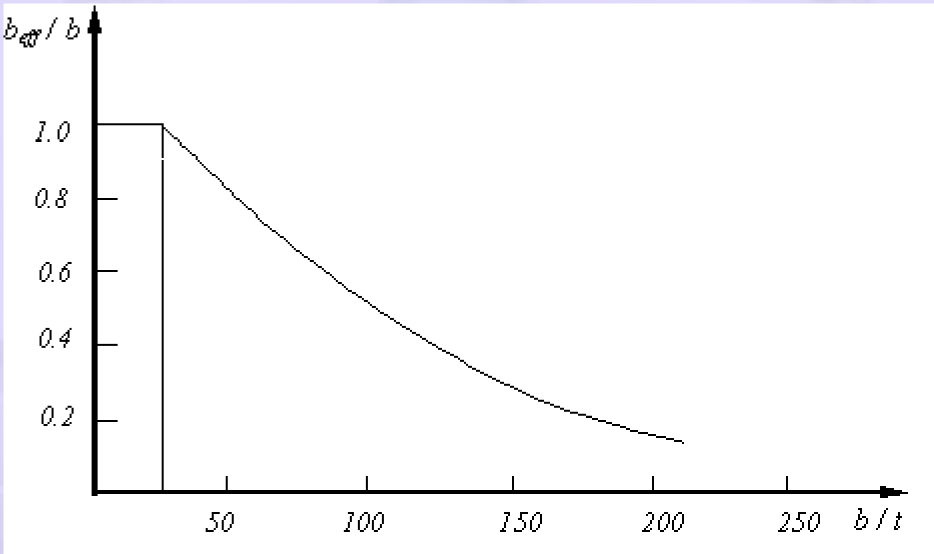


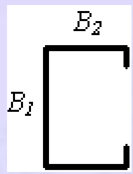
Fig.5.7 Ratio of effective width to flat width (f_y = 280 N/mm^2) of compression plate with simple edge supports

It is emphasised that in employing eqn. (5.2a), the value of K (to compute p_{cr}) could be 4.0 for a stiffened element or 0.425 for an unstiffened element.

BS5950, part 5 provides for a modification for an unstiffened element under uniform compression (Refer clause 4.5.1). The code also provides modifications for elements under combined bending and axial load (ref. Clause

4.5.2). Typical formula given in BS 5950, Part 5 for computing K values for a channel element is given below for illustration. (see BS 5950, Part 5 for a complete list of buckling coefficients).

1. Lipped channel.



The buckling coefficient K_1 for the member having a width of B_1 in a lipped channel of the type shown above is given by

$$K_1 = 7 - \frac{1.8h}{0.15 + h} - 1.43h^3 \qquad \textbf{\textit{(5.3a)}}$$

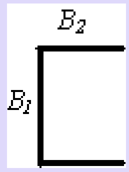
Where $h = B_2 / B_1$

For the member having the width of B_2 in the above sketch.

$$K_2 = K_2 h^2 \left(\frac{t_1}{t_2} \right)^2 \qquad \textbf{\textit{(5.3b)}}$$

Where t_1 and t_2 are the thicknesses of element width B_1 and B_2 respectively. (Note: normally t_1 and t_2 will be equal). The computed values of K_2 should not be less than 4.0 or 0.425 as the case may be.

2. Plain channel (without lips)



The buckling coefficient K_1 for the element of width B_1 is given by

$$K_1 = \frac{2}{\left(1+15h^3\right)^{0.5}} + \frac{2+4.8h}{\left(1+15h^3\right)} \qquad \textbf{\textit{(5.4)}}$$

K_2 is computed from eqn.. 5.3(b) given above.

5.3.4.1 Maximum width to thickness ratios

IS: 801 and BS 5950, Part 5 limit the maximum ratios of (b/t) for compression elements as follows:

- Stiffened elements with one longitudinal edge connected to a flange or web element and the other stiffened by a simple lip 60
- Stiffened elements with both longitudinal edges connected to other stiffened elements 500
- Unstiffened compression elements 60

However the code also warns against the elements developing very large deformations, when b/t values exceed half the values tabulated above.

5.3.5 Treatment of elements with stiffeners

5.3.5.1 Edge stiffeners

As stated previously, elements having b/t ? 60 and provided with simple lip having one fifth of the element width may be regarded as a stiffened element. If b/t > 60, then the width required for the lip may become too large and the lip itself may have stability problems. Special types of lips (called "compound" lips) are designed in such cases and these are outside the scope of this chapter.

5.3.5.2 Intermediate stiffeners

A wide and ineffective element may be transformed into a highly effective element by providing suitable intermediate stiffeners (having a minimum moment of inertia (I_{min}) about an axis through the element mid surface). The required minimum moment of inertia of the stiffener about the axis 0-0 in Fig. 5.8 is given by:

$$I_{min} = 0.2t^4 . \left(\frac{w}{t}\right)^2 . \left(\frac{f_y}{280}\right) \qquad \textbf{(5.5)}$$

Where w = larger flat width of the sub element (see Fig. 5.8) between stiffeners (in mm)

t = thickness of the element (mm)

f_y = yield stress (N/mm^2)

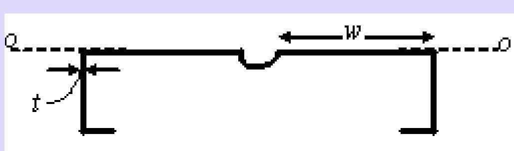


Fig.5.8 Intermediate stiffener

If the sub-element width/thickness ratio (w/t) does not exceed 60, the total effective area of the element may be obtained by adding effective areas of the sub-elements to the full areas of stiffeners.

When (w/t) is larger than 60, the effectiveness of the intermediately stiffened elements is somewhat reduced due to shear lag effects. (Refer to BS5950, Part 5, clauses 4.7.2 and 4.7.3) If an element has a number of stiffeners spaced closely (b/t ? 30), and then generally all the stiffeners and sub elements can be considered to be effective. To avoid introducing complexities at this stage, shear lag effects are not discussed here.

5.3.6 Effective section properties

In the analysis of member behaviour, the effective section properties are determined by using the effective widths of individual elements. As an example, let us consider the compression member ABCDEF shown in Fig.5.9. The effective portions of the member are shown darkened (i.e. 1-B, B-2, 3-C, C-4, 5-D, D-6, 7-E, and E-8). The parts A-1, 2-3, 4-5, 6-7 and 8-F are regarded as being ineffective in resisting compression. As a general rule, the portions located close to the supported edges are effective (see Fig.5.5c) . Note that in the case of compression members, all elements are subject to reductions in width.

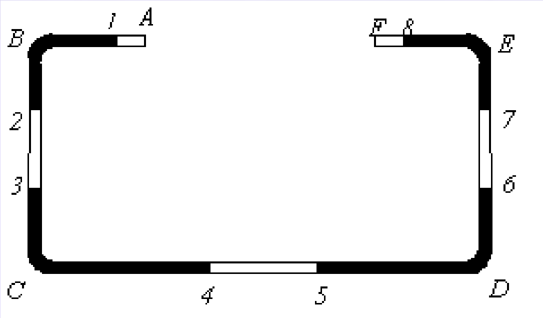


Fig. 5.9 Effective widths of compression elements

In the case of flexural members, in most cases, only the compression elements are considered to have effective widths. Some typical effective sections of beams are illustrated in Fig.5.10.

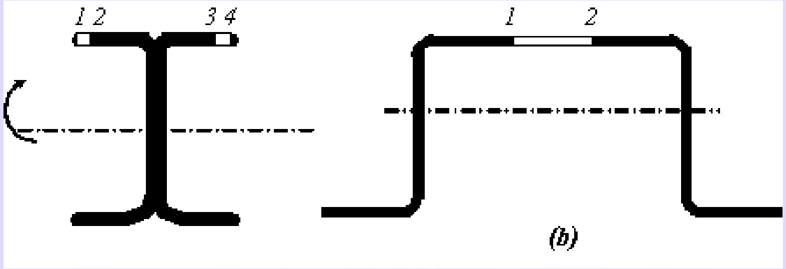


Fig. 5.10 Effective flexural sections

As in the previous example, fully effective sections in compression elements are darkened in Fig.5.10. The portions 1-2 and 3-4 in Fig. 5.10(a) and the portion 1-2 in Fig. 5.10 (b) are regarded as ineffective in resisting compression. Elements in tension are, of course, not subject to any reduction of width, as the full width will resist tension

5.3.7 Proportioning of stiffeners

The performance of unstiffened elements could be substantially improved by introducing stiffeners (such as a lip). Similarly very wide elements can be divided into two or more narrower sub elements by introducing intermediate stiffeners formed during the rolling process; the sum of the "effective widths" of individual sub elements will enhance the efficiency of the section.

According to BS 5950, Part 5 an unstiffened element (when provided with a lip) can be regarded as a stiffened element, when the lip or the edge stiffener has a moment of inertia about an axis through the plate middle surface equal to or greater than

$$I_{min} = \frac{b^3 t}{375} \qquad \textbf{\textit{(5.6)}}$$

Where t and b are the thickness and breadth of the full width of the element to be stiffened.

For elements having a full width b less than or equal to 60 t, a simple lip of one fifth of the element width (i.e. b/5) can be used safely. For lips with b > 60 t, it would be appropriate to design a lip to ensure that the lip itself does not develop instability.

A maximum b/t ratio of 90 is regarded as the upper limit for load bearing edge stiffeners.

The Indian standard IS: 801-1975 prescribes a minimum moment of inertia for the lip given by $I_{min} = 1.83t^4 \sqrt{\left(w/t\right)^2 - 281200/F_y}$ but not less than 9.2 t^4 .

Where I_{min} = minimum allowable moment of inertia of stiffener about its own centroidal axis parallel to the stiffened element in cm^4

w / t = flat width - thickness ratio of the stiffened element.

F_y = Yield stress in kgf/cm^2

For a simple lip bent at right angles to the stiffened element, the required overall depth d_{min} is given by $d_{min} = 2.8t \sqrt[6]{\left(w/t\right)^2 - 281200/F_y}$ but not less than 4.8 t

Note that both the above equations given by the Indian standard are dependent on the units employed.

5.3.7.1 Intermediate stiffeners.

Intermediate stiffeners are used to split a wide element into a series of narrower and therefore more effective elements. The minimum moment of inertia about an axis through the element middle surface required for this purpose (according to BS 5950, Part 5) is given in equation (5.5) above.

The effective widths of each sub element may be determined according to equation 5.2 (a) and eqn..5.2(b) by replacing the sub element width in place of the element width b.

When w / t < 60, then the total effective area of the element is obtained as the sum of the effective areas of each sub element to the full areas of stiffeners.

When the sub elements having a larger w / t values are employed (w/ t > 60), the performance of intermittently stiffened elements will be less efficient. To model this reduced performance , the sub element effective width must be reduced to b_{er} given by,

$$\frac{b_{er}}{t} = \frac{b_{eff}}{t} - 0.1\left(\frac{w}{t} - 60\right) \qquad \textbf{(5.7)}$$

The effective stiffener areas are also reduced when w / t > 90 by employing the equation:

$$A_{eff} = A_{st} \cdot \frac{b_{er}}{w} \qquad \textbf{(5.8)}$$

Where A_{st} = the full stiffener area and

A_{eff} = effective stiffener area.

For w / t values between 60 and 90, the effective stiffener area varies between A_{st} and A_{eff} as given below:

$$A_{eff} = A_{st}\left[3 - 2\frac{b_{er}}{w} - \frac{1}{30}\left(1 - \frac{b_{er}}{w}\right)\frac{w}{t}\right] \qquad \textbf{\textit{(5.9)}}$$

It must be noted that when small increases in the areas of intermediate stiffeners are provided, it is possible to obtain large increases in effectiveness and therefore it is advantageous to use a few intermediate stiffeners, so long as the complete element width does not exceed 500 t.

When stiffeners are closely spaced, i.e. w < 30 t, the stiffeners and sub elements may be considered to be fully effective. However there is a tendency for the complete element (along with the stiffeners) to buckle locally. In these circumstances, the complete element is replaced for purposes of analysis by an element of width b and having fictitious thickness t_s given by

$$t_s = \left(\frac{12\ I_s}{b}\right)^{1/3} \qquad \textbf{\textit{(5.10)}}$$

Where I_s = Moment of inertia of the complete element including stiffeneres, about its own neutral axis.

IS: 801- 1975 also suggests some simple rules for the design of intermediate stiffeners.

When the flanges of a flexural member is unusually wide, the width of flange projecting beyond the web is limited to

$$w_f = \sqrt{\frac{126500td}{f_{uv}}} \times \sqrt[4]{\frac{100\,c_f}{d}} \qquad \textbf{(5.10a)}$$

Where t = flange thickness

d = depth of beam

c_f = the amount of curling

f_{av} = average stress in kgf/cm^2 as specified in IS: 801 – 1975.

The amount of curling should be decided by the designer but will not generally exceed 5 % of the depth of the section.

Equivalent thickness of intermediate stiffener is given by

$$t_s = \sqrt[3]{\frac{12\ I_s}{w_s}} \qquad \textbf{(5.10b)}$$

5.4 Beams

As stated previously, the effect of local buckling should invariably be taken into account in thin walled members, using methods described already. Laterally stable beams are beams, which do not buckle laterally. Designs may be carried out using simple beam theory, making suitable modifications to take account of local buckling of the webs. This is done by imposing a maximum compressive stress, which may be considered to act on the bending element. The maximum value of the stress is given by

$$p_0 = \left[1.13 - 0.0019\frac{D}{t}\sqrt{\frac{f_y}{280}}\right] p_y \ngtr f_y \qquad \textbf{\textit{(5.11)}}$$

Where p_o = the limiting value of compressive stress in N/mm^2

D/t = web depth/thickness ratio

f_y = material yield stress in N/mm^2.

p_y = design strength in N/mm^2

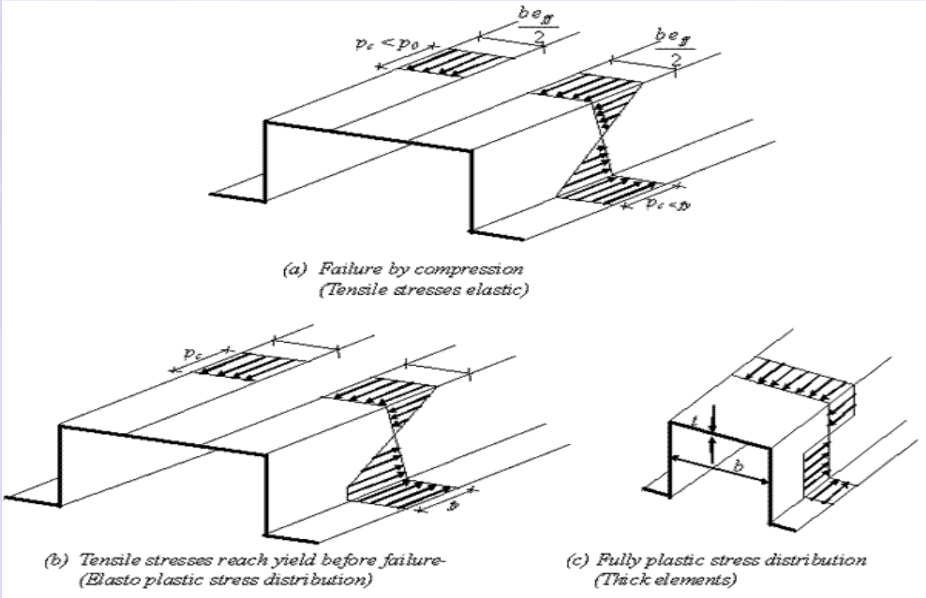


Fig.5.11 Laterally stable beams: Possible stress patterns

For steel with f_y = 280 N/mm^2, $p_0 = f_y$ when (D/t) ? 68.

For greater web slenderness values, local web buckling has a detrimental effect. The moment capacity of the cross section is determined by limiting the maximum stress on the web to p_0. The effective width of the compression element is evaluated using this stress and the effective section properties are evaluated. The ultimate moment capacity (M_{ult}) is given by

$$M_{ult} = Z_c.p_0 \qquad \textbf{\textit{(5.11a)}}$$

Where Z_c = effective compression section modulus

This is subject to the condition that the maximum tensile stress in the section does not exceed f_y (see Fig.5.11a).

If the neutral axis is such that the tensile stresses reach yield first, then the moment capacity is to be evaluated on the basis of elasto-plastic stress distribution (see Fig.5.11b). In elements having low (width/thickness) ratios, compressive stress at collapse can equal yield stress (see Fig. 5.11c). In order to ensure yielding before local buckling, the maximum (width/thickness) ratio of stiffened elements is $\leq 25\sqrt{\frac{280}{f_y}}$ and for unstiffened elements, it is $\leq 8\sqrt{\frac{280}{f_y}}$

5.4.1 Other beam failure criteria

5.4.1.1 Web crushing

This may occur under concentrated loads or at support point when deep slender webs are employed. A widely used method of overcoming web crushing problems is to use web cleats at support points (See Fig.5.12).

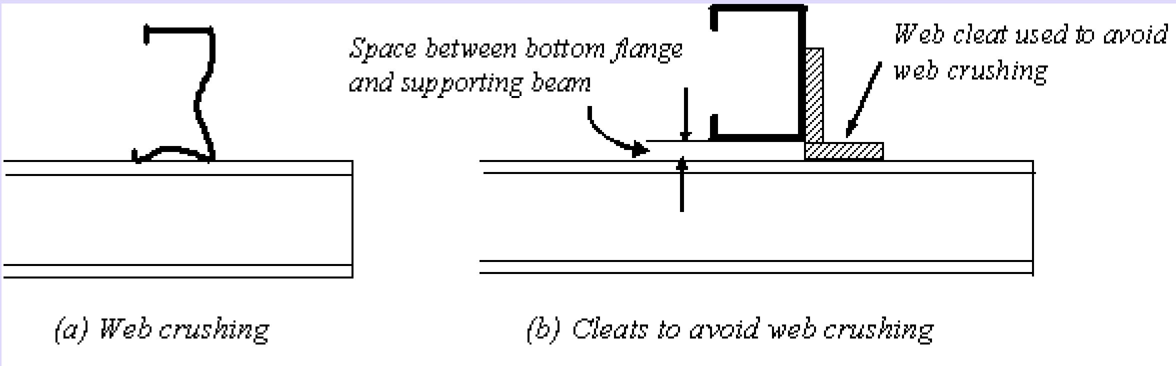


Fig.5.12 Web crushing and how to avoid it

5.4.1.2 Shear buckling

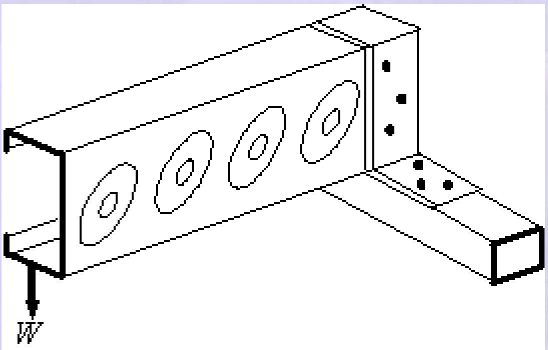


Fig. 5.13 Web buckling

The phenomenon of shear buckling of thin webs has been discussed in detail in the chapter on "Plate Girders". Thin webs subjected to predominant shear will buckle as shown in Fig.5.13. The maximum shear in a beam web is invariably limited to 0.7 times yield stress in shear. In addition in deep webs, where shear buckling can occur, the average shear stress (p_v) must be less than the value calculated as follows:

$$p_v \leq \left(\frac{1000\,t}{D}\right)^2 \qquad \textit{(5.12)}$$

Where p = average shear stress in N/mm^2.

t and D are the web thickness and depth respectively (in mm)

5.4.2 Lateral buckling

The great majority of cold formed beams are (by design) restrained against lateral deflections. This is achieved by connecting them to adjacent elements, roof sheeting or to bracing members. However, there are circumstances where this is not the case and the possibility of lateral buckling has to be considered.

Lateral buckling will not occur if the beam under loading bends only about the minor axis. If the beam is provided with lateral restraints, capable of resisting a lateral force of 3% of the maximum force in the compression flange, the beam may be regarded as restrained and no lateral buckling will occur.

As described in the chapter on "Unrestrained Beams'", lateral buckling occurs only in "long" beams and is characterised by the beam moving laterally and twisting when a transverse load is applied. This type of buckling is of importance for long beams with low lateral stiffness and low torsional stiffness (See Fig.5.14); such beams under loading will bend about the major axis.

The design approach is based on the "effective length" of the beam for lateral buckling, which is dependent on support and loading conditions. The effective length of beams with both ends supported and having restraints against twisting is taken as 0.9 times the length, provided the load is applied at bottom flange level. If a load is applied to the top flange which is unrestrained laterally, the effective length is increased by 20%. This is considered to be a "destabilising load", i.e. a load that encourages lateral instability.

The elastic lateral buckling moment capacity is determined next. For an I section or symmetrical channel section bent in the plane of the web and loaded through shear centre, this is

$$M_E = \frac{\pi^2 A.E.D}{2\left(l_e / r_y\right)^2} . C_b \sqrt{1 + \frac{1}{20}\left(\frac{l_e}{r_y} . \frac{t}{D}\right)^2} \qquad \textbf{\textit{(5.13)}}$$

Where,

A = cross sectional area, in mm^2

D = web depth, in mm

t = web thickness, in mm

r_y = radius of gyration for the lateral bending of section

C_b = 1.75 - 1.05 β + 0.3 β^2 x 2.3.

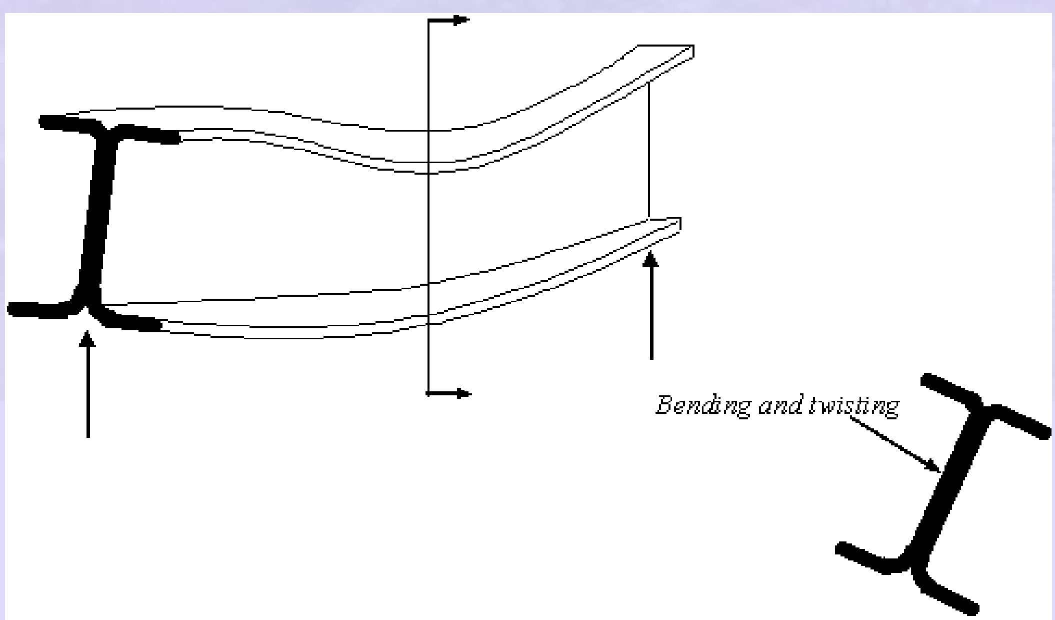


Fig.5.14 Lateral buckling

Where β = ratio of the smaller end moment to the larger end moment M in an unbraced length of beam. β is taken positive for single curvature bending and negative for double curvature (see Fig. 5.15)

To provide for the effects of imperfections, the bending capacity in the plane of loading and other effects, the value of M_E obtained from eq. (5.13) will need to be modified. The basic concept used is explained in the chapter on Column Buckling where the failure load of a column is obtained by employing the Perry-Robertson equation for evaluating the collapse load of a column from a knowledge of the yield load and Euler buckling load.

M_E = Elastic lateral buckling resistance moment given by equation (5.13)

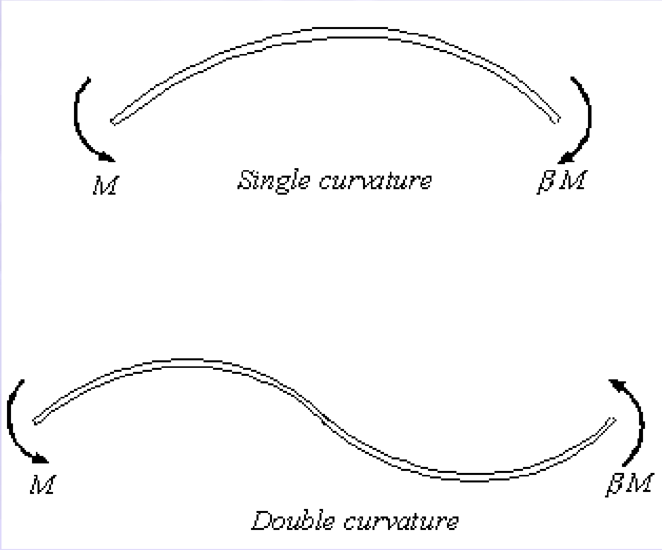


Fig. 5.15 Single and double curvature bending

A similar Perry-Robertson type equation is employed for evaluating the Moment Resistance of the beam

$$M_b = \frac{1}{2}\left[\left\{M_y + (1+\eta)M_E\right\} - \sqrt{\left[M_y + (1+\eta)M_E\right]^2 - 4M_y.M_E}\right] \qquad (5.14)$$

M_y = First yield moment given by the product of yield stress (f_y) and the Elastic Modulus (Z_c) of the gross section.

η = Perry coefficient, given by

$$\text{When } \frac{l_e}{r_y} < 40C_b, \ \eta = 0.$$

$$\text{When } \frac{l_e}{r_y} < 40C_b, \ \eta = 0.002\left(\frac{l_e}{r_y} - 40C_b\right)$$

l_e = effective length

r_y = radius of gyration of the section about the y - axis.

When the calculated value of M_b exceed M_{ult} calculated by using equation (5.11.a), then M_b is limited to M_{ult}. This will happen when the beams are "short".

5.5 Axially compressed column

As pointed out in the last chapter, local buckling under compressive loading is an extremely important feature of thin walled sections. It has been shown that a compressed plate element with an edge free to deflect does not perform as satisfactorily when compared with a similar element supported along the two opposite edges. Methods of evaluating the effective widths for both edge support conditions were presented and discussed.

In analysing column behaviour, the first step is to determine the effective area (A_{eff}) of the cross section by summing up the total values of effective areas for all the individual elements.

The ultimate load (or squash load) of a short strut is obtained from

$$P_{cs} = A_{eff} \,.\, f_{yd} = Q.\, A.\, f_{yd} \qquad \textbf{\textit{(5.15)}}$$

Where

P_{cs} = ultimate load of a short strut

A_{eff} = sum of the effective areas of all the individual plate elements

Q = the ratio of the effective area to the total area of cross section at yield stress

In a long column with doubly - symmetric cross section, the failure load (P_c) is dependent on Euler buckling resistance (P_{EY}) and the imperfections present. The method of analysis presented here follows the Perry-Robertson approach presented in the chapter on "Introduction to Column Buckling". Following that approach, the failure load is evaluated from

$$P_c = \frac{1}{2}\left\{\left[P_{cs} + (1+\eta)P_{Ey}\right] - \sqrt{\left[P_{cs} + (1+\eta)P_{Ey}\right]^2 - 4P_{cs}.P_{Ey}}\right\}$$

$$\text{Where } \eta = 0.002\left(\frac{l_e}{r_y} - 20\right), \quad \text{for } \frac{l_e}{r_y} > 20 \qquad \textbf{\textit{(5.16)}}$$

$$\eta = 0, \quad \text{for } \frac{l_e}{r_y} > 20$$

P_{EY} = the minimum buckling load of column = $\dfrac{\pi^2 E\, I_{min}}{l_e^{\,2}}$

and r_y = radius of gyration corresponding to P_{EY}.

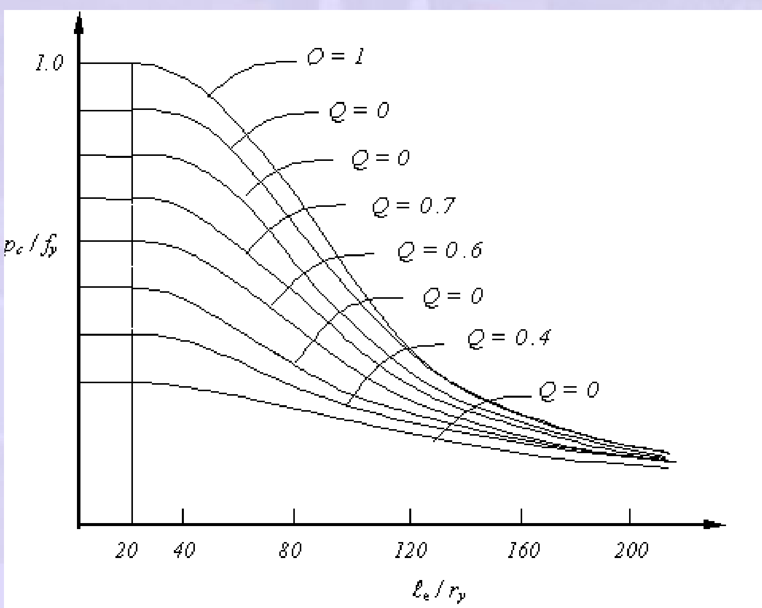


Fig.5.16 Column Strength (non- dimensional) for different Q factors

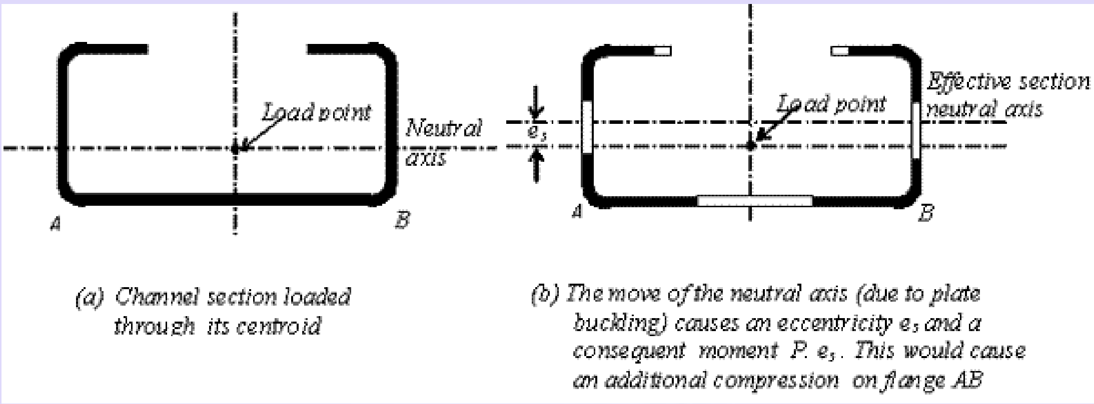


Fig.5.17 Effective shift in the loading axis in an axially compressed column

Fig. 5.16 shows the mean stress at failure ($p_c = P_c$ / cross sectional area) obtained for columns with variation of l_e /r_y for a number of "Q" factors. (The y-axis is non dimensionalised using the yield stress, f_y and "Q" factor is the ratio of effective cross sectional area to full cross sectional area). Plots such as Fig.7.16 can be employed directly for doubly symmetric sections.

5.5.1 Effective shift of loading axis

If a section is not doubly symmetric (see Fig. 5.17) and has a large reduction of effective widths of elements, then the effective section may be changed position of centroid. This would induce bending on an initially concentrically loaded section, as shown in Fig.5.17. To allow for this behaviour, the movement of effective neutral axis (e_s) from the geometric neutral axis of the cross section must be first determined by comparing the gross and effective section properties. The ultimate load is evaluated by allowing for the interaction of bending and compression using the following equation:

$$P_{ult} = \frac{P_c . M_c}{M_c + P_c . e_5} \qquad \textbf{\textit{(5.17)}}$$

Where P_c is obtained from equation (5.16) and M_c is the bending resistance of the section for moments acting in the direction corresponding to the movement of neutral axis; e_s is the distance between the effective centroid and actual centroid of the cross section.

5.5.2 Torsional - flexural buckling

Singly symmetric columns may fail either (a) by Euler buckling about an axis perpendicular to the line of symmetry (as detailed in 5.5.1 above) or (b) by a combination of bending about the axis of symmetry and a twist as shown in Fig.5.18. This latter type of behaviour is known as Torsional-flexural behaviour. Purely torsional and purely flexural failure does not occur in a general case.

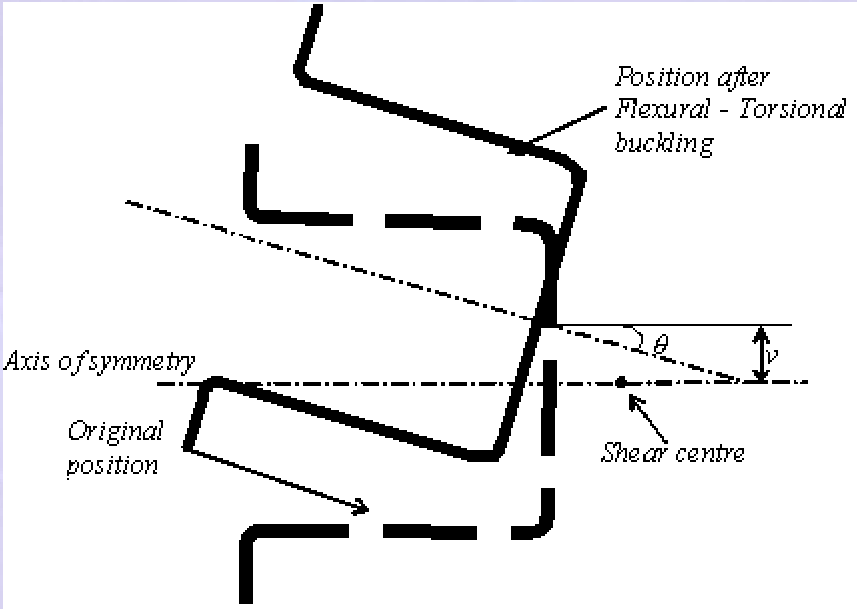


Fig.5.18 Column displacements during Flexural - Torsional buckling

Theoretical methods for the analysis of this problem was described in the chapters on Beam Columns. Analysis of torsional-flexural behaviour of cold formed sections is tedious and time consuming for practical design. Codes deal with this problem by simplified design methods or by empirical methods based on experimental data.

As an illustration, the following design procedure, suggested in BS5950, Part 5 is detailed below as being suitable for sections with at least one axis of symmetry (say x - axis) and subjected to flexural torsional buckling.

Effective length multiplication factors (known as α factors) are tabulated for a number of section geometries. These α factors are employed to obtain increased effective lengths, which together with the design analysis prescribed in 5.5.1 above can be used to obtain torsional buckling resistance of a column.

$$\begin{aligned} &\text{For} \quad P_{EY} \le P_{TF}, \quad \alpha = 1 \\ &\text{For} \quad P_{EY} > P_{TF}, \quad \alpha = \sqrt{\frac{P_{EY}}{P_{TF}}} \end{aligned} \qquad \textbf{\textit{(5.18)}}$$

α Values can be computed as follows:

Where P_{EY} is the elastic flexural buckling load (in Newton's) for a column about the y- axis, i.e.

$$\frac{\pi^2 EI_y}{l_e^{\,2}}$$

le = effective length (in mm) corresponding to the minimum radius of gyration

P_{TF} = torsional flexural buckling load (in Newtons) of a column given by

$$P_{TF} = \frac{1}{2\beta}\left[\left(P_{EX} + P_T\right) - \left\{\left(P_{EX} + P_T\right)^2 - 4\beta P_{EX} P_T\right\}^{1/2}\right] \qquad \textbf{\textit{(5.19)}}$$

where P_{EX} = Elastic flexural buckling load of the column (in Newton's) about the x- axis given by

$$\frac{\pi^2 EI_y}{l_e{}^2}$$

P_T = Torsional buckling load of a column (In Newton's) given by

$$P_T = \frac{1}{r_0^2}\left(GJ + \frac{2\pi^2.ET}{l_e^2}\right) \qquad \textbf{\textit{(5.20)}}$$

β is a constant given by $\beta = 1 - \left(\frac{x_0}{r_0}\right)$ ***(5.21)***

In these equations,

r_o = polar radius of gyration about the shear centre (in mm) given by

$$r_0 = \left(r^2{}_x + r^2{}_y + x^2{}_0\right)^{1/2} \qquad \textbf{\textit{(5.22)}}$$

Where

r_x, r_y are the radii of gyration (in mm) about the x and y- axis

G is the shear modulus (N/mm^2)

x_0 is the distance from shear centre to the centroid measured along the x axis (mm)

J St Venants' Torsion constant (mm^4) which may be taken as $\sum \frac{bt^3}{3}$ summed up for all elements,

Where b = flat width of the element and t = thickness (both of them measure in mm)

I_x the moment of inertia about the x axis (mm^4)

Γ Warping constant for all section.

5.5.3 Torsion behaviour

Cold formed sections are mainly formed with "open" sections and do not have high resistance to torsion. Hence the application of load which would cause torsion should be avoided where possible. Generally speaking, by adjusting the method of load application, it is possible to restrain twisting so that torsion does not occur to any significant extent.

In general, when examining torsional behaviour of thin walled sections, the total torsion may be regarded as being made up of two effects:

- St. Venant's Torsion or Pure Torsion
- Warping torsion.

St.Venant's torsion produces shear stresses, which vary linearly through the material thickness. Warping torsion produces in-plane bending of the elements of a cross section, thus inducing direct (i.e. normal) stresses and the angle of twist increases linearly.

Since cold formed sections are thin walled, they have very little resistance to St. Venant's Torsion and will twist substantially. The extent of warping torsion in a thin walled beam is very much dependent on the warping restraint afforded by the supports as well as the loading conditions and the type of section.

If the beam ends are restrained from warping, then short beams exhibit high resistance to warping torsion and the total torque acting on such a beam will be almost completely devoted to overcoming warping resistance, the St Venant's Torsion being negligible. Conversely, the resistance to warping torsion becomes low for long beams and warping stresses and degrees of twist become very large.

A detailed theoretical treatment of beams subject to bending and torsion is given in another chapter. As stated previously, particular care and attention should be paid to the detailing of the connections and the method of load application so that the design for torsion does not pose a serious problem.

5.6 Combined bending and compression

Compression members which are also subject to bending will have to be designed to take into account the effects of interaction. The following checks are suggested for members which have at least one axis of symmetry: (i) the local capacity at points of greatest bending moment and axial load and (ii) an overall buckling check.

5.6.1 Local Capacity Check

The local capacity check is ascertained by satisfying the following at the points of greatest bending moment and axial load:

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1 \qquad \textbf{(5.23)}$$

F_c = applied axial load

P_{cs} = short strut capacity defined by $A_{eff}.P_{yd}$ (eqn.7.15)

M_x, M_y = applied bending moments about x and y axis

M_{cx} = Moment resistance of the beam about x axis in the absence of F_c and M_y

M_{cy} = Moment resistance of the beam about y axis in the absence of F_c and M_x.

5.6.2 Overall buckling check

For members not subject to lateral buckling, the following relationship should be satisfied:

$$\frac{F_c}{P_c} + \frac{M_x}{C_{bx}.M_{cx}\left(1-\frac{F_c}{P_{EX}}\right)} + \frac{M_y}{C_{by}.M_{cy}\left(1-\frac{F_c}{P_{EY}}\right)} \leq 1 \qquad \textbf{(5.24)}$$

For beams subject to lateral buckling, the following relationship should be satisfied:

$$\frac{F_c}{P_c} + \frac{M_x}{M_b} + \frac{M_y}{C_{by}.M_{cy}\left(1 - \frac{F_c}{P_{EY}}\right)} \leq 1 \qquad \textit{(5.25)}$$

Where

P_c = axial buckling resistance in the absence of moments (see eq. 5.16)

P_{EX}, P_{EY} = flexural buckling load in compression for bending about the x- axis and for bending about the y-axis respectively.

C_{bx}, C_{by} = C_b factors (defined in the previous chapter) with regard to moment variation about x and y axis respectively.

M_b = lateral buckling resistance moment about the x axis defined in the previous chapter.

5.7 Tension members

If a member is connected in such a way as to eliminate any moments due to connection eccentricity, the member may be designed as a simple tension member. Where a member is connected eccentrically to its axis, then the resulting moment has to be allowed for.

The tensile capacity of a member (P_t) may be evaluated from

$$P_t = A_e \cdot P_y \qquad (5.26)$$

Where

A_e is the effective area of the section making due allowance for the type of member (angle, plain channel, Tee section etc) and the type of connection (eg. connected through one leg only or through the flange or web of a T- section).

p_y is design strength (N/mm^2)

Guidance on calculation of A_e is provided in Codes of Practice (eg. BS 5950, Part 5). The area of the tension member should invariably be calculated as its gross area less deductions for holes or openings. (The area to be deducted from the gross sectional area of a member should be the maximum sum of the sectional areas of the holes in any cross section at right angles to the direction of applied stress).

Reference is also made to the chapter on "Tension Members" where provision for enhancement of strength due to strain hardening has been incorporated for hot rolled steel sections. The Indian code IS: 801-1975 is in the process of revision and it is probable that a similar enhancement will be allowed for cold rolled steel sections also.

When a member is subjected to both combined bending and axial tension, the capacity of the member should be ascertained from the following:

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1 \qquad \textbf{\textit{(5.27)}}$$

and

$$\frac{M_x}{M_{cx}} \leq 1 \qquad \textbf{\textit{(5.28)}}$$

and

$$\frac{M_y}{M_{cy}} \leq 1 \qquad \textbf{\textit{(5.29)}}$$

Where F_t = applied load

P_t = tensile capacity (see eqn. 5.12)

M_x , M_y , M_{cx} and M_{cy} are as defined previously.

5.8 Design on the basis of testing

While it is possible to design many cold formed steel members on the basis of analysis, the very large variety of shapes that can be formed and the complex interactions that occur make it frequently uneconomical to design members and systems completely on theoretical basis. The behaviour of a component or system can often be ascertained economically by a test and suitable modifications incorporated, where necessary.

Particular care should be taken while testing components, that the tests model the actual loading conditions as closely as possible. For example, while these tests may be used successfully to assess the material work hardening much caution will be needed when examining the effects of local buckling. There is a possibility of these tests giving misleading information or even no information regarding neutral axis movement. The specimen lengths may be too short to pick up certain types of buckling behaviour.

Testing is probably the only realistic method of assessing the strength and characteristics of connections. Evaluating connection behaviour is important as connections play a crucial role in the strength and stiffness of a structure.

In testing complete structures or assemblies, it is vital to ensure that the test set up reflects the in-service conditions as accurately as possible. The method of load application, the type of supports, the restraints from adjacent structures and the flexibility of connections are all factors to be considered carefully and modeled accurately.

Testing by an independent agency (such as Universities) is widely used by manufacturers of mass produced components to ensure consistency of quality. The manufacturers also provide load/span tables for their products, which can be employed by structural designers and architects who do not have detailed knowledge of design procedures. An advantage to the manufacturers in designing on the basis of proof testing is that the load/span tables obtained are generally more advantageous than those obtained by analytical methods; they also reassure the customers about the validity of their load/span tables.

5.9 Empirical methods

Some commonly used members such as Z purlins are sometimes designed by time-tested empirical rules; such rules are employed when theoretical analysis may be impractical or not justified and when prototype tests data are not available. (Members designed by proven theoretical methods or by prototype testing need not comply with the empirical rules). As an illustration the empirical rules permitted by BS 5950, Part 5 is explained below.

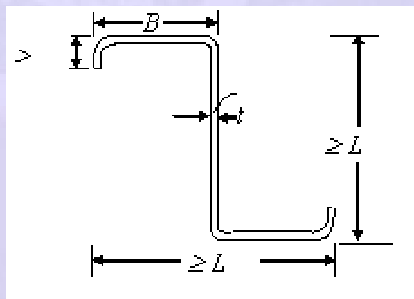


Fig.5.19 Z Purlins

5.9.1 Z Purlins

A Z purlin used for supporting the roofing sheet is sketched in Fig. 5.19. In designing Z purlins with lips using the simplified empirical rules the following recommendations are to be complied with:

- Unfactored loads should be used for designing purlins

- Imposed loads should be taken to be at least 0.6 kN /mm^2

- Claddings and fixings should be checked for adequacy to provide lateral restraint to the purlin and should be capable of carrying the component of load in the plane of the roof slope.

• The purlin should be considered to carry the load normal to roof slope (and a nominal axial load due to wind or restraint forces)

• These rules apply to purlins up to 8 m span in roof slopes up to $22\frac{1}{2}$

• Antisag bars should be provided to ensure that laterally unsupported length of the purlin does not exceed 3.8 m. These should be anchored to rigid apex support or their forces should be transferred diagonally to main frames.

• Purlin cleats should provide adequate torsional restraint.

5.9.2 Design rules

The following design rules apply with reference to Fig. 5.19

• The overall depth should be greater than 100 t and not less than L /45.

• Overall width of compression flange / thickness ratio should be less than 35.

• Lip width should be greater than B /5

• Section Modulus $\geq \frac{WL}{1400}\text{cm}^3$ for simply supported purlins and $\geq \frac{WL}{1400}\text{cm}^3$ for continuous or semi rigidly jointed purlins.

In the above,

L = span of the purlin (in mm)

W = Normal component of unfactored (distributed dead load + imposed load) in kN

B = Width of the compression flange in mm

T = thickness of the purlin in mm.

• The net allowable wind uplift in a direction normal to roof when purlins are restrained is taken as 50% of the (dead + imposed) load.

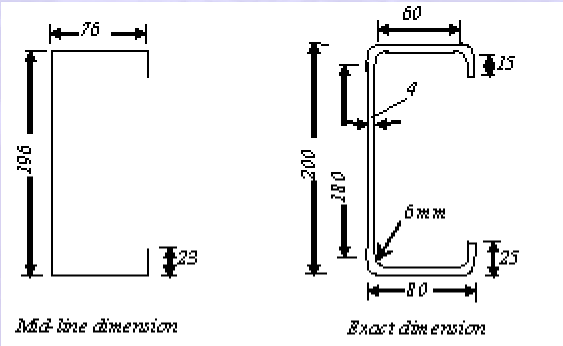
5.10 Examples

5.10.1 Analysis of effective section under compression

To illustrate the evaluation of reduced section properties of a section under axial compression.

Section: 200 x 80 x 25 x 4.0 mm

Using mid-line dimensions for simplicity. Internal radius of the corners is 1.5t.



Effective breadth of web (flat element)

h = B_2 / B_1 = 60 / 180 = 0.33

$$K_1 = 7 - \frac{1.8h}{0.15 + h} - 1.43h^3$$
$$= 7 - \frac{1.8 \,x\, 0.33}{0.15 + 0.33} - 1.43 \,x\, 0.33^3$$

= 5.71 or 4 (minimum) = 5.71

p_{cr} = 185000 K_1 (t / b)^2

= 185000 x 5.71 x (4 / 180)^2 = 521.7 N / mm^2

$$\frac{f_{cr}}{p_{cr} \,x\, \gamma_m} = \frac{240}{521.7 \,x\, 1.15} = 0.4 > 0.123$$

$$\frac{b_{eff}}{b} = \left[1 + 14\left\{\sqrt{f_{cr} / \left(p_{cr}\ x\ \gamma_m\right)} - 0.35\right\}^4\right]^{-0.2}$$

$$= \left[1 + 14\left\{\sqrt{0.4} - 0.35\right\}^4\right]^{-0.2} \qquad = 0.983$$

or b_{eff} = 0.983 x 180 = 176.94 mm

Effective width of flanges (flat element)

$K_2 = K_1\ h^2\ (\ t_1 / t_2\ )^2$

$= K_1\ h^2\ (t_1 = t_2\ )$

$= 5.71 \times 0.33^2 = 0.633$ or 4 (minimum) = 4

$p_{cr} = 185000 \times 4 \times (\ 4 / 60\ )^2 = 3289\ N/mm^2$

$$\frac{f_c}{p_{cr}\ x\ \gamma_m} = \frac{240}{3289\ x\ 1.15} = 0.063 > 0.123$$

$$\therefore \quad \frac{b_{eff}}{b} = 1 \qquad b_{eff} = 60\ mm$$

Effective width of lips (flat element)

K = 0.425 (conservative for unstiffened elements)

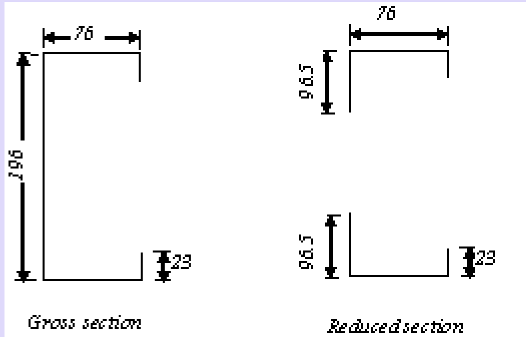
$p_{cr} = 185000 \times 0.425 \times (\ 4 / 15\ )^2 = 5591\ N/mm^2$

$$\frac{f_c}{p_{cr}\ x\ \gamma_m} = \frac{240}{5591\ x\ 1.15} = 0.04 > 0.123$$

$$\therefore \quad \frac{b_{eff}}{b} = 1 \qquad b_{eff} = 15\ mm$$

Effective section in mid-line dimension

As the corners are fully effective, they may be included into the effective width of the flat elements to establish the effective section.



The calculation for the area of gross section is tabulated below:

	A_i (mm^2)
Lips	2 x 23 x 4 = 184
Flanges	2 x 76 x 4 = 608
Web	196 x 4 = 784
Total	1576

The area of the gross section, A = 1576 mm^2

The calculation of the area of the reduced section is tabulated below:

	A_i (mm^2)
Lips	2 x 15 x 4 = 120
Corners	4 x 45.6 = 182.4
Flanges	2 x 60 x 4 = 480
Web	176.94 x 4 = 707.8
Total	1490.2

The area of the effective section, A_{eff} = 1490.2 mm^2

Therefore, the factor defining the effectiveness of the section under compression,

$$Q = \frac{A_{eff}}{A} = \frac{1490}{1576} = 0.95$$

The compressive strength of the member = Q A f_y / γ_m

= 0.95 x 1576 x 240 / 1.15

= 313 kN

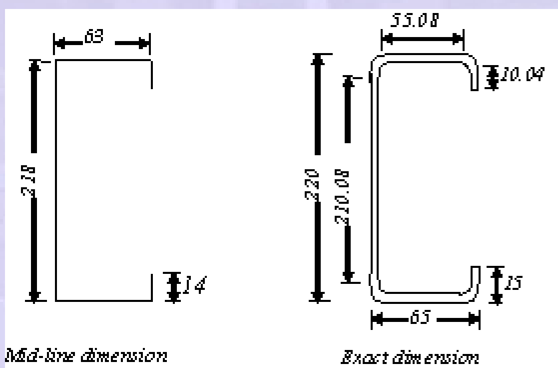
5.10.2 Analysis of effective section under bending

To illustrate the evaluation of the effective section modulus of a section in bending.

We use section: 220 x 65 x 2.0 mm Z28 Generic lipped Channel (from "Building Design using Cold Formed Steel Sections", Worked Examples to BS 5950: Part 5, SCI PUBLICATION P125)

Only the compression flange is subject to local buckling.

Using mid-line dimensions for simplicity. Internal radius of the corners is 1.5t.



Thickness of steel (ignoring galvanizing), t = 2 - 0.04 = 1.96 mm

Internal radius of the corners = 1.5 x 2 = 3 mm

Limiting stress for stiffened web in bending

$$p_0 = \left\{1.13 - 0.0019\frac{D}{t}\sqrt{\frac{f_y}{280}}\right\}p_y$$

and p_y = 280 / 1.15 = 243.5 N / mm^2

$$p_0 = \left\{1.13 - 0.0019 \text{ x } \frac{220}{1.96}\sqrt{\frac{280}{280}}\right\}\frac{280}{1.15}$$

= 223.2 N / mm^2

Which is equal to the maximum stress in the compression flange, i.e.,

f_c = 223.2 N / mm^2

Effective width of compression flange

h = B_2 / B_1 = 210.08 / 55.08 = 3.8

$$K_1 = 5.4 - \frac{1.4h}{0.6+h} - 0.02h^3$$

$$= 5.4 - \frac{1.4 \text{ x } 3.8}{0.6 + 3.8} - 0.02 \text{ x } 3.8^3$$

= 3.08 or 4 (minimum) = 4

$$p_{cr} = 185000 \text{ x } 4 \text{ x} \left(\frac{1.96}{55.08}\right)^2 = 937 \text{ N/mm}^2$$

$$\frac{f_c}{p_{cr}} = \frac{223.2}{937} = 0.24 > 0.123$$

$$\frac{b_{eff}}{b} = \left[1 + 14\left\{\sqrt{f_c / p_{cr}} - 0.35\right\}^4\right]^{-0.2}$$

$$= \left[1 + 14\left\{\sqrt{0.24} - 0.35\right\}^4\right]^{-0.2} = 0.998$$

b_{eff} = 0.99 x 55 = 54.5

Effective section in mid-line dimension:

The equivalent length of the corners is 2.0 x 2.0 = 4 mm

The effective width of the compression flange = 54.5 + 2 x 4 = 62.5

The calculation of the effective section modulus is tabulated as below:

Elements	A_i (mm^2)	y_i (mm)	A_i y_i (mm^3)	I_g + A_i y_i^2 (mm^4)
Top lip	27.44	102	2799	448 + 285498
Compression flange	122.5	109	13352.5	39.2 + 1455422.5
Web	427.3	0	0	39.2 + 1455422.5
Tension flange	123.5	-109	-13459.3	39.5 + 1467064
Bottom lip	27.4	-102	-2799	448 + 285498
Total	728.2		-106.8	5186628.4

The vertical shift of the neutral axis is

$$\overline{y} = \frac{-106.8}{728.2} = -0.15\,\text{mm}$$

The second moment of area of the effective section is

I_{xr} = (5186628.4 + 728.2 x 0.15^2) x 10^{-4}

= 518.7 cm^4 at p_0 = 223.2 N / mm^2

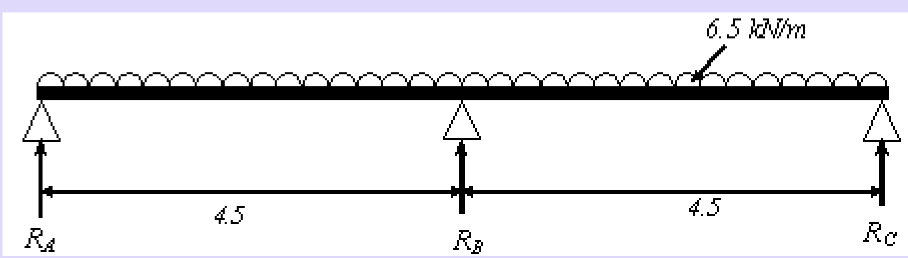
or = $518.7 \text{ x } \dfrac{223.2 \text{ x } 1.15}{280} = 475.5\,\text{cm}^4$ at p_y = 280 / 1.15 N / mm^2

The effective section modulus is,

$$Z_{xr} = \frac{475.5}{(109 + 0.15)/10} = 43.56\,\text{cm}^3$$

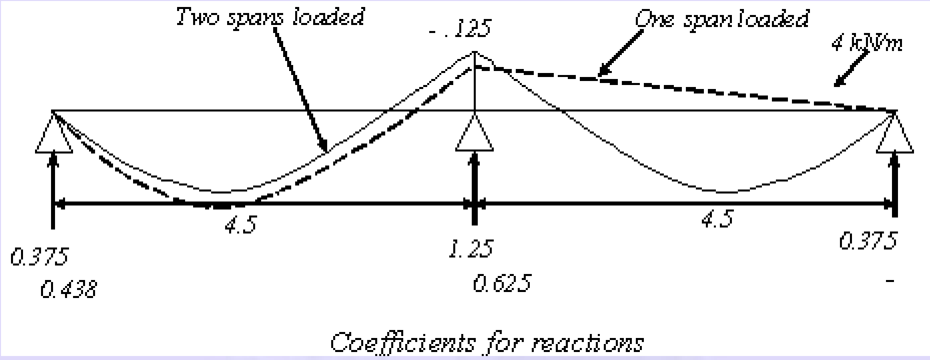
5.10.3 Two span design

Design a two span continuous beam of span 4.5 m subject to a UDL of 4kN/m as shown in Fig.1.



Factored load on each span = 6.5 x 4.5 = 29.3 kN

Bending Moment



Maximum hogging moment = 0.125 x 29.3 x 4.5 = 16.5 kNm

Maximum sagging moment = 0.096 x 29.3 x 4.5 = 12.7 kNm

Shear Force

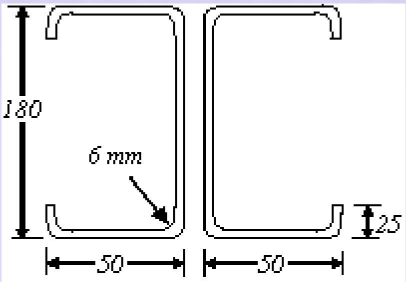
Two spans loaded: R_A = 0.375 x 29.3 = 11 kN

R_B = 1.25 x 29.3 = 36.6 kN

One span loaded: R_A = 0.438 x 29.3 = 12.8 kN

Maximum reaction at end support, $F_{w,max}$ = 12.8 kN

Maximum shear force, $F_{v,max}$ = 29.3 - 11 = 18.3 kN



Try 180 x 50 x 25 x 4 mm Double section (placed back to back)

Material Properties: E = 205 kN/mm

p_y = 240 / 1.15

= 208.7 N/mm^2

Section Properties: t = 4.0 mm

D = 180 mm

r_{yy} = 17.8 mm

$I_{xx} = 2 \times 518 \times 10^4$ mm^4

$Z_{xx} = 115.1 \times 10^3$ mm^3

Only the compression flange is subject to local buckling

Limiting stress for stiffened web in bending

$$p_0 = \left\{1.13 - 0.0019\frac{D}{t}\sqrt{\frac{f_y}{280}}\right\}p_y$$

and p_y = 240 / 1.15 = 208.7 N / mm^2

$$p_0 = \left\{1.13 - 0.0019 \, x \, \frac{180}{4}\sqrt{\frac{240}{280}}\right\} x \, 208.7$$

= 219.3 N / mm^2

Which is equal to the maximum stress in the compression flange, i.e.,

f_c = 219.3 N / mm^2

Effective width of compression flange

h = B_2 / B_1 = 160 / 30 = 5.3

$$K_1 = 5.4 - \frac{1.4h}{0.6 + h} - 0.02h^3$$
$$= 5.4 - \frac{1.4 \text{ x } 3.8}{0.6 + 3.8} - 0.02 \text{ x } 5.3^3$$

= 1.1 or 4 (minimum) = 4

$$p_{cr} = 185000 \text{ x } 4 \text{ x} \left(\frac{4}{30}\right)^2 = 13155 \text{ N/mm}^2$$
$$\frac{f_c}{p_{cr}} = \frac{219.3}{13155} = 0.017 > 0.123$$
$$\frac{b_{eff}}{b} = 1$$

b_{eff} = 30 mm

i.e. the full section is effective in bending.

I_{xr} = 2 x 518 x 10^4 mm^4

Z_{xr} = 115.1 x 10^3 mm^3

Moment Resistance

The compression flange is fully restrained over the sagging moment region but it is unrestrained over the hogging moment region, that is, over the internal support.

However unrestrained length is very short and lateral torsional buckling is not critical.

The moment resistance of the restrained beam is:

$$M_{cx} = Z_{xr}\, p_y$$

$= 115.1 \times 103 \times (240 / 1.15)\ 10^{-6} = 24$ kNm $>$ 16.5 kNm

O.K

Shear Resistance

Shear yield strength,

$$p_v = 0.6\ p_y = 0.6 \times 240 / 1.15 = 125.2\ N/mm^2$$

Shear buckling strength, $q_{cr} = \left(\frac{1000t}{D}\right)^2 = \left(\frac{1000 \times 4}{180}\right)^2 = 493.8\ N/mm^2$

Maximum shear force, $F_{v,max} = 18.3$ kN

Shear area = 180 x 4 = 720 mm^2

Average shear stress $f_v = \frac{18.3 \times 10^3}{720} = 25.4\ N/mm^2 < q_{cr}$

O.K

Web crushing at end supports

Check the limits of the formulae.

$$\frac{D}{t} = \frac{180}{4} = 45 \le 200 \qquad \therefore \text{O.K}$$

$$\frac{r}{t} = \frac{6}{4} = 1.5 \le 6 \qquad \therefore \text{O.K}$$

At the end supports, the bearing length, N is 50 mm (taking conservatively as the flange width of a single section)

For c=0, N/ t = 50 / 4 = 12.5 and restrained section.

C is the distance from the end of the beam to the load or reaction.

Use

$$P_w = 2\,x\,t^2\,C_r\,\frac{f_y}{\gamma_m}\left\{8.8 + 1.11\sqrt{N/t}\right\}$$

$$C_r = 1 + \frac{D/t}{750}$$

$$= 1 + \frac{45}{750} = 1.06$$

$$P_w = 2\,x\,4^2\,x\,1.06\,x\,\frac{240}{1.15}\left\{8.8 + 1.11\sqrt{12.5}\right\}10^{-3}$$

Web Crushing at internal support

t the internal support, the bearing length, N, is 100mm (taken as the flange width of a double section)

For c > 1.5D, N / t = 100 / 4 = 25 and restrained section.

$$P_w = t^2\,C_5 C_6\,\frac{f_y}{\gamma_m}\left\{13.2 + 1.63\sqrt{N/t}\right\}$$

$$k = \frac{f_y}{228\,x\,\gamma_m} = \frac{240}{1.15\,x\,228} = 0.9$$

C_5 = (1.49 - 0.53 k) = 1.49 - 0.53 x 0.92 = 1.0 > 0.6

C_6 = (0.88 - 0.12 m)

m = t / 1.9 = 4 / 1.9 = 2.1

C_6 = 0.88 - 0.12 x 2.1 = 0.63

$$\therefore\ P_w = 2\,x\,4^2\,x\,1\,x\,0.63\,x\,\frac{240}{1.15}\left\{13.2 + 1.63\sqrt{25}\right\}10^{-3}$$

= 89.8 kN > R_B (= 36 kN)

Deflection Check

A coefficient of $\frac{3}{384}$ is used to take in account of unequal loading on a double span. Total unfactored imposed load is used for deflection calculation.

$$\delta_{max} = \frac{3}{384}\frac{W L^3}{E I_{av}}$$

$$I_{av} = \frac{I_{xx} + I_{xy}}{2} = \frac{1036+1036}{2} = 1036 \, x \, 10^4 \text{ mm}^4$$

W = 29.3 / 1.5 = 19.5 kN

$$\delta_{max} = \frac{3}{384}\frac{19.5 \, x \, 10^3 \, x \, 4500^3}{205 \, x \, 10^3 \, x \, 1036 \, x \, 10^4} = 6.53 \text{ mm}$$

Deflection limit = L / 360 for imposed load

= 4500 / 360 = 12.5 mm > 6.53 mm O.K

In the double span construction: Use double section 180 x 50 x 25 x 4.0 mm lipped channel placed back to back.

5.10.4 Column design

Design a column of length 2.7 m for an axial load of 550 kN.

Axial load P = 550 kN

Length of the column, L = 2.7 m

Effective length, le = 0.85L = 0.85 x 2.7 = 2.3 m

Try 200 x 80 x 25 x 4.0 mm Lipped Channel section

Material Properties: E = 205 kN/mm^2

f_y = 240 N/ mm^2

p_y = 240 / 1.15 = 208.7 N/mm^2

Section Properties: A = 2 x 1576 = 3152 mm^2

I_{xx} = 2 x 903 x 10^4 mm^4

I_{yy} = 2 [124 x 10^4 + 1576 x 24.8^2]

= 442 x 10^4 mm^4

$r_{min} = \sqrt{\dfrac{442 \times 10^4}{2 \times 1576}}$ = 37.4 mm

Load factor Q = 0.95 (from worked example 1)

The short strut resistance, P_{cs} = 0.95 ? 2 ? 1576 ? 240/ 1.15 = 625 kN

P = 550 kN < 625 kN O. K

Axial buckling resistance

Check for maximum allowable slenderness

$$\frac{l_e}{r_y} = \frac{2.3 \times 10^3}{37.4} = 61.5 < 180 \qquad \text{O.K}$$

In a double section, torsional flexural buckling is not critical and thus α = 1

Modified slenderness ratio,

$$\bar{\lambda} = \frac{\alpha \dfrac{l_e}{r_y}}{\lambda_y}$$

$$\lambda_y = \pi\sqrt{\frac{E}{p_y}} = \pi\sqrt{\frac{2.05 \times 10^5}{208.7}} = 98.5$$

$$\therefore \bar{\lambda} = \frac{1 \times 61.5}{98.5} = 0.62$$

$$\frac{P_c}{P_{cs}} = 0.91$$

P_c = 0.91 x 625 = 569 kN > P O. K

5.11 Concluding remarks

In this chapter the difference between cold rolled steel and hot rolled steel has been discussed and the merits of the former are outlined. The concepts of "effective width" and "effective section" employed in the analysis and design of cold rolled section have been explained. The difference between "stiffened" and "unstiffened" elements has been explained. Considerations in the design of cold rolled beams have been explained and formulae employed for the limit state design of beams made of cold rolled sections have been provided.

In the two preceeding chapters on cold rolled steel, a detailed discussion of design of elements made from it has been provided, the major differences between the hot rolled steel products and cold rolled steel products outlined and the principal advantages of using the latter in construction summarized. Design methods, including methods based on prototype testing and empirical design procedures have been discussed in detail.

Thin steel products are extensively used in building industry in the western world and this range from purlins and lintels to roof sheeting and decking. Light steel frame construction is often employed in house building and is based on industrialized manufacture of standardized components, which ensure a high quality of materials of construction. The most striking benefit of all forms of light steel framing is their speed of construction, ease of handling and savings in site supervision and elimination of wastage in site, all of which contribute to overall economy.

In the Indian context, industrialized methods of production and delivery of cold rolled steel products to site have the potential to build substantially more houses than is otherwise possible, with the same cash flow, thus freeing capital and financial resources for other projects. Other advantages include elimination of shrinkage and movement cracks, greater environmental acceptability and less weather dependency. Properly constructed light steel frames are adaptable to future requirements and will provide high acoustic performance and a high degree of thermal insulation. Provided the sheets are pre-galvanised, the members provide adequate corrosion protection when used close to the boundaries of the building envelope. The design life of these products exceeds 60 years.

5.12 References

1. BS5950, Part 5: Structural Use of Steelwork in Building, British Standards Institution, London 1987.

2. J. Rhodes and R.M. Lawson "Design of Structures using Cold Formed Steel Sections, SCI Publication 089, The Steel Construction Institute, U.K. 1992.

3. Rhodes, J. and Lawson, R. M., Design of Structures using Cold Formed Steel Sections, The Steel Construction Institute, Ascot, 1992.

4. Chung, K. F Worked examples to BS 5950, Part 5, 1987, The Steel Construction Institute, Ascot, 1993.

5. Wei-Wen Yu , Cold Formed Steel Structures by, Mc Graw Hill Book Company, 1973.

6. MICROWAVE TOWERS

6.1 Introduction

In the present era the technology in communications has developed to a very large extent. The faster growth demands advances in the design and implementation of the communication towers. There are different types of communication towers present now-a-days in the cellular business. The present paper covers the issues related to the types of towers, codal provisions for the communication towers, foundation design of the green field and roof top towers and optimization of the towers through expert ware.

Cold-formed sections are used in many industries and are often specially shaped to suit the particular application. In building uses, the most common sections are the C and the Z shapes. There are, however, a whole range of variants of these basic shapes, including those with edge lips, internal stiffeners and bends in the webs.

Other section shapes are the "top-hat" section and the modified I section. The common range of cold-formed sections that are marketed is illustrated in Figure. The sections can also be joined together to form compound members.

The reason for the additional lips and stiffeners is because unstiffened wide thin plates are not able to resist significant compression and consequently the use of steel in the section becomes inefficient. However, a highly stiffened section is less easy to form and is often less practicable from the point of view of its connections. Therefore, a compromise between section efficiency and practicability is often necessary.

High strength for a given section depth
Ability to provide long spans (up to 10 m)
Dimensional accuracy
Long term durability (if galvanized) in internal environments
Freedom from creep and shrinkage
Can be formed to a particular shape or application
Lightness, particularly important in poor ground conditions
Dry envelope
Delivered to site cut to length and with pre-punched holes, requiring no further fabrication
Ability to be prefabricated into panels etc.
Robust and sufficiently light for site handling

***Examples of the structural use of cold-formed sections which utilize these features are as follows*:**

Roof and wall members

Traditionally, a major use of cold-formed steel in the UK has been as purlins and side rails to support the cladding in industrial type buildings. These are generally based on the Z section (and its variants) which facilitates incorporation of sleeves and overlaps to improve the efficiency of the members in multi-span applications. Special shapes are made for eaves members etc.

Steel framing

An increasing market for cold-formed steel sections is in site-assembled frames and panels for walls and roofs, and stand-alone buildings. This approach has been used in light industrial and commercial buildings and in mezzanine floors of existing buildings.

Wall partitions

A special application is for very light sections used in conjunction with plaster board panels in stud wall partitioning to form a thin robust wall.

Large panels for housing

Storey-high panels can be factory-built and assembled into housing units on site. This is an extension of the approach used for timber framing.

Lintels

A significant market for specially formed cold formed sections is as lintels over doors and windows in low rise masonry walls (Figure 6). These products are often powder coated for extra corrosion protection.

Floor joists

Cold formed sections may be used as an alternative to timber joists in floors of modest span in domestic and small commercial buildings.

Modular frames for commercial buildings.

A prefabricated modular framing system panel system using cold formed channels and lattice joists has been developed for use in buildings up to 4 storeys height (Figure 7). Although primarily developed for commercial building this modular system has broad application in such as educational and apartment buildings.

Trusses

There are a number of manufacturers of lattice girder and truss systems using cold formed steel sections.

Space frames

A space frame (a three-dimensional truss) using cold formed steel sections has recently been marketed in the UK.

Curtain walling

A modem application is in cladding framing to multi-storey mullions and transoms in standard glazing systems, steel buildings, and as mullions and transoms in standard glazing systems.

Prefabricated buildings

The transportable prefabricated building unit (such as the ubiquitous site hut) is a common application of the use of cold-formed steel. Other applications are as prefabricated "toilet pod" units in multi-storey buildings.

Frameless steel buildings

Steel folded plates, barrel vaults and truncated pyramid roofs are examples of systems that have been developed as so-called frameless buildings (i.e. those without beams and which rely partly on â€˜stressed skin" action).

Storage racking

Storage racking systems for use in warehouses and industrial buildings are made from cold formed steel sections. Most have special clip attachments, or bolted joints for easy assembly.

Applications in general civil engineering include:

Lighting and transmission towers

These are often made from thin tubular or angle sections.

Motorway crash barriers

These thin steel members are primarily designed for strength but also have properties of energy absorbtion by permitting gross deformation.

Silos for agricultural use

Silo walls are often stiffened and supported by cold-formed steel sections.

The main structural use of cold-formed steel not listed above is that of floor decking which is usually sold as a galvanised product. In particular, â€˜composite" decking is designed to act in conjunction with in situ concrete floors in steel framed buildings to form composite slabs. Composite decking is usually designed to be unpropped during construction and typical spans are 3.0m to 3.6m.

Other major non-structural applications of cold formed steel in building include such diverse uses as garage doors, and ducting for heating and ventilating systems.

6.2 Types of communication towers

The different types of communication towers are based upon their structural action, their cross-section, the type of sections used and on the placement of tower.

A brief description is as given below:

6.2.1 Based on structural action.

Towers are classified into three major groups based on the structural action. They are:

- Self supporting towers
- Guyed towers
- Monopole.

6.2.1.1. Self supporting towers.

The towers that are supported on ground or on buildings are called as self-supporting towers. Though the weight of these towers is more they require less base area and are suitable in many situations. Most of the TV, MW, Power transmission, and flood light towers are self-supporting towers.

6.2.1.2. Guyed towers.

Guyed towers provide height at a much lower material cost than self-supporting towers due to the efficient use of high-strength steel in the guys. Guyed towers are normally guyed in three directions over an anchor radius of typically 2/3 of the tower height and have a triangular lattice section for the central mast. Tubular masts are also used, especially where icing is very heavy and lattice sections would ice up fully. These towers are much lighter than self-supporting type but require a large free space to anchor guy wires. Whenever large open space is available, guyed towers can be provided. There are other restrictions to mount dish antennae on these towers and require large anchor blocks to hold the ropes.

6.2.1.3 Monopole.

It is single self-supporting pole, and is generally placed over roofs of high raised buildings, when number of antennae required is less or height of tower required is less than 9m.

6.2.2. Based on cross section of tower.

Towers can be classified, based on their cross section, into square, rectangular, triangular, delta, hexagonal and polygonal towers.

Open steel lattice towers make the most efficient use of material and enables the construction of extremely light-weight and stiff structures by offering less exposed area to wind loads. Most of the power transmission, telecommunication and broadcasting towers are lattice towers.

Triangular Lattice Towers have less weight but offer less stiffness in torsion. With the increase in number of faces, it is observed that weight of tower increases. The increase is 10% and 20% for square and hexagonal cross sections respectively. If the supporting action of adjacent beams is considered, the expenditure incurred for hexagonal towers is somewhat less.

6.2.3 Based on the type of material sections.

Based on the sections used for fabrication, towers are classified into angular and hybrid towers (with tubular and angle bracings).

Lattice towers are usually made of bolted angles. Tubular legs and bracings can be economic, especially when the stresses are low enough to allow relatively simple connections. Towers with tubular members may be less than half the weight of angle towers because of the reduced wind load on circular sections. However the extra cost of the tube and the more complicated connection details can exceed the saving of steel weight and foundations.

6.2.4 Based on the placement of tower.

Based on this placement, Communication towers are classified as follows:

	Green Field Tower	**Roof Top Tower**
Erection	Erected on natural ground with suitable foundation.	Erected on existing building with raised columns and tie beams.
Height	30 – 200 m	9 – 30 m
Usual Location	Rural Areas	Urban Areas
Economy	Less	More

6.2.5 Based on the number of segments.

The towers are classified based on the number of segments as Three slope tower; Two slope tower; Single slope tower; Straight tower.

6.3 Ladders and platforms

6.3.1 Ladder

In communication towers the climbing facility can be provided by two ways.

a) by providing climbing ladder with or without safety ring and

b) by providing step bolts confirming to IS 10238:1982.

Generally for communication towers it is usual practice to provide climbing ladder with safety cage. The exposed area of ladder shall be considered while calculating the wind load on tower. The position of ladder will have impact on weight of tower. If ladder is provided internally its effect will be less and if it is provided externally it will have more effect. Protection ring is a safety requirement and may be replaced by fall arrest safety system.

Cable ladder is provided to support the cable wave-guide running from antenna to the equipment shelter. The cable ladder is provided inside and along the slope of the tower.

Step bolts are provided only for specific cases of narrow based towers of smaller heights as per user's requirements. The step bolts should be capable of withstanding a vertical load of not less than 1.5 kN.

6.3.2 Platforms

The platforms shall be provided at different levels as rest platforms or working platforms. The rest platforms are provided with chequered plate or welded wire mesh along with suitable railing inside the tower and are provided for every 10m height, for all towers of height exceeding 20m.

The working platforms may be internal or external and these are provided with railings of 1000mm with toe, knee and hand rail protection.

6.4 Codal provisions in design of communication towers

The following are the steps involved in design of communication tower.

a. Selection of configuration of tower

b. Computation of loads acting on tower

c. Analysis of tower for above loads

d. Design of tower members according to codes of practices.

Selection of configuration of a tower involves fixing of top width, bottom width, number of panels and their heights, type of bracing system and slope of tower.

6.4.1 Wind load on tower

The wind load on tower can be calculated using the Indian standards IS: 875(Part 3)-1987[3] and BS: 8100 (Part 1)-1996[4].

The designer should select the basic wind speed depending on the location of tower. The design wind speed is modified to induce the effect of risk factor (k_1), terrain coefficient (k_2) and local topography (k_3) to get the design wind speed V_z. ($V_z = k_1k_2k_3V_b$).

The design wind pressure P_z at any height above mean ground level is $0.6V_z^2$.

The coefficient 0.6 in the above formula depends on a number of factors and mainly on the atmospheric pressure and air temperatures.

Solidity ratio is defined as the ratio of effective area (projected area of all the individual elements) of a frame normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind direction.

Force coefficient for lattice towers of square or equilateral triangle section with flat sided members for wind blowing against any face shall be as given in Table 30 of IS:875(Part-3)-1987.

Force coefficients for lattice towers of square section with circular members and equilateral triangle section with circular members are as given in tables 31 and 32 of IS: 875(Part-3)-1987 respectively.

Table 2 of IS:875(Part-3)-1987 gives the factors to obtain design wind speed variation with height in different terrains for different classes of structures such as class A, class B, class C.

The wind load acting on a tower can be computed as $F = C_{dt}A_eP_zk_2$.

For circular sections the force coefficient depends upon the way in which the wind flows around it and is dependent upon the velocity and kinematic viscosity of the wind and diameter of the section. The force coefficient is usually quoted against a non-dimensional parameter, called the Reynolds number, which takes account of the velocity and viscosity of the medium and the member diameter.

6.4.2 Wind load on antennae

Wind load on antennae shall be considered from Andrew's catalogue. In the Andrew's catalogue the wind loads on antennas are given for 200kmph wind speed. The designer has to calculate the antenna loads corresponding to design wind speed.

6.4.3 Design of tower members

According to the clause 5.1 of IS-802(Part-1/sec2)[5] the estimated tensile stresses on the net effective sectional areas in various members shall not exceed minimum guaranteed yield stress of the material. However in case the angle section is connected by one leg only, the estimated tensile stress on the net effective sectional area shall not exceed F_y, where F_y is the minimum guaranteed yield stress of the material. For structural steels confirming to IS-226[6] and IS-2062[7] the yield strength is 250 MPa. Generally yst25 grade tubes confirming IS-1161[8] are used for tower members.

As per IS-802 part1/sec2 estimated compressive stresses in various members shall not exceed the values given by the formulae in clause 5.2.2. of IS-802 code.

6.4.4 Limiting slenderness ratios

a. As per clause 6.3 of IS-802(Part1/sec2)-1992 the limiting values KL / r shall be as follows:

Leg members	120
Redundant members and those carrying nominal stresses	250
Other members carrying computed stresses	200

b. As per clause 6.4 of IS-802(Part1/sec2) Slenderness ratio *L / r* of a member carrying axial tension only, shall not exceed 400.

c. Similarly for tubular sections as per clause 6.4.2 of IS-806-1968[9] – The ratio of effective length (*l*) to the appropriate radius of gyration(r) of a compression member shall not exceed the following values.

Type of member	*l/r*
Carrying loads resulting from dead loads and superimposed loads	180
Carrying loads resulting from wind or seismic forces only provided the deformation of such members does not adversely; affect the stress in any part of the structure.	250
Normally acting as a tie in a roof truss but subject to possible reversal of stress resulting from the action of wind.	350

As per clause 6.4.1 of IS-806-1968 the effective length (l) of a compression member for the purpose of determining allowable axial stresses shall be assumed in accordance with table 7 of IS-806-1968.

As per clause 7.2 of IS-802(Part1/sec2) Gusset plates shall be designed to resist the shear, direct and flexural stresses acting on the weakest or critical section. Re – entrant cuts shall be avoided as far as practical. Minimum thickness of gusset shall be 2mm more than lattice it connects only in case when the lattice is directly connected on the gusset outside the leg member. In no case the gusset shall be less than 5mm in thickness.

6.5 References:

[1]. G.A. Savitskii “Calculation of Antenna Installations, Physical Principles”

[2]. A.R. Santhakumar, S.S. Murthy “Transmission Line Structures” McGraw-Hill Book Co. 1990.

[3]. IS: 875(Part-3):1987 “Code of practice for design loads (other than earthquake) for buildings and structures”.

[4]. BS: 8100 (Part-1)-1996 “Lattice towers and Masts”.

[5]. IS: 802(Part-1)-1977 “Code of practice for use of structural steel in overhead transmission line towers”.

[6]. IS: 226-1975 “Structural Steel (Standard Quality)”.

[7]. IS: 2062 – 1992 “Steel for general structural purposes”.

[8]. IS: 1161 – 1998 “Steel tubes for structural purposes”.

[9]. IS: 806 – 1968 “Code of practice for use of steel tubes in general building construction”.

Examples

Example1 Basic wind pressure - calculation

A Power house building 25m high is to be designed in Darbhanga city. Compute the basic wind pressure.

Basic wind speed in Darbhanga (from appendix A)

P. 53 Code V_b = 55m/sec

An industrial building can be grouped under all general buildings and structures so should be designed for 50 years of design life

Risk coefficient from table 1. P. 11 code

$k_1 = 1$

Assuming the terrain is in city industrial area with numerous closely spaced obstructions. It can be grouped under category 3. P.8 code. Since the height of the building is 25m this falls under class B P.11 code. The terrain factor k_2 can be got from table 2 P.12 code. For category 3, class B interpolating between 20m and 30m

$k_2 = 1.005$

The ground is assumed to be plain so the topography factor k_3 is 1 + cs P. 56 code

where c = Z / L

Since the terrain assumed is plain. Read clause 5.3.3.1 P.12 code

$k_3 = 1$

Design wind speed $(V_z) = V_b\ k_1\ k_2\ k_3$

= 55 (1) (1.005) (1)

= 55.275 m/sec

Design wind pressure = $0.6\ V_Z^2$

$= 0.6\ (55.275)^2$

$= 1833.2\ N/m^2$

Example2

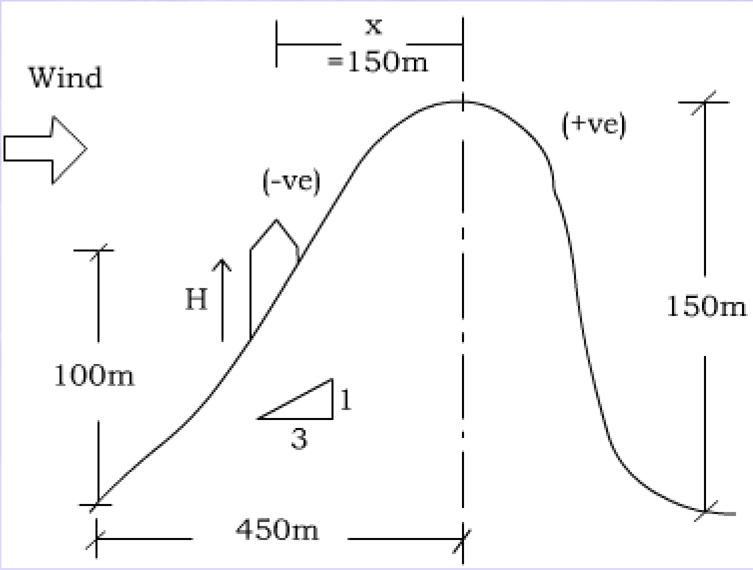
If the above building has to be constructed on a hillock where the height of the hill is 150m having a slope of 1:3 and the building is proposed at a height of 100m from the base on hte windward side, find the design wind

Basic wind speed at Darbhanga = 55m/sec

Risk coefficient k_1 =1

Terrain factor k_2 = 1.005

To find the topography factor k_3 Ref. appendix C.   P. 56 code



Z = height of the hill (feather) = 150m

θ = slope in 3 $\tan^{-1}(1/3) = 18.43^o$

L = Actual length of upwind slope in the wind direction = 150(3) = 450m

L_e = Effective horizontal length of the hill for $\theta > 17^o$ $L_e = Z / 0.3 = 150 / 0.3 =$ 500m

Values of C for $\theta = 18.43^o$ (i.e.) $> 17^o$

C = 0.36

Height of the building = 25m

To find x (i.e) the horizontal distance of the building from the crest measured +ve towards the leeward side and -ve towards the windward side.

$k_3 = 1 + cs$

To get s Fig 14 and 15 are used

x = -150m

$x / L_e = -150 / 500 = -0.3$ $H / L_e = 25 / 500 = 0.05$

Referring to figure 15 hill and ridge for $x / L_e = -0.3$ and $H / L_e = 0.05$ on the upwind direction

s = 0.58

$k_3 = 1 + (0.36)(0.58)$

$k_3 = 1.21$

Design wind speed $V_z = V_b\, k_1\, k_2\, k_3$

$= 55 (1) (1.005) (1.21)$

$= 66.9$ m/sec

Design wind pressure $P_Z = 0.6\, V_Z^2$

$= 0.6 (66.9)^2$

$= 2685.4$ N/m^2

Example 3:

A memorial building is proposed at Sriperumbudur - Madras on a hill top. The size of the building is 40m x 80m and height is 10m. The hill is 300m high with a gradiant of 1 in 5. The building is proposed at a distance of 100m from the crest on the downwind slope. Calculate the design wind pressure on the building.

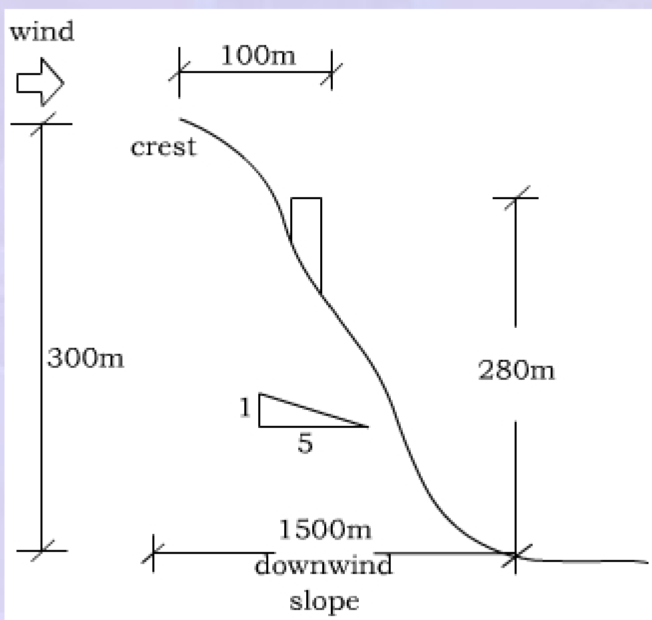
Basic wind velocity at madras is 50m/sec Ref. Appendix A. P.53 code

Risk coefficient k_{s1} = 1.08 for a memorial building of 100 years design life

Terrain factor k_2 for category 3 and class C since dimension of building 750m

k_2 = 0.82

Topography factor k_3



Z = effective height of the hill = 300m

θ = 1 in 5 $\tan^{-1}$ (1/5) = 11.31^o

L = Actual length of upward slope in the wind direction = 1500m

L_e = effective horizontal length of the hill

For θ = 11.31^o L_e = L = 1500m

Topography factor $k_3 = 1 + cs$

where $c = 1.2\ (Z/L)$ since $\theta = 11.31^o$ $3^o < \theta < 17^o$

$c = 1.2\ (300/1500) = 0.24$

x is the distance of the building from the crest + on downwind side

- on upward side $x = +100m$

The non dimensional factors are

$x / L_e = 100 / 1500 = 0.067$; $H / L_e = 10 / 1500 = 0.0067$

s = 1 from fig 15. P.57

$k_3 = 1 + (0.24)\ (1)$;

$k_3 = 1.24$

Design wind speed $V_z = V_b\ k_1\ k_2\ k_3$

$= 50\ (1.08)\ (0.82)\ (1.24)$

$= 54.91$ m/sec

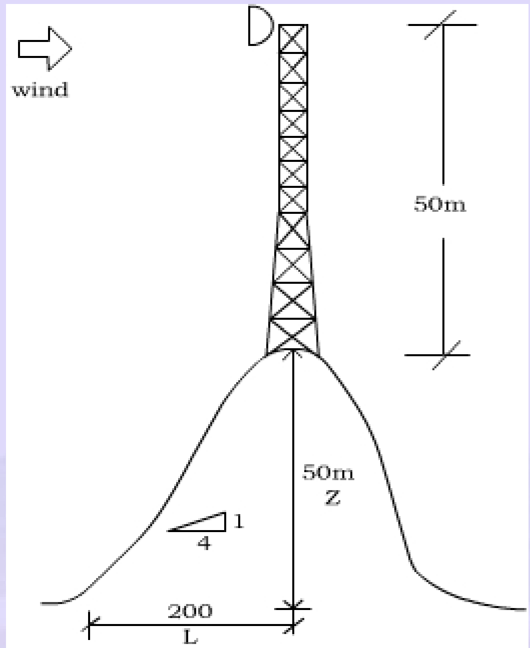
Design wind pressure $P_Z = 0.6\ V_Z^2$

$= 0.6\ (54.91)^2$

$= 1809.1\ N/m^2$

Example 4: Wind pressure on tower on a hill

A microwave tower of 50m height is proposed over a hill top. The height of the hill is 50m with a gradiant of 1 in 4. The terrain category is 3. The tower is proposed at coimbatore. Compute the design wind pressure:



Basic wind speed at CBE is 39m/sec

Risk factor $k_1 = 1.06$

Terrain factor (k_2) for category 3 class B - height between 20 and 50

$k_2 = 1.09$ table 2, P.12

Topography factor (k_3) Ref. P.56

Z - effective height of the hill = 50m

θ - slope 1 in 4 $\tan^{-1}(1/4) = 14.04^o$

L - Actual length of the upwind slope = 200m

L_e - Effective horizontal length of the hill $\theta = 14.04^o < 17$

$L_e = L = 200m$

$k_3 = 1 + cs$

$\theta < 17$, $c = 1.2 (Z/L) = 1.2 (50/200) = 0.3$

$x / L_e = 0/200 = 0$; $H / L_e = 50/200 = 0.25$

Ref. Fig.15 s = 0.6; $k_3 = 1 + (0.3)(0.6)$

$k_3 = 1.18$

Design wind speed $V_z = V_b\, k_1\, k_2\, k_3$

$= 39\ (1.06)\ (1.09)\ (1.18)$

$= 53.17$ m/sec

Design wind pressure $P_Z = 0.6\ V_Z^2$

$= 0.6\ (53.17)^2$

$= 1696.23\ N/m^2$

Example 5:

If the 50m tower given in previous example is mounted with a hollow hemispherical dome of 2m diameter weighing 10kN. Compute the forces and stresses in members of various panels. The elevation of the tower is as shown below

Data given: Height of the tower = 50m

Base width = 6m

Top width = 2m

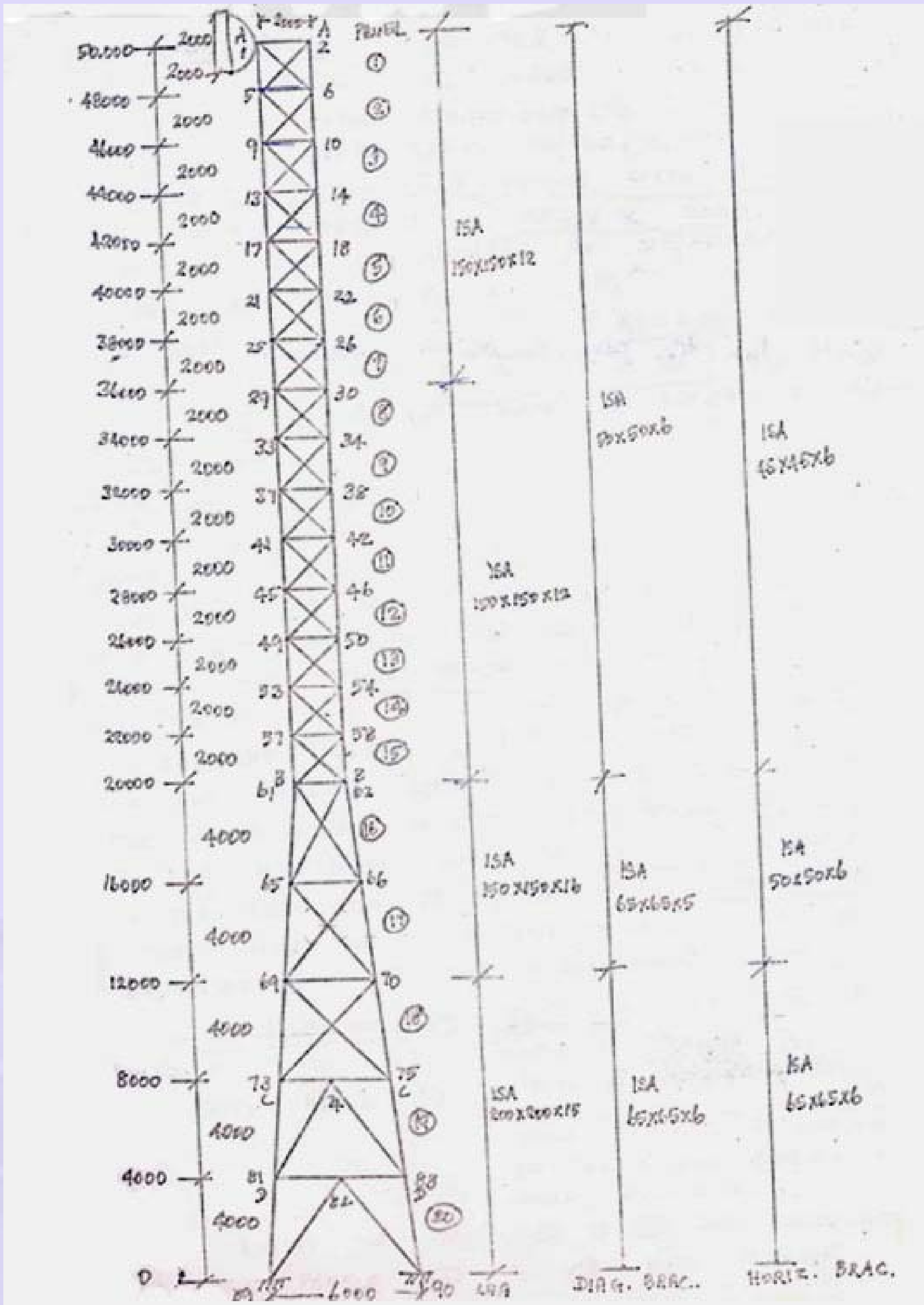
No. of panels = 20

Disk size = 2m diameter

Step 1: Wind force - From the previous example

Basic wind speed = 39m/sec

Risk coefficient (k_1) = 1.06



Topography factor (k_3) = 1.2

Terrain factor (k_2), varies with the height of the tower Ref, P.12 Table 2 code

The design wind pressures at different heights are computed as

$$P_Z = 0.6\ V_Z^2$$

$$= 0.6\ (39 \times 1.06 \times 1.2 \times k_2)^2$$

$=1476.6\ k_2^2\ N/m^2$

The values of k_2 at different height is chosen from Table 2

Step2: Basic assumptions:

1. Self weight of the members are equally distributed to the two joints connected by the members

2. No load is applied at the middle of the k-braced joint but allocated to column joint

3 Dead and wind loads are increased by 15% for each joints to account for Gussets, bolts and nuts

4. Secondary members are assumed to be provided in the panel where batter starts (below the waist level in our case panels 16 to 20. So an additional load of 10% is accounted for in the case of provision of secondary members

5. The wind loads on the members are equally distributed to the connecting joints.

Step3: Calculation of solidity ratios: Ref P.7 code

Solidity ratio for different panels are calculated

$$\text{Solidity ratio}\left(\phi\right) = \frac{\text{Projected area of all the individual elements}}{\text{Area enclosed by the boundary of the frame normal to the wind direction}}$$

Solidity ratios of panel 1 to 15 are calculated once as panels 1 to 15 are similar

$$\phi_{1-15} = \frac{15 \text{ x } 2\left(2 \text{ x } 0.15\right) + 15 \text{ x } 2\left(\sqrt{2} \text{ x } 2 \text{ x } 0.05\right) + 16 \text{ x } 2 \text{ x } 0.045}{30 \text{ x } 2}$$

$$\phi_{1-15} = 0.245 \text{ Similarly for } \phi_{16}$$

$$\phi_{1-16} = \frac{2 \text{ x } 4.04 \text{ x } 0.15 + 2 \text{ x } 4.68 \text{ x } 0.065 + 2.8 \text{ x } 0.05}{\left(\frac{2+2.8}{2}\right) \text{x } 4}$$

$$\phi_{16} = 0.204$$

$$\phi_{17} = \frac{2 \text{ x } 4.04 \text{ x } 0.15 + 2 \text{ x } 5.14 \text{ x } 0.065 + 1 \text{ x } 3.6 \text{ x } 0.065}{\left(\frac{2+3.6}{2}\right) \text{x } 4}$$

$$\phi_{17} = 0.165$$

$$\phi_{18} = \frac{2 \text{ x } 4.04 \text{ x } 0.2 + 2 \text{ x } 5.67 \text{ x } 0.065 + 1 \text{ x } 4.4 \text{ x } 0.065}{\left(\frac{3.6+4.4}{2}\right) \text{x } 4}$$

$$\phi_{18} = 0.165$$

$$\phi_{19} = \frac{2 \text{ x } 4.04 \text{ x } 0.2 + 2 \text{ x } 4.79 \text{ x } 0.065 + 1 \text{ x } 5.2 \text{ x } 0.065}{\left(\frac{4.4+5.2}{2}\right) \text{x } 4}$$

$$\phi_{19} = 0.134$$

$$\phi_{20} = \frac{2 \text{ x } 4.04 \text{ x } 0.2 + 2 \text{ x} 5.016 \text{ x } 0.065}{\left(\frac{5.2+6}{2}\right) \text{x } 4}$$

$$\phi_{20} = 0.101$$

Step4 : Calculation of bowl wind pressure

Ref. Fig6 P.44 code. Bowl wind coeffs. are

c_f = 1.4 for wind from front

c_f = 0.4 for wind from rear

wind pressure at 50m above GL

Design wind pressure P_Z =1476.6 (1.09)^2

= 1.754 kN/m^2

Wind loads on dish are on front face F_{DISH} 1 = $c_f.A_e.p_d$

Ref. P.36 clause 6.3 code

F_{DISH} 1 = 1.4 x π/4 x 2_2 x 1.754

= 7.71 kN

On rear face

F_{DISH} 2 = 0.4 x π/4 x 2_2 x 1.754

= 2.20 kN

Step5:

The terrain factor (k_2), the solidity ratio and the design wind pressures at various heights are tabulated as shown - category 3 class B

Panel from top	Height in 'm' from top	Terrain size,HT. coeff. k_2	Design wind pressure P_Z =1476.6 (k_2^2) N/m^2	Solidity ratio	Overall force coeff. c_f Table30 P.47	P_Z . c_f N/m^2
1 to 5	10	1.09 = 1.075 1.06	1706.4	0.245	3.075	5247.2
6 to 10	20	1.06 = 1.045 1.03	1612.5	0.245	3.075	4958.4
11 to 15	30	1.03 = 1.005 0.98	1491.4	0.245	3.075	4586.1
16	34	0.98 = 0.964 0.948	1372.2	0.204	3.28	4500.8
17	38	0.948 = 0.926 0.904	1266.1	0.165	3.475	4399.7
18	42	0.904 = 0.88 0.856	1143.5	0.165	3.475	3975.7
19	46	0.856 = 0.832 0.808	1022.1	0.134	3.630	3710.2
20	50	0.808	964.0	0.101	3.795	3658.4

Step6: Calculation of forces at different joints

The forces from the dish are transferred to two top most joints 1 and 4. The dish weight and wind force on the dish are equally distributed at the two joints.

Panel 1 Leg: Length of the leg = 2m

Width of the leg = 0.15m

Since 4 Nos of ISA 150 x 150 x 12 @ 0.272 kN/m

Self weight of legs = 4 x 2 x 0.272 = 2.176 kN

No. of legs exposed to wind = 2

Wind obstruction area = 2 x 2 x 0.15

= 0.6 m^2

wind load on leg = 0.6 x 5247.2

= 3.148 kN

Diagonal bracing : No. of diagonal bracings = 8

No. of obstructing wind = 2

Size of diagonal bracing ISA 50 x 50 x 6 @ 0.045 kN/m.

Self weight = 8 x $\sqrt{2}$ x 2 x 0.045

= 1.018 kN

Wind obstruction area = 2 x $\sqrt{2}$ x 2 x 0.05

= 0.283 m^2

Wind load on diag. Brac = 0.283 x 5247.2

= 1.485 kN

Horizontal bracing: ISA 45 x 45 x 6

No. of horizontal bracings = 8

No. of obstructing wind = 2

Self weight of horizontal bracing = 8 x 2 x 0.04

= 0.64 kN

Wind obstruction area = 2 x 2 x 0.045

= $0.18 m^2$

Wind load on horizontal brac = 0.18 x 5247.2

= 0.945 kN

Total self weight of leg, diag. brac and horizontal brac

F_v = 2.176 + 1.018 + 0.64 = 3.834 kN

Total wind load on leg, diag and Hor. bracs

F_H = 3.148 + 1.485 + 0.945 = 5.578 kN

These load are to be distributed to all the 8 joints connecting the elements (i.e. joints 1 to 8)

Load at each joint is increased by 15% to account for gussets, bolts and washers

F_{v1} vertical load on joints 1 to 8 = 1.15 x 3.834 / 8

= 0.551 kN

F_{H1} wind load on joints 1 to 8 = 1.15 x 5.576 / 8

= 0.802 kN

The self weight of the dish is shared by joints 1 and 4

$F_{V\ DISH}$ = 10/2 kN = 5kN

Wind load on the dish is shared by joints 1, 2, 3 and 4, $F_{H\ DISH}$ = 7.71 / 4 = 1.93 kN

Panel 2: Self weight of legs = 2.176 kN

wind load on legs = 3.148 kN

Self weight of diag. Bracs = 1.018 kN

Wind load on Diag. Brac = 1.485 kN

No. of horizontal bracings = 4

No. of obstructing wind = 4

Self weight of horizontal bracing = 4 x 2 x 0.04

= 0.32 kN

Wind obstruction area = 1 x 2 x 0.045

= 0.09 m^2

Wind load on hor. brac. = 0.09 x 5247.2 = 472.2 N

Vertical load due to leg and diag. brac carried by joints 5 to 12 = 1.15 (2.176 + 1.018) / 8

= 0.46 kN

Vertical load due to hor.brac. carried by joints 9, 10, 11 and 12 = 1.15 x (0.32)/4 = 0.092 kN

Wind load carried by joints 5 to 12 = 1.15 (3.148 + 1.485) / 8

= 0.666 kN

Wind load carried by joints 9, 10, 11 and 12 = 1.15 x 0.472/4

= 0.136 kN

Computation of loads at different joints are made for panel to panel from panel 2 to panel 5 are tabulated

Panel 6: Self weight of legs = 4 x 2 x 0.272 = 2.176 kN

Wind load = 0.6 x 4958.4 = 2.975 kN

Self weight of Diag. Brac. = 1.018 kN

Wind load = 0.283 x 4958.4 = 1.403 kN

Self weight of hor. bracings = 0.32 kN

Wind load = 0.09 x 4958.4 = 0.446 kN

Vertical load carried by joints 21 to 28 = (2.176 + 1.018) 1.15 / 8

= 0.46 kN

Wind load carried by joints 21 to 28 = (2.975 + 1.403) 1.15 / 8

= 0.63 kN

Vertical load due to Hor. Brac. carried by joints 25, 26, 27 and 28 = 1.15 x (0.32)/4

= 0.092 kN

Wind load carried by joints 25, 26, 27 and 28 = 1.15 x (0.446)/4

= 0.128 kN

Computations of loads at different joints were done from 6 to 10 and are tabulated.

Panel 11: Vertical load carried by joints 41 to 48 = 0.46 kN

Wind load on the legs = 0.6 x 4586.1

= 2.75 kN

Wind load on the Diag. Brac. = 0.283 x 4586.1

= 1.3 kN

Vertical load due to Hor. Brac carried by joints 45, 46, 47 and 48 = 0.092 kN

Wind load carried by joints 41 to 48 = 1.15 (2.75 + 1.3)/8

= 0.582 kN

Wind load carried by joints 45 to 48 due to Hor. Brac. = (0.09 x 4586.1)/4

Computation of loads at different joints were done from panel 11 to 15 and are tabulated

Panel 16: Leg: ISA 150 x 150 x 15 @ 0.336 kN/m

Length of the leg (L) = 4.04m

Width of the leg (B) = 0.15m

Self weight of legs = 4 x 4.04 x 0.336

= 5.43 kN

No. of legs exposed to wind = 2

Wind obstruction area = 2 x 4.04 x 0.15

= 1.212 m^2

Wind load on leg = 1.212 x 4500.8

= 5.454 kN

Diag. Brac: ISA 65 x 65 x 5 @ 0.049 kN/m

No. of bracing = 8

No. of obstructing wind = 2

Self weight of diagonal brac. = 8 x 4.68 x 0.049

= 1.835 kN

Wind obstruction area = 2 x 4.68 x 0.065

= 0.6084 m^2

Wind load on Diag. Brac = 0.6084 x 4500.8

= 2.74 kN

Horizontal Brac: ISA 65 x 65 x 5 @ 0.045 kN/m

No. of bracing = 4

No. of obstructing wind = 1

Self weight of Hor. brac. = 4 x 2.8 x 0.045

= 0.504 kN

Wind obstruction area = 1 x 2.8 x 0.050

= 0.14 kN

Wind load on Hor. Brac = 0.14 x 4500.8

= 0.63 kN

Secondary bracings are accounted for so DL and WL is increased by 10%

Vertical load carried by joints 61 to 68 = (1.25 / 5.43 + 1.835)/8

= 1.135 kN

Vertical load carried by joints 65 to 68 due to Hor. Brac. = 1.25 (0.504)/4

= 0.158 kN

Wind load carried by joints 61 to 68 = 1.25 (5.454 + 2.74)/8

= 1.28 kN

Wind load carried by joints 65 to 68 due to Hor. Brac = 1.25 (0.63) / 4

= 0.197 kN

Panel 17: Leg: ISA 150 x 150 x 16 @ 0.336 kN/m

Self weight of legs = 4 x 4.04 x 0.336

= 5.43 kN

Wind obstruction area = 2 x 4.04 x 0.15

= 1.212 m^2

Wind load on leg = 1.212 x 4399.7

= 5.332 kN

Diag. Brac: ISA 65 x 65 x 5 @ 0.049 kN/m

Self weight of diagonal brac. = 8 x 5.14 x 0.049

= 2.015 kN

Wind obstruction area = 2 x 5.14 x 0.065

= 0.6682 m^2

Wind load on Diag. Brac = 0.6682 x 4399.7

= 2.94 kN

Horizontal Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of Hor. brac. = 4 x 3.6 x 0.058

= 0.835 kN

Wind obstruction area = 1 x 3.6 x 0.065

= 0.234 m^2

Wind load on Hor. Brac = 0.234 x 4399.7

= 1.03 kN

Secondary bracings should be accounted for in this panel

Vertical load carried by joints 69 to 72 = 1.25 (5.43 + 2.015)/8

= 1.163 kN

Vertical load carried by (Due to horizontal brac.) joints 69 to 72 = 1.25 (0.835)/4

= 0.261 kN

Wind load carried by joints 65 to 72 = 1.25 (5.332 + 2.94)/8

= 1.29 kN

Wind load carried by joints 69 to 72 due to Hor. Brac = 1.25 (1.03) / 4

= 0.332 kN

Panel 18 : Leg: ISA 200 x 200 x 15 @ 0.454 kN/m

Self weight of legs = 4 x 4.04 x 0.454

= 7.34 kN

Wind obstruction area = 2 x 4.04 x 0.2

= 1.616 m^2

Wind load on leg = 1.616 x 3973.7

= 6.42 kN

Diag. Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of diagonal brac. = 8 x 5.67 x 0.058

= 2.63 kN

Wind load on Diag. Brac = 2 x 5.67 x 0.065 x 3973.7

= 2.93 kN

Horizontal Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of Hor. brac. = 4 x 4.4 x 0.058

= 1.02 kN

Wind load on Hor. Brac = 1 x 4.4 x 0.065 x 3973.7

= 1.14 kN

Vertical load carried by joints 69 to 79 except 74, 76, 78, 80 = 1.25 (7.34 + 2.68)/8

= 1.56 kN

Vertical load carried by joints 73, 75, 77, 79 (Due to horizontal brac.) = 1.25 (1.02)/4

= 0.32 kN

Wind load carried by joints 65 to 79 except 74, 76, 78, 80 = 1.25 (6.42 + 2.93)/8

= 1.46 kN

Wind load carried by joints 73, 75, 77, 79 due to Hor. Brac = 1.25 (1.14) / 4

= 0.356 kN

Panel 19: Leg: ISA 200 x 200 x 15 @ 0.454 kN/m

Self weight of legs = 4 x 4.04 x 0.454

= 7.34 kN

Wind load on leg = 2 x 4.04 x 0.2 x 3710.2

= 6 kN

Diag. Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of diagonal brac. = 8 x 4.79 x 0.058

= 2.22 kN

Wind load on Diag. Brac = 2 x 4.79 x 0.065 x 3710.2

= 2.31 kN

Horizontal Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of Hor. brac. = 4 x 5.2 x 0.058

= 1.21 kN

Wind load on Hor. Brac = 1 x 5.2 x 0.065 x 3710.2

= 1.254 kN

Vertical load carried by joints 73 to 88 except 74, 76, 78, 80, 82, 84, 86, 88 = 1.25 (7.34 + 2.22)/8

= 1.494 kN

Vertical load carried by joints 81, 83, 85, 87 (Due to horizontal brac.) = 1.25 (1.21)/4

= 0.378 kN

Wind load carried by joints 73, 75, 77, 79, 81, 83, 85, 87 = 1.25 (6 + 2.31)/8

= 1.3 kN

Wind load carried by joints 81, 83, 85, 87 due to Hor. Brac = 1.25 (1.254) / 4

= 0.392 kN

Panel 20: Leg: ISA 200 x 200 x 15 @ 0.454 kN/m

Self weight of legs = 4 x 4.04 x 0.454

= 7.34 kN

Wind load on leg = 2 x 4.04 x 0.2 x 3658.4

= 5.91 kN

Diag. Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of diagonal brac. = 8 x 5.02 x 0.058

= 2.33 kN

Wind load on Diag. Brac = 2 x 5.02 x 0.065 x 3658.4

= 2.39 kN

Vertical load carried by joints 81, 83, 85, 87, 89, 90, 91, 92 = 1.25 (7.34 + 2.33)/8 = 1.51 kN

Wind load carried by joints 81, 83, 85, 87, 89, 90, 91, 92 = 1.25 (5.91 + 2.39)/8 = 1.3 kN

Computation of loads at different joints are made panel by panel and the nodal loads are superposed and tabulated in the following sections. The tower is symmetrically loaded in the XY plane and so nodal loads are tabulated for joints which are in the front plane.

Calculation of forces in the members

By symmetry the two planes are identical the front plane is analysed and forces are resolved. The tower is analysed for three basic static loads

- Self weight of the tower
- Superimposed load from Hemispherical Dome
- Wind Loads

- Acting parallel to face
- Acting diagonal to the tower

Tabulation of joint forces

Joint No	Self WT.(kN)	Wind load (kN)	Joint No	Self WT (kN)	Wind load (kN)
1	5 + 0.551 = 5.551	0.802 + 1.93 = 2.732	2	0.551	0.802 + 1.93 = 2.732
5	0.551 + 0.46 = 1.011 6.562	0.802 + 0.666 = 1.468	6	0.551 + 0.46 = 1.011 1.562	0.802 + 0.666 = 1.468
9	0.46 + 0.092 + 0.46 = 1.012 7.574	0.666 + 0.136 + 0.666 = 1.468	10	0.46 + 0.092 + 0.46 = 1.012 2.574	0.666 + 0.136 + 0.666 = 1.468
13	0.46 + 0.092 + 0.46 = 1.012 8.586	0.666 + 0.136 + 0.666 = 1.468	14	0.46 + 0.092 + 0.46 = 1.012 3.586	0.666 + 0.136 + 0.666 = 1.468
17	0.46 + 0.092 + 0.46 = 1.012 9.598	0.666 + 0.136 + 0.666 = 1.468	18	0.46 + 0.092 + 0.46 = 1.012 4.598	0.666 + 0.136 + 0.666 = 1.468
21	0.46 + 0.092 + 0.46 = 1.012 10.61	0.666 + 0.136 + 0.63 = 1.432	22	0.46 + 0.092 + 0.46 = 1.012 5.61	0.666 + 0.136 + 0.63 = 1.432
25	0.46 + 0.092 + 0.46 = 1.012 11.622	0.63 + 0.128 + 0.63 = 1.388	26	0.46 + 0.092 + 0.46 = 1.012 6.622	0.63 + 0.128 + 0.63 = 1.388
29	0.46 + 0.092 + 0.46 = 1.012 12.634	0.63 + 0.128 + 0.63 = 1.388	30	0.46 + 0.092 + 0.46 = 1.012 7.634	0.63 + 0.128 + 0.63 = 1.388
33	0.46 + 0.092 + 0.46 = 1.012 13.646	0.63 + 0.128 + 0.63 = 1.388	34	0.46 + 0.092 + 0.46 = 1.012 8.646	0.63 + 0.128 + 0.63 = 1.388
37	0.46 + 0.092 + 0.46 = 1.012 14.658	0.63 + 0.128 + 0.63 = 1.388	38	0.46 + 0.092 + 0.46 = 1.012 9.658	0.63 + 0.128 + 0.63 = 1.388
41	0.46 + 0.092 + 0.46 = 1.012 15.67	0.63 + 0.128 + 0.63 = 1.34	42	0.46 + 0.092 + 0.46 = 1.012 10.67	0.63 + 0.128 + 0.63 = 1.34
45	0.46 + 0.092 + 0.46 = 1.012	0.582 + 0.103 + 0.582 = 1.267	46	0.46 + 0.092 + 0.46 = 1.012	0.582 + 0.103 + 0.582 =

	16.682			11.682	1.267
49	0.46 + 0.092 + 0.46 = 1.012 17.694	0.582 + 0.103 + 0.582 = 1.267	50	0.46 + 0.092 + 0.46 = 1.012 12.694	0.582 + 0.103 + 0.582 = 1.267
53	0.46 + 0.092 + 0.46 = 1.012 18.706	0.582 + 0.103 + 0.582 = 1.267	54	0.46 + 0.092 + 0.46 = 1.012 13.706	0.582 + 0.103 + 0.582 = 1.267
57	0.46 + 0.092 + 0.46 = 1.012 19.718	0.582 + 0.103 + 0.582 = 1.267	58	0.46 + 0.092 + 0.46 = 1.012 14.718	0.582 + 0.103 + 0.582 = 1.267
61	0.46 + 0.092 + 1.135 = 1.687 21.405	0.582 + 0.103 + 1.28 = 1.965	62	0.46 + 0.092 + 1.135 = 1.687 16.405	0.582 + 0.103 + 1.28 = 1.965
65	1.135 + 0.158 + 1.163 = 2.456 23.861	1.28 + 0.197 + 1.29 = 2.767	66	1.135 + 0.158 + 1.163 = 2.456 18.861	1.28 + 0.197 + 1.29 = 2.767
69	1.163 + 0.261 + 1.56 = 2.984 26.845	1.29 + 0.322 + 1.46 = 3.072	70	1.163 + 0.261 + 1.56 = 2.984 21.845	1.29 + 0.322 + 1.46 = 3.072
73	1.56 + 0.32 + 1.494 = 3.374 30.219	1.46 + 0.356 + 1.3 = 3.116	75	1.56 + 0.32 + 1.494 = 3.374 25.219	1.46 + 0.356 + 1.3 = 3.116
81	1.494 + 0.378 + 1.51 = 3.382 33.601	1.3 + 0.392 + 1.3 = 2.99	83	1.494 + 0.378 + 1.51 = 3.382 28.601	1.3 + 0.392 + 1.3 = 2.99
89	1.51 35.111	1.3	90	1.51 30.111	1.3

Panel 15: 1. Considering self weight of the tower

The leg ISA 150 x 150 x 12 will be maximum stressed in this panel. So this panel is chosen. The self weight acting on joints 61 and 62 is taken.

The leeward leg 2 will be in compression and also the windward leg 1

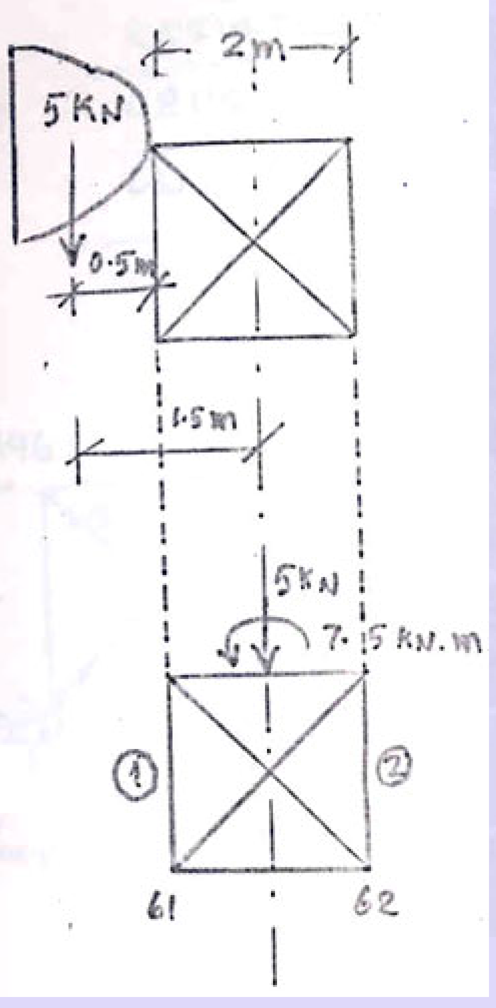
$F_1 = F_2 = 16.405$ kN (compression)

2. Considering superimposed load from hemispherical dome:

The front plane takes half the self weight = 5kN

The self weight of the dome will create a moment with respect to centre of planar truss. The eccentric load of 5 kN is transferred as a concentric load of 5 kN acting at the centre of planar truss and an anticlockwise moment of 7.5 kN.m as shown. Due to self weight both the legs F_1 and F_2 will be in compression

$$F_1 = F_2 = 2.5 \text{ kN (compression)}$$



The moment will cause compression on the windward side and tension on the leeward side.

$F_1 = 7.5 / 2 = 3.75$ kN (compression)

$F_2 = 7.5 / 2 = 3.75$ kN (tension)

Net force on F_1 = 3.75 + 2.5 = 6.25 kN (compression)

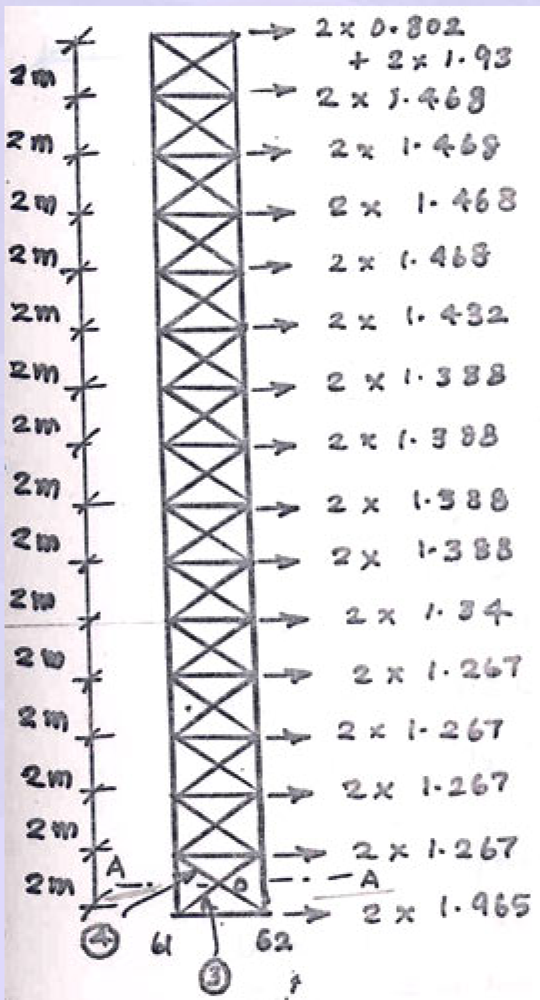
Net force on F_2 = -3.75 + 2.5 = 1.25 kN (tension)

The moment due to dome and self weight are carried entirely by legs.

3. Considering wind load condition

(i) Wind parallel to the face of the frame

The sum of the wind forces upto panel 15 and also the bending moment due to wind load about point 0 (the point of intersection of Diag. Brac.) is taken



Total wind load above the level 'AA'

F_{LAT1} = 2 x 0.802 + 2 x 1.93 + 4 x 2 x 1.468 + 2 x 1.432 + 4 x 2 x 1.388 + 2 x 1.34 + 4 x 2 x 1.267

F_{LAT1} = 43.992 kN

Moment due to wind

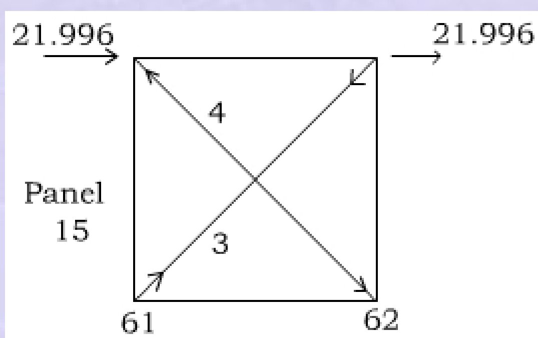
M_{W1} = (1.604 + 3.86) x 29 + 2.936 x 27 + 2.936 x 25 + 2.936 x 23 + 2.936 x 21 + 2.864 x 19 + 2.776 (17 + 15 + 13 + 11) + 2.68 x 9 + 2.534 (7 + 5 + 3 + 1)

M_{W1} = 714.85 kN.m

This external wind moment has to be resisted by internal couple. this moment will cause tension of the windward leg and comp on the leeward leg

$F_1 = M_{W1} / 2$ = 714.85 / 2 = 357.43 kN

F_1 = 357.43 kN (tension) F_2 = 357.43 kN (compression)



The lateral force of 43.992 kn is shared by the diagonal bracings equally and the tension diagonal is considered as effective taking moment about joint 62

$43.992 = \sqrt{2}\, F_3$

F_3 = 31.11 kN tension

F_4 = 31.11 kN compression

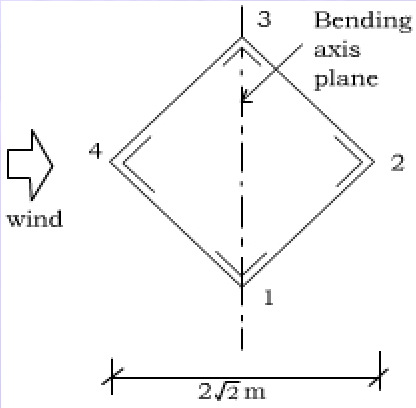
(**ii) Wind wards acting along diagonal:**

when the wind is parallel to the diagonal, the wind pressure coeff. is taken 1.2 times that of parallel to the plane Ref. clause 6.3.3.5 P.47 - IS 875

However the wind pressure on hte dish is reduced as the wind is at 45^o to the front of the dish.

Wind pressure on the dish = 2 x 3.86 x Sin 45^o

= 5.46 kN



Considering the tower as a space frame:

The wind load on the four joints together can be obtained. By multiplying the loads by 1.2

So total horizontal load due to wind

$F_{LAT\ 2}$ = 5.46 + 1.2 x 2 (43.992 - 3.86)

$F_{LAT\ 2}$ = 101.78 kN

Similarly the bending moment of all the wind forces along the diagonal about point 0

M_{W2} = 1.2 x 2 {714.85 - (3.86 x 29)} + 5.46 x 29

M_{W2} = 1605.32 kN.m Since the legs are upright, the horizontal force is registered by the braces and the forces in the braces will be equal and opposite.

The forces have to be resolved in the horizontal plane and then parallel to the diagonal.

Let F_D = force in each brace (tension or compression)

The total force from braces in the horizontal plane along the tower diagonal is

$= 8\ F_D \cos45^o . \sin45^o$

$= 4\ F_D$

Equilibrium in the horizontal direction gives

$4\ F_D = 101.78$ kN

$F_D = 25.45$ kN

This value is less than that of case 1. Therefore the forces in braces are controlled by the load condition wind parallel to the frame. The bending moment is resisted by the pair of extreme legs 2 and 4. Forces in legs 3 and 1 will be zero as they lie in the bending axis Ref. Fig.

$F_1 = F_3 = 0$

$F_2 = M_{W2} / 2\sqrt{2} \qquad = 1605.32 / 2\sqrt{2}$

$F_2 = 567.57$ kN (compression)

$F_4 = 567.57$ kN (tension)

Maximum compressive force on the leg = 567.57 + 16.405 - 1.25

= 582.73 kN

Leg ISA 150 x 150 x 12 @ 0.272 kN / m

$A = 3459\ mm^2$; $r_{min} = 29.3$ mm

$L_{eff} = 0.85 \times 2000 = 1700mm$; $L_{eff} / r_y = 1700 / 29.3 = 58.02$

σ_{ac} from table 5.1 = 124 N/mm^2 can be raised by 25%. Since wind is considered: σ_{ac} = 1.25 x 124 = 155 N/mm^2

Actual stress σ_c = (582.73 x 10^3) / 3459 = 168.5 N/mm^2

Diag. Brac: The tension member is considered effective.

Force in the bracing = 31.11 kN

Size ISA 50 x 50 x 6 mm A = 568 mm^2

Check the adequacy of the section as a tension member

Panel 20: Leg: ISA 200 x 200 x 15 @ 0.454 kn/m

1. Self weight acting at the bottom most panels

$F_1 = F_2$ = 30.111 kn (compression)

The leg is checked at the mid height as buckling will occur midway between the nodes

2. Considering superimposed load from hemispherical dome

Due to moment F_1 = 7.5 / 5.6 = 1.34 kn (compression)

F_2 = 1.34 kN (tension)

Due to self weight F_1 = 2.5 kN (compression)

F_2 = 2.5 kN (compression)

Net forces F_1 = 1.34 + 2.5 = 3.84 kN (compression)

F_2 = -1.34 + 2.5 = 1.16 kN (compression)

3. Considering wind load condition:

(a) Wind parallel to the face of the frame:

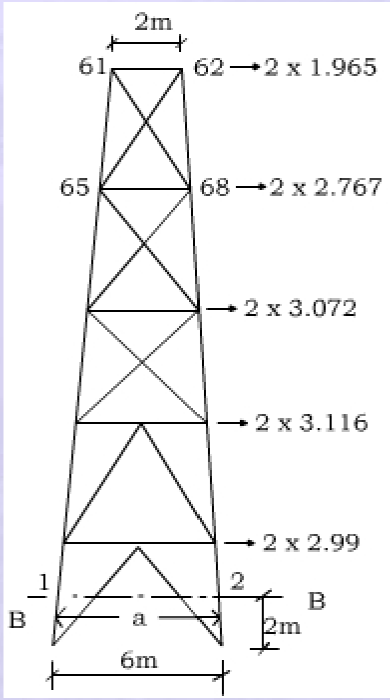
Total wind load above level 'BB'

$F_{LAT\ 3} = 43.992 + 2 \times 1.965 + 2 \times 2.767 + 2 \times 3.072 + 2 \times 3.116 + 2 \times 2.99$

$F_{LAT\ 3} = 71.812$ kN

$M_{W3} = (1.604 + 3.86) \times 48 + 2.936\ (46 + 44 + 42 + 40) + 2.864 \times 38 + 2.776\ (36 + 34 + 32 + 30) + 2.68 \times 28 + 2.534\ (26 + 24 + 22 + 20) + 3.93 \times 18 + 5.534 \times 14 + 6.144 \times 10 + 6.232 \times 6 + 5.98 \times 2$

$M_{W3} = 1809.704$ kN.m



Force in the legs and braces

$F_1 = M_{W3} / a = 1809.704 / 5.6 = 323.16$ kN

$F_1 = 323.16$ kN (tension)

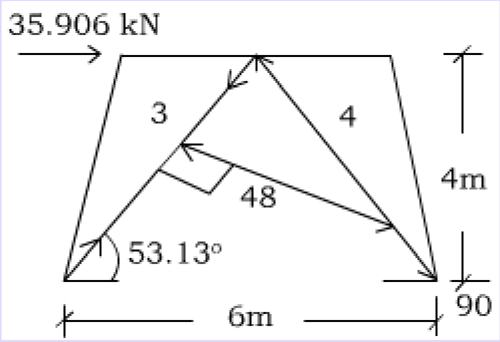
$F_2 = 323.16$ kN (compression)

The lateral force of 71.812 kN is shared by the diagonal bracings equally and the tension diagonal is considered effective taking moment about joint 90

$35.906 \times 4 = F_3 \times 4.8$

$F_3 = 29.92$ kN (tension)

$F_4 = 29.92$ kN (compression)



(b) Wind acting parallel to the diagonal:

Wind load is increased by 1.2 times that of parallel to the frame. P.47 code. However wind pressure on the dish is reduced as the wind is 45^o to the front of the dish

Wind pressure on dish = 5.46 kN

Considering the tower as a space frame the wind load on the four joints together can be obtained by multiplying the load by 1.2

So, total horizontal load due to wind

$F_{LAT\ 4}$ = 5.46 + 1.2 x 2 (71.812 - 3.86)

$F_{LAT\ 4}$ = 168.55 kN

Similarly the bending moment of all the wind forces along section 'BB'

M_{W4} = 1.2 x 2 {1809.704 - (3.86 x 48)} + 5.46 x 48

M_{W4} = 4160.7 kN.m

The horizontal forces are resisted by the braces these forces have to be resolved in the horizontal plane and then parallel to the diagonal.

Let F_d be the force in each brace tension or compression. The total force is resisted by these 8 braces

$4F_d$ cos 53.13^o (cos 37.47^o + cos 52.59^o) = 168.55

F_d = 50.12 kN (tension or compression)

This is more than the value with wind parallel to the frame. The bending moment M_{W4} is resisted by the pair of extreme legs which does not lie on the bending axis

$F_1 = F_3 = 0$

$F_2 = M_{W4} / a\sqrt{2} = 4160.7 / 5.6\sqrt{2} = 525.4$ kN

$F_2 = 525.4$ kN (compression)

$F_4 = 525.4$ kN (tension)

Maximum compressive force will be on leg 2

$= 30.111 + 1.16 + 525.4$

$F_2 = 556.67$ kN (compression)

Leg ISA 200 x 200 x 15 @ 0.454 kN/m

$A = 5780$ mm^2; $r_y = 39.1$ mm

$L_{ef} = 0.85 \times 4040 = 3434$mm

$L_{ef} / r_y = 3434 / 39.1 = 87.83$ Refer Table 5.1

$\sigma_{ac} = 86$ N / mm^2

Since wind is considered allowable stresses are raised by 25%. So $\sigma_{ac} = 1.25 \times 86 = 107.5$ N / mm^2

Actual stress $\sigma_c = 556.67 / 5780 = 96.31$ N / mm^2

σ_{ac} and σ_c Safe

7. TRANSMISSION TOWERS

7.1 Introduction

In every country, developed and developing, the elastic power consumption has continued to rise, the rate of growth being greater in the developing countries on account of the comparatively low base. This in turn had led to the increase in the number of power stations and their capacities and consequent increase in power transmission lines from the generating stations to the load centres. Interconnections between systems are also increasing to enhance reliability and economy. The transmission voltage, while dependent on th quantum of power transmitted, should fit in with the long-term system requirement as well as provide flexibility in system operation. It should also conform to the national and international standard voltage levels.

In the planning and design of a transmission line, a number of requirements have to be met. From the electrical point of view, the most important requirement is insulation and safe clearances to earthed parts. These, together with the cross-section of conductors, the spacing between conductors, and the relative location of ground wires with respect to the conductors, influence the design of towers and foundations. The conductors, ground wires, insulation, towers and foundations constitute the major components of a transmission line.

7.2 Material properties, clearances and tower configurations

7.2.1 Material properties

Classification of steel

The general practice with reference to the quality of steel is to specify the use of steel for tower members, although some authorities have instead specified the use of steel manufactured by either the open hearth or electric furnace process for tower members, although some athorities have instead specified the use of steel manufactured by either the open hearth or electric furnace process. The usual standards specified are ASTM A-7, BSS 15, and German Steel Standard St 37. IS: 226-1975, Specification for structural Steel (Revised), is currently adopted in India.

In so far as standard structural steel is concerned, reference to IS: 226-1975 shows that

Steel manufactured by the open-hearth, electric, basic oxygen or a combination of the processes is acceptable for structural use and that in case any other process is employed, prior approval of the purchaser should be obtained.

In addition to standard structural steel (symbol A), high tensile steel conforming to IS: 226-1975 may be used for transmission line towers for greater economy. The chemical composition and mechanical properties of steel covered by IS: 226-1975 for structural steel and IS: 961-1975 for high tensile steel are shown in Tables 7.1 to 7.4.

Suitability for welding

The standard structural mild steel is suitable for welding, provided the thickness of the material does not exceed 20mm. When the thickness exceeds 20mm, special precautions such as double Vee shaping and cover plates may be required.

St 58-HT is intended for use in structures where fabrication is done by methods other than welding. St 55-HTw is used where welding is employed for fabrication.

In the past, transmission line structures in India were supplied by firms like Blaw Knox, British Insulated Callender Cables (BICC), etc. from the United Kingdom. Later, towers from SAE, Italy, were employed for some of the transmission lines under the Damodar Valley Corporation. In recent times, steel from the USSR and some other East European countries were partly used in the transmission line industry. Currently, steel conforming g to IS: 961 and IS: 226 and manufactured in the country are almost exclusively use for towers.

A comparison of mechanical properties of standard and high tensile steels conforming to national standards of the countries mentioned above is given in Table 7.5.

Properties of structural steel

A typical stress-strain curve of mild steel is shown in Figure 7.1. Steels for structural use are classified as: Standard quality, high strength low carbon steel and alloy steel. The various properties of steel will now be briefly discussed.

Behavior up to elastic limit

Table 7.1 Chemical composition

Constituent	Percent (Max)		
	Mild steel	High tensile steel	
		St 58-HT	St 55-HT
Carbon for thickness/dia 20mm and below for thickness/dia over 20mm	0.23 0.25	0.27	0.20
Sulphur	0.055	0.055	0.055
Phosphorus	0.055	0.055	0.055

Table 7.2 Mechanical properties of mild steel

Class of steel product	Nomial thickness/diameter mm	Tensile strength kgf/mm2	Yield stress, Min. kgr/mm2	Percentage elongation Min.
Plates, sections (angles, tees, beams, channels, etc.) and flats	$6 \le x \le 20$	42-54	26.0	23
	$20 < x \le 40$	42-54	24.0	23
	40 < x	42-54	23.0	23

Up to a well-defined point, steel behaves as a perfectly elastic material. Removal of stress at levels below the yield stress causes the material to regain its unstressed dimension. Figure 7.2 shows typical stress-strain curves for mild steel and high tensile steel. Mild steel has a definite yield point unlike the high-tensile steel; in the latter case, the yield point is determined by using 0.2 percent offset[1].

Table 7.3 Mechanical properties of high tensile steel St 58-HT

Class of steel product	Nomial thickness/diameter mm	Tensile strength kgf/mm2	Yield stress, Min. kgr/mm2	Percentage elongation Min.
Plates, sections, flats and bars	$6 \leq X \leq 28$	58	36	20
	$28 < X \leq 45$	58	35	20
	$45 < X \leq 63$	58	33	20
	63<x	55	30	20

Table 7.4 Mechanical properties of high tensile steel St 55-HTw

Class of steel product	Nomial thickness/diameter mm	Tensile strength kgf/mm2	Yield stress, Min. kgr/mm2	Percentage elongation Min.
Plates, sections, flats and bars	$6 < X \leq 16$	55	36	20
	$16 < X \leq 32$	55	35	20
	$32 < X \leq 63$	52	34	20
	$63 < X$	50	29	20

Table 7.5 Comparison of mechanical properties of standard and high tensile steels

Sl. No.	Origin	High tensile steel				Standard steel			
		No. of standard	Ultimate tensile stress kg/mm2	Minimum yield stress kg/mm2	Minimum elongation %	No. of standard	Ultimate tensile stress kg/mm2	Minimum yield stress kg/mm2	Minimum elongation %
1	India	IS : 961 1975	58	36	20	IS :226 1975	42-54	23-26	23
2	USSR	CT5 20L2	50-62	28	15-21	CT4	45-52	26	19-25
3	Italy	UNI	50-60	34-38	22	UNI	37-45	24-28	25
4	UK	BS : 548 1934	58-68	30-36	14	BS : 15 1948	44-52	23.2-24	16-20

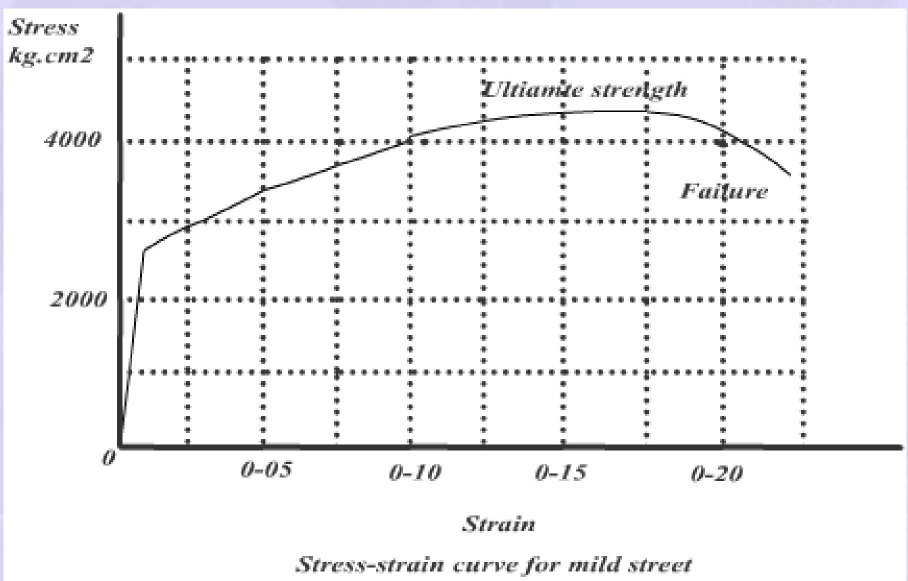


Figure 7.1 Stress-strain curve for mild steel

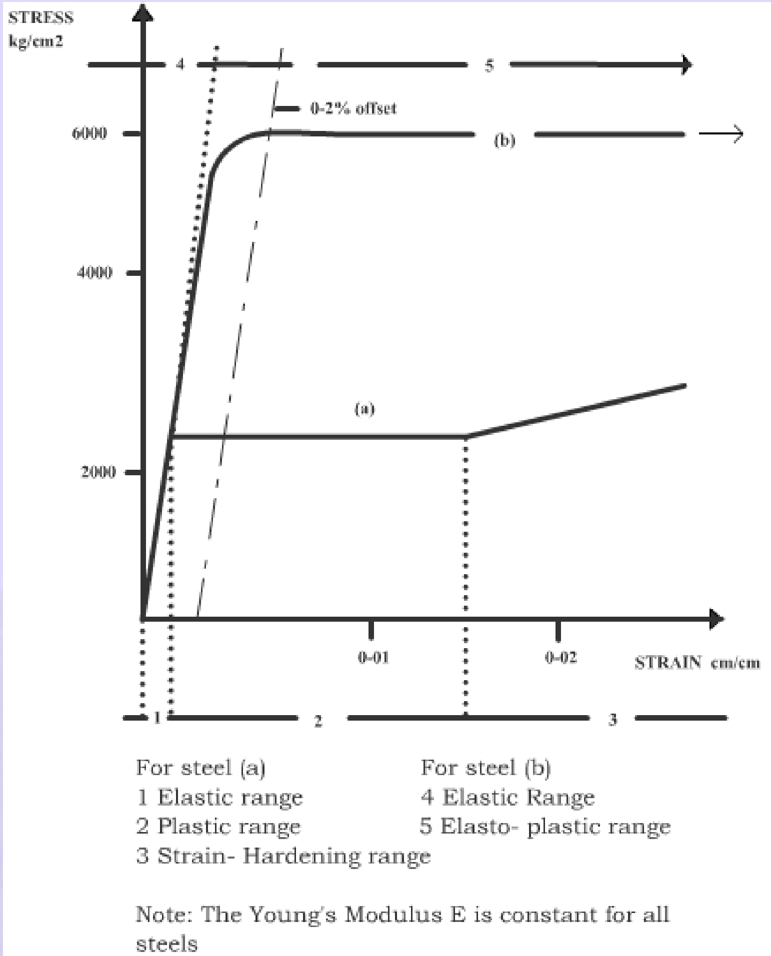


Figure 7.2 Stress-strain curves for various types of steel

Tensile strength

The applied stress required to cause failure is greater than the yield stress and is generally defined as tensile strength.

Ductility

This is important property of steel which enables it to undergo large deformations after yield point without fracture.

Design span lengths

In transmission line calculations, the following terms are commonly used

1. Basic or normal span
2. Ruling oe equivalent span
3. Average span
4. Wind span
5. Weight span

Table 7.6a properties of SAIL-MA steels (kg/cm^2)

	Type of steel			
	IS:226 Mild steel	SAIL-MA 300	SAIL- MA 350	SAIL-MA 410
Yield strength	2,600	3,100	3,600	4,200
Allowable stress in tension	1,500	1,850	2,125	2,450
Allowable stress in bending section other than palte girders	1,650	1,950	2,285	2,650
Allowable stress in shear	1,100	1,350	1,575	1,800
Allowable stress in bearing	1,890	2,300	2,675	3,100

Table 7.6b Allowable stresses of SAIL-MA steel in axial copression (kg/cm^2)

L/r	Type of steel			
	IS:226 Mild steel	SAIL-MA 300	SAIL- MA 350	SAIL-MA 410
0	1,250	1,537	1,785	2,083
20	1,239	1,520	1,762	2,055
40	1,203	1,464	1,685	1,946
60	1,130	1,317	1,522	1,710
80	1,007	1,155	1,255	1,352
100	840	920	960	1,005

Basic or normal span

The normal span is tha most economic span for which the line is designed over level ground, so that the requisite ground clearance is obtained at the maximum specified temperature

Ruling span

The ruling span is the assumed design span that will produce, between dead ends, the best average tension throughout a line of varying span lengths with changes in temperature and loading. It is the weighted average of the varying span lengths, calculated by the formula:

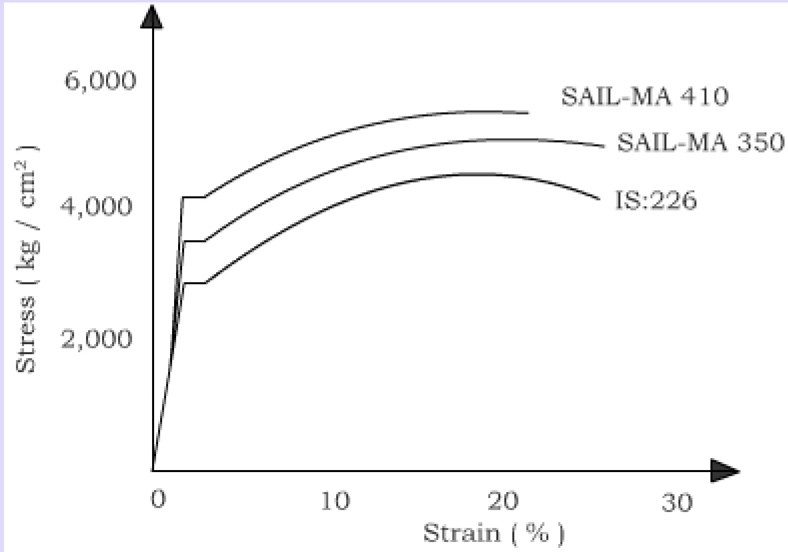


Figure 7.3 Stress strain curves of SAIL-MA 350 and 410 and IS: 226 steels

$$\text{Ruling span} = \sqrt{\frac{l_1^3 + l_2^3 + \ldots . l_n^3}{l_1 + l_2 + l_n}}$$

Where l_1, l_2 ... l_n are the first, second and last span lengths in sections. The erection tension for any line section is calculated for this hypothetical span.

Tower spotting on the profile is done by means of a sag template, which is based on the ruling span. Therefore, this span must be determined before the template can be made.

The ruling span is then used to calculate the horizontal component of tension, which is to be applied to all the spans between the anchor points

Average span

The average span is the mean span length between dead ends. It is assumed that the conductor is freely suspended such that each individual span reacts to changes in tension as a single average span. All sag and tension calculations are carried out for the average span, on this assumption.

Wind span

The wind span is that on which the wind is assumed to act transversely on the conductors and is taken as half the sum of the two spans, adjacent to the support (Figure 7.4). In order to take full advantage of towers located on elevated ground, it is usual to allow a wind span of 10 to 15 percent in excess of the normal span. This additional strength can be used in taking a samll angle of deviation on an intermediate tower, where the actual wind span is less than the design wind span. The angle of deviation to be taken in such cases is approximately given by:

$$\theta = \frac{wl}{\pi T} x 180$$

Where w = total ind load per unit run of span length of all conductor carried by the tower,

l = difference between the wind span used for design and the actual wind span, and

T = the total maximum working tension of all conductors carried by the tower.

Weight span

The weight span is the horizontal distance between the lowest point of the conductors, on the two spans adjacent to the tower (figure 7.4). The lowest point is defined as the point at which the tangent to the sag curve, or to the sag curve produced, is horizontal. The weight span is used in the design of cross-arms.

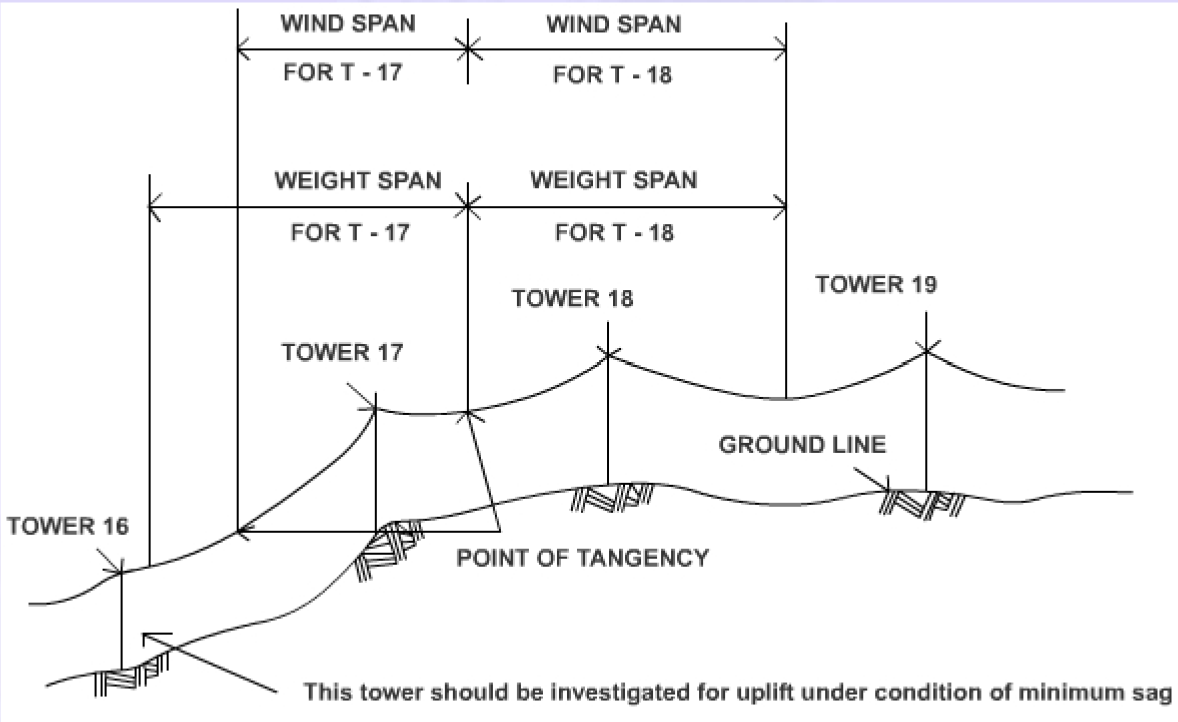


Figure 7.4 Wind span and weight span

7.2.2 Tower configurations

Depending upon the requirements of the transmission system, various line configurations have to be considered - ranging from single circuit horizontal to double circuit vertical structures and with single or V strings in all phases, as well as any combination of these.

The configuration of a transmission line tower depends on:

1. the length of the insulator assembly

2. the minimum clearances to be maintained between conductors, and between conductors and tower

3. the location of ground wire or wires with respect to the outermost conductor

4. the mid-span clearance required from considerations of the dynamic behaviour of conductors and lightning protection of the line

5. the minimum clearance of the lowest conductor above ground level.

The tower outline is determined essentially by three factors: tower height, base width, and top hamper width.

Determination of tower weight

The factors governing the height of the tower are:

1. Minimum permissible ground clearance (h_1)
2. Maximum sag (h_2)
3. Vertical spacing between conductors (h_3)
4. Vertical clearance between ground wire and top conductor (h_4)

Thus the total height of the tower is given by

$$H = h_1 + h_2 + h_3 + h_4$$

in the case of double circuit tower with vertical configuration of conductors (figure 7.5)

The calculation of sags (h_2) is covered later. The principles and practices in regard to the determination of ground clearance and spacings between conductors and between the ground wire and top conductor will now be outlined.

Minimum permissible ground clearance

For safety considerations, power conductors along the route of the transmission line should maintain requite clearance to ground in open country, national highways, rivers, railways tracks, telecommunication lines, other power lines, etc.., as laid down in the Indian Electricity Rules, or Standards or codes of practice in vogue.

Rule 77(4) of the Indian Electricity Rules, 1956, stipulates the following clearances above ground of the lowest point of the conductor:

For extra- high voltage lines, the clearance above ground shall not be less than 5.182 metres plus 0.305 metres for every 33,000 volts or part there of by which the voltage of the line exceeds 33,000 volts.

Accordingly, the values for the various voltages, 66kV to 400 kV, are:

66kV - 5.49m

132kV - 6.10m

220kV - 7.01m

400kV - 8.84m

The above clearances are applicable to transmission lines running in open country.

Power line crossings

In crossings over rivers, telecommunication lines, railway tracks, etc.., the following clearances are maintained:

1. Crossing over rivers

   a. Over rivers which are not navigable. The minimum clearance of conductor is specified as 3.05 over maximum flood level.

   b. Over navigable rivers: Clearances are fixed in relation to the tallest mast, in consultation with the concerned navigation authorities.

2. Crossing over telecommunication lines. The minimum clearances between the conductors of a power line and telecommunication wires are

   66 kV - 2,440mm

   132 kV - 2,740mm

   220 kV - 3,050mm

   400 kV - 4,880mm

3. Crossing over railway tracks: The minimum height over the rail level, of the lowest portion of any conductor under conditions of maximum sag, as stipulated in the regulations for Electrical Crossings of Railway Tracks, 1963, is given in Table 7.7.

4. Between power lines

   a. Between power lines L.T up to 66 kV - and 66 kV line 2.44m

   b. Between power lines L.T up to 132 kV - and 132kV line 2.75m

   c. Between power lines L.T up to 220kV - and 220kV line 4.55m

d. Between power lines L.T up to 400kV - and 400kV line 6.00m(Tentative)

Spacing of conductors

Considerable differences are found in the conductor spacings adopted in different countries and on different transmission systems in the same country.

Table 7.7 Minimum height of power conductors over railway tracks

1. For unelectrified tracks or tracks electrified on 1,500 volts D.C. system

	Broad gauge		Metre and Narrow gauge	
	inside station limits	outside station limits	inside station limits	outside
66 kV	10.3	7.9	9.1	6.7
132 kV	10.9	8.5	9.8	7.3
220 kV	11.2	8.8	10.0	7.6
400 kV	13.6	11.2	12.4	10.0

2. Tracks electrified on 25 kV A.C. system

	for Broad, Metre and Narrow gauge	
	Inside station limits	Outside station limits
66 kV	13.0	11.0
132kV	14.0	12.0
220 kV	15.3	13.3
400 kV	16.3	14.3

The spacing of conductors is determined by considerations which are partly mechanical. The material and diameter of the conductors should also be considered when deciding the spacing, because a smaller conductor, especially if made of aluminium, having a small weight in relation to the area presented to a crosswind, will swing out of the vertical plane father than a conductor of large cross-section. Usually conductors will swing synchronously (in phase) with the wind, but with long spans and small wires, there is always a possibility of the conductors swinging non-synchronously, and the size of the conductor and the maximum sag at the centre of the span are factors which should be taken in to account in determining the distance apart at which they should be strung.

There are a number of empirical formulae in use, deduced from spacings which have successfully operated in practice while research continues on the minimum spacings which could be employed.

The following formulae are in general use:

1. Mecomb's formula

Spacing in cm = 0.3048V + 4.010 $\frac{D}{w}\sqrt{S}$

Where V = Voltage in kV,

D = Conductor diameter in cm,

S = sag in cm, and

W = Weight of conductor in kg/m.

2. VDE (verbandes Deutscher electrotechnischer) formula

Spacing in cm = $7.5\sqrt{S}+\frac{V^2}{200}$

where S= sag in cm, and V= Voltage in kV.

3. Still's formula

Distance between conductors (cm)

$$= 50.8 + 1.8.14\,V + \left[\frac{l}{27.8}\right]^2$$

Where l = average span length in metres ,and V = line voltage between conductors in kV.

The formula may be used as a guide in arriving at a suitable value for the horizontal spacing for any line voltage and for the value spans between 60 and 335 meters

4. NESC, USA formula

Horizontal spacing in cm

$$= A + 3.681\sqrt{S + \frac{L}{\sqrt{2}}}$$

Where A= 0.762 cm per kV line voltage S = Sag in cm, and

L = Length of insulator string in cm

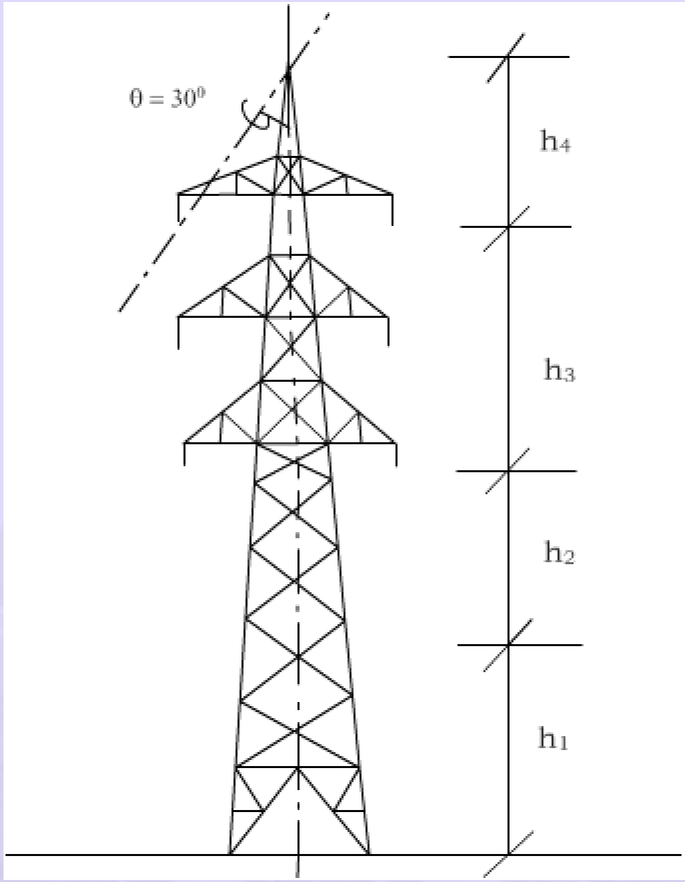


Figure 7.5 Determination of the tower height

5. Swedish formula

Spacing in cm = 6.5 $\sqrt{S}$ + 0.7E

Where S = Sag in cm, and E = Line voltage in kV

6. French formula

Spacing in cm = $8\sqrt{S+L}+\frac{E}{1.5}$

Where S = Sag in cm, L = Length of insulator string in cm, and

E = Line voltage in kV.

Offset of conductors (under ice-loading conditions)

The jump of the conductor, resulting from ice dropping off one span of an ice-covered line, has been the cause of many serious outages on long-span lines where conductors are arranged in the same vertical plane. The 'sleet jump' has been practically cleared up by horizontally offsetting the conductors. Apparently, the conductor jumps in practically a vertical plane, and this is true if no wind is blowing, in which cases all forces and reactions are in a vertical plane. In double circuit vertical configuration, the middle conductors are generally offset in accordance with the following formula:

$$\text{Offset in cm} = 60 + \text{Span in cm} / 400$$

The spacing commonly adopted on typical transmission lines in India are given in the table

Vertical clearance between ground wire and top conductor

This is governed by the angle of shielding, ie., the angle which the line joining the ground wire and the outermost conductor makes with the vertical, required for the interruption of direct lightning strokes at the ground and the minimum midspan clearance between the ground wire and the top power conductor. The shield angle varies from about 25^{o} to 30^{o}, depending on the configuration of conductors and the number of ground wires (one or two) provided.

Table 7.8 Vertical and horizontal spacings between conductors

Type of tower	Vertical spacing between conductors (mm)	horizontal spacing between conductors (mm)
1. 66 kV: Single circuit		
A(0-2^o)	1,030	4,040
B(2-30^o)	1,030	4,270
C(30-60^o)	1,220	4,880
2. 66 kV: Double circuit		
A(0-2^o)	2,170	4,270
B(2-30^o)	2,060	4,880
C(30-60^o)	2,440	6,000
3. 132 kV: Single circuit		
A(0-2^o)	4,200	7,140
B(2-15^o)	4,200	6,290
C(15-30^o)	4,200	7,150
D(30-60^o)	4,200	8,820
4. 132 kV: Double circuit		
A(0-2^o)	3,965	7,020
B(2-15^o)	3,965	7,320
C(15-30^o)	3,965	7,320
D(30-60^o)	4,270	8,540
5. 220 kV: Single circuit		
A(0-2^o)	5,200	8,500
B(2-15^o)	5,250	10,500
C(15-30^o)	6,700	12,600
D(30-60^o)	7,800	14,000
6. 220 kV: double circuit		
A(0-2^o)	5,200	9,900
B(2-15^o)	5,200	10,100
C(15-30^o)	5,200	10,500
D(30-60^o)	6,750	12,600
7. 400 kV: Single circuit horizontal configuration		
A(0-2^o)	7,800	12,760
B(2-15^o)	7,800	12,640
C(15-30^o)	7,800	14,000
D(30-60^o)	8,100	16,200

Determination of base width

The base width at the concrete level is the distance between the centre of gravity at one corner leg and the centre of gravity of the adjacent corner leg. There is a particular base width which gives the minimum total cost of the tower and foundations

Ryle has given the following formula for a preliminary determination of the economic base width:

$$B = 0.42\sqrt{M} \text{ or } 0.013\sqrt{m}$$

Where B = Base width in meters,

M = Overturning moment about the ground level in tonne-meters, and

M= Overturning moment about the ground level in kg.meters.

The ratio of base width to total tower height for most towers is generally about one-fifth to one-tenth from large-angle towers to tangent towers.

The following equations have been suggested9,based on the best fit straight line relationship between the base width B and $\sqrt{M}$

$$B = 0.0782\sqrt{M} + 1.0$$
$$B = 0.0691\sqrt{M} + 0.7$$

Equations are for suspension and angle towers respectively.

It should be noted that Ryle's formula is intended for use with actual external loads acting on the tower whereas the formulae in Equations take into account a factor of safety of 2.0.

Narrow-base towers are commonly used in Western Europe, especially Germany, mainly from way-leave considerations. British and American practices generally favor the wide base type of design, for which the total cost, of tower and foundations is a minimum. In the USA, a continuous wide strip of land called the ‘right of way’ has usually to be acquired along the line route. In Great Britain, the payments made for individual tower way-leaves are generally reasonably small and not greatly affected by tower base dimensions. Therefore, it has been possible to adopt a truly economical base width in both the United States and Great Britain.

A wider taper in the tower base reduces the foundation loading and costs but increases the cost of the tower and site. A minimum cost which occurs with a tower width, is greater with bad soil than with good soil. A considerable saving in foundation costs results from the use of towers with only three legs, the tower being of triangular section throughout its height. This form of construction entails tubular legs or special angle sections. The three-footing anchorage has further advantages, e.g., greater accessibility of the soil underneath the tower when the land is cultivated.

Determination of top hamper width

The width at top hamper is the width of the tower at the level of the lower cross-arm in the case of barrel type of towers (in double circuit towers it may be at the middle level) and waist line in the case of towers with horizontal configuration of conductors.

The following parameters are considered while determining the width of the tower at the top bend line:

1. Horizontal spacing between conductors based on the midspan flashover between the power conductor under the severest wind and galloping conditions and the electrical clearance of the line conductor to tower steel work.

2. The sloe of the legs should be such that the corner members intersect as near the center of gravity (CG) of the loads as possible. Then the braces will be least loaded. Three cases are possible depending upon the relative position of the CG of the loads and intersection of the tower legs as shown in Figure 7.6.

In Case (1) the entire shear is taken up by the legs and the bracings do not carry any stress. Case (2) shows a condition in which the resultant of all loads O' is below the inter-section of tower legs O. The shear here is shared between legs and bracings which is a desirable requirement for an economical tower design. In Case (3), the legs have to withstand greater forces than in cases (1) and (2) because the legs intersect below the center of gravity of the loads acting on the tower. This outline is uneconomical.

The top hamper width is generally found to be about one-third of the base width for tangent and light angle towers and about 1.35 of the base width for medium and heavy angle towers. For horizontal configurations, the width at the waistline is, however, found to vary from 1/1.5 to ½.5 of the base width.

7.2.3 Types of towers

Classification according to number of circuits

The majorities of high voltage double circuit transmission lines employ a vertical or near vertical configuration of conductors and single circuit transmission lines a triangular arrangement of conductors. Single circuit lines, particularly at 400kv and above, generally employ a horizontal arrangement of conductors. The arrangement of conductors and ground wires in this configuration is shown in Figure7.8

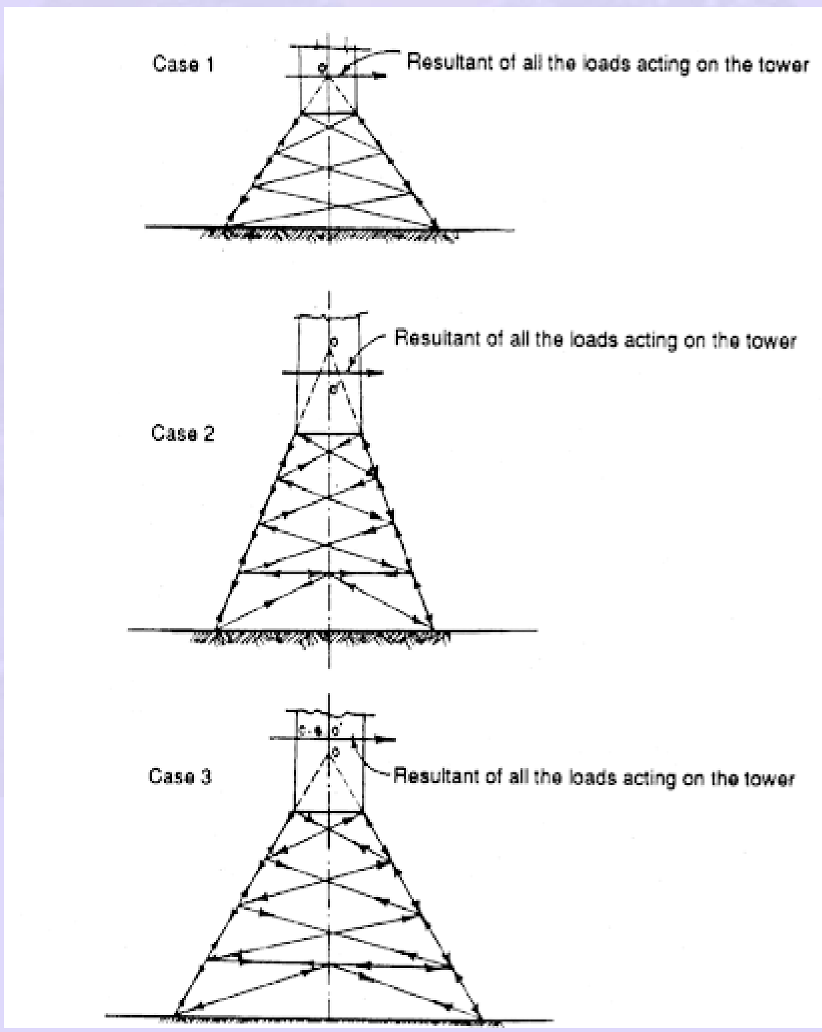


Figure 7.6 Relative position of C.G of loads intersection of tower legs

The number of ground wires used on the line depends on the isoceraunic level (number of thunderstorm days/hours per year) of the area, importance of the line, and the angle of coverage desired. Single circuit line using horizontal configuration generally employ tow ground wires, due to the comparative width of the configuration; whereas lines using vertical and offset arrangements more often utilize one ground wire except on higher voltage lines of 400 kv and above, where it is usually found advantageous to string tow ground wires, as the phase to phase spacing of conductors would require an excessively high positioning of ground wire to give adequate coverage.

Classification according to use

Towers are classified according to their use independent of the number of conductors they support.

A tower has to withstand the loadings ranging from straight runs up to varying angles and dead ends. To simplify the designs and ensure an overall economy in first cost and maintenance, tower designs are generally confined to a few standard types as follows.

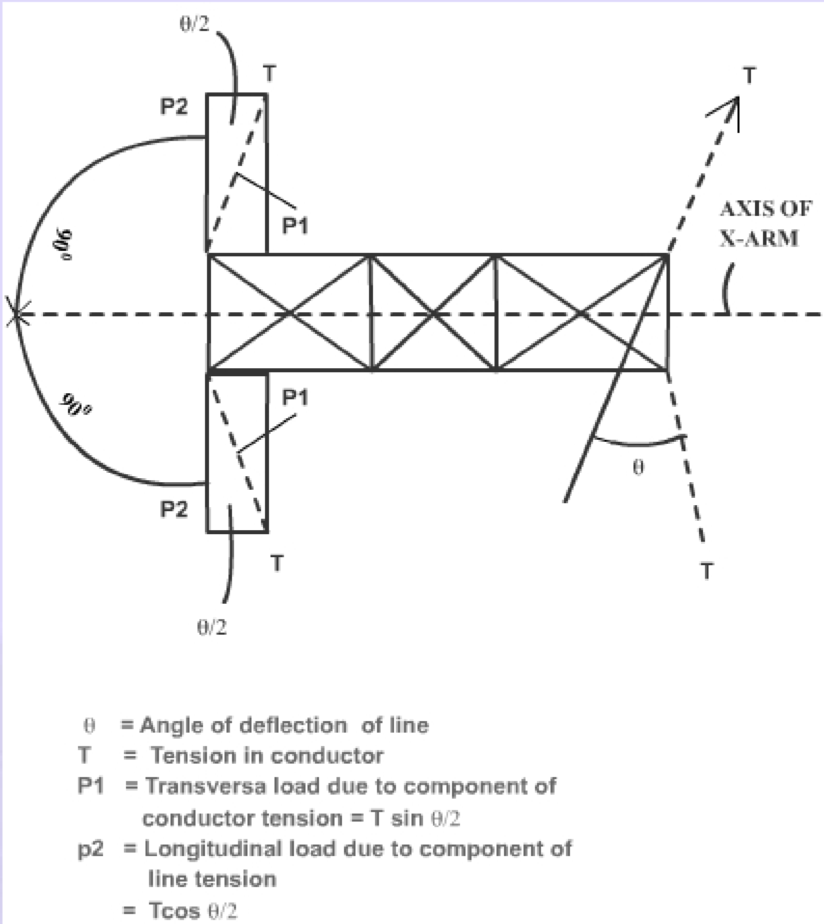


Figure 7.7 Orientation of tower in an angle

Tangent suspension towers

Suspension towers are used primarily on tangents but often are designed to withstand angles in the line up to tow degrees or higher in addition to the wind, ice, and broken-conductor loads. If the transmission line traverses relatively flat, featureless terrain, 90 percent of the line may be composed of this type of tower. Thus the design of tangent tower provides the greatest opportunity for the structural engineer to minimize the total weight of steel required.

Angle towers

Angle towers, sometimes called semi-anchor towers, are used where the line makes a horizontal angle greater than two degrees (Figure). As they must resist a transverse load from the components of the line tension induced by this angle, in addition to the usual wind, ice and broken conductor loads, they are necessarily heavier than suspension towers. Unless restricted by site conditions, or influenced by conductor tensions, angle towers should be located so that the axis of the cross-arms bisects, the angle formed by the conductors.

Theoretically, different line angles require different towers, but for economy there are a limiting number of different towers which should be used. This number is a function of all the factors which make up the total erected cost of a tower line. However, experience has shown that the following angle towers are generally suitable for most of the lines:

1. Light angle - 2 to 15^{o} line deviation
2. Medium angle - 15 to 30^{o} line deviation
3. Heavy angle - 30 to 60^{o} line deviation

(And dead end)

While the angles of line deviation are for the normal span, the span may be increased up to an optimum limit by reducing the angle of line deviation and vice versa. IS: 802(Part I)-1977 also recommends the above classification.

The loadings on a tower in the case of a 60^{0} angle condition and dead-end condition are almost the same. As the numbers of locations at which 60^{0} angle towers and dead-end towers are required are comparatively few, it is economical to design the heavy angle towers both for the 60^0 angle condition and dead-end condition, whichever is more stringent for each individual structural member.

For each type of tower, the upper limit of the angle range is designed for the same basic span as the tangent tower, so that a decreased angle can be accommodated with an increased span or vice versa.

In India, then angle towers are generally provided with tension insulator strings.

Appreciable economies can be affected by having the light angle towers (2 to 15^0) with suspension insulators, as this will result in lighter tower designs due to reduced longitudinal loads to which the tower would be subjected under broken-wire conditions because of the swing of the insulator string towards the broken span. It would be uneconomical to use 30^0 angle tower in locations where angle higher than 2^0 and smaller than 30^0 are encountered. There are limitations to the use of 2^0 angle towers at higher angles with reduced spans and the use of 30^0 angle towers with smaller angles and increased spans. The introduction of a 15^0 tower would bring about sizeable economies.

It might appear that the use of suspension insulators at angle locations would result in longer cross-arms so as to satisfy the clearance requirements under increased insulator swings because of the large line deviation on the tower. In such a case, it is the usual practice to counteract the excessive swing of insulator string by the use of counter weights (in some countries counter weights up to 250kg have been used) and thus keep the cross-arm lengths within the economic limits. It is the practice in Norway and Sweden to use suspension insulators on towers up to 20^0 angles and in France up to as much as 40^0. The possibilities of conductor breakdown in modern transmission lines equipped with reliable clamps, anti-vibration devices, etc., are indeed rare, and should the contingency of a breakdown arise, the problems do not present any more difficulties than those encountered in the case of plain terrain involving tangent towers over long stretches.

Calculation of counterweights

The calculation for the counterweights (Figure 7.8) to be added to limit the sing of the insulator string is quite simple and is illustrated below:

Let $\theta_1 =$ Swing of insulator string with-out counterweight.

$\theta_2 =$ Desired swing of insulator string (with suitable counterweight).

H = Total transverse load due to wind on one span of conductor and line deviation.

W_1 and W_2 = Weight of one span of the conductor, insulators, etc., corresponding to insulator swings $\theta 1$ and $\theta 2$ respectively

Now, $\tan\theta_1 = H / W_1$

and $\tan\theta_2 = H / W_2$

Therefore, the magnitude of the counterweight required to reduce the insulator swing from θ_1 to $\theta_2 = W_2 - W_1$.

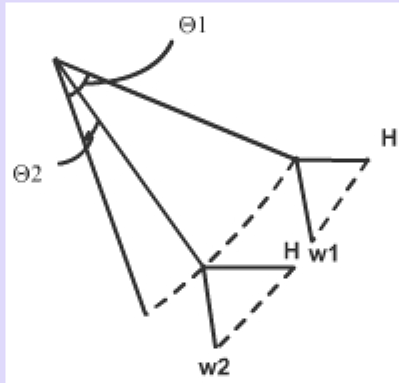


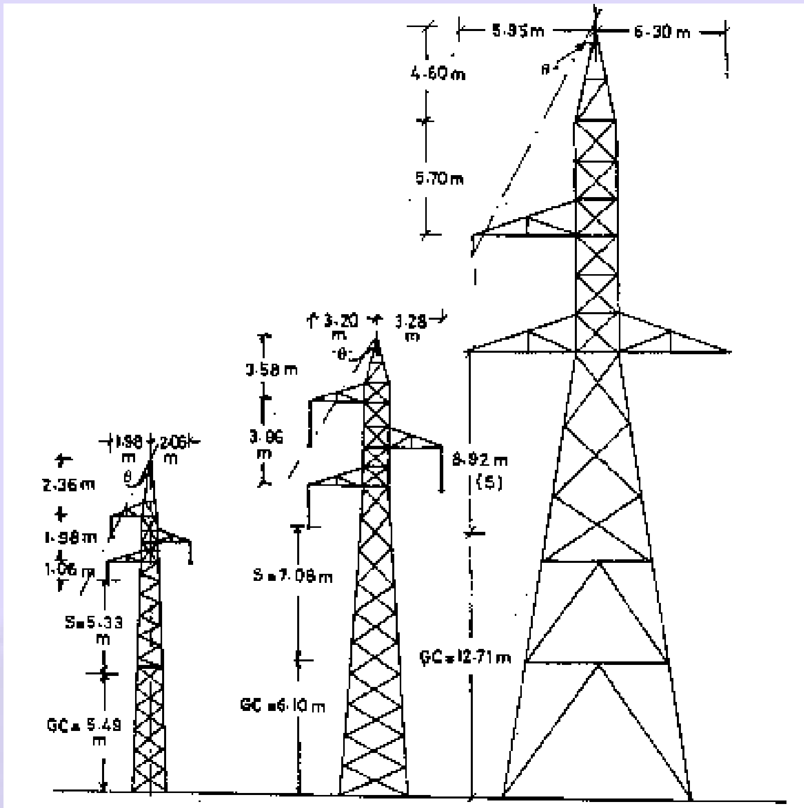
Figure 7.8 Insulator swing using counterweight

Unequal cross-arms

Another method to get over the difficulty of higher swings (if suspension strings are used for 150 line deviations) is to have unequal cross-arms of the tower. The main differences in the design aspects between this type of tower and the usual towers (with equal cross-arms) are:

1. The tower will be subjected to eccentric vertical loading under normal working conditions.
2. For calculation of torsional loads, the conductor on the bigger half of the cross-arm should be assumed to be broken, as this condition will be more stringent.

These features can be taken care of at the design stage. An example of unequal cross-arms widely used in the USSR. Note also the rectangular section used for the tower.

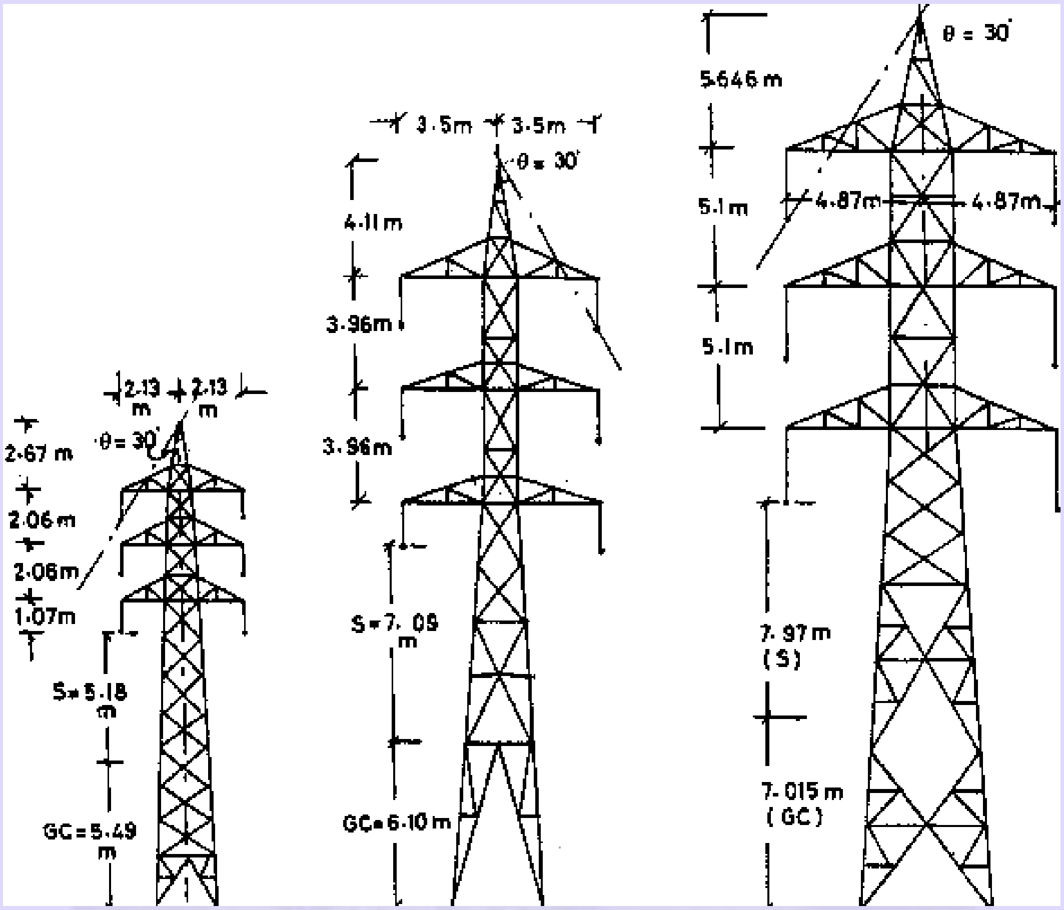


Normal span = 250m Conductor : 6/4.72mm Al + 7/1.57mm St Ground wire: 7/2.55mm (110kg/mm^2 quality)	Normal span = 350m Conductor : 30/3mm Al + 7/3mm St Ground wire: 7/3.15mm (110kg/mm^2 quality)	Normal span = 350m Conductor : 54/3.18mm Al + 7/3.18mm St Ground wire: 7/3.15mm (110kg/mm^2 quality)

Figure 7.9 66kV, 132 kV, 220 kV Single circuit tangent towers

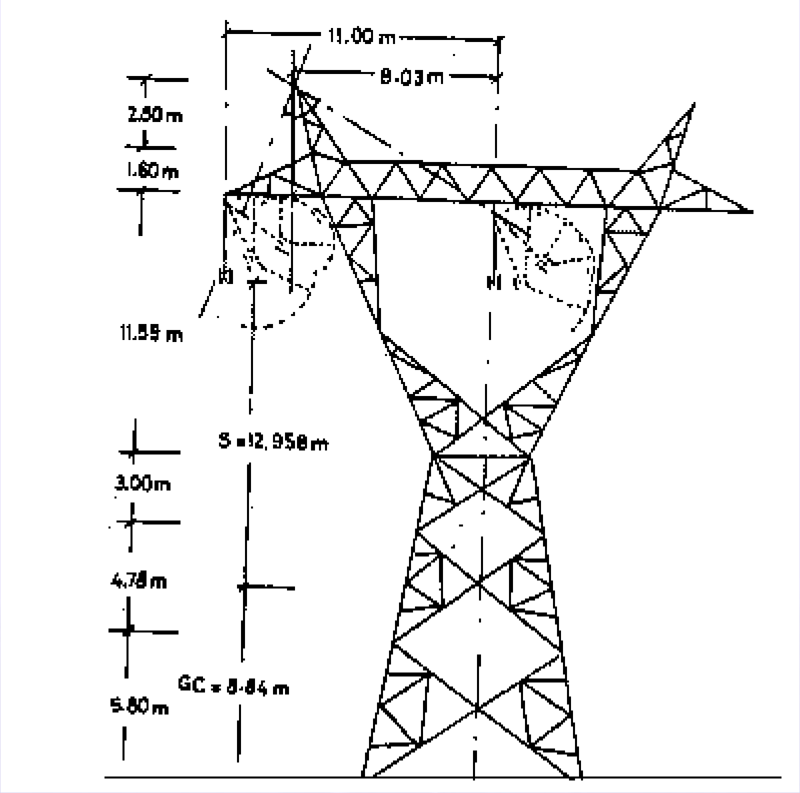
The standardized 400 kV towers presently employed in France are given in Figure 7.9 together with the corresponding weights, sizes of the conductor and ground wire employed and the ruling span. The extension and

S = Maximum Sag GC = Minimum Ground Clearance



Normal span = 245m Conductor : 6/4.72mm Al + 7/1.57mm St Ground wire: 7/2.5mm (110kg/mm^2 quality)	Normal span = 350m Conductor : 30/3mm Al + 7/3mm St Ground wire: 7/3.15mm (110kg/mm^2 quality)	Normal span = 320m Conductor : 54/3.18mm Al + 7/3.18mm St Ground wire: 7/3.15mm (110kg/mm^2 quality)

Figure 7.10 66 kV, 132 kV, 220 kV double circuit tangent towers



Normal span: 400m

Conductor: "Moose"

(54/3.53mm Al + 7/3.53mm St)

Ground wire: 7/4mm (110 kg/mm^2 quality)

Figure 7.11 400kV single circuit tangent towers (2^o)

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^{o}C without external load shall not exceed the following percentages of the ultimate tensile strength of the conductor:

Intial 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 stromy 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 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 contion of 32^{o}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

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

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 occuring 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

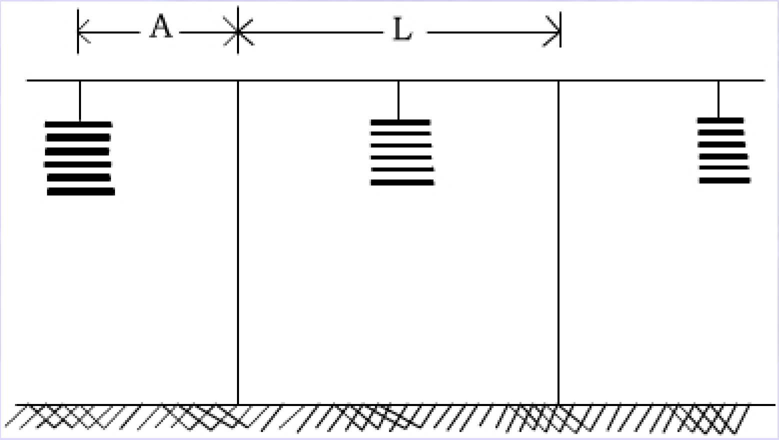


Figure 7.12 Limits of deflection (USSR practice)

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.

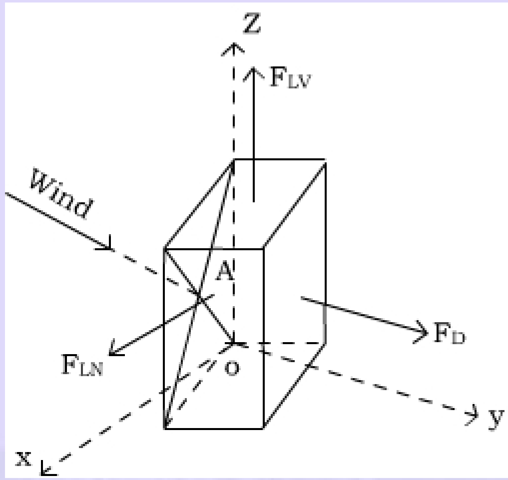


Figure 7.13 Components of wind force on a structure

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

$$F = (C_p P)\,A \qquad \textbf{\textit{(7.1)}}$$

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

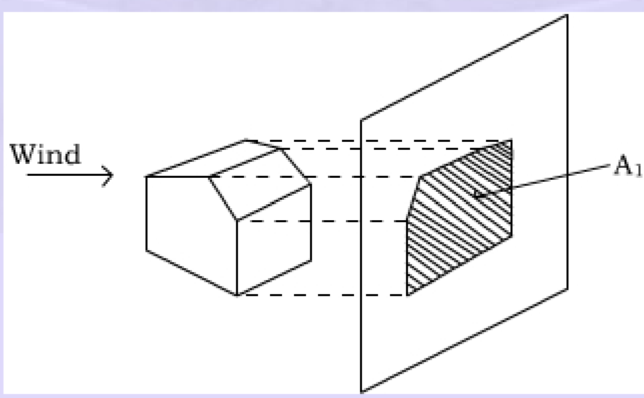


Figure 7.14(a) Frontal area of a structure

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

$$F=(C_f q)A_1 \qquad \textbf{\textit{(7.2)}}$$

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,ϕ.

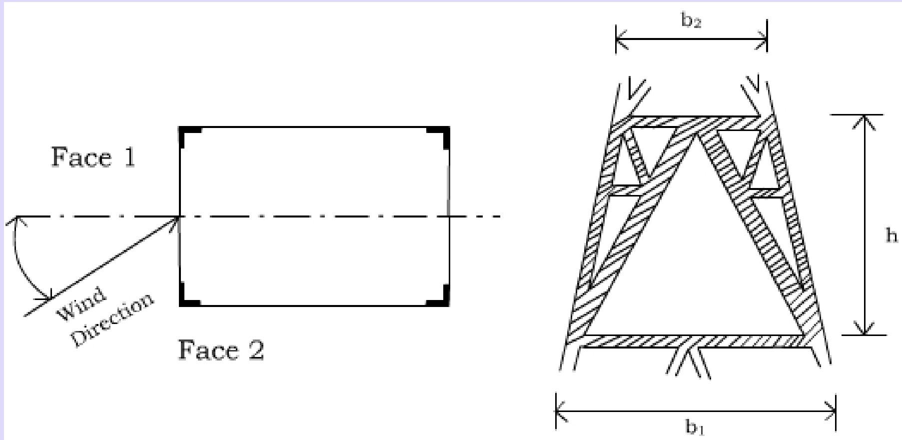
Solidity ratio ϕ is defined as the ratio of effective area of the frame normal to the wind direction S_T to the area enclosed by the projected frame boundary (Figure 7.14(b))

$$\phi = \frac{S_T \text{ x } 2}{h\left(b_1 + b_2\right)} \qquad \textbf{\textit{(7.3)}}$$

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 ψ 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)

ϕ: Solidity Ratio

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

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 ϕ and shielding factor ψ . Thus

C_f for windward truss = $C_f^1\,\phi$ ***(7.4)***

C_f for leeward truss = $C_f^1\,\psi\phi$ ***(7.5)***

C_f for tower = $C_f^1\,\phi\,(1 + \psi)$ ***(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]

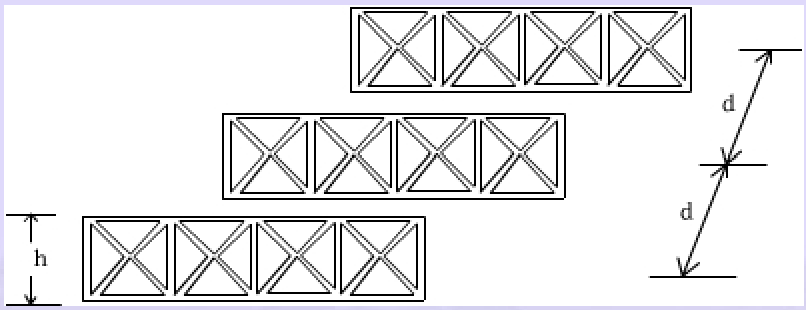


Figure 7.15 Spacing ratio for leeward truss

The wind force F on latticed towers is given by

$$F = C_f P A_e \qquad (7.7)$$

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

$$F = \alpha C_f A_e \frac{V^2}{16} \qquad \textbf{\textit{(7.8)}}$$

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 α 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 1.1

For diameters less than 20mm 1.2

For supports:

For lattice metallic supports according to the Table 6.13.

Values of the coefficient α

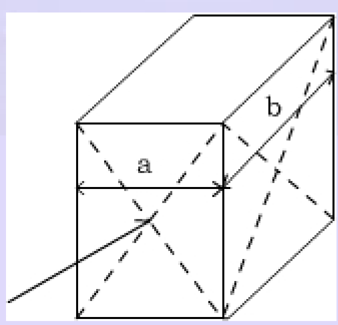


Figure 7.16 Definition of aspect ratio a / b

The values of α 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 110kV 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 25m/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 40m/sec., the minimum velocity assumed in the designs is 25m/sec. For lines up to and including 220 kV, and 27m/sec. For 330kV 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

Country, Code		ψ			
France, Regles NV65 Italy, CNR UNI 10012 $\phi \le 0.6 \begin{Bmatrix} d/h \le 2 \\ 2 < d/h \le 5 \end{Bmatrix}$		$1-1.2\phi$ $1-0.4\phi\left(5-\frac{d}{h}\right) \le 1$			
Soviet Union,SNIP II-A.11 - 62	d/h	1	2	4	5
	ϕ				
	0.1	1.00	1.00	1.00	1.00
	0.2	0.85	0.90	0.93	0.97
	0.3	0.68	0.75	0.80	0.85
	0.4	0.50	0.60	0.67	0.73
	0.5	0.33	0.45	0.53	0.62
	0.6	0.15	0.30	0.40	0.50
	1.0	0.15	0.30	0.40	0.50

Table 7.11 Overall force coefficients C_f for square-section towers

Country,code	Flat-side members	Rounded members	
France, Regles NV65 $0.08<\phi<0.35$	$3.2 - 2\phi$	$0.7\,(3.2 - 2\phi)$	
Great Britain, CP3: Ch V:Part 2:1972		Subcritical flow $dV<6m^2s^{-1}$	Supercritical flow $dV>6m^2s^{-1}$
$\phi = \begin{Bmatrix} 0.1 & 3.8 & 2.2 & 1.2 \\ 0.2 & 3.3 & 1.9 & 1.3 \\ 0.3 & 2.8 & 1.7 & 1.4 \\ 0.4 & 2.3 & 1.6 & 1.4 \\ 0.5 & 2.1 & 1.4 & 1.4 \end{Bmatrix}$			

Table 7.12 Force coefficients, C_f for wires and cables

Country,Code	Description	C_f
France, Regles NV65 Smooth surface members of circular section	$d \geq 0.28$ $d < 0.28 \begin{cases} d\sqrt{p} \leq 0.5 \\ 0.5 < d\sqrt{p} < 1.5 \\ d\sqrt{p} \geq 1.5 \end{cases}$	$+0.6$ $+1.0$ $+1.2 - 0.4d\sqrt{p}$ $+0.6$
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:Part2: 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

Ratio of width of the surface over which wind is acting to width of the surface perpendicular to direction of wind (fig3.7)	Aerodynamic coefficients C_f corresponding to ratio of area of bracing to area of panel			
Aspect ratio = a/b	0.15	0.25	0.35	0.45
0.5 - 0.7	3	2.6	2.2	1.8
1	3	2.7	2.3	2.0
1.5 - 2.0	3	3.0	2.6	2.2

Table 7.14 Space coefficient a for conductors and ground wires

At wind velocities	coefficient α
Up to 20m/sec	1.00
Up to 25m/sec	0.85
Up to 30m/sec	0.75
Up to 35m/sec and above	0.70

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 ths 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).

Thus $V_Z = V_b\ k_1 k_2 k_3$ ***(7.9)***

Where k_1, k2, and k_3 represent multiplying factor to account for choosen probobility 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 probobility factor (k1)

Table 7.15 Risk coefficients for different classes of structures

Class of structure	Mean probable design life of structure in years	k_1 for each basic wind speed					
		33	39	44	47	50	55
1. All general buildings and structures	50	1.0	1.0	1.0	1.0	1.0	1.0
2. Temporary sheds, structures such as those used during construction operation (for example, form-work and falsework), structures in construction stages and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
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	25	0.94	0.92	0.91	0.90	0.90	0.89
4. Important buildings & structures like hospitals, communications buildings or towers, power plant structures.	100	1.05	1.06	1.07	1.07	1.08	1.08

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 (k_2 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:

$$V_z = V_{gs}\left(\frac{Z}{Z_g}\right)^k = 1.35V_b\left(\frac{Z}{Z_g}\right)^k \qquad \textbf{\textit{(7.10)}}$$

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 givenin 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 commencemnt 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 (θ) is greater than 3^o, 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^o.

The influence of topographic feature is considered to extend 1.5 L_e upwind and 2.5L_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 ($\theta < 3^o$) 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

Category	Description
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

$$k_3 = 1 + Cs \qquad \textbf{\textit{(7.11)}}$$

Where C has tha 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

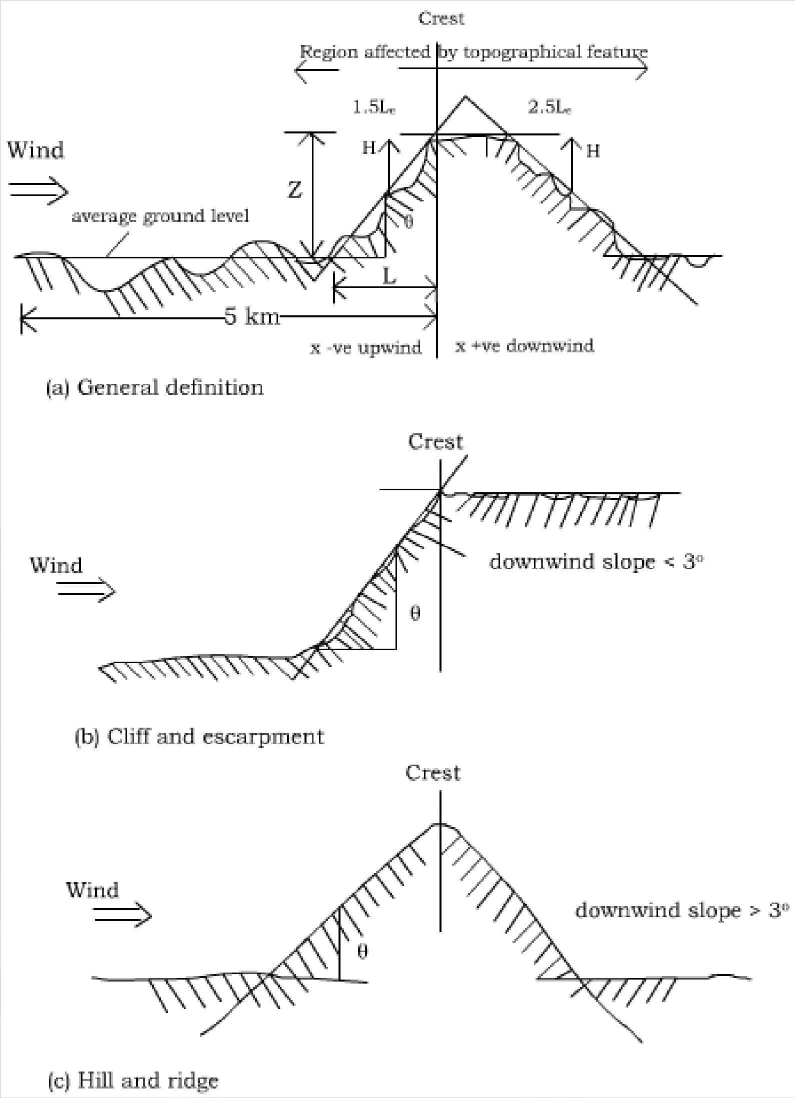


Figure 7.17 Definition of topographical dimensions

The design wind pressure p_z at any height above means groundlevel is obtained by the following relationship between wind pressure and wind velocity:

$$p_z = 0.6\, V_z^2 \qquad \textbf{\textit{(7.12)}}$$

Wher p_z = design wind pressure in N/m^2, and

V_z = 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 caonditions in which the sea level temperature is higher and the sea level pressure slightly lower than in temperate zones.

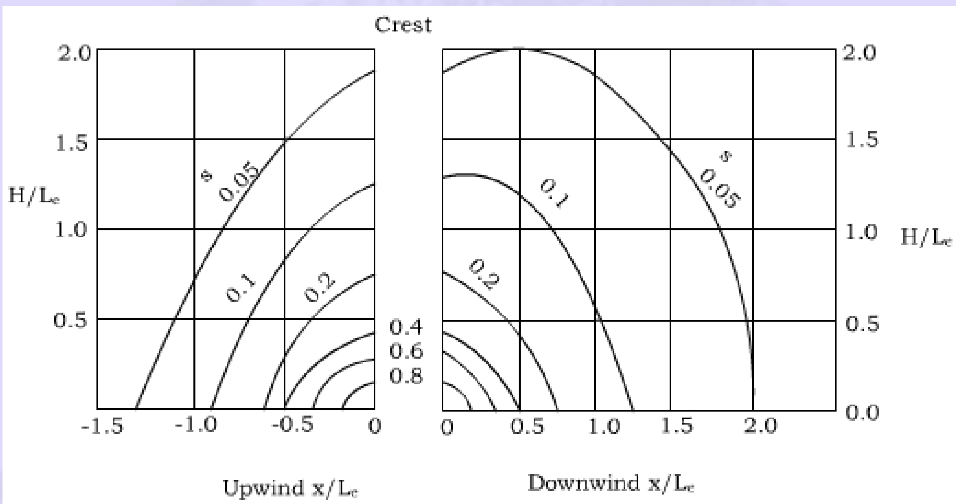


Figure 7.18 Factors for cliff and escarpment

Table 7.17 Factors to obtain design wind speed variation with height in different terrains for different classes of buildings structures

Height	Terrain Category 1 class			Terrain Category 2 class			Terrain Category 3 class			Terrain Ctegory 4 class		
(m)	A	B	C	A	B	C	A	B	C	A	B	C
10	1.05	1.03	0.99	1.00	0.98	0.93	0.91	0.88	0.82	0.80	0.76	0.67
15	1.09	1.07	1.03	1.05	1.02	0.97	0.97	0.94	0.87	0.80	0.76	0.67
20	1.12	1.10	1.06	1.07	1.05	1.00	1.01	0.98	0.91	0.80	0.76	0.67
30	1.15	1.13	1.09	1.12	1.10	1.04	1.06	1.03	0.96	0.97	0.93	0.83
50	1.20	1.18	1.14	1.17	1.15	1.10	1.12	1.09	1.02	1.10	1.05	0.95
100	1.26	1.24	1.20	1.24	1.22	1.17	1.20	1.17	1.10	1.20	1.15	1.05
150	1.30	1.28	1.24	1.28	1.25	1.21	1.24	1.21	1.15	1.24	1.20	1.10
200	1.32	1.30	1.26	1.30	1.28	1.24	1.27	1.24	1.18	1.27	1.22	1.13
250	1.34	1.32	1.28	1.32	1.31	1.26	1.29	1.26	1.20	1.30	1.26	1.17
300	1.35	1.34	1.30	1.34	1.32	1.28	1.31	1.28	1.22	1.30	1.26	1.17
350	1.37	1.35	1.31	1.36	1.34	1.29	1.32	1.30	1.24	1.31	1.27	1.19
400	1.38	1.36	1.32	1.37	1.35	1.30	1.34	1.31	1.25	1.32	1.28	1.20
450	1.39	1.37	1.33	1.39	1.37	1.32	1.36	1.33	1.28	1.34	1.30	1.21
500	1.40	1.38	1.34	1.39	1.37	1.32	1.36	1.33	1.28	1.34	1.30	1.22

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 θ

Slope θ	L_e
$3° < \theta \le 17°$	L
$> 17°$	Z/0.3

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 θ

Slope θ	Factor C
$3^\circ < \theta \leq 17^\circ$	1.2 (Z/L)
$> 17^\circ$	0.36

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.

$\tan\theta = 10/35 = 0.2857$; $\theta = 15.94$

X = +5 L = 10m H = 20m

X/L = +5/10 = +0.5 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 x 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.

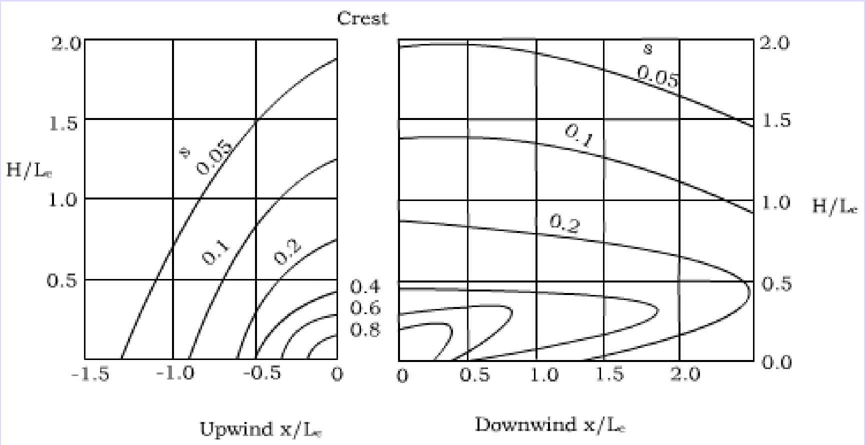


Figure 7.19 Factors for ridge and hill

Wind force on the structure

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

$$F = C_f A_e p_d \qquad (7.13)$$

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

ϕ - solidity ratio

Wind force on multiple frames

The wind force on the winddard 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 ψ, which is dependent upon the solidity ratio of windward frame, the types of members and tha 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 β to be adopted is as follows:

Aerodynamic solidity ratio β = solidity ratio ϕ for flat-sided members. ***(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 with circular members are given in the Table 7.22.

Table 7.19 Force coefficients for single frames

Solidity Ratio ϕ	Force coefficient C_f for		
	Flat-sided members	Circular section	
		Subcritical flow $DV_d < 6m^2/s$	Supercritical flow $DV_d \geq 6m^2/s$
0.1	1.9	1.2	0.7
0.2	1.8	1.2	0.8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8
0.75	1.6	1.5	1.4
1.0	2.0	2.0	2.0

Table 7.20 Shielding factors for multiple frames

Effective solidity ratio β	Frame spacing ratio				
	<0.5	1.0	2.0	4.0	>8.0
0	1.0	1.0	1.0	1.0	1.0
0.1	0.9	1.0	1.0	1.0	1.0
0.2	0.8	0.9	1.0	1.0	1.0
0.3	0.7	0.8	1.0	1.0	1.0
0.4	0.6	0.7	1.0	1.0	1.0
0.5	0.5	0.6	0.9	1.0	1.0
0.7	0.3	0.6	0.8	0.9	1.0
1.0	0.3	0.6	0.6	0.8	1.0

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

$$F = C_f\,(1 + \psi)A_e\, p_d \qquad \textbf{\textit{(7.15)}}$$

Tables 7.19 and 7.20 give the values of C_f and y respectively.

In the case of rectangular towers, the wind force can be calculated base 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 ψ 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

Solidity Ratio ϕ	Force coefficient for	
	Square towers	Equilateral triangular towers
0.1	3.8	3.1
0.2	3.3	2.7
0.3	2.8	2.3
0.4	2.3	1.9
0.5	2.1	1.5

Tables 7.22 Overall force coefficient for square towers composed of rounded members

Solidity ratio of front face ϕ	Force coefficient for			
	Subcritical flow $DV_d < 6m^2/s$		Supercritical flow $DV_d \geq 6m^2/s$	
	on to face	on to corner	on to face	on to corner
0.05	2.4	2.5	1.1	1.2
0.1	2.2	2.3	1.2	1.3
0.2	1.9	2.1	1.3	1.6
0.3	1.7	1.9	1.4	1.6
0.4	1.6	1.9	1.4	1.6
0.5	1.4	1.9	1.4	1.6

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

Solidity ratio of front face ϕ	Force coefficient for	
	Subcritical flow $DV_d < 6m^2/s$	Supercritical flow $DV_d \geq 6m^2/s$
0.05	1.8	0.8
0.1	1.7	0.8
0.2	1.6	1.1
0.3	1.5	1.1
0.4	1.5	1.1
0.5	1.4	1.2

Table 7.24 Force coefficients for wires and cables (VD > 100)

Flow regime	Force coefficient C_f for			
	Smooth surface wire	Moderately smooth wire (galvanized or painted)	Fine stranded cables	Thick stranded cables
$DV_d < 0.6m^2/s$	-	-	1.2	1.3
$DV_d \geq 0.6m^2/s$	-	-	0.9	1.1
$DV_d < 6m^2/s$	1.2	1.2	-	-
$DV_d \geq 6m^2/s$	0.5	0.7	-	-

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 $0.6m^2/s$, 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 (λ = 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 singlr factor called tha '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:

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

Where α 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:

$$\text{Seismic lateral force } P = Ma \qquad \textbf{\textit{(7.16) a}}$$

$$= (W/g)\, a = W\,(a/g) \qquad \textbf{\textit{(7.16) b}}$$

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:

$$P = \alpha_H W \text{ in which } \alpha_H = \beta I F_o \,(Sa/g) \qquad \textbf{\textit{(7.16c)}}$$

$$= \beta I \alpha_o \qquad \textbf{\textit{(7.16d)}}$$

Where β = 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,

(Sa/g) = average acceleration coefficient which takes into account the period of vibration of the structure and damping characteristics to be read from Figure 7.20,

α_H = Seismic coefficient, and

α_o = 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 α_o = 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 Wof 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.6 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 EI 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):

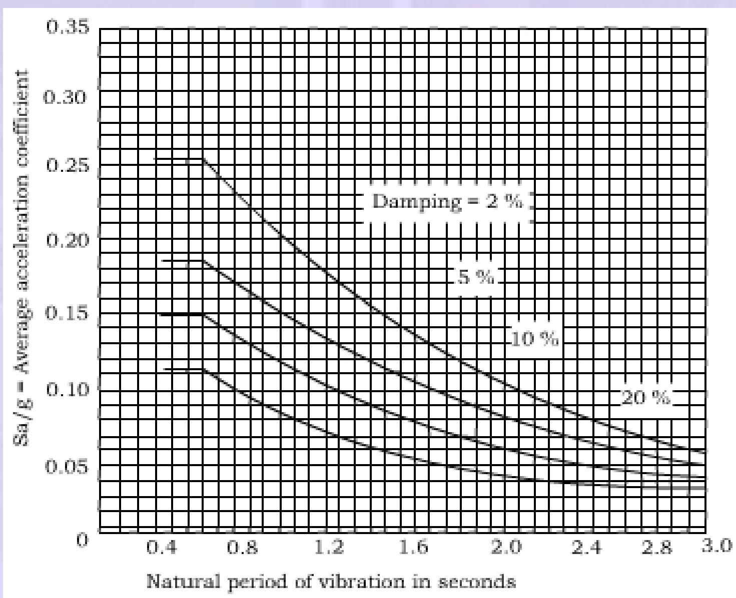


Figure 7.20 Average acceleration spectra

Example:

Let the period of the tower be two seconds and damping five percent critical. Further, the soil-foundation system gives a factor of β = 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 -8eismic Zones of India), the horizontal seismic coefficient

$$\alpha_H = \beta I F_o \text{ (Sa/g)}$$

$$= 1.2 \times 1 \times 0.4 \times 0.06$$

$$= 0.0288$$

Therefore, the horizontal seismic force for a tower weighing 5,000kg 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°to30°):

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 sup- ports and ground wire supports are designed for the broken-wire conditions.

For 400 k V 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 equa1 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 s pan, 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 bro- ken, or two phases and anyone ground wire on the same span is broken, whichever combination is more stringent for a particular member.

± 500 k V 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 bro- ken-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 net- works 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 maxi- mum 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 bro- ken 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).

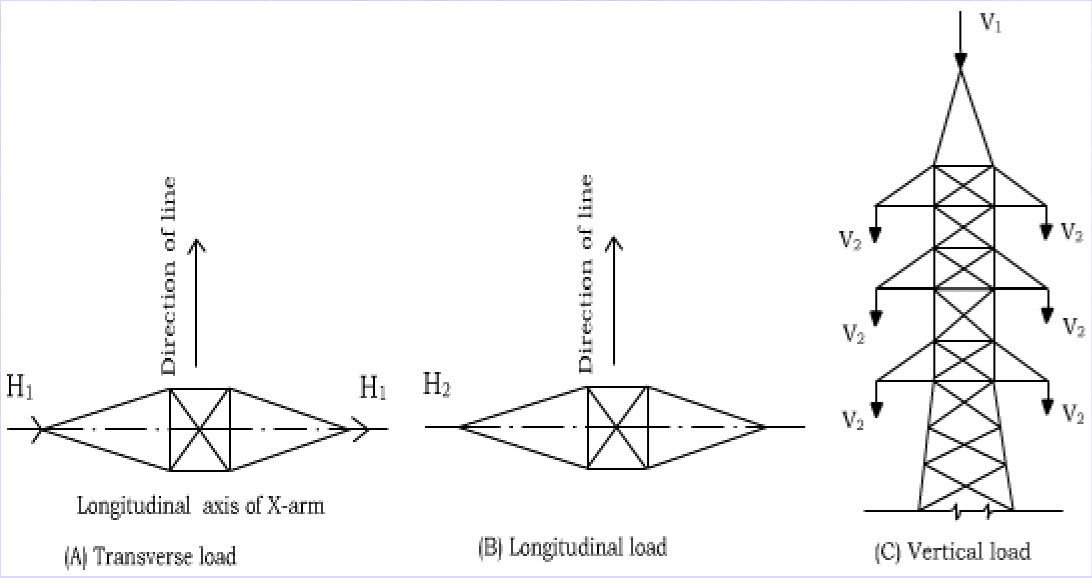


Figure 7.21 Loadings on tower

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 k V) 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 I-metre length of conductor,

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

(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,

$$P = \frac{2}{3} x \frac{(d+2t)}{1000} p\,\text{kg} \qquad \textbf{\textit{(7.17b)}}$$

Where p = wind pressure in kg/m2 on two-thirds the projected area of conductor.

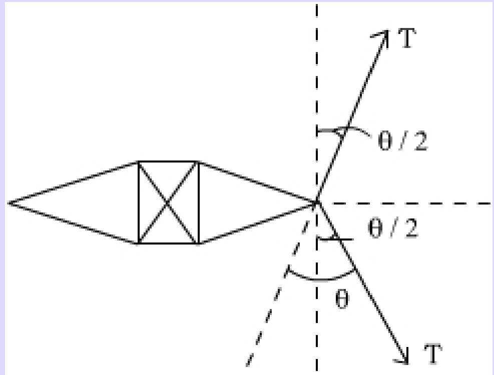


Figure 7.22 Transverse load on the cross-arm due to line deviation

Resultant force per metre length, R

$$= \sqrt{W^2 + P^2} \qquad \textbf{\textit{(7.17c)}}$$

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 of100 kg/m^2 is given in Table 7.25. The Table also gives the approximate weight of the insulator string normally used.

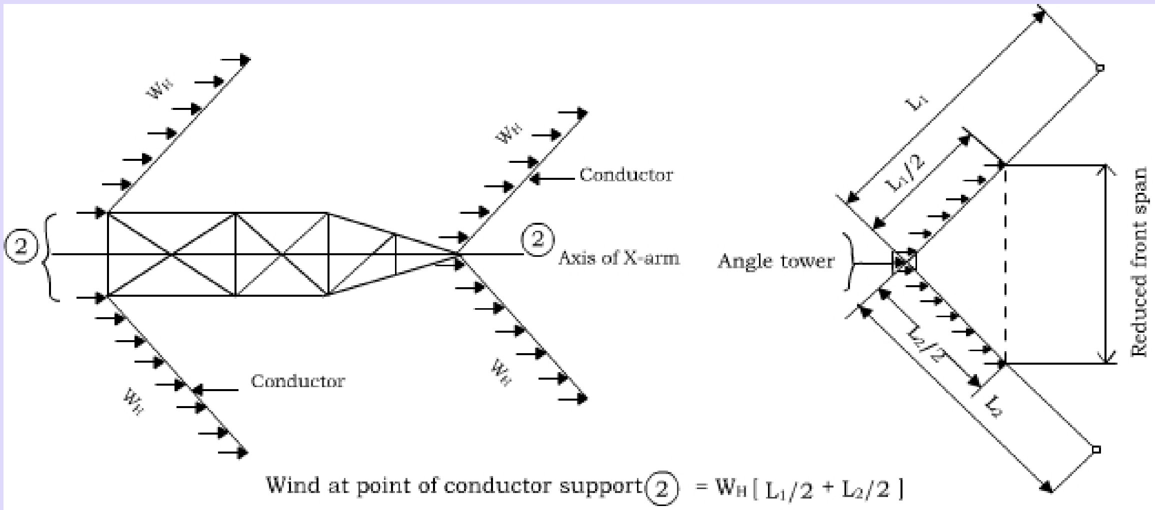


Figure 7.23 Wind on conductor of angle tower

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 trans- verse load = 2TSinθ/2 where θ 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)

Voltage(kV) 1	Length of suspension insulator string (cm) 2	Diameter of each disc (cm) 3	Projected area of the string (Cot.2 x Cot.3) (sq.m) 4	Effective area for wind load (assumed 50% of Cot.4)(m^2) 5	Computed wind load on insulator string in kg for wind pressure of 100kg/m^2 6	Weight of insulator string (kg) 7
66	107	25.5	0.2718	0.1359	13.6	31.8
132	168	25.5	0.4267	0.2134	21.4	57.1
220	265	25.5	0.6731	0.3366	33.7	122.5
400	415	25.5	1.0541	0.5271	52.7	200.0

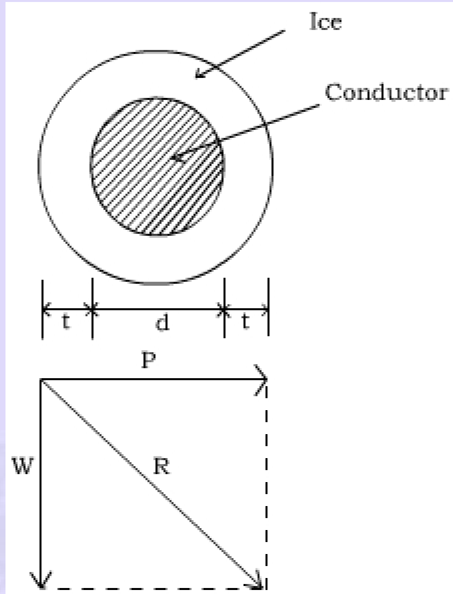


Figure 7.24 Resultant load on conductor

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 ac- counting the wind force on the leeward face, a factor ofl.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.

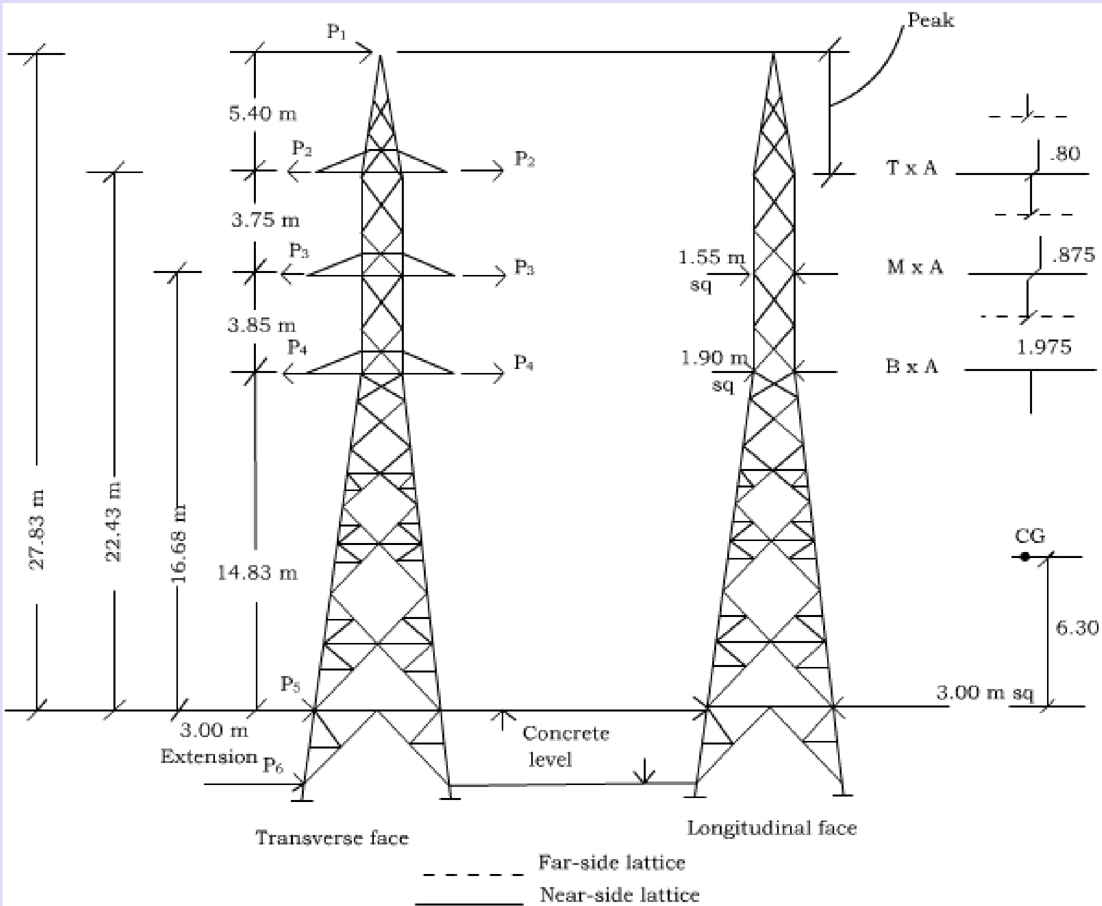


Figure 7.25 Equivalent wind load on transmission line tower

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

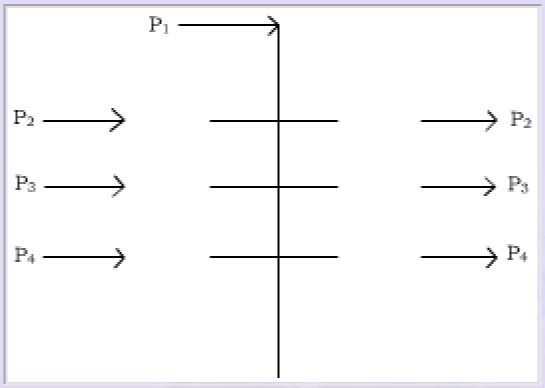
3. waist level (also portal base level if desired) in the case of corset type towers.

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. How- ever, 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

Tower details		Method 1	Method 2	Method 3	Method 4
		Load in Kg			
	P_1	0	0	205	90
	P_2	253	200	175	108
	P_3	253	200	195	135
	P_4	253	400	305	626
	Total	1,518	1,600	1,555	1,828
	Actual wind load	2,940			

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 al- lowed 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

$$W = KH\sqrt{M} \qquad \textbf{(7.18)}$$

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$\sqrt{M}$ 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 fonnula in the form $\sqrt{PH}$. 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,

$$W = 0.1993\,H\sqrt{M} + 495 \qquad \textbf{\textit{(7.19)}}$$

For angle towers

$$W = 0.2083\,H\sqrt{M} + 400 \qquad \textbf{\textit{(7.20)}}$$

Since there is no appreciable difference in the above two equations, the common equation given below may be used for both the tower types:

$$W = 0.205\, H \sqrt{M} + 450 \qquad \textbf{\textit{(7.21)}}$$

A more detailed evaluation of tower weight due to Walter BIlckner 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

$$w = Al\gamma = \frac{P}{\sigma_K} l\gamma \qquad \textbf{\textit{(7.22)}}$$

Where A is the cross-section of strut,

l is the unsupported length,

γ 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 σ_K. For geometrically similar sections

$$\sigma_K = \left(C\sqrt{P}\right)/l \qquad \textbf{\textit{(7.23)}}$$

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 $\left(\sqrt{P}\right)/l$, 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

$$g_m = \frac{\gamma}{l_E}\sum\frac{lP}{\sigma_K} + g_z \qquad \textbf{\textit{(7.24)}}$$

Where n = number of members,

P = truss force,

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

l_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:

$$G_T = Q\left(\frac{h^2}{2b_0} + \frac{Q}{\Delta_b}\right) + 4.4\sqrt{\frac{M_T}{\Delta_b}}\left(b_m^{3/2} - b_0^{3/2}\right) \qquad \textbf{\textit{(7.25)}}$$

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

Q = M_{bmax} / h; M_b= maximum normal mo- ment at ground level (x = h) in tonne- metres,

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

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

Δ_b= taper in metres/metre, and

M_T= 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 kVto 400 kV, together with the spans and sizes of conductor and ground wire used on the lines, are given in Table 7.27. Assum- ing that 80 percent are tangent towers, 15 percent 30^o towers and 5 percent 60^o towers and dead-end towers, and allowing 15 percent extra for exten- sions and stubs, the weights of towers for a 10km line are also given in the Table.

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

Span (m)	400kV Single circuit	220kV Double circuit	220kV Single circuit	132kV Double circuit	132kV Single circuit	66kV Double circuit	66kV Single circuit
	400	320	320	320	320	245	245
Conductor:	Moose 54/3.53mm Al. + 7/3.53mm St	Zebra 54/3.18mm Al + 7/3.8mm St	Zebra 54/3.18mm Al + 7/3.8mm St	Panther 30/3mm Al + 7/3mm St	Panther 30/3mm Al + 7/3mm St	Dog 6/4.72mm Al + 7/1.57mm St	Dog 6/4.72mm Al + 7/1.57mm St
Groundwire:	7/4mm 110kgf/mm^2 quality	7/3.15mm 110kgf/mm^2 quality	7/3.15mm 110kgf/mm^2 quality	7/3.15mm 110kgf/mm^2 quality	7/3.15mm 110kgf/mm^2 quality	7/2.5mm 110kgf/mm^2 quality	7/2.5mm 110kgf/mm^2 quality
Tangent Tower	7.7	4.5	3.0	2.8	1.7	1.2	0.8
30^o Tower	15.8	9.3	6.2	5.9	3.5	2.3	1.5
60^o and Dead-end Tower	23.16	13.4	9.2	8.3	4.9	3.2	2.0
Weight of towers for a 10-km line	279	202	135	126	76	72	48

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 andd broken-wire conditions for a typical 400kV line

Tower type	Longitudinal loads		Transverse loads	
	Normal Condition	Broken-wire condition	Normal Condition	Broken-wire condition
A	0.0	0.5 x T x Cos $\phi/2$	WC + WI + D	0.6WC + WI + 0.5DA
B	0.0	1.0 x MT x Cos $\phi/2$	WC + WI + D	0.6WC + WI + 0.5D
B Section Tower	0.0	1.0 x MT	WC + WI + 0.0	0.6WC + WI + 0.0
C	0.0	1.0 x MT x Cos $\phi/2$	WC + WI + D	0.6WC + WI + 0.5D
C Section Tower	0.0	1.0 x MT	WC + WI + 0.0	0.6WC + WI + 0.0
D 60^{o}	0.0	1.0 x MT x Cos $\phi/2$	WC + WI + D	0.6WC + WI + 0.5D
Dead-end with slack span (slak span side broken)	0.7 MT	1.0 x MT	0.65WC + WI	0.6WC + WI
Dead-end with slack span line side broken	0.7 MT	0.3 MT x Cos 15^{o}	0.65WC + WI + 0.3MT x Sin 15^{o}	0.25WC + WI + 0.3MT x Sin 15^{o}
Dead-end	MT	Nil	0.5WC + WI + 0.0	0.1WC + WI + 0.0

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 = ϕ

Load due to deviation of 'A' type tower under BWC = DA = 2 x T x Sin (ϕ / 2)

Load due to deviation for others = D = 2 x MT x Sin (ϕ / 2)

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. Nonnal span: 335 m
2. Wind pressure

a. Tower (on 11/2 times the exposed area of one face): 200 kg/m^2

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 mm^2

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 : 54.5 mm^2
   ground wire

4. Ultimate tensile : 5,710 kg
   strength

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) = 335 x45 x 21 / 1,000

= 317 kg

Wind load on conductor (broken-wire condition) = 0.6x317= 190 kg

b. Wind load on ground wire (Normal condition) = 335 x 9.45 x45 / 1,000

= 142 kg

Wind load on ground wire (Broken-wire condition) = 0.6 x 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 nsulator 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^o x 0.5

= 33 kg

(2) Ground wire point = 2,500 x sin 1^o

= 44 kg

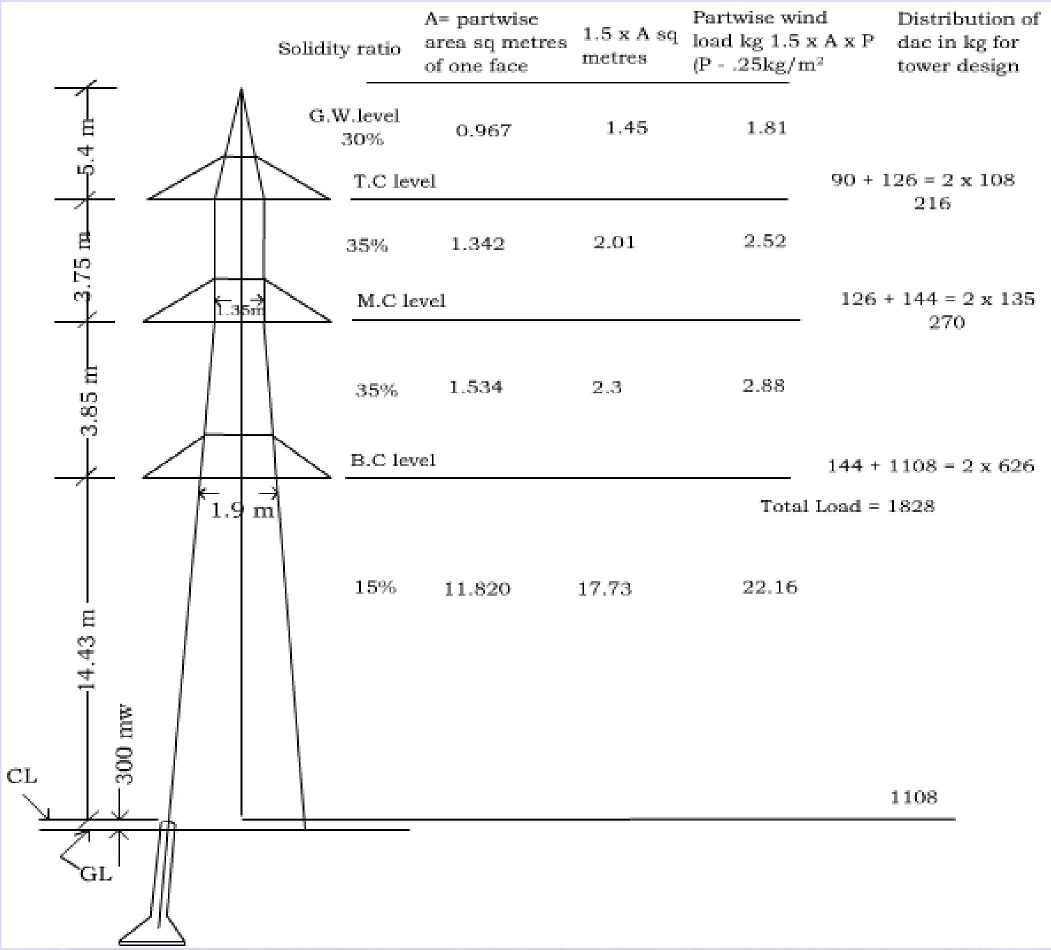


Figure 7.26 Method of wind load calculation on tower

2. Longitudinal load

Longitudinal load under broken-conductor condition = 3,800 x cos 1^{o} x 0.5

= 1,900 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

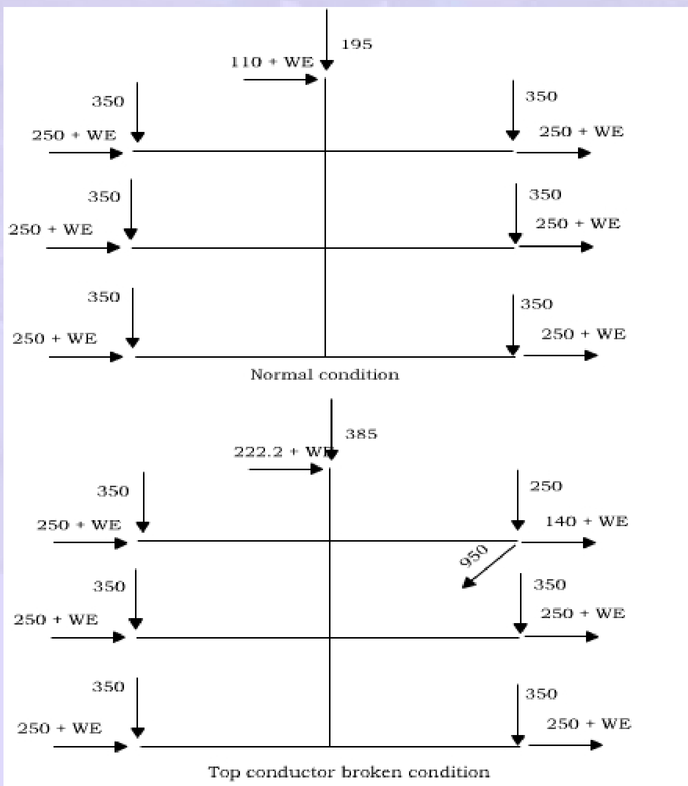
= 299 kg

4. Torsional shear

Torsional shear per face at the top conductor position = 1,900 x 3.5 / 2 x 1.75 =1,900kg 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

Details	Conductor		Ground wire	
	Normal condition	Broken-wire condition	Normal condition	Broken-wire condition
1. For tower design				
Due to wind on conductors	317	190	142	85
Due to deviation	133	33	87	44
Equivalent wind on tower	W_e	W_e	W_e	W_e
Wind load on insulator string	50	50	-	-
Transverse load (Total)	500 + W_e	273 + W_e	229 + W_e	129 + W_e
Transverse load (rounded)	500 + W_e	280 + W_e	230 + W_e	130 + W_e
Vertical load	700	500	390	299
Longitudinal load	-	1,900	-	-
2. For cross-arm design				
Transverse load	Same as for tower design		Note: The loads indicated in Figure3.18 are half the above loads as they represent loads on one face of the tower	
Vertical load				
Longitudinal load				



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

Figure 7.28

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

$$P_1 = T \sin \theta/2 \text{ and } P_2 = T \cos \theta/2 \qquad \textbf{\textit{(7.26)}}$$

Where, as in Figure 7.16, T is the conductor tension and θ is the line angle. If θ 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 θ_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 θ, if

$$W_h L_n + 2T\sin \theta_n/2 = W_h L + 2T \sin \theta/2 \qquad \textbf{\textit{(7.27)}}$$

This equation relating L and θ, 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 θ for conductor

$$1.83L + 2 \times 4{,}080 \sin \theta/2 = 1.83 \times 300 + 2 \times 4{,}080 \times 0.02618$$

$$= 762.63$$

For θ = zero, L = maximum tangent span

$$= 762.63 / 1{:}83 = 416.7 \text{ say, } 420 \text{ metres.}$$

If the minimum span length required = 150 metres,

$$8{,}160 \sin \theta/2 = 762.63 - 1.83 \times 150$$

$$\sin \theta/2 = 488.13 / 8{,}160 = 0.05982$$

$$\theta = 6°52'$$

Similarly for the ground wire:

With θ = zero, L = 396 metres, say, 400 metres.

If the minimum span length requiredis 150 metres, as before, θ = 7°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.

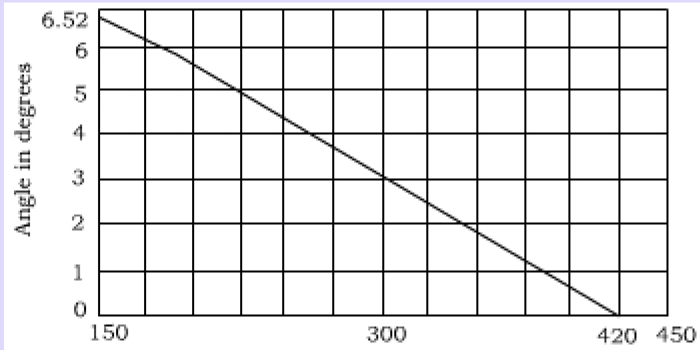


Figure 7.28 Span angle diagram

7.4 Tower Design

Once the external loads acting on the tower are determined, one proceeds with an analysis of the forces in various members with a view to fixing up their sizes. Since axial force is the only force for a truss element, the member has to be designed for either compression or tension. When there are multiple load conditions, certain members may be subjected to both compressive and tensile forces under different loading conditions. Reversal of loads may also induce alternate nature of forces; hence these members are to be designed for both compression and tension. The total force acting on any individual member under the normal condition and also under the broken- wire condition is multiplied by the corresponding factor of safety, and it is ensured that the values are within the permissible ultimate strength of the particular steel used.

Bracing systems

Once the width of the tower at the top and also the level at which the batter should start are determined, the next step is to select the system of bracings. The following bracing systems are usually adopted for transmission line towers.

Single web system (Figure 7.29a)

It comprises either diagonals and struts or all diagonals. This system is particularly used for narrow-based towers, in cross-arm girders and for portal type of towers. Except for 66 kV single circuit towers, this system has little application for wide-based towers at higher voltages.

Double web or Warren system (Figure 7.29b)

This system is made up of diagonal cross bracings. Shear is equally distributed between the two diagonals, one in compression and the other in tension. Both the diagonals are designed for compression and tension in order to permit reversal of externally applied shear. The diagonal braces are connected at their cross points. Since the shear perface is carried by two members and critical length is approximately half that of a corresponding single web system. This system is used for both large and small towers and can be economically adopted throughout the shaft except in the lower one or two panels, where diamond or portal system of bracings is more suitable.

Pratt system (Figure 7.29c)

This system also contains diagonal cross bracings and, in addition, it has horizontal struts. These struts are subjected to compression and the shear is taken entirely by one diagonal in tension, the other diagonal acting like a redundant member.

It is often economical to use the Pratt bracings for the bottom two or three panels and Warren bracings for the rest of the tower.

Portal system (Figure 7.29d)

The diagonals are necessarily designed for both tension and compression and, therefore, this arrangement provides more stiffness than the Pratt system. The advantage of this system is that the horizontal struts are supported at mid length by the diagonals.

Like the Pratt system, this arrangement is also used for the bottom two or three panels in conjuction with the Warren system for the other panels. It is specially useful for heavy river-crossing towers.

Where

p = longitudinal spacing (stagger), that is, the distance between two successive holes in the line of holes under consideration,

g = transverse spacing (gauge), that is, the distance between the same two consecutive holes as for p, and

d = diameter of holes.

For holes in opposite legs of angles, the value of 'g' should be the sum of the gauges from the back of the angle less the thickness of the angle.

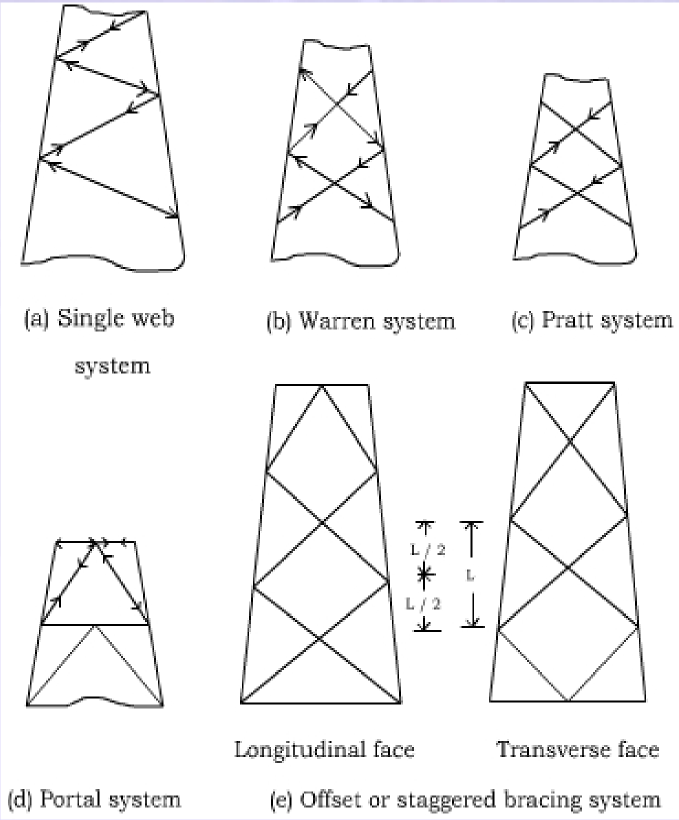


Figure 7.29 Bracing syatems

Net effective area for angle sections in tension

In the case of single angles in tension connected by one leg only, the net effective section of the angle is taken as

$$A_{eff} = A + Bk \qquad \textbf{(7.28)}$$

Where

A = net sectional area of the connected leg,

B = area of the outstanding leg = (l -t)t,

l = length of the outstanding leg,

t = thickness of the leg, and

$$k = \frac{1}{1 + 0.35\frac{B}{A}}$$

In the case of a pair of angles back to back in tension connected by only one leg of each angle to the same side of the gusset,

$$k = \frac{1}{1 + 0.2\frac{B}{A}}$$

The slenderness ratio of a member carrying axial tension is limited to 375.

7.4.1 Compression members

While in tension members, the strains and displacements of stressed material are small, in members subjected to compression, there may develop relatively large deformations perpendicular to the centre line, under certain criticallol1ding conditions.

The lateral deflection of a long column when subjected to direct load is known as buckling. A long column subjected to a small load is in a state of stable equilibrium. If it is displaced slightly by lateral forces, it regains its original position on the removal of the force. When the axial load P on the column reaches a certain critical value P_{cr}, the column is in a state of neutral equilibrium. When it is displaced slightly from its original position, it remains in the displaced position. If the force P exceeds the critical load P_{cr}, the column reaches an unstable equilibrium. Under these circum- stances, the column either fails or undergoes large lateral deflections.

Table 7.30 Effective slenderness ratios for members with different end restraint

Type of member	KL / r
a) Leg sections or joint members bolted at connections in both faces.	L/r
b) Members with eccentric loading at both ends of the unsupported panel with value of L / r up to and including 120	L/r
c) Members with eccentric loading at one end and normal eccentricities at the other end of unsupported panel with values of L/r up to and including 120	30+0.75 L/r
d) Members with normal framing eccentricities at both ends of the unsupported panel for values of L/r up to and including 120	60+0.5 L/r
e) Members unrestrained against rotation at both end of the unsupported panel for values of L/r from 120 to 200.	L/r
f) Members partially restrained against rotation at one end of the unsupported panel for values of L/r over 120 but up to and including 225	28.6+0.762 L/r
g) members partially restrained against rotation at both ends of unsupported panel for values of L/r over 120 up to and including 250	46.2+0.615 L/r

Slenderness ratio

In long columns, the effect of bending should be considered while designing. The resistance of any member to bending is governed by its flexural rigidity EI where I =Ar2. Every structural member will have two principal moments of inertia, maximum and minimum. The strut will buckle in the direction governed by the minimum moment of inertia. Thus,

$$I_{min} = Ar_{min}^2 \qquad \textbf{\textit{(7.29)}}$$

Where r_{min} is the least radius of gyration. The ratio of effective length of member to the appropriate radius of gyration is known as the slenderness ratio. Normally, in the design procedure, the slenderness ratios for the truss elements are limited to a maximum value.

IS: 802 (Part 1)-1977 specifies the following limiting values of the slenderness ratio for the design of transmission towers:

Leg members and main members in the cross-arm in compression	150
Members carrying computed stresses	200
Redundant members and those carrying nominal stresses	250
Tension members	350

Effective length

The effective length of the member is governed by the fixity condition at the two ends.

The effective length is defined as 'KL' where L is the length from centre to centre of intersection at each end of the member, with reference to given axis, and K is a non-dimensional factor which accounts for different fixity conditions at the ends, and hence may be called the restraint factor. The effective slenderness ratio KL/r of any unbraced segment of the member of length L is given in Table 7.30, which is in accordance with 18:802 (Part 1)-1977.

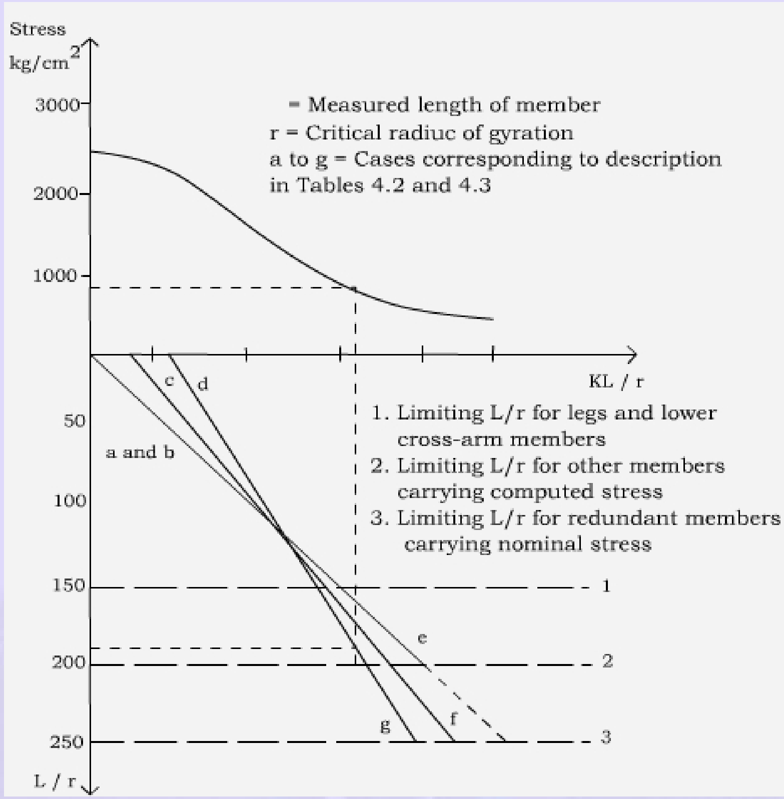


Figure 7.30 Nomogram showing the variation of the effective slenderness ratio kl / rL / r and the corresponding unit stress

Figure 7.30 shows the variation of effective slenderness ratio KL / r with L / r of the member for the different cases of end restraint for leg and bracing members.

The value of KL / r to be chosen for estimating the unit stress on the compression strut depends on the following factors:

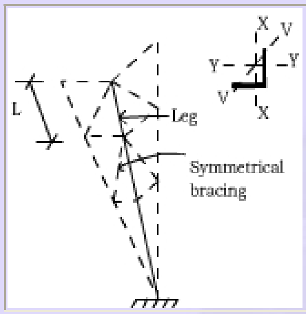
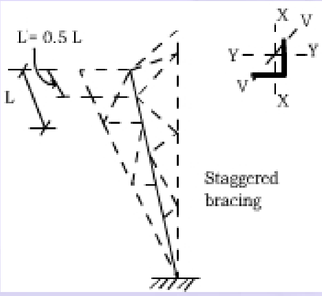
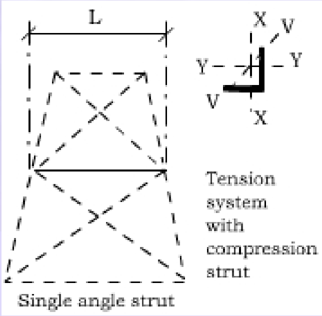
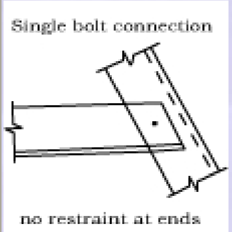
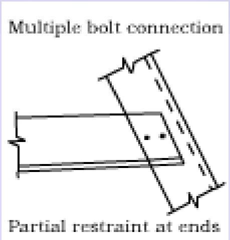
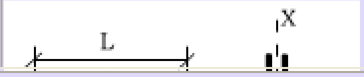
1. the type of bolted connection

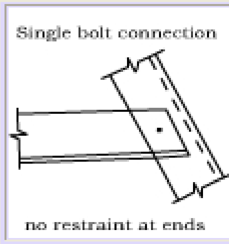
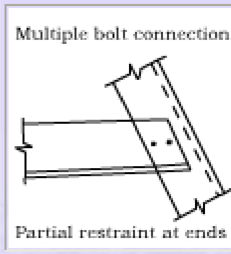
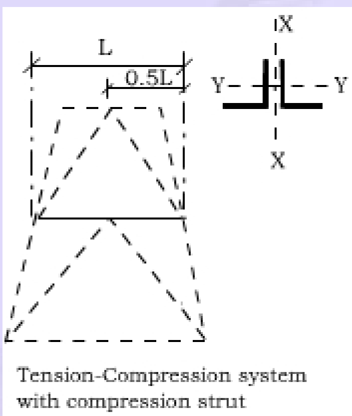
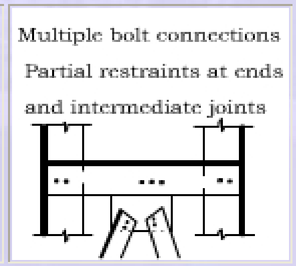
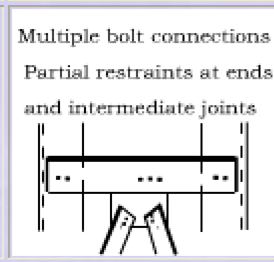
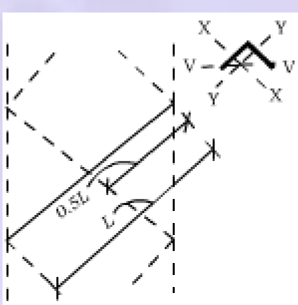
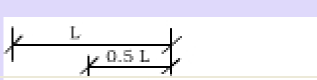
2. the length of the member

3. the number of bolts used for the connection, i.e., whether it is a single-bolted or mul- tiple-bolted connection

4. the effective radius of gyration

Table 7.31 shows the identification of cases mentioned in Table 7.30 and Figure 7.30 for leg and bracing members normally adopted. Eight different cases of bracing systems are discussed in Table 7.31.

Sl. No 1	Member 2	Method of loading 3	Rigidity of joint 4	L/r ratio 5	Limiting values of L/r 6	Categorisation of member 7	KL/r 8
1		Concentric	No restraint at ends	L/r_w	0 to 120	Case (a)	L/r
					120 to 150	Case (e)	L/r
2		Concentric	No restraint at ends	L/r_{xx} or L/r_{yy} or $0.5L/r_w$	0 to 120	Case (a)	L/r
				L/r_{xx} or L/r_{yy} or $0.5L/r_w$	120 to 150	Case (e)	L/r
3		eccentric	unsupported panel-no restraint at ends	L/r_w	0 to 120	Case (d)	60 +0.5L/r
				L/r_w	120 to 200	Case (e)	L/r
				L/r_w	120 to 250	Case (g)	46.2 + 0.615L/r
4		concentric	No restraint at ends	max of L/r_{xx} or L/r_{yy}	0 to 120	Case (b)	L/r

				max of L/r_{xx} or L/r_{yy}	120 to 200	Case (e)	L/r
				max of L/r_{xx} or L/r_{yy}	120 to 250	Case (g)	46.2 + 0.615L/r
5		concentric at ends and eccentric at intermediate joints in both directions		$0.5L/r_{yy}$ or L/r_{xx}	0 to 120	Case (e)	30 + 0.75L/r
		concentric at ends and intermediate joints		$0.5L/r_{yy}$ or L/r_{xx}	0 to 120	Case (a)	L/r
		concentric at ends	Multiple bolt connections Partial restraints at ends and intermediate joints	$0.5L/r_{yy}$ or L/r_{xx}	120 to 250	Case (g)	46.2 + 0.615L/r
6		eccentric (single angle)	Single bolt No restraint at ends	$0.5L/r_w$ or $0.75L/r_{xx}$	0 to 120	Case (c)	30 + 0.75L/r
			Single bolt No restraint at ends	$0.5L/r_w$ or $0.75L/r_{xx}$	120 to 200	Case (e)	L/r
		concentric (Twin angle)	Multiple bolt connections Partial restraints at ends and intermediate joints	$0.5L/r_w$ or $0.75L/r_{xx}$	120 to 250	Case (g)	46.2 + 0.615L/r
7		eccentric (single angle)	Single or multiple bolt connection	$0.5L/r_w$ or L/r_{xx}	0 to 120	Case (g)	60 + 0.5L/r

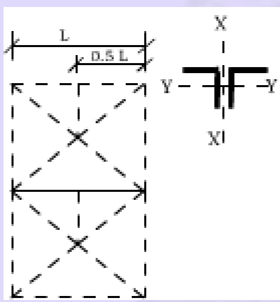
			Single bolt connection, no restraint at ends and at intermediate joints.	$0.5L/r_w$ or L/r_{xx}	120 to 200	Case (e)	L/r
			Multiple bolt at ends and single bolt at intermediate joints	$0.5L/r_w$	120 to 225	Case (f)	28.6 + .762L/r
			Multiple bolt at ends and at intermediate joints Partial restraints at both ends	L/r_{xx}	120 to 250	Case (g)	46.2 + 0.615L/r
			Partial restraints at ends and at intermediate joints	$0.5L/r_w$ or L/r_{xx}	120 to 250	Case (g)	46.2 + 0.615L/r
8		eccentric (single angle)	Single or multiple bolt connection	$0.5L/r_{yy}$ or L/r_{xx}	0 to 120	Case (a)	L/r
			Single bolt connection, no restraint at ends and at intermediate joints.	$0.5L/r_{yy}$ or L/r_{xx}	120 to 200	Case (e)	L/r
			Multiple bolt at ends and single bolt at intermediate joints	$0.5L/r_{yy}$	120 to 200	Case (f)	28.6 + .762L/r
			Multiple bolt connection Partial restraints at both ends	L/r_{xx}	120 to 250	Case (g)	46.2 + 0.615L/r
			Partial restraints at ends and at intermediate joints	$0.5L/r_{yy}$ or L/r_{xx}	120 to 250	Case (g)	46.2 + 0.615L/r

Table 7.31 Categorisation of members according to eccentricity of loading and end restraint conditions

Euler failure load

Euler determined the failure load for a perfect strut of uniform cross-section with hinged ends. The critical buckling load for this strut is given by:

$$P_{cr} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 EA}{\left(\frac{L}{r}\right)^2} \qquad \textit{(7.30)}$$

The effective length for a strut with hinged ends is L.

At values less than π^2 EL/L^2 the strut is in a stable equilibrium. At values of P greater than π^2 EL/L^2 the strut is in a condition of unstable equilibrium and any small disturbance produces final collapse. This is, however, a hypothetical situation because all struts have some initial imperfections and thus the load on the strut can never exceed π^2 EL/L^2. If the thrust P is plotted against the lateral displacement Δ at any section, the P - Δ relationship for a perfect strut will be as shown in Figure 7.31 (a).

In this figure, the lateral deflections occurring after reaching critical buckling load are shown, that is $P_{cr} \geq \pi^2 EI/L^2$, When the strut has small imperfections, displacement is possible for all values of P and the condition of neutral equilibrium P = π^2 EL/L^{2}is never attained. All materials have a limit of proportionality. When this is reached, the flexural stiffness decreases initiating failure before P = π^2 EL/L^2 is reached (Figure 7.31 (b))

Empirical formulae

The following parameters influence the safe compressive stress on the column:

1. Yield stress of material
2. Initial imperfectness
3. (L/r) ratio
4. Factor of safety
5. End fixity condition
6. (b/t) ratio (Figure 7.31)) which controls flange buckling

Figure 7.31 (d) shows a practical application of a twin-angle strut used in a typical bracing system.

Taking these parameters into consideration, the following empirical formulae have been used by different authorities for estimating the safe compressive stress on struts:

1. Straight line formula
2. Parabolic formula
3. Rankine formula
4. Secant or Perry's formula

These formulae have been modified and used in the codes evolved in different countries.

IS: 802 (Part I) -1977 gives the following formulae which take into account all the parameters listed earlier.

For the case b / t $\leq$13 (Figure 7.30 (c)),

$$F_a = \left\{ 2600 - \frac{\left(\frac{KL}{r}\right)^2}{12} \right\} \text{kg/cm}^2 \qquad \textbf{\textit{(7.31)}}$$

Where KL / r $\leq$120

$$F_a = \frac{20 \times 10^6}{\left(\frac{KL}{r}\right)^2} \text{kg/cm}^2 \qquad \textbf{\textit{(7.32)}}$$

Where KL / r > 120

$$F_{cr} = 4680 - 160(b / t) \text{ kg/cm}^2$$

Where 13 < b/t < 20 ***(7.33)***

$$F_{cr} = \frac{590000}{\left(\frac{b}{t}\right)^2} \text{kg} / \text{cm}^2$$

Where b / t > 20 ***(7.34)***

Where

F_a = buckling unit stress in compression,

F_{cr} = limiting crippling stress because of large value of b / t,

b = distance from the edge of fillet to the extreme fibre, and

t = thickness of material.

Equations (7.31) and (7.32) indicate the failure load when the member buckles and Equations (7.33) and (7.34) indicate the failure load when the flange of the member fails.

Figure 7.30 gives the strut formula for the steel with a yield stress of 2600 kg/sq.cm. with respect to member failure. The upper portion of the figure shows the variation of unit stress with KL/r and the lower portion variation of KL/r with L/r. This figure can be used as a nomogram for estimating the allowable stress on a compression member.

An example illustrating the procedure for determining the effective length, the corresponding slenderness ratio, the permissible unit stress and the compressive force for a member in a tower is given below.

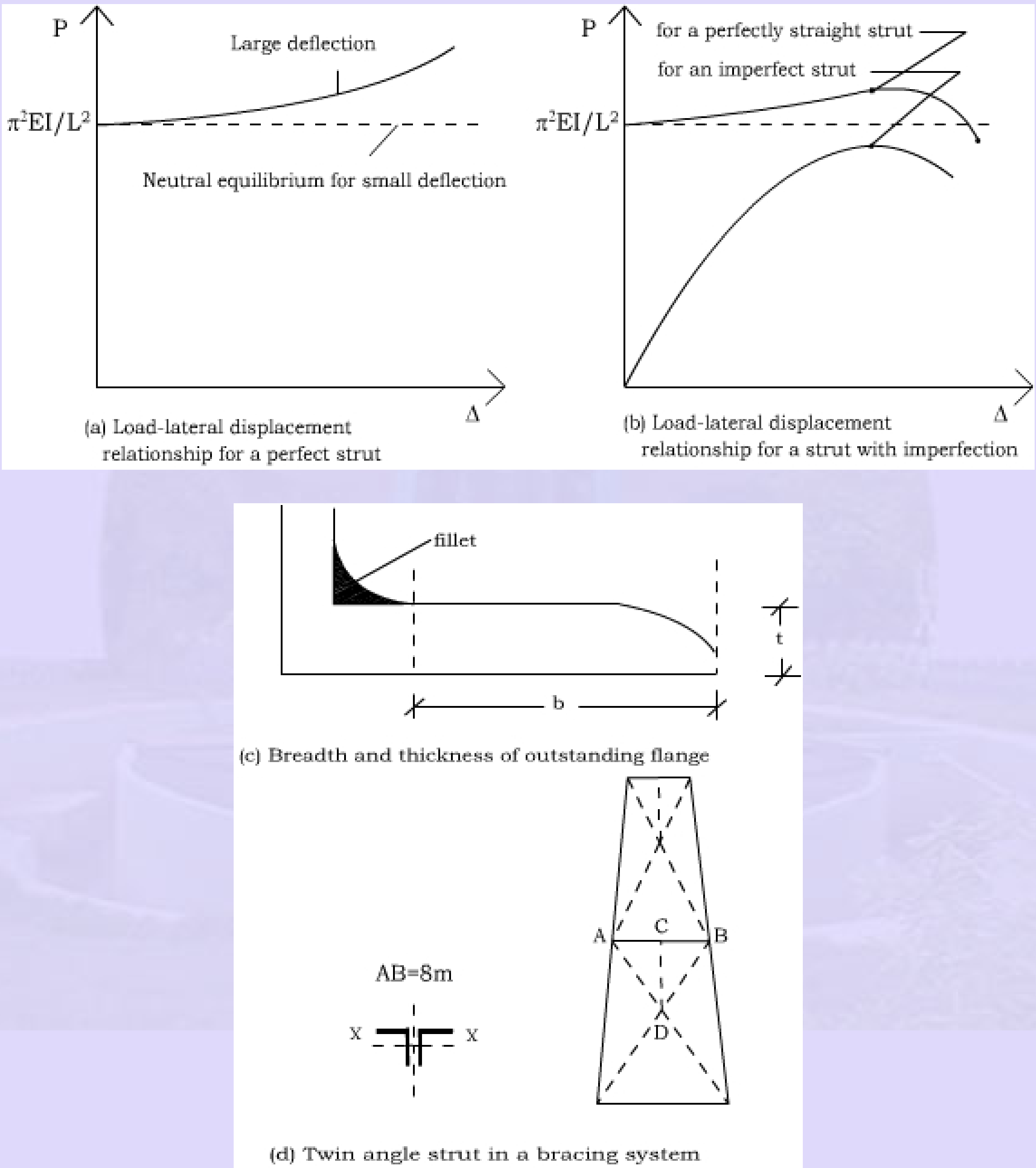


Figure 7.31

Example

Figure 7.31 (d) shows a twin angle bracing system used for the horizontal member of length L = 8 m. In order to reduce the effective length of member AB, single angle CD has been connected to the system. AB is made of two angles 100 x 100mm whose properties are given below:

r_{xx} = 4.38 cm

r_{yy} = 3.05 cm

Area = 38.06 sq.cm.

Double bolt connections are made at A, Band C. Hence it can be assumed that the joints are partially restrained. The system adopted is given at SL. No.8 in Table 7.31. For partial restraint at A, B and C,

$L/r = 0.5\ L/r_{yy}$ or L/r_{xx}

$= 0.5 \times 800/3.05$ or $800/4.38$

= 131.14 or 182.64

The governing value of L/r is therefore182.64, which is the larger of the two values obtained. This value corresponds to case (g) for which KL/r

= 46.2 + 0.615L/r

= 158.52

Note that the value of KL/r from the curve is also 158.52 (Figure 7.30). The corresponding stress from the curve above is 795 kg/cm^2, which is shown dotted in the nomogram. The value of unit stress can also be calculated from equation (7.32). Thus,

$$F_a = \frac{20 \times 10^6}{\left(\frac{KL}{r}\right)^2} \, kg/cm^2$$

$= 20 \times 10^6 / 158.52 \times 158.52$

$= 795 \ kg/cm^2$.

The safe compression load on the strut AB is therefore

F = 38.06 x 795

= 30,257 kg

7.4.2 Computer-aided design

Two computer-aided design methods are in vogue, depending on the computer memory. The first method uses a fixed geometry (configuration) and minimizes the weight of the tower, while the second method assumes the geometry as unknown and derives the minimization of weight.

Method 1: Minimum weight design with assumed geometry

Power transmission towers are highly indeterminate and are subjected to a variety of loading conditions such as cyclones, earthquakes and temperature variations.

The advent of computers has resulted in more rational and realistic methods of structural design of transmission towers. Recent advances in optimisation in structural design have also been incorporated into the design of such towers.

While choosing the member sizes, the large number of structural connections in three dimensions should be kept in mind. The selection of members is influenced by their position in relation to the other members and the end connection conditions. The leg sections which carry different stresses at each panel may be assigned different sizes at various levels; but consideration of the large number of splices involved indicates that it is usually more economical and convenient, even though heavier, to use the same section for a number of panels. Similarly, for other members, it may be economical to choose a section of relatively large flange width so as to eliminate gusset plates and correspondingly reduce the number of bolts.

In the selection of structural members, the designer is guided by his past experience gained from the behavior of towers tested in the test station or actually in service. At certain critical locations, the structural members are provided with a higher margin of safety, one example being the horizontal members where the slope of the tower changes and the web members of panels are immediately below the neckline.

Optimisation

Many designs are possible to satisfy the functional requirements and a trial and error procedure may be employed to choose the optimal design. Selection of the best geometry of a tower or the member sizes is examples of optimal design procedures. The computer is best suited for finding the optimal solutions. Optimisation then becomes an automated design procedure, providing the optimal values for certain design quantities while considering the design criteria and constraints.

Computer-aided design involving user-ma- chine interaction and automated optimal design, characterized by pre-programmed logical decisions, based upon internally stored information, are not mutually exclusive, but complement each other. As the techniques of interactive computer-aided design develop, the need to employ standard routines for automated design of structural subsystems will become increasingly relevant.

The numerical methods of structure optimisation, with application of computers, automatically generate a near optimal design in an iterative manner. A finite number of variables has to be established, together with the constraints, relating to these variables. An initial guess-solution is used as the starting point for a systematic search for better designs and the process of search is terminated when certain criteria are satisfied.

Those quantities defining a structural system that are fixed during the automated design are- called pre-assigned parameters or simply parameters and those quantities that are not pre-assigned are called design variables. The design variables cover the material properties, the topology of the structure, its geometry and the member sizes. The assignment of the parameters as well as the definition of their values is made by the designer, based on his experience.

Any set of values for the design variables constitutes a design of the structure. Some designs may be feasible while others are not. The restrictions that must be satisfied in order to produce a feasible design are called constraints. There are two kinds of constraints: design constraints and behavior constraints. Examples of design constraints are minimum thickness of a member, maximum height of a structure, etc. Limitations on the maximum stresses, displacements or buck- ling strength are typical examples of behavior constraints. These constraints are expressed ma- thematically as a set of inequalities:

$$g_j\left(\{X\}\right) \le 0 \qquad j = 1, 2, ..., m \qquad \textbf{\textit{(7.35a)}}$$

Where {X} is the design vector, and

m is the number of inequality constraints.

In addition, we have also to consider equality constraints of the form

$$h_j\left(\{X\}\right) \le 0 \qquad j = 1, 2, ..., k \qquad \textbf{\textit{(7.35b)}}$$

Where k is the number of equality constraints.

Example

The three bar truss example first solved by Schmit is shown in Figure 7.32. The applied loadings and the displacement directions are also shown in this figure.

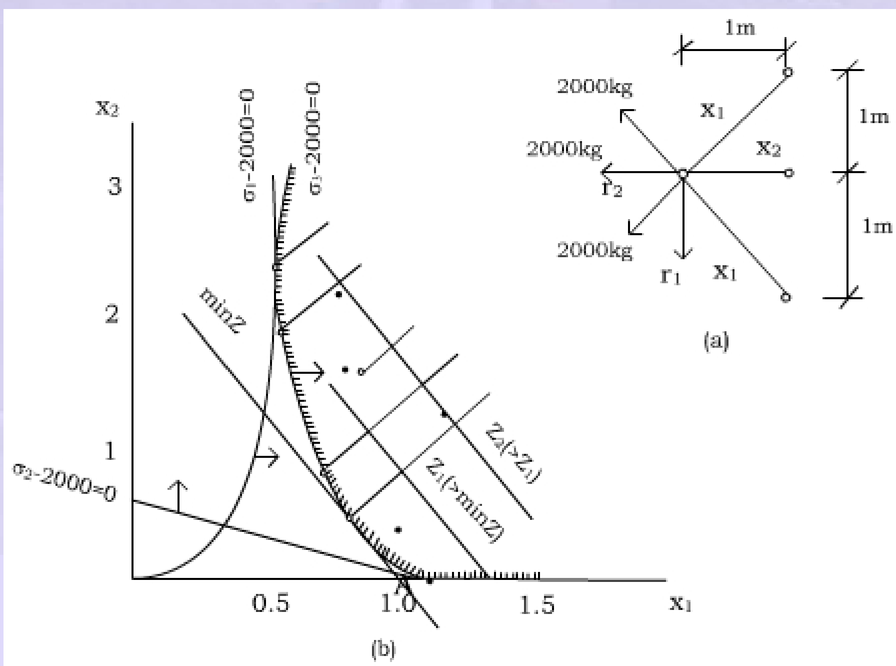


Figure 7.32 Two dimensional plot of the design variables X_1 and X_2

1. **Design constraints:** The condition that the area of members cannot be less than zero can be expressed as

$$g_1 \equiv -X_1 \le 0$$
$$g_2 \equiv -X_2 \le 0$$

2. Behaviour constraints: The three members of the truss should be safe, that is, the stresses in them should be less than the allowable stresses in tension (2,000 kg/cm^2) and compression (1,500kg/cm^2). This is expressed as

$g_3 \equiv \sigma_1 - 2,000 \le 0$ Tensile stress limitation in member 1

$g_4 \equiv -\sigma_1 - 1,500 \le 0$

$g_5 \equiv \sigma_2 - 2,000 \le 0$

$g_6 \equiv -\sigma_2 - 1,500 \le 0$ Compressive stress limitation in member 2 and so on

$g_7 \equiv \sigma_3 - 2,000 \le 0$

$g_8 \equiv -\sigma_3 - 1,500 \le 0$

3. Stress force relationships: Using the stress-strain relationship σ = [E] {Δ} and the force-displacement relationship F = [K] {Δ}, the stress-force relationship is obtained as {s} = [E] [K]$^{-1}$[F] which can be shown as

$$\sigma_1 = 2000\left(\frac{X_2 + \sqrt{2}X_1}{2X_1X_2 + \sqrt{2}X_1^2}\right)$$
$$\sigma_2 = 2000\left(\frac{\sqrt{2}X_1}{2X_1X_2 + \sqrt{2}X_1^2}\right)$$
$$\sigma_1 = 2000\left(\frac{X_2}{2X_1X_2 + \sqrt{2}X_1^2}\right)$$

4. Constraint design inequalities: Only constraints g_3, g_5, g_8 will affect the design. Since these constraints can now be expressed in terms of design variables X_1 and X_2 using the stress force relationships derived above, they can be represented as the area on one side of the straight line shown in the two-dimensional plot (Figure 7.32 (b)).

Design space

Each design variable X_1, X_2 ...is viewed as one- dimension in a design space and a particular set of variables as a point in this space. In the general case of n variables, we have an n-dimensioned space. In the example where we have only two variables, the space reduces to a plane figure shown in Figure 7.32 (b). The arrows indicate the inequality representation and the shaded zone shows the feasible region. A design falling in the feasible region is an unconstrained design and the one falling on the boundary is a constrained design.

Objective function

An infinite number of feasible designs are possible. In order to find the best one, it is necessary to form a function of the variables to use for comparison of feasible design alternatives. The objective (merit) function is a function whose least value is sought in an optimisation procedure. In other words, the optimization problem consists in the determination of the vector of variables X that will minimise a certain given objective function:

$$Z = F(\{X\}) \qquad \textbf{\textit{7.35(c)}}$$

In the example chosen, assuming the volume of material as the objective function, we get

$$Z = 2(141\ X_1) + 100\ X_2$$

The locus of all points satisfying F ({X}) = constant, forms a straight line in a two-dimensional space. In this general case of n-dimensional space, it will form a surface. For each value of constraint, a different straight line is obtained. Figure 7.32 (b) shows the objective function contours. Every design on a particular contour has the same volume or weight. It can be seen that the minimum value of F ({X}) in the feasible region occurs at point A.

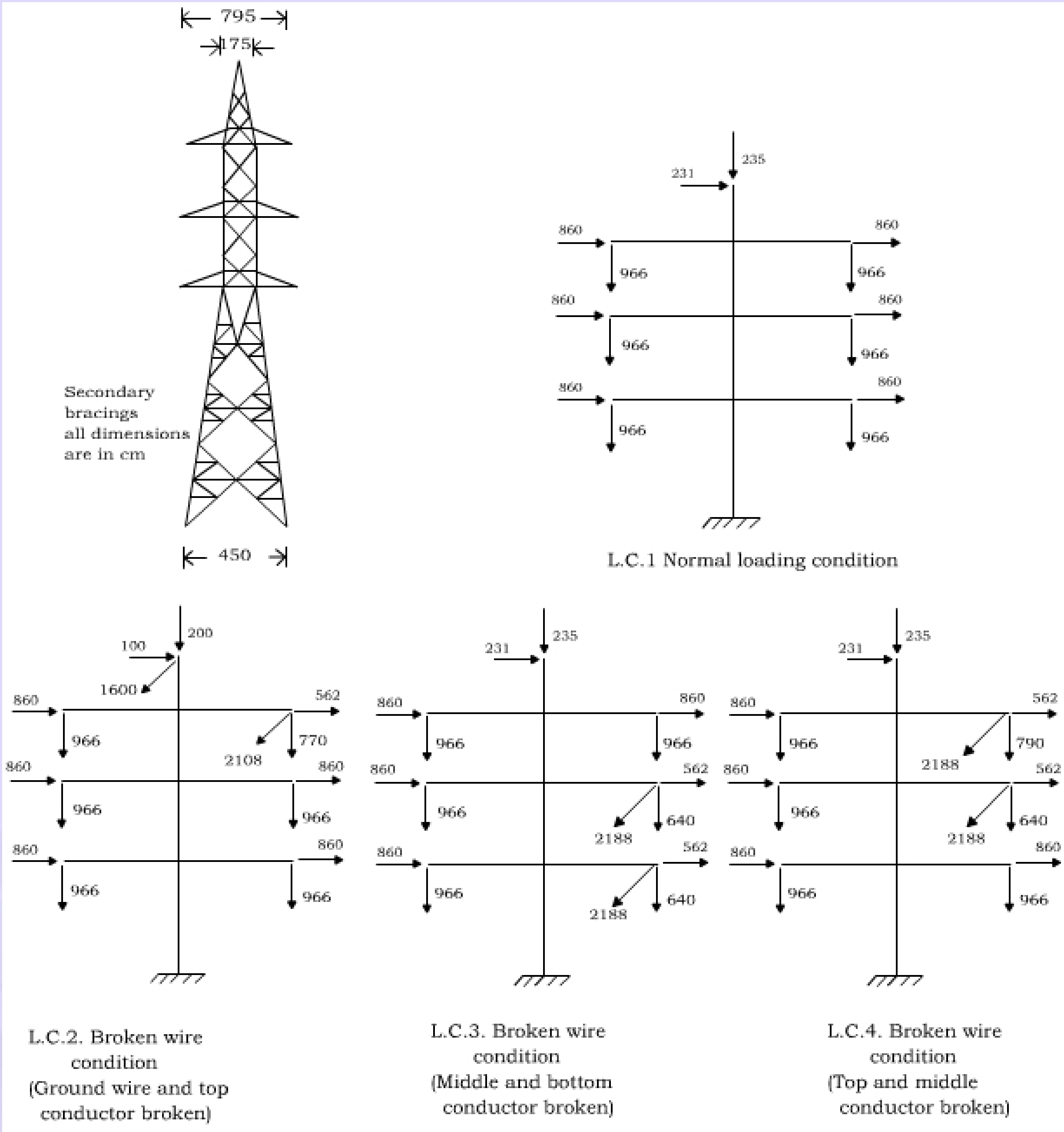


Figure 7.33 Configuration and loading condition for the example tower

There are different approaches to this problem, which constitute the various methods of optimization. The traditional approach searches the solution by pre-selecting a set of critical constraints and reducing the problem to a set of equations in fewer variables. Successive reanalysis of the structure for improved sets of constraints will tend towards the solution. Different re-analysis methods can be used, the iterative methods being the most attractive in the case of towers.

Optimality criteria

An interesting approach in optimization is a process known as optimality criteria. The approach to the optimum is based on the assumption that some characteristics will be attained at such optimum. The well-known example is the fully stressed design where it is assumed that, in an optimal structure, each member is subjected to its limiting stress under at least one loading condition.

The optimality criteria procedures are useful for transmission lines and towers because they constitute an adequate compromise to obtain practical and efficient solutions. In many studies, it has been found that the shape of the objective function around the optimum is flat, which means that an experienced designer can reach solutions, which are close to the theoretical optimum.

Mathematical programming

It is difficult to anticipate which of the constraints will be critical at the optimum. Therefore, the use of inequality constraints is essential for a proper formulation of the optimal design problem.

The mathematical programming (MP) methods are intended to solve the general optimisation problem by numerical search algorithms while being general regarding the objective function and constraints. On the other hand, approximations are often required to be efficient on large practical problems such as tower optimisation.

Optimal design processes involve the minimization of weight subject to certain constraints. Mathematical programming methods and structural theorems are available to achieve such a design goal.

Of the various mathematical programming methods available for optimisation, the linear programming method is widely adopted in structural engineering practice because of its simplicity. The objective function, which is the minimisation of weight, is linear and a set of constraints, which can be expressed by linear equations involving the unknowns (area, moment of inertia, etc. of the members), are used for solving the problems. This can be mathematically expressed as follows.

Suppose it is required to find a specified number of design variables x_1, x_2.....x_n such that the objective function

$$Z = C_1 x_1 + C_2 x_2 + ...C_n x_n$$

is minimised, satisfying the constraints

$$\begin{aligned} &a_{11}x_1 + a_{12}x_2 + ...a_{1n}x_n \le b_1 \\ &a_{21}x_1 + a_{22}x_2 + ...a_{2n}x_n \le b_2 \\ &. \\ &. \\ &. \\ &a_{m1}x_1 + a_{m2}x_2 + ...a_{mn}x_n \le b_m \end{aligned} \qquad \textbf{\textit{(7.36)}}$$

The simplex algorithm is a versatile procedure for solving linear programming (LP) problems with a large number of variables and constraints.

The simplex algorithm is now available in the form of a standard computer software package, which uses the matrix representation of the variables and constraints, especially when their number is very large.

The equation (7.36) is expressed in the matrix form as follows:

Find $X = \begin{Bmatrix} x_1 \\ x_2 \\ - \\ - \\ x_n \end{Bmatrix}$ which minimises the objective function

$$f\left(x\right) = \sum_{i-1}^{n} C_i x_i \qquad \textbf{\textit{(7.37)}}$$

subject to the constraints,

$$\begin{aligned} &\sum_{k-1}^{n} a_{jk} x_k = b_j, \;\; j = 1, 2, ...m \\ &\text{and} x_i \geq 0, \;\; i = 1, 2, ...n \end{aligned} \qquad \textbf{\textit{(7.38)}}$$

where C_i, a_{jk} and b_j are constants.

The stiffness method of analysis is adopted and the optimisation is achieved by mathematical programming.

The structure is divided into a number of groups and the analysis is carried out group wise. Then the member forces are determined. The critical members are found out from each group. From the initial design, the objective function and the constraints are framed. Then, by adopting the fully stressed design (optimality criteria) method, the linear programming problem is solved and the optimal solution found out. In each group, every member is designed for the fully stressed condition and the maximum size required is assigned for all the members in that group. After completion of the design, one more analysis and design routine for the structure as a whole is completed for alternative cross-sections.

Example

A 220 k V double circuit tangent tower is chosen for study. The basic structure, section plan at various levels and the loading conditions are tentatively fixed. The number of panels in the basic determinate structure is 15 and the number of members is 238. Twenty standard sections have been chosen in the increasing order of weight. The members have been divided into eighteen groups, such as leg groups, diagonal groups and horizontal groups, based on various panels of the tower. For each group a section is specified.

Normal loading conditions and three broken- wire conditions has been considered. From the vertical and horizontal lengths of each panel, the lengths of the members are calculated and the geometry is fixed. For the given loading conditions, the forces in the various members are computed, from which the actual stresses are found. These are compared with allowable stresses and the most stressed member (critical) is found out for each group. Thereafter, an initial design is evolved as a fully stressed design in which critical members are stressed up to an allowable limit. This is given as the initial solution to simplex method, from which the objective function, namely, the weight of the tower, is formed. The initial solution so obtained is sequentially improved, subject to the constraints, till the optimal solution is obtained.

In the given solution, steel structural angles of weights ranging from 5.8 kg/m to 27.20 kg/m are utilised. On the basis of the fully stressed design, structural sections of 3.4 kg/m to 23.4 kg/m are indicated and the corresponding weight is 5,398 kg. After the optimal solution, the weight of the tower is 4,956 kg, resulting in a saving of about 8.1 percent.

Method 2: Minimum weight design with geometry as variable

In Method 1, only the member sizes were treated as variables whereas the geometry was assumed as fixed. Method 2 treats the geometry also as a variable and gets the most preferred geometry. The geometry developed by the computer results in the minimum weight of tower for any practically acceptable configuration. For solution, since an iterative procedure is adopted for the optimum structural design, it is obvious that the use of a computer is essential.

The algorithm used for optimum structural design is similar to that given by Samuel L. Lipson which presumes that an initial feasible configuration is available for the structure. The structure is divided into a number of groups and the externally applied loadings are obtained. For the given configuration, the upper limits and the lower limits on the design variables, namely, the joint coordinates are fixed. Then (k-1) new configurations are generated randomly as

$$x_{ij} = l_i + r_{ij}(u_i - l_i) \qquad \textbf{\textit{(7.39)}}$$

$$i = 1, 2 \ldots n$$

$$j = 1, 2 \ldots k$$

where k is the total number of configurations in the complex, usually larger than (n + 1), where n is the number of design variables and r_{ij} is the random number for the i^{th} coordinate of the j^{th} point, the random numbers having a uniform distribution over the interval 0 to 1 and u_i is the upper limit and L_i is the lower limit of the i^{th} independent variable.

Thus, the complex containing k number of feasible solutions is generated and all these configurations will satisfy the explicit constraints, namely, the upper and lower bounds on the design variables. Next, for all these k configurations, analysis and fully stressed designs are carried out and their corresponding total weights determined. Since the fully stressed design concept is an eco nomical and practical design, it is used for steel area optimisation. Every area optimisation problem is associated with more than one analysis and design. For the analysis of the truss, the matrix method described in the previous chapter has been used. Therefore, all the generated configurations also satisfy the implicit constraints, namely, the allowable stress constraints.

From the value of the objective function (total weight of the structure) of k configurations, the vector, which yields the maximum weight, is searched and discarded, and the centroid c of each joint of the k-1 configurations is determined from

$$x_{ic} = \frac{1}{K-1}\left\{ K\sum_{j-1}\left(x_{ij}\right) - x_{iw} \right\} \qquad \textbf{\textit{(7.40)}}$$

i = 1, 2, 3 ... n

in which x_{ic} and x_{iw} are the i^{th} coordinates of the centroid c and the discarded point w.

Then a new point is generated by reflecting the worst point through the centroid, x_{ic}

That is, $x_{iw} = x_{ic} + \alpha (x_{ic} - x_{iw})$

(7.41)

i = 1,2,..... n where α is a constant.

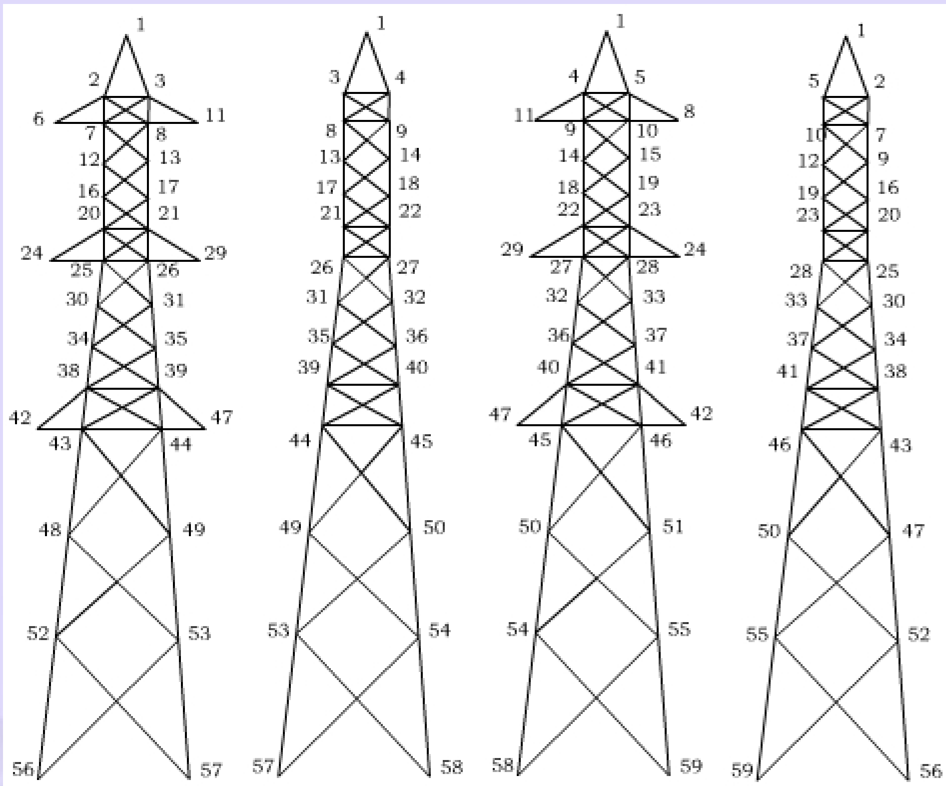


Figure 7.34 Node numbers

This new point is first examined to satisfy the explicit constraints. If it exceeds the upper or lower bound value, then the value is taken as the corresponding limiting value, namely, the upper or lower bound. Now the area optimisation is carried out for the newly generated configuration and the functional value (weight) is determined. If this functional value is better than the second worst, the point is accepted as an improvement and the process of developing the new configuration is repeated as mentioned earlier. Otherwise, the newly generated point is moved halfway towards the centroid of the remaining points and the area optimisation is repeated for the new configuration.

This process is repeated over a fixed number of iterations and at the end of every iteration, the weight and the corresponding configuration are printed out, which will show the minimum weight achievable within the limits (l and u) of the configuration.

Example

The example chosen for the optimum structural design is a 220 k V double-circuit angle tower. The tower supports one ground wire and two circuits containing three conductors each, in vertical configuration, and the total height of the tower is 33.6 metres. The various load conditions are shown in Figure 7.33.

The bracing patterns adopted are Pratt system and Diamond system in the portions above and below the bottom-most conductor respectively. The initial feasible configuration is shown on the top left corner of Figure 7.33. Except x, y and z coordinates of the conductor and the z coordinates of the foundation points, all the other joint coordinates are treated as design variables. The tower configuration considered in this example is restricted to a square type in the plan view, thus reducing the number of design variables to 25.

In the initial complex, 27 configurations are generated, including the initial feasible configuration. Random numbers required for the generation of these configurations are fed into the comJ7llter as input. One set containing 26 random numbers with uniform distribution over the interval 0 to 1 are supplied for each design variable. Figure 7.34 and Figure 7.35 show the node numbers and member numbers respectively.

The example contains 25 design variables, namely, the x and y coordinates of the nodes, except the conductor support points and the z coordinates of the support nodes (foundations) of the tower. 25 different sets of random numbers, each set containing 26 numbers, are read for 25 design variables. An initial set of27 configuration is generated and the number of iterations for the development process is restricted to 30. The weight of the tower for the various configurations developed during optimisation procedure is pictorially represented in Figure 7.36. The final configuration is shown in Figure 7.37a and the corresponding tower weight, including secondary bracings, is 5,648 kg.

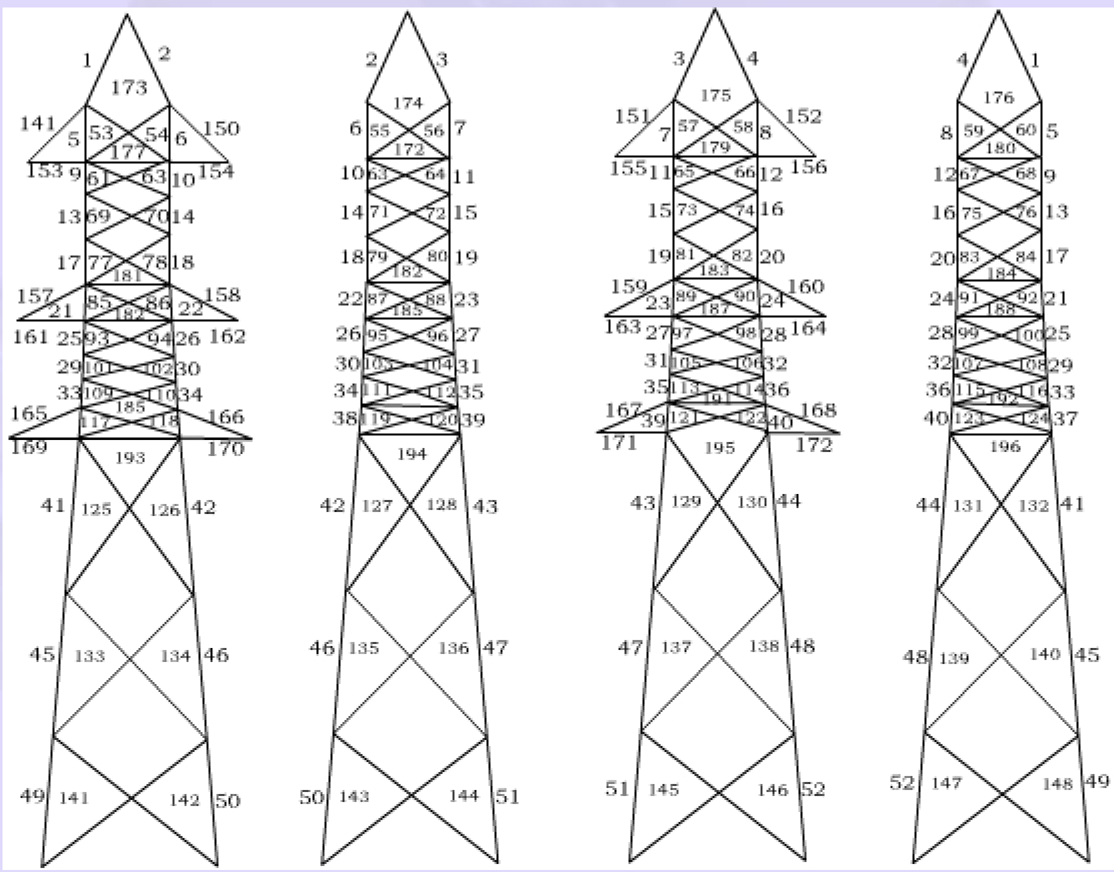


Figure 7.35 Member numbers

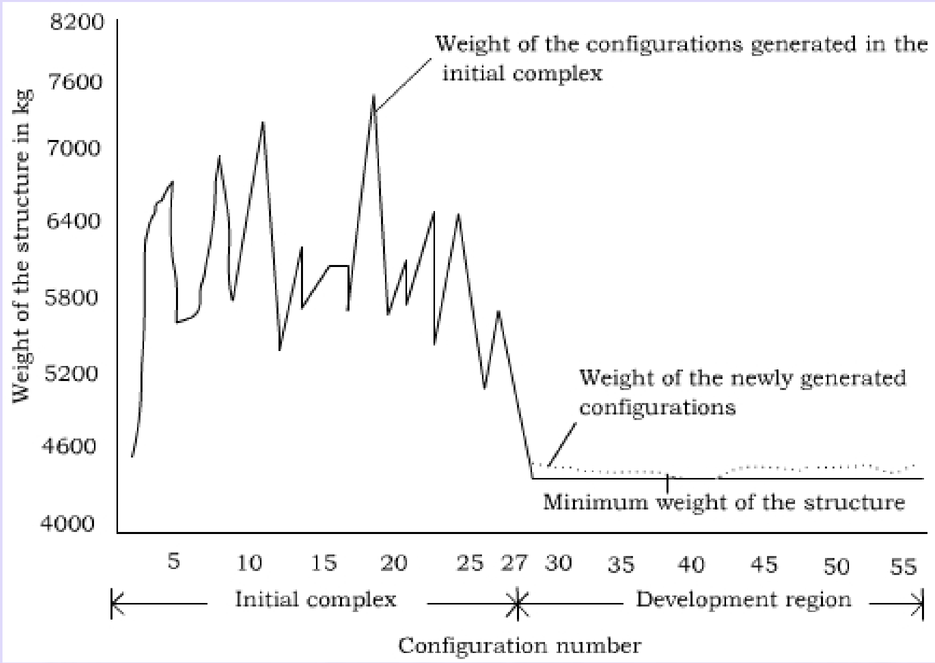


Figure 7.36 Tower weights for various configurations generated

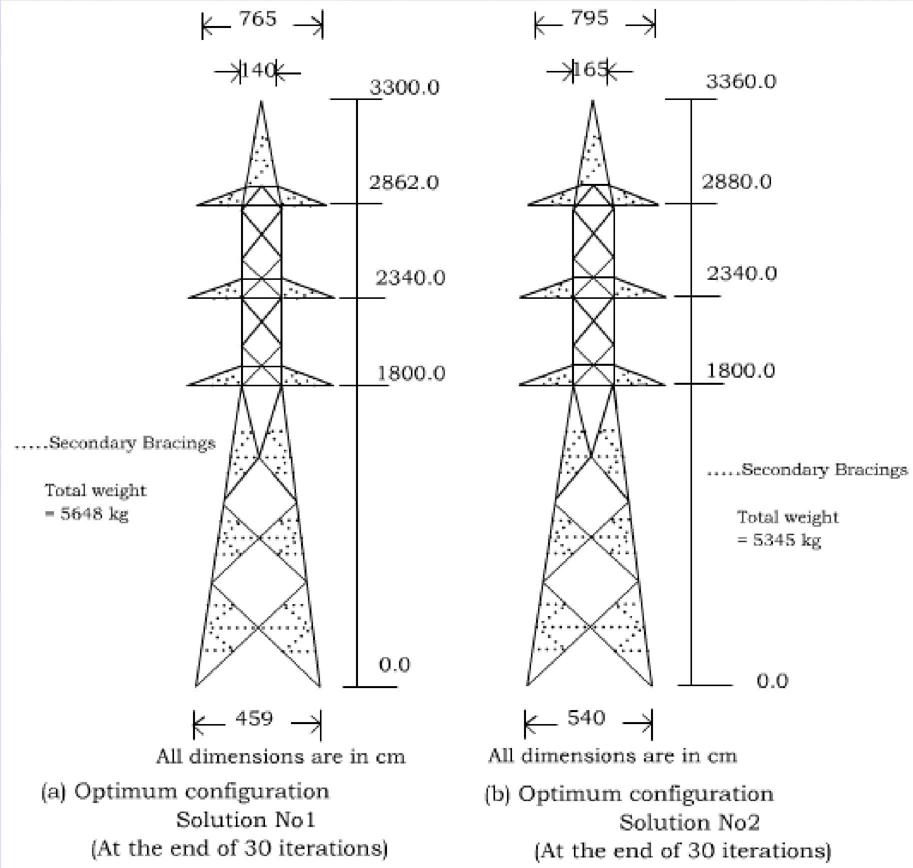


Figure 7.37

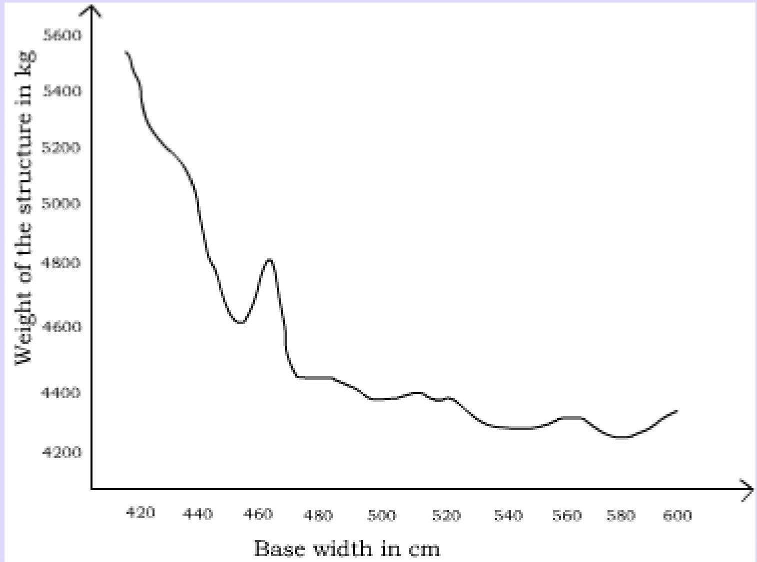


Figure 7.38 Variation of tower weight with base width

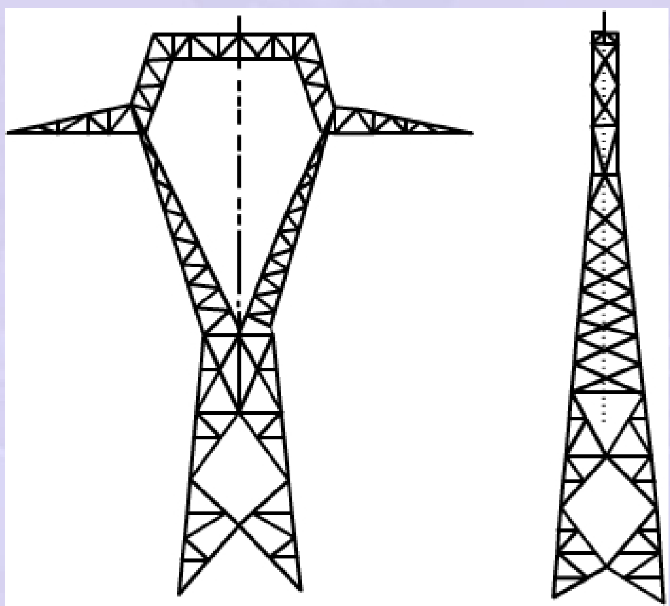


Figure 7.39 Tower geometry describing key joints and joints obtained from key joints

This weight can further be reduced by adopting the configuration now obtained as the initial configuration and repeating the search by varying the controlling coordinates x and z. For instance, in the present example, by varying the x coordinate, the tower weight has been reduced to 5,345 kg and the corresponding configuration is shown in Figure 7.37b. Figure 7.38 shows the variation of tower weight with base width.

In conclusion, the probabilistic evaluation of loads and load combinations on transmission lines, and the consideration of the line as a whole with towers, foundations, conductors and hardware, forming interdependent elements of the total sys- tem with different levels of safety to ensure a preferred sequence of failure, are all directed towards achieving rational behaviour under various uncertainties at minimum transmission line cost. Such a study may be treated as a global optimisation of the line cost, which could also include an examination of alternative uses of various types of towers in a family, materials to be employed and the limits to which different towers are utilised as discrete variables and the objective function as the overall cost.

7.4.3 Computer software packages

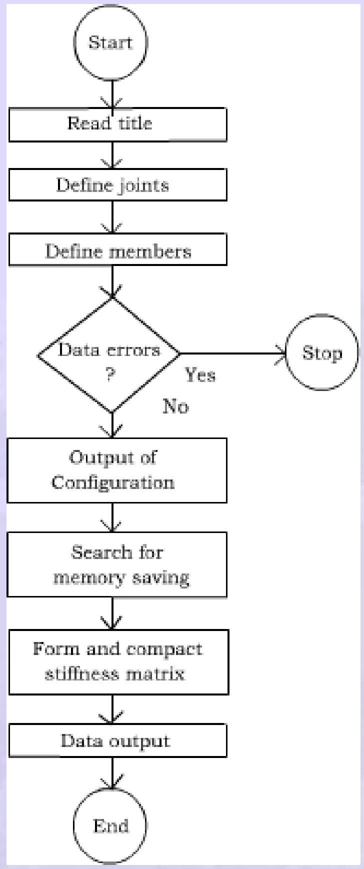


Figure 7.40 Flowchart for the development of tower geometry in the OPSTAR program

The general practice is to fix the geometry of the tower and then arrive at the loads for design purposes based on which the member sizes are determined. This practice, however, suffers from the following disadvantages:

1. The tower weight finally arrived at may be different from the assumed design weight.

2. The wind load on tower calculated using assumed sections may not strictly correspond to the actual loads arrived at on the final sections adopted.

3. The geometry assumed may not result in the economical weight of tower.

4. The calculation of wind load on the tower members is a tedious process.

Most of the computer software packages available today do not enable the designer to overcome the above drawbacks since they are meant essentially to analyse member forces.

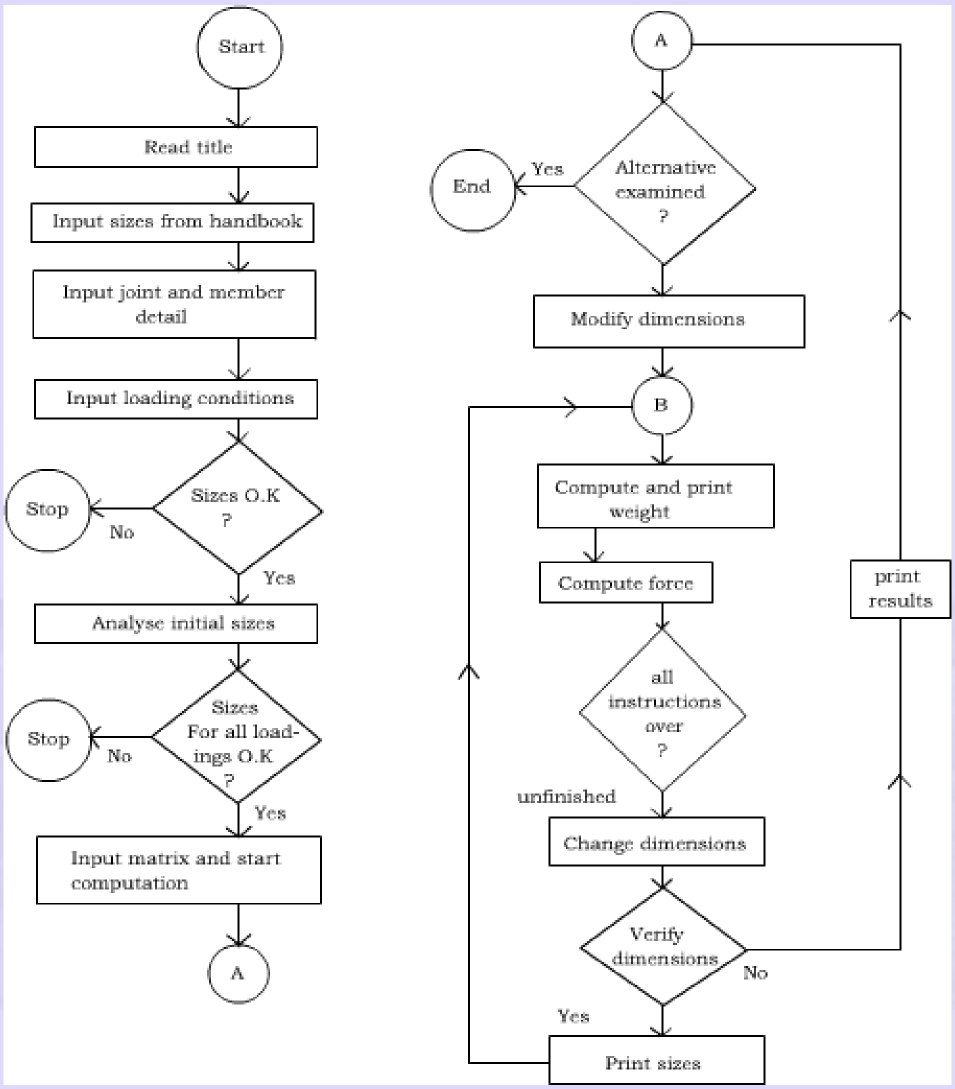


Figure 7.41 Flowchart for the solution sequence (opstar programme)

In Electricite de France (EDF), the OPSTAR program has been used for developing economical and reliable tower designs. The OPSTAR program optimises the tower member sizes for a fixed configuration and also facilitates the development of new configurations (tower outlines), which will lead to the minimum weight of towers. The salient features of the program are given below:

Geometry: The geometry of the tower is described by the coordinates of the nodes. Only the coordinates of the key nodes (8 for a tower in Figure 7.39) constitute the input. The computer generates the other coordinates, making use of symmetry as well as interpolation of the coordinates of the nodes between the key nodes. This simplifies and minimises data input and aids in avoiding data input errors.

Solution technique: A stiffness matrix approach is used and iterative analysis is performed for optimisation.

Description of the program: The first part of the program develops the geometry (coordinates) based on data input. It also checks the stability of the nodes and corrects the unstable nodes. The flow chart for this part is given in Figure 7.40.

The second part of the program deals with the major part of the solution process. The input data are: the list of member sections from tables in handbooks and is based on availability; the loading conditions; and the boundary conditions.

The solution sequence is shown in Figure 7.41. The program is capable of being used for either checking a tower for safety or for developing a new tower design. The output from the program includes tower configuration; member sizes; weight of tower; foundation reactions under all loading conditions; displacement of joints under all loading conditions; and forces in all members for all loading conditions.

7.4.4 Tower accessories

Designs of important tower accessories like Hanger, Step bolt, Strain plate; U-bolt and D-shackle are covered in this section. The cost of these tower accessories is only a very small fraction of the S overall tower cost, but their failure will render the tower functionally ineffective. Moreover, the towers have many redundant members whereas the accessories are completely determinate. These accessories will not allow any load redistribution, thus making failure imminent when they are overloaded. Therefore, it is preferable to have larger factors of safety associated with the tower accessories than those applicable to towers.

Hanger (Figure 7.42)

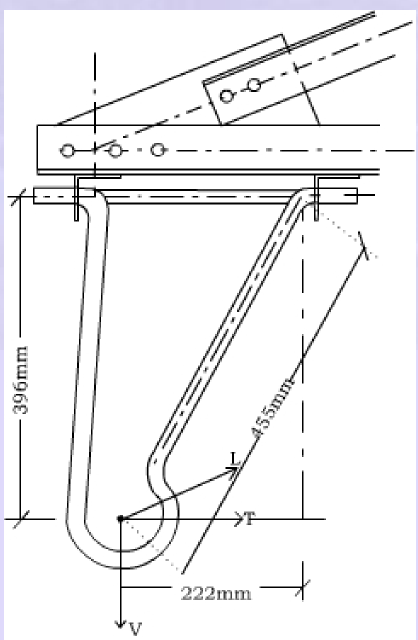


Figure 7.42 Hanger

The loadings coming on a hanger of a typical 132 kV double-circuit tower are given below:

Type of loading	NC	BWC
Transverse	480kg	250kg
Vertical	590kg	500kg
Longitudinal	-	2,475kg

Maximum loadings on the hanger will be in the broken-wire condition and the worst loaded member is the vertical member.

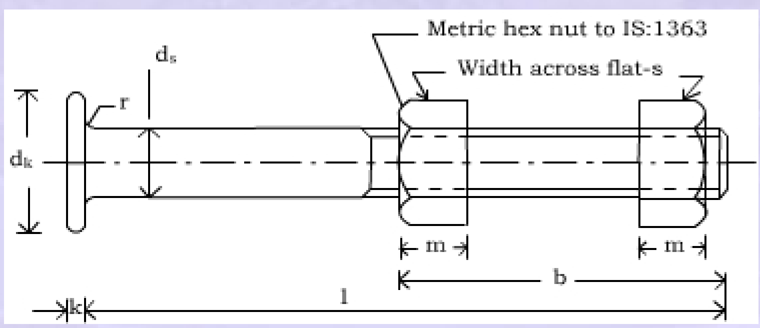
Diameter of the hanger leg = 21mm

Area = p x $(21)^2$ / 4 x 100 = 3.465 sq.cm.

Maximum allowable tensile stress for the steel used = 3,600 kg/cm^2

Allowable load = 3,600 x 3.465

= 12,474 kg.



Dimensions															
Nom bolt dia	threads	Shank dia d_s		Head dia d_k		Head thickness k		Neck radius (app) r	Bolt length l		Thread length b		Width across flats s		Nut thickness m
Metric Serious (dimensions in mm before galvanising)															
16	m 16	16	+1.10 -0.43	35	+2 -0	6	+1 -0	3	175	+3 -0	60	+5 -0	24	+0 -0.84	13 ± 0.55

Bolts	Nuts
1. Tensile strength - 400 N/mm^2 min.	1. Proof load stress - 400 N/mm^2
2. Brinell Hardness- HB 114/209	2. Brinell Hardness- HB 302 max
3. Cantilever load test - with 150kg	

Figure 7.43 Dimensions and mechanical properties of step bolts and nuts

Loads in the vertical leg

1. Transverse load (BWC) = 250 / 222 x 396

   = 446kg.

2. Longitudinal load = 2,475 kg.

3. Vertical load = 500 kg.

   Total = 3,421kg.

It is unlikely that all the three loads will add up to produce the tension in the vertical leg. 100 percent effect of the vertical load and components of longitudinal and transverse load will be acting on the critical leg to produce maximum force. In accordance with the concept of making the design conservative, the design load has been assumed to be the sum of the three and hence the total design load = 3,421 kg.

Factor of safety = 12,474 / 3,421 = 3.65 which is greater than 2, and hence safe

Step bolt (Figure 7.43)

Special mild steel hot dip galvanised bolts called step bolts with two hexagonal nuts each, are used to gain access to the top of the tower structure. The design considerations of such a step bolt are given below.

The total uniformly distributed load over the fixed length = 100 kg (assumed).

The maximum bending moment

100 x 13 / 2 = 650 kg cm.

The moment of inertia = p x 16^4 / 64 = 0.3218 cm^4

Maximum bending stress = 650 x 0.8 / 0.3218

= 1,616 kg/cm^2

Assuming critical strength of the high tensile steel = 3,600 kg/cm^2,

factor of safety = 3,600 / 1,616 = 2.23, which is greater than 2, and hence safe.

Step bolts are subjected to cantilever load test to withstand the weight of man (150kg).

Strain plate (Figure 7.44)

The typical loadings on a strain plate for a 132 kV double-circuit tower are given below:

Vertical load = 725kg

Transverse load = 1,375kg

Longitudinal load = 3,300kg

Bending moment due to vertical load = 725 x 8 / 2 = 2,900kg.cm.

I_{xx} = 17 x $(0.95)^3$ / 12 = 1.2146 cm^4

y (half the depth) = 0.475cm.

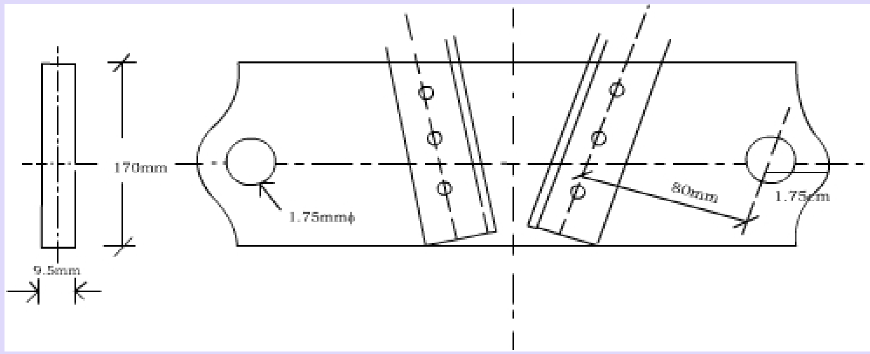


Figure 7.44 Strain plate

Section modulus Z_{xx} = 1.2146 / 0.475 = 2.5568

Bending stress f_{xx} = 2,900 / 2.5568 = 1,134 kg/cm^2

Bending moment due to transverse load = 1.375 x 8 / 2 = 5,500 kg.cm.

Actually the component of the transverse load in a direction parallel to the line of fixation should be taken into account, but it is safer to consider the full transverse load.

I_{yy} = 0.95 x 17^3 / 12 = 389 cm^4

Z_{yy} = 389 / 8.5 = 45.76

Bending stress f_{yy} = 5500 / 45.76 = 120 kg/cm^2

Total maximum bending stress

$f_{xx} + f_{yy}$ = 1,134 + 120 = 1,254 kg/cm^2

Direct stress due to longitudinal load = longitudinal load / Cross-sectional area

= 3,300 / 13.5 x 0.95

= 257.3 kg/cm^2

Check for combined stress

The general case for a tie, subjected to bending and tension, is checked using the following interaction relationship:

$$\frac{f_b}{F_b} + \frac{f_t}{F_T} \leq 1 \qquad \textbf{\textit{(7.36)}}$$

Where f_t = actual axial tensile stress,

f_b = actual bending tensile stress

F_t = permissible axial tensile stress, and

F_b = permissible bending tensile stress.

Assuming F_t = 1,400 kg/cm^2 and F_b = 1,550 kg/cm^2. The expression reduces to

= 1,254 / 1,550 + 257.3 / 1,400

= 0.9927 <1, hence safe.

Check for the plate in shear

Length of the plate edge under shear = 1.75 cm

Area under shear = 2 x 1.75 x 0.95

= 3.325 sq.cm.

Shearing stress =3,300 / 3,325 = 992 kg/cm^2

Permissible shear stress = 1,000 kg/cm^2

Hence, it is safe in shear.

Check for the plate in bearing

Pin diameter = 19mm

Bearing area = 1.9 x 0.95 = 1.805 cm^2

Maximum tension in the conductor = 3,300 kg.

Bearing stress = = 1,828 kg/cm^2

Permissible bearing stress = 1860 kg/cm^2

Hence, it is safe in bearing.

Check for bolts in shear

Diameter of the bolt = 16mm

Area of the bolt = 2.01 sq.cm.

Shear stress = 3,300 / 3 x 2.01 = 549 kg/cm^2

Permissible shearing stress = 1,000 kg/cm^2

Hence, three 16mm diameter bolts are adequate.

U-bolt (Figure 7.45)

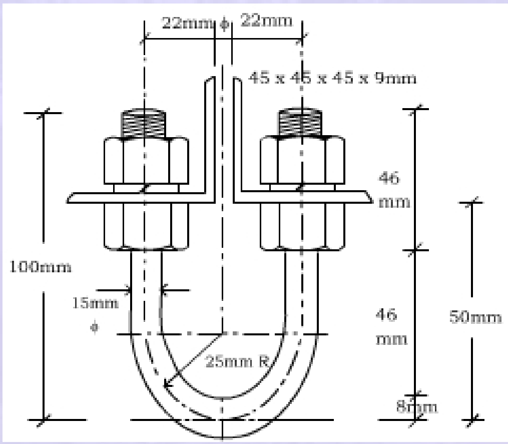


Figure 7.45 U-bolt

The loadings in a U-bolt for a typical 66 kV double circuit tower are given below:

	NC	BWC
Transverse load	= 216	108
Vertical load	= 273	227
Longitudinal load	= -	982

Permissible bending stress for mild steel = 1,500 kg/cm^2

Permissible tensile stress = 1,400 kg/cm^2

Let the diameter of the leg be 16mm.

The area of the leg = 2.01 sq.cm.

1. Direct stress due to vertical load = 273 / 2.01 x 2

= 67.91 kg/cm^2

2. Bending due to transverse load (NC)

Bending moment = 216 x 5 = 1,080 kg.cm

Section Modulus = 2 x πd^3 / 32 = 2 x 3.14 x 1.6^3 / 32

= 0.804

Bending stress = 1,080 / 0.804

= 1,343 kg/cm^2< 1,500 kg/cm^2

Hence safe.

3. Bending due to longitudinal load (BWC)

Bending moment = 982 x 5 = 4,910 kg.cm

$$I_{xx} = \left(\frac{\pi d^4}{64} + \frac{\pi d^2}{4} \text{ x } 2.5^2\right)^2 = 25.77 \text{cm}^4$$

y = 2.5 + 0.8

Bending stress = 4, 90 / 25.77 x (2.5 + 0.8) = 629 kg/cm^2

In the broken-wire condition total bending stress = 1,343 / 2 + 629

= 1,300 kg/cm^2

Hence, the worst loading will occur during normal condition.

For safe design,

$$\frac{f_b}{F_b} + \frac{f_t}{F_t} \le 1$$

$$67.91 / 1400 + 1343 / 1500 = 0.9365 < 1$$

Hence safe.

Bearing strength of the angle-bolt connection

Safe bearing stress for the steel used = 4,725 kg/cm^2

Diameter of hole = 16mm + 1.5mm = 17.5mm

Thickness of the angle leg = 5mm

Under normal condition

Bearing stress = (216 + 273)/1.75 x 0.5 = 558.85 kg/cm^2

Factor of safety = 4,725 / 558.85 = 8.45

Under broken-wire condition

Bearing stress = (108+227+982) / 1.75 x 0.5

= 1,505.14 kg/cm^2

Therefore, factor of safety = 4,725 /1,505.14 = 3.13

Hence safe.

D-shackle (Figure 7.46)

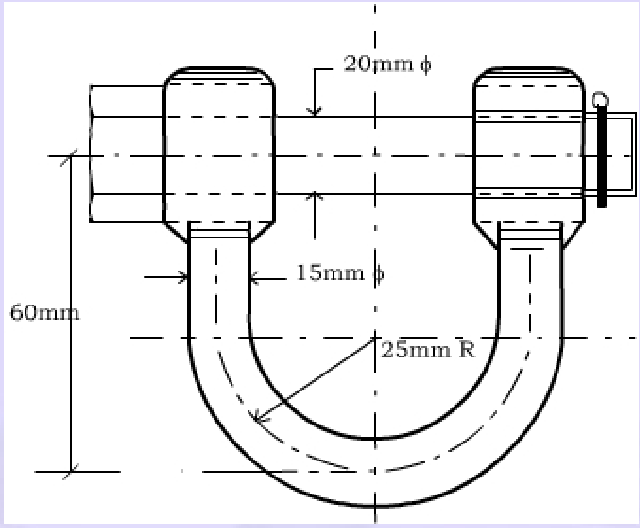


Figure 7.46 D-Shackle

The loadings for a D-shackle for a 132 kV single circuit tower are given below:

	NC	BWC
Transverse load	597	400
Vertical load	591	500
Longitudinal load	-	1945

The D-shackle is made of high tensile steel. Assume permissible stress of high tensile steel as 2,500kg/cm^2 and 2,300kg/cm^2 in tension and bearing respectively.

Normal condition

Area of one leg = π / 4 x (1.6) = 2.01 sq.cm.

Assuming the total load to be the sum of vertical and transverse loads (conservative), the design load

= 597 + 591

= 1,188

Tensile stress = 1,188 / 2 x 1 / 2.01 = 295.5 kg/cm^2

Factor of safety = 2,500 / 295.5 = 8.46

Shearing stress in the bolt = $\dfrac{597}{\frac{\pi}{4} \text{ x } 2^2}$

= 190 kg/cm^2

Factory of safety = 2,300 / 190 = 12.1

Broken-wire condition

Assuming the total load to be sum of the loads listed for broken-wire condition,

Tensile stress in shackle = 2,845 / 2 x 1 / 2.01

= 707.711 kg/cm^2

Factor of safety = 2,500 / 707.7 = 3.53

Shearing stress in the bolt = $\dfrac{2{,}845}{2 \text{ x } \frac{\pi}{4} \text{ x } (2)^2}$

= 452.8

Factor of safety = 2,300 / 452.8 = 5.07

Hence safe.

8. FOUNDATION OF TOWERS

8.1 Introduction

Foundation of any structure plays an important role in safety and satisfactory performance of the structure as it transmits the loads from structure to earth. Without having a sound and safe foundation, structure cannot perform the functions for which it has been designed. Therefore, the design of foundation needs to be given top priority.

Past records show that failures of tower foundations are responsible for collapse of towers. These failures have usually been associated with certain deficiencies either in the design or classification or construction of foundations. Many times, foundation cast are over safe because of inappropriate classification, resulting in wastage of resources. From engineering point of view, the task of design and selection of most suitable type of tower foundation is challenging because of the variety of soil conditions at the site of construction of tower. The foundations in various types of soils have to be designed to suit the soil conditions of particular type.

As the concept of safe value for properties of soil has been dispensed with in the design of foundation, limit value of properties of soil should be obtained from soil investigation report.

8.2 Types of load on foundations

The foundations of towers are normally subjected to three types of forces. These are

a. The compression or downward thrust;

b. The tension or uplift and

c. The lateral forces or side thrusts in both transverse and longitudinal directions.

The magnitudes of each of these forces depend on the types of tower. The magnitudes of limit loads for foundations should be taken 10% higher than those for the corresponding towers.

8.3 Basic design requirements

To meet the varying needs in respect of soil conditions and loading quantum, several types of tower foundation have been used for the communication towers. Design philosophy of tower foundation should be closely related to the principles adopted for the design of towers, which the foundation has to support. It should take care of all the loads such as dead load, live loads, wind loads, seismic loads, erection loads etc. causing vertical thrust, uplift as well as horizontal reactions. For satisfactory performances, it should be stable and structurally adequate and be able to transmit these forces to the soil such that the limit soil bearing capacities are not exceeding.

8.4 Soil parameters

For designing the foundations, following parameters are required:

a. Limit bearing capacity of soil

b. Density of soil and

c. Angle of earth frustum.

These soil properties are normally obtained either by conducting in-situ or Laboratory tests on soil samples collected from the field during soil investigation or from available testing record of the area. The importance of above soil parameters in foundation design is discussed below in brief.

Limit bearing capacity

This parameter is vital from the point of view of establishing the stability of Foundation against shear failure of soil and excessive settlement of foundation when Foundation is subjected to total downward loads and moments due to horizontal shears and/or eccentricities as applicable.

Recommended limit bearing capacities of various types of soil are given in Table 9.1 of Annexure for guidance.

Density of soil

This parameter is required to calculate the uplift resistance of the foundation. Recommended values of angle of earth frustum for different types of soils/rocks are given in Table.1 of Annexure.

Angle of earth frustum

This parameter is required for finding out the uplift resistance of the foundation. Recommended values of angle of earth frustum for different types of soils/rocks are given in Table 8.1 of Annexure.

Table 8.1: Soil properties to be considered in foundation design for various types of soil

SI No	Types of soil	Angle of earth Frustum (Degrees)	Unit Wt.of soil (Kg/cum)	Limit bearing capacity (kg/sq.m)
1	Normal dry soil (a) Without undercut (b) With undercut	 30 30	 1440 1600	 25000 25000
2	Wet soil due to presence of sub soil Water/Surface Water	15	940	12500
3	Black cotton soil (a) In Dry portion (b) In Wet portion	 0 0	 1440 940	 12500 12500
4	Sandy soil (a) With clay content 0-5% (b) With clay content 5-10%	 10 20	 1440 1440	 25000 25000
5	Fissured Rock/Soft rock (with undercut) (a) In Dry portion (b) In Wet portion	 20 10	 1700 940	 62500 62500
6	Hard rock	-	-	125000
7	Normal hard dry soil (Morrum) with undercut	30	1600	40000

Note:

1. Limit bearing capacity of soil has been arrived at taking FOS 2.5 over the safe bearing capacity values. Soil research institute will be approached to furnish the limit bearing capacities of soil. If and when such data are available the above values can be reviewed.

2. Where clay content is more than 10% but less than 15% the soil will be classified as normal dry soil.

3. Angle of earth frustum shall be taken with respect to vertical.

(Source: Transmission line manual - Central board of irrigation and power)

Types of foundations

The foundations are designed for the uplift force, down thrust, lateral forces and over turning moments for varieties of soils.

Depending upon the ground water table and type of soil and rock, the foundations can be classified as follows.

Normal dry soil foundations

When water table is below foundation level and when soil is cohesive and homogeneous up to the full depth having clay content of 10-15%.

Wet soil foundations

When water table is below foundation level and up to 1.5m below ground level. the foundation in the soils which have standing surface water for a long period with penetration not exceeding 1.0m below ground level (e.g. paddy fields) are also classified as wet foundations.

Partially submerged foundations

When water table is at a depth between 1.5m and 0.75 below ground level and when the soil is normal and cohesive.

Fully submerged foundations

When water table is within 0.75m below ground and the soil is normal and cohesive.

Black cotton soil foundations

When the soil is cohesive having inorganic clay exceeding 15% and characterized by high shrinkage and swelling property (need not be always black in colour)

Partial black cotton foundations

When the top layer of soil up to 1.5m is black cotton and thereafter it is normal dry cohesive soil.

Soft rock/Fissured rock foundations

When discomposed or fissured rock, hard gravel or any other similar nature is met this can be executed without blasting. Under cut foundation is to be used at these locations.

Hard rock foundations

Where chiseling, drilling and blasting are required for execution.

Sandy soil foundations

Soil with negligible cohesion because of its low clay content (0-10%)

The above categorization of foundations has been done for economizing the design of foundations; uplift resistance of foundations is a critical design factor which is greatly affected by the location of water table and the soil surrounding the foundation.

8.5 Structural arrangement of foundation

Based on structural arrangement of foundations, the various type of foundations are possible. The necessity of erecting towers on a variety of soils has made it possible and necessary for the designers to adopt new innovations and techniques. As a result, several types of tower foundations have been devised and successfully used. Some of the more common types of foundations are described below.

P.C.C. types

This type of foundation consists of a plain concrete footing pad with reinforced chimney. They is as shown in figure. In this type of foundation, the stub angle is taken inside and effectively anchored to the bottom pad by cleat angles and / or keying rods and the chimney with reinforcement and stub angle inside works as a composite member. The pad may be either pyramidal in shape as shown in Figure 8.1(a) or stepped as shown in Figure 8.1(b). Stepped footings will require less shuttering materials but need more attention during construction to avoid cold joints between the steps. The pyramidal footings on the other hand will require somewhat costlier formwork. In this pad and chimney type footing, where the chimney is comparatively slender, the lateral load acting at the top of the chimney will cause bending moment and, therefore the chimney should be checked for combined stress due to direct pull / thrust and bending.

If the soil is very hard, conglomerate of soil, containing stones, rubbles, kankar which can be loosened with the help of pick-axe or if the soil is of composite nature i.e. combination of normal dry soil, hard murrum, fissured rock which will not get unified easily with the parent soil after back filling, pyramid chimney type foundations having 150m side clearance are not advisable and in such cases undercut / stepped footings without side clearance should be adopted.

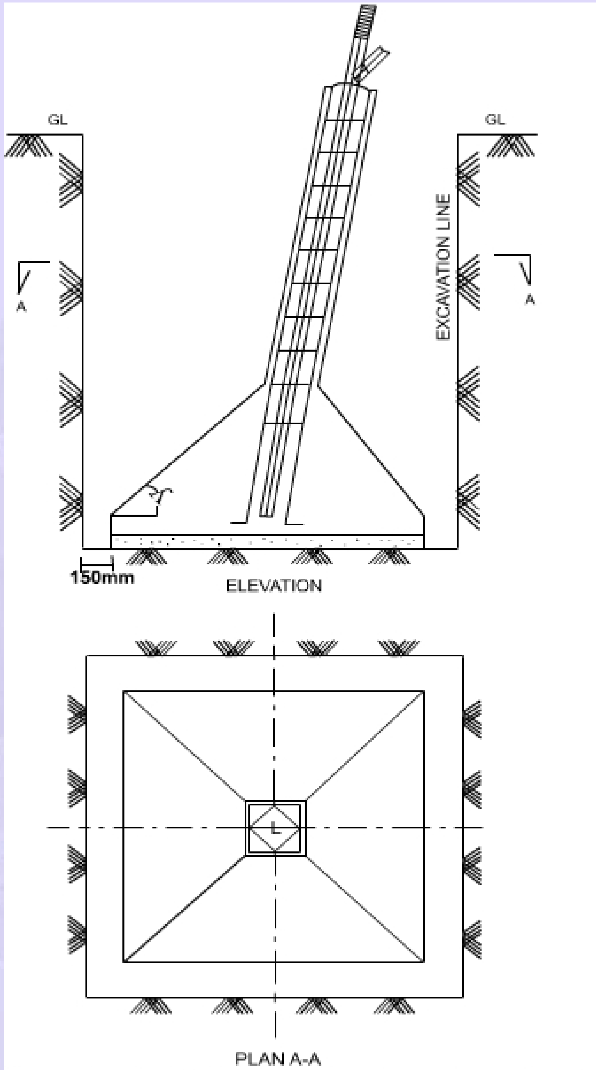


Figure : PYRAMID CHIMNEY TYPE FOUNDATION (P.C.C)
A= ∢ 45^{O}

Figure 8.1(a)

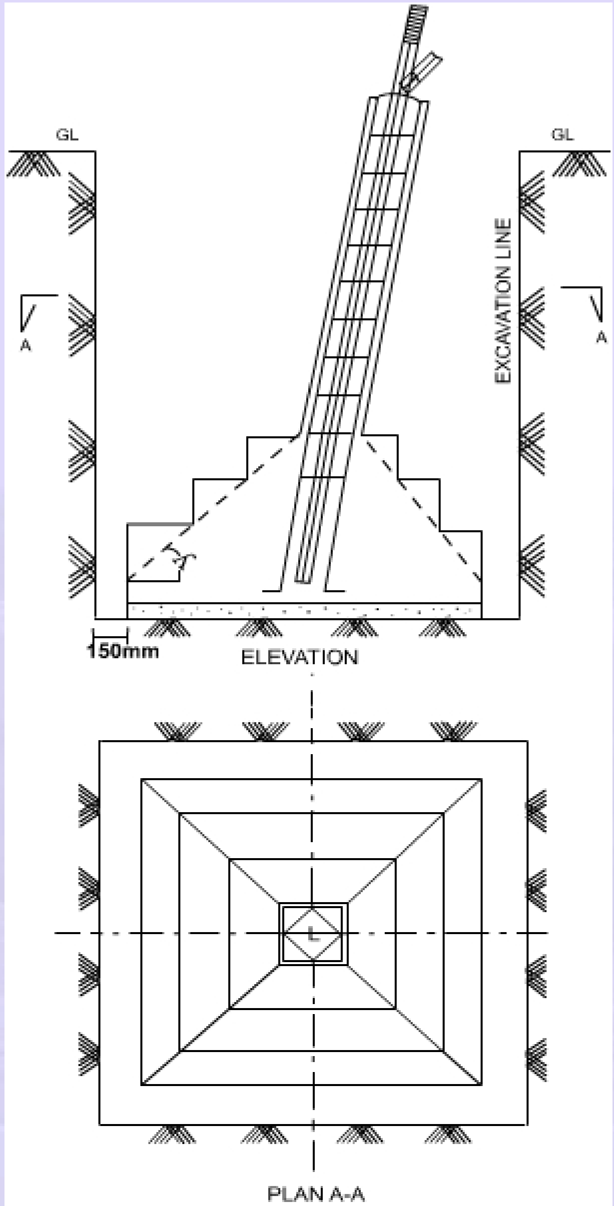


Figure : P.C.C. TYPE STEPPED FOUNDATION
A= ∡ 45°

Figure 8.1(b)

R.C.C. spread type

Typical types of R.C.C spread footings are shown in figure 8.2. It consists of a R.C.C base slab or mat and requires a square chimney.

There are several types of R.C.C spread footings which can be designed for tower foundations. The three most common types of these are shown in figure 8.2(a), (b) and (c). As shown in figures, this type of foundation can be either single step type or multiple step type and / or chamfered step type.

The R.C.C spread type footing can be suitably designed for variety of soil conditions. R.C.C footings in some situations may be higher in cost although structurally these are the best.

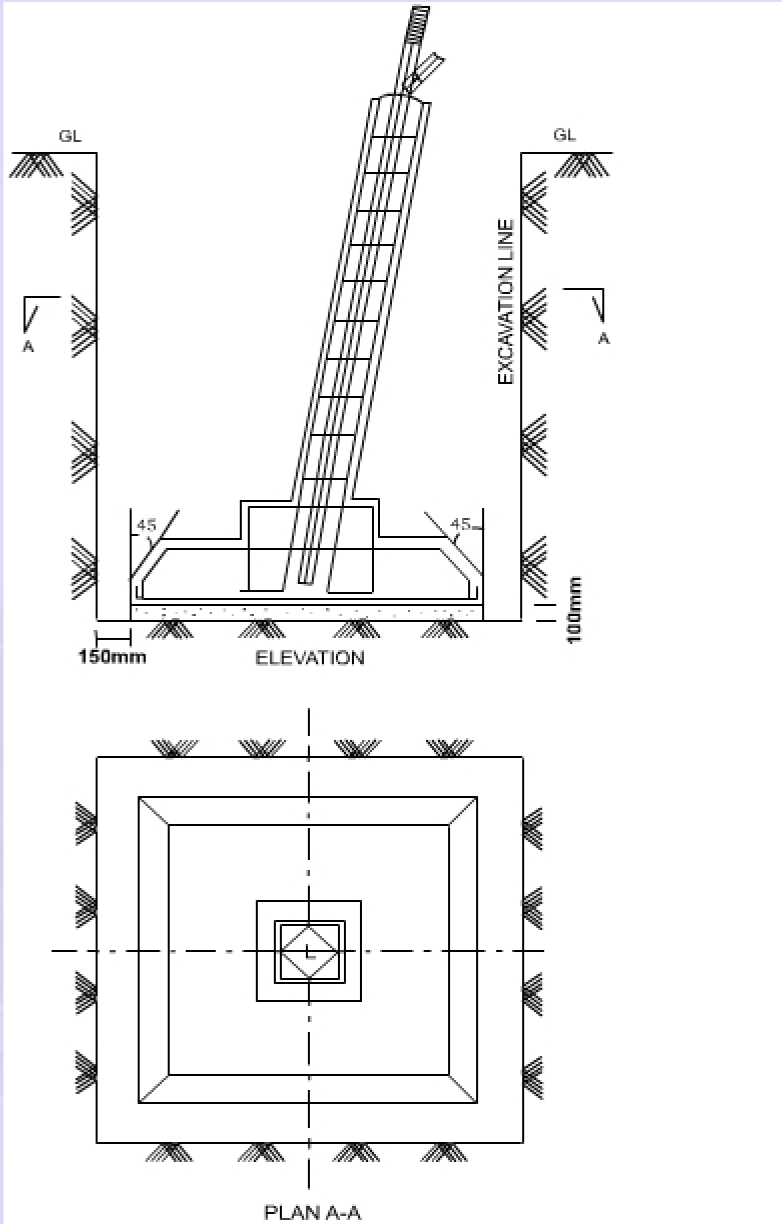


Figure : P.C.C. SPREAD TYPE FOUNDATION (CHAMFERED TYPE)
WITH 15mm WORKING CLEARANCE

Figure 8.2(a)

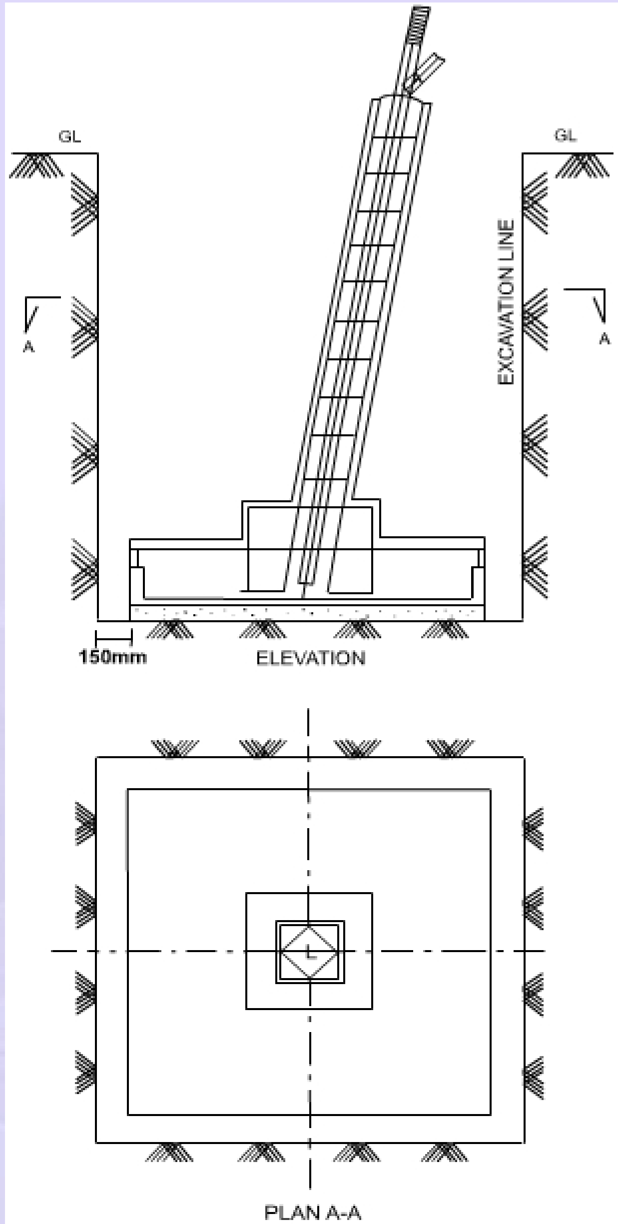


Figure : P.C.C. SPREAD TYPE FOUNDATION (STEP) WITH 15mm WORKING CLEARANCE

Figure 8.2(b)

When loads on foundations are heavy and / or soil is poor, the pyramid type foundations may not be feasible from techno-economical considerations and under such situations, R.C.C spread type footings are technically superior and also economical. R.C.C spread footing with bottom step/slab when cast in contact with inner surface of excavated soil will offer higher uplift resistance as compared to the footing having 150mm side clearance as shown in figure 8.2(c)

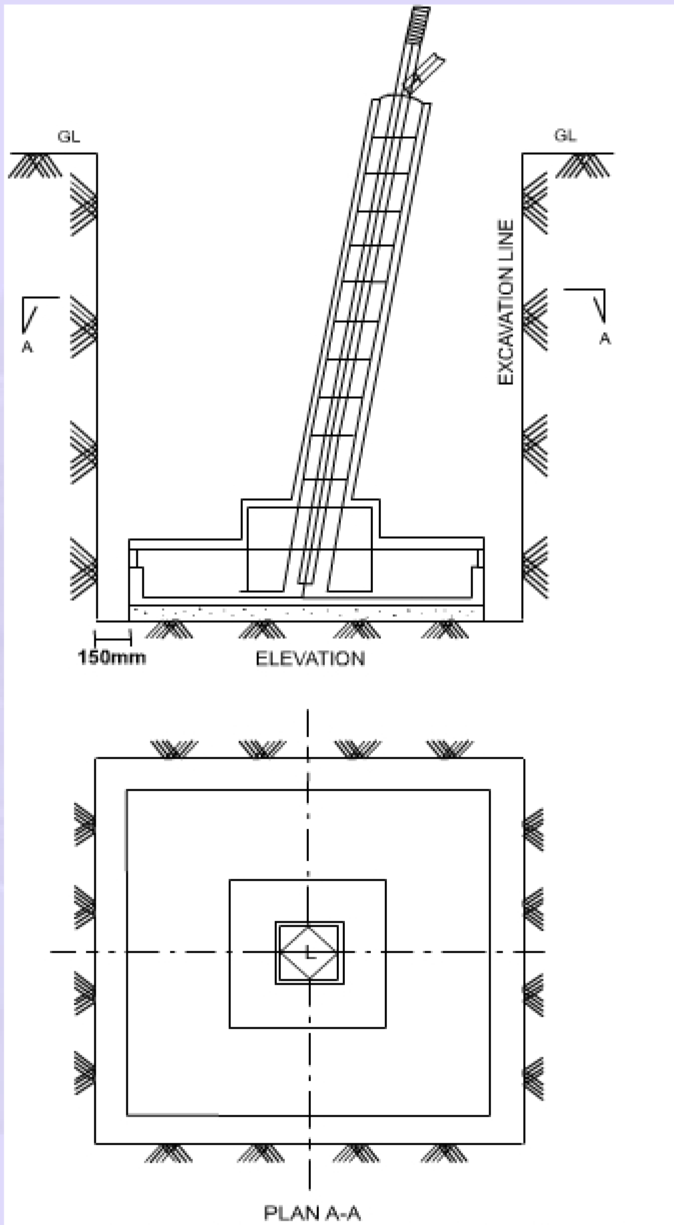


Figure : R.C.C. SPREAD TYPE FOUNDATION (STEP) CAST DIRECTLY CONTACT WITH THE SOIL & WITHOUT WITH 15mm WORKING CLEARANCE

Figure 8.2 (c)

Block type

This type of foundation is shown in fig 8.3 and fig 8.5(a). It consists of a chimney and block of concrete. This type of foundation is usually provided where soft rock and hard rock are strata are encountered at the tower location. In this type of foundation, concrete is poured in direct contact with the inner surfaces of the excavated rock so that concrete develops bond with rock. The bond between concrete and rock provides the uplift resistance in this type of footing. The thickness and size of the block is decided based on uplift capacity of foundation and bearing area required.

It is advisable to have footing with a minimum depth of about 1.5m below ground level and check this foundation for the failure of bond between rock and concrete. The values of ultimate bond stress between the rock and the concrete to be considered for various types of rocks are given in Table 8.2 of Annexure for guidance. However, the actual bond stress between rock and concrete can be decided by tests.

Block type foundations are being provided by some power utilities for soft and hard rock strata. However, under cut type of foundations for soft rock and rock anchor type of foundations for hard rock are sometimes preferred by some power utilities because of their soundness though these are more costly in comparison with Block type foundations.

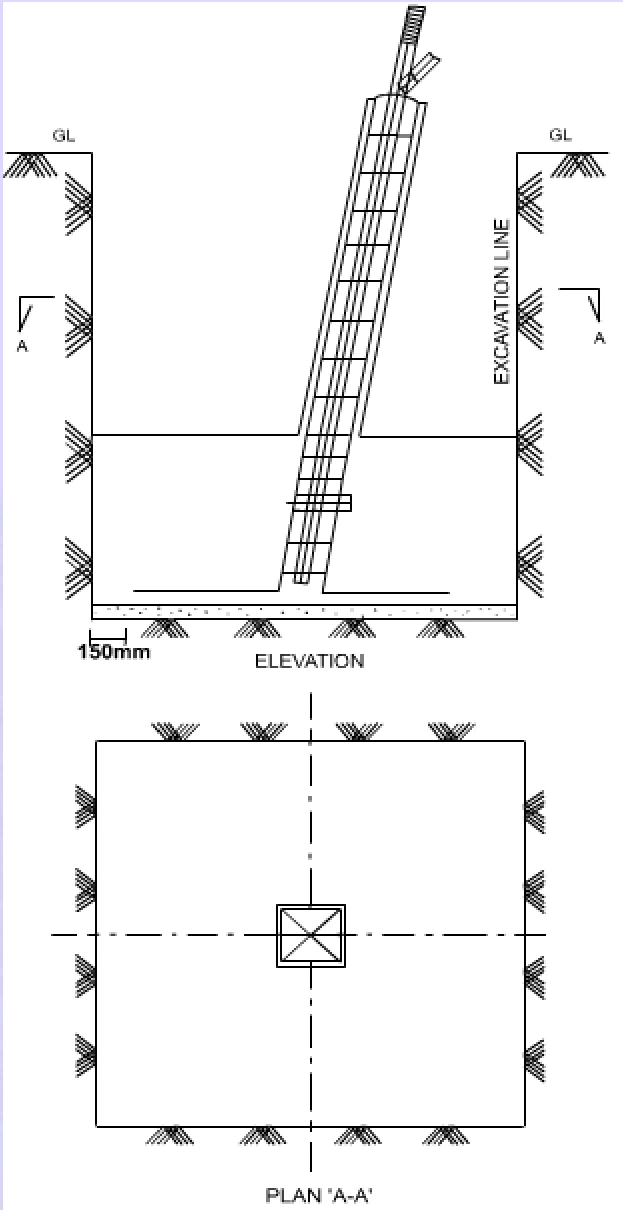


Figure : BLOCK FOUNDATION (FRICTION TYPE)

Figure 8.3

Table: 8.2 Bond stress as per IS: 456-2000

(1) Limit bond stress between concrete and reinforcement steel deformed in tension of grade Fe415 (conforming to IS:1786-1985 and 1139-1165) (a) With M15 Mix (b) With M 20 Mix Note: For bars in compression the above values shall be increased by 25%	 16.0 kg/cm^2 19.5 kg/cm^2
(2) Limit bond stress between concrete and stubs in tension with (a) M15 Mix (b) M20 Mix For compression above values will be increased by 25%	 10.0 kg/cm^2 12.0 kg/cm^2
(3) Limit bond stress between rock and concrete (a) In Fissured rock (b) In Hard rock	 1.5 kg/cm^2 4.0 kg/cm^2
(4) Limit bond stress between hard rock and grout	2.0 kg/cm^2

Under cut type

These types of foundations are shown in figures 8.4(a), (b), (c). These are constructed by making under-cut in soil / rock at foundation level. This type of foundation is very useful in normal dry cohesive soil, hard murrum, fissured / soft rock, soils mixed with clinker, where soil is not collapsible type i.e. it can understand by itself. A footing with an under-cut generally develops higher uplift resistance compared to that of an identical footing without under-cut. This is due to hte anchorage in un disturbed virgin soil. The size of under-cut shall not be less than 1.50mm. At the discretion of utility and based on the cohesiveness of the normal dry soil, the owner may permit undercut type of foundation for normal and cohesive soil.

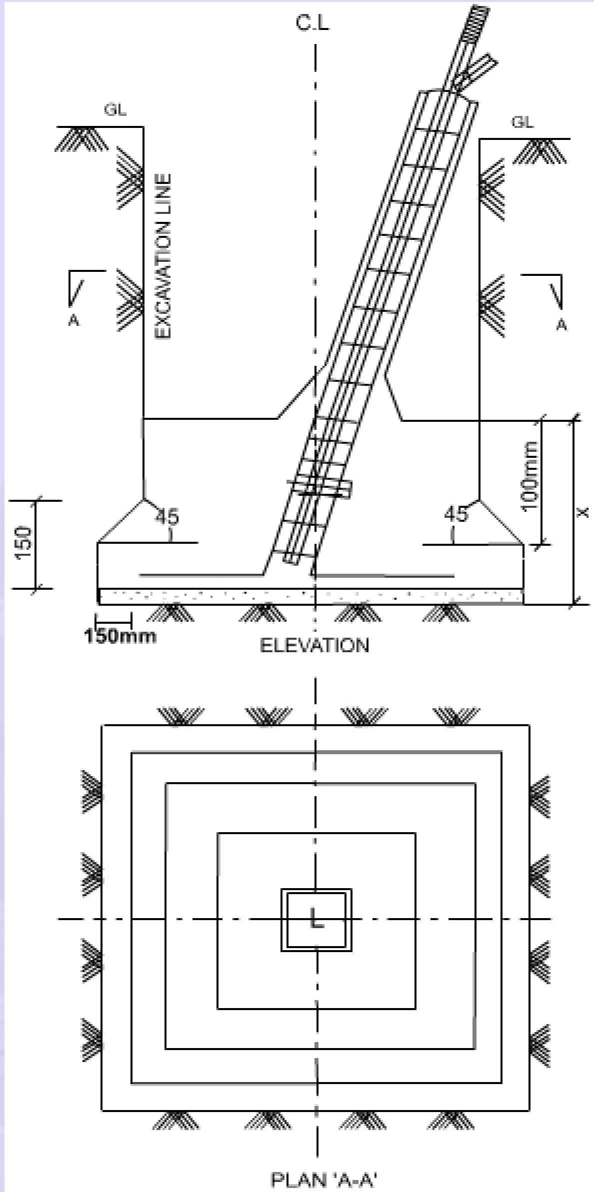


Figure : PYRAMID TYPE FOUNDATION
(WITH UNDER CUT)

Figure 8.4(a)

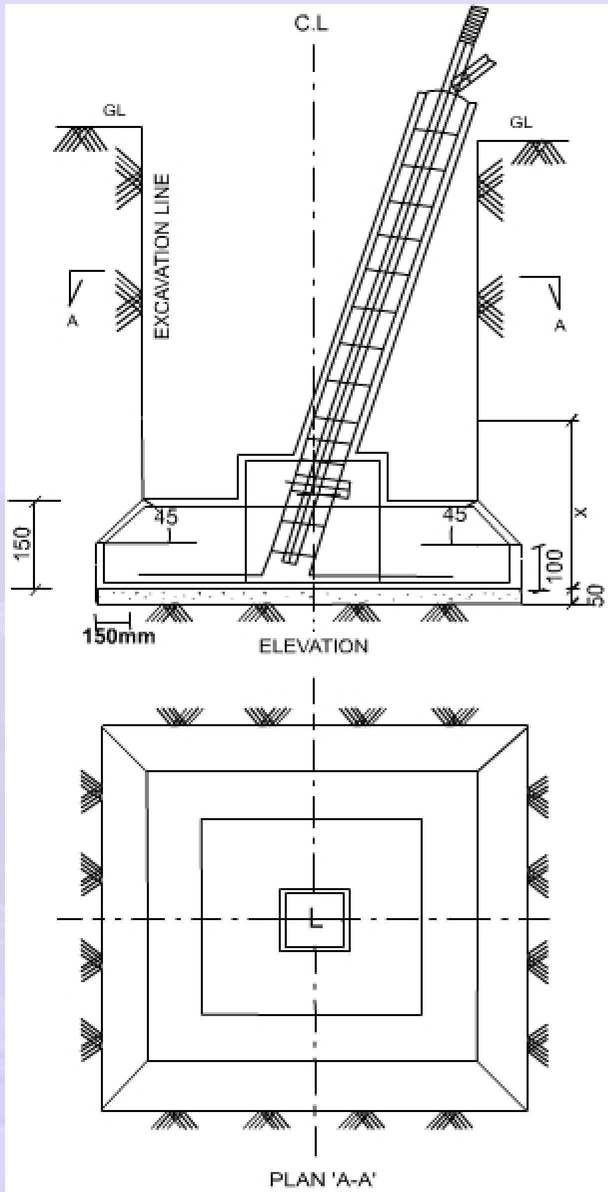


Figure : R.C.C SPREAD TYPE FOUNDATION
(UNDER CUT TYPE)

Figure 8.4(b)

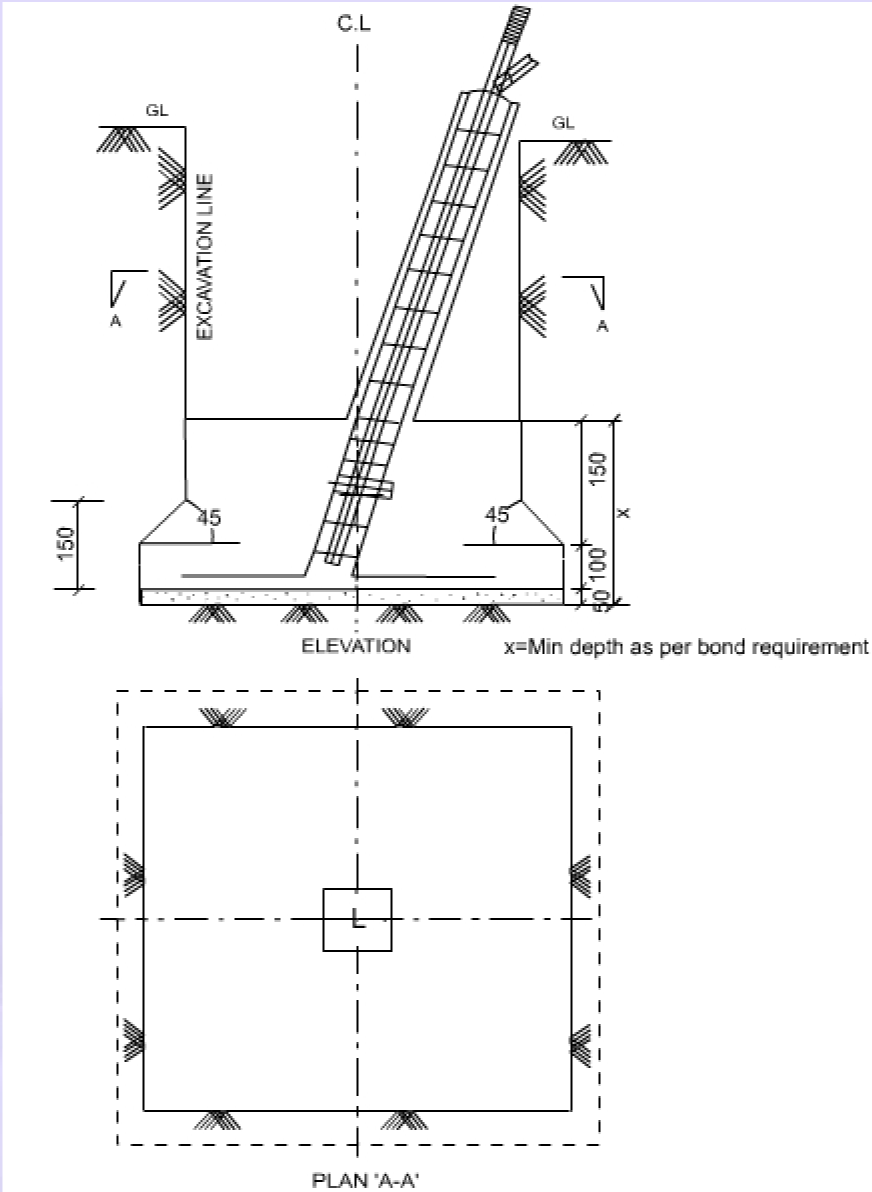


Figure : BLOCK FOUNDATION (UNDER CUT TYPE)

Figure 8.4(c)

Grouted rock and Rock anchor type

Typical Grouted rock and Rock anchor type footing is shown in figure 8.5(b). This type of footing is suitable when the rock is very hard. It consists of two parts viz., Block of small depth followed by anchor bars embedded in the grouted anchor holes. The top part of the bar is embedded in the concrete of the shallow block. The depth of embedment, diameter and number of anchor bars will depend upon the uplift force on the footing. The diameter shall not be less than 12mm. The grouting hole shall normally be 20mm more than diameter of the bar.

The determination of whether a rock formation is suitable for installation of rock anchors is an engineering judgement based on rock quality. Since, the bearing capacity of rock is usually much greater, care must be exercised in designing for uplift. The rock surfaces may be roughened grooved or shaped to increase the uplift capacity.

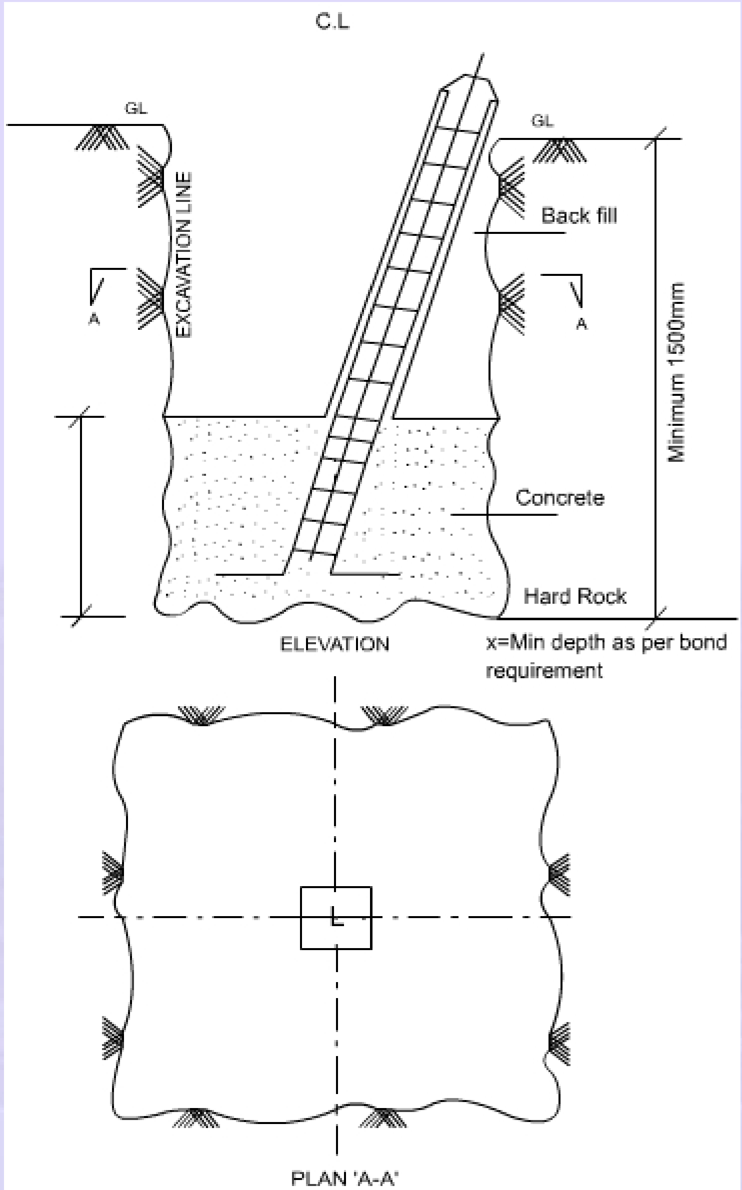


Figure : HARD ROCK FOUNDATION (BLOCK TYPE)

Figure 8.5(a)

The uplift resistance will be determined by considering the bond reinforcement bar and grout / concrete. However, an independent check for uplift resistance should be carried out by considering the bond between rock and concrete block which in turn will determine the minimum depth of concrete block to be provided in hard rock. Anchor strength can be substantially increased by provision of mechanical anchorages, such as use of eye-bolt, fox bolt, or threaded rods as anchoring bars or use of keying rods in case or stub angle anchoring . The effective anchoring strength should preferably be determined by testing.

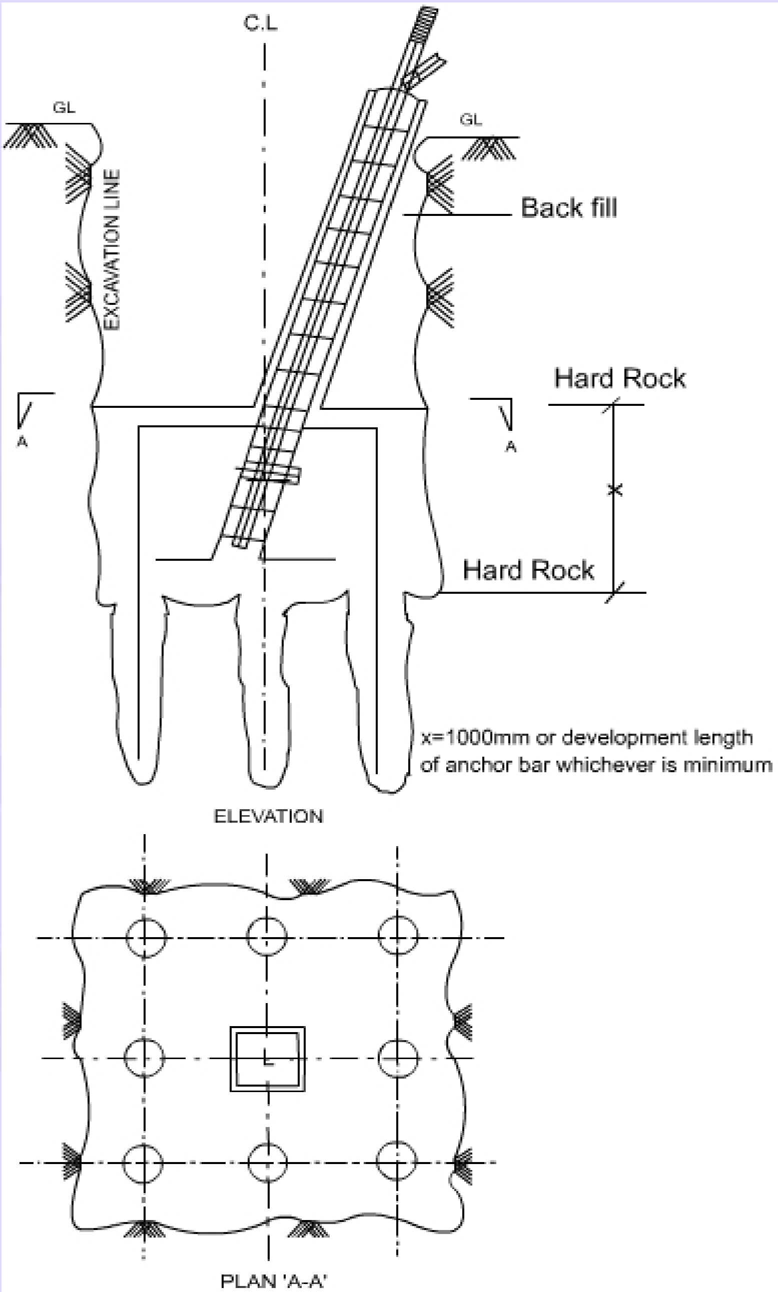


Figure : ROCK ANCHOR TYPE FOUNDATION

Figure 8.5(b)

Augur type / under-reamed pile type

Typical types of foundation are shown in figure 8.6(a). The cast-in-siut reinforced concrete augured footins has been extensively used in some western countries like USA, Canada and many Asian countries. The primary benefits derived from this type of foundations are the saving in time and manpower. Usually a truck mounted power augur is utilized to drill a circular hole of required diameter, the lower portion of this may be belled, if required, to a larger diameter to increase the uplift resistance of the footing. Holes can be driven upto one meter in diameter and six meter deep. Since, the excavated hole has to stand for some time before reinforcing bars and cage can be placed in position and concrete poured. Usually, stiff clays and dense sands are capable of being drilled and standing up sufficiently long for concreting works and installation of stub angle or anchor bolts, whereas loose granular materials may give trouble during construction of these footings. Betonies slurry or similar material is used to stabilize the drilled hole. In soft soils, a steel casing can also be lowered into the hole as the excavation proceeds to hold the hole open.

The friction along the surface of the shalt alone provides uplift resistance of augured footing without bell and hence its capacity to resist uplift is limited. Augured footing can be constructed according to the requirements, vertical or battered and with or without expanded base.

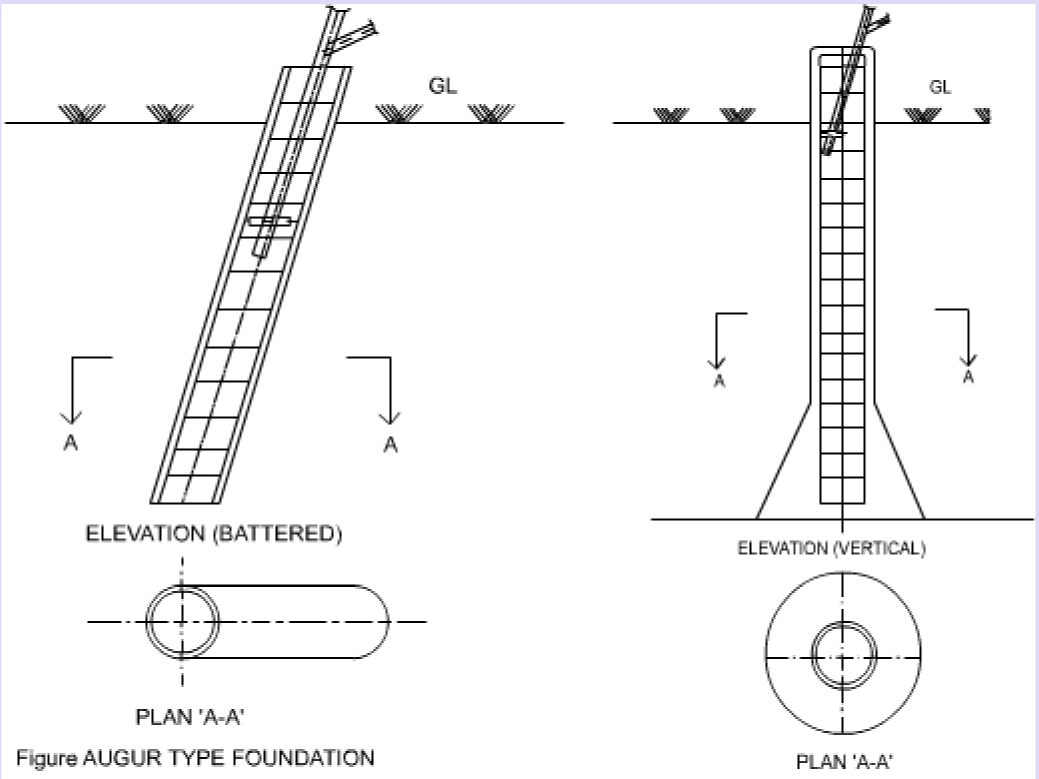


Figure 8.6(a)

Under-reamed pile type

The under-reamed piles are more or less similar to augured footings except that they have under reaming above bottom of shaft. These can be generally constructed with hand augur. The bore is drilled vertically or at a batter with the augur, having an arrangement of cutting flanges (edges) to be opened by the lever. This arrangement makes it possible to make under-reams at various levels of bores as shown in fig 8.6(b). The advantage of this foundation is faster construction.

The load carrying capacity of these footings, both for downward and uplift forces should be established by tests. The safe loads allowed on under-reamed piles of length 3.50m and under-reamed to 2.5 times shaft diameter in clayey, black cotton and medium dense sandy soils may be taken from IS: 4091 for guidance.

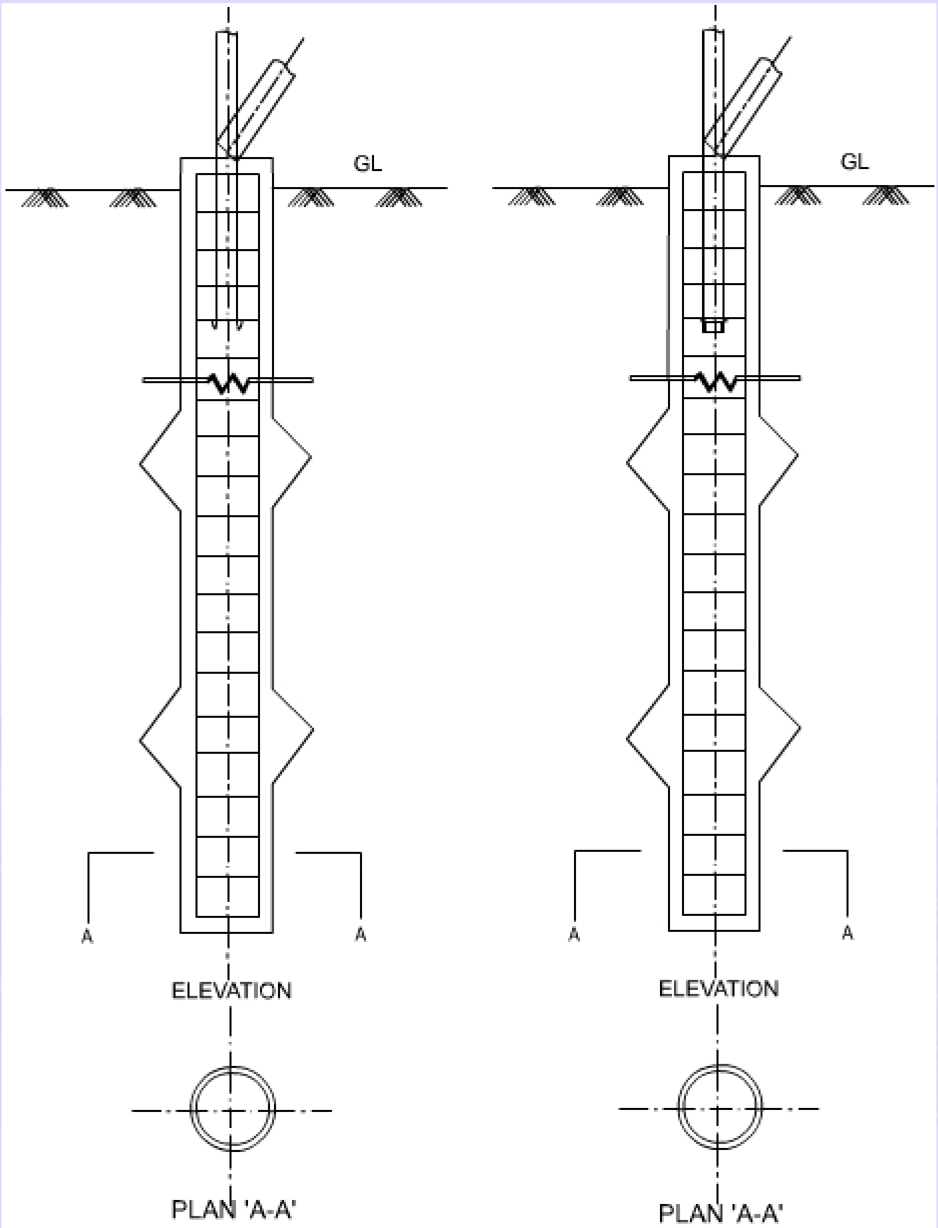


Figure AUGUR TYPE FOUNDATION (UNDER REAMED PILE TYPE)

Figure 8.6(b)

Pile type

A typical pile type is shown in Figure 8.7. This type of foundation is usually adopted when soil isvery weak and has very poor bearing capacity or foundation has to be located in filled-up soil or sea mud to a large depth or where tower location falls within river bed and creek bed which are likely to get scoured during floods. The pile foundations are designed based on the data of soil exploration at the tower location. The important parameter for the design of pile foundation the type of soil, angle of internal friction, cohesion and unit weight of soil at various depths along the shaft of pile, maximum discharge of the river, maximum velocity of water, high flood level, scour depth etc.

Pile foundation usually costs more and may be adopted only after the detailed examination of the site condition and soil data. The downward vertical load on the foundation is carried by the dead weight of the concrete in piles and pile caps and frictional resistance between pile and soil surrounding the pile. For carrying heavy lateral loads, battered piles may be advantageously used. Piles are of different types such as driven pre-cast files, cast-in-situ concrete bored piles and cast-in-situ concrete driven piles, concrete driven piles whether pr-cast or cast-in-situ, require heavy machinery for their construction and as such may no be possible to use for transmission line foundations because of the remoteness of the sites and small volume of work. Mostly, cast-in-situ concrete bored piles are provided in transmission line projects since, they do not require heavy machinery for their construction.

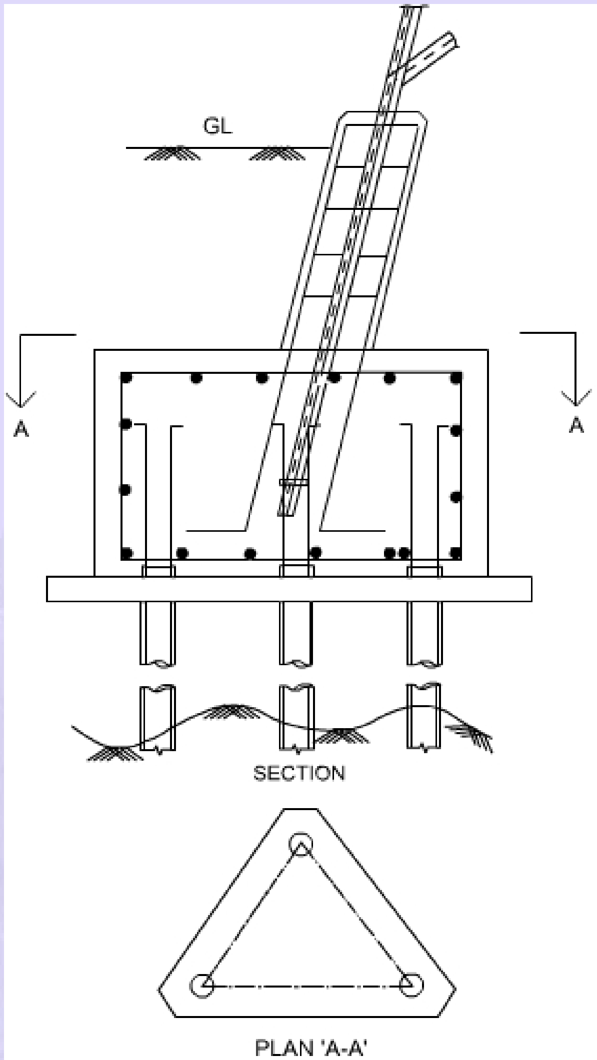


Figure PILE TYPE FOUNDATION

Figure 8.7

8.6 Soil resistances for designing foundation

As discussed earlier, the foundations of towers are subjected to three types of loads viz., the downward trust (compression), the uplift (tension) and the side trust (horizontal shear). The soil resistances available for transferring the above forces to earth are described below.

Uplift resistance

The soil surrounding a tower foundation has to resist a considerable amount of upward force (tension). In fact, in the case of self-supporting towers, the available uplift resistance of the soil becomes the most decisive factor for selection for a particular location.

It is generally considered that the resistance to uplift is provided by the shear strength of the surrounding soil and the weight of the foundation. Various empirical relationships linking ultimate uplift capacity of foundation to the physical properties of soil like angle of internal friction (0) and cohesion (C) as well as the dimensions and depth of the footing have been proposed on the basis of experimental results. However, the angle of earth frustum is considered for calculating the uplift resistance of soil. Typical value of angle of earth frustum; are given in table of annexure for guidance the angle of earth frustum is taken as 2/3 of the angle of internal friction or the value given in the table whichever is smaller for the type of soil under consideration. The uplift resistance is estimated by computing the weight of the earth contained in an inverted frustum of cone whose sides make an angle (0) with the vertical equal to the angle of earth frustum.

It should, however, be noted that effective uplift resistance, apart from being a function of the properties of soil like angle of internal friction (0) and cohesion (C) is greatly affected by the degree of compaction and the ground water table. When the backfill is under consolidated with non-cohesive material, the effective uplift resistance will be greatly reduced. In case of foundation under water table, the buoyant weights of concrete and backfill are only considered to be effective. The uplift resistance of footing with under cut projections within undisturbed soils in firm non-cohesive soils and fissured/soft rock generally is larger than that of conventional footings.

Lateral soil resistance

In foundation design of towers, the side thrusts on the foundation are considered to be resisted by the passive earth pressures mobilized the adjoining soils due to rotation of the footing. Passive pressure/resistance of soil is calculated based on Rankine's formula for frictional soils and unconfirmed compressive strength for cohesive soils.

Bearing capacity

The downward compressive loads acting on the foundation including moments due to horizontal shears and / or eccentricities, where existing, are transferred from foundation to earth through bearing capacity of soil. The limiting bearing capacity of soil is the maximum downward intensity of load which the soil can resist without shear failure or excessive settlement.

8.7. Design procedure for foundation

The design of any foundation consists of following two parts.

8.7.1 Stability analysis

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

Check for bearing capacity

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

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

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

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

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

Where

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

'B' is the dimension of the footing base;

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

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

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

Check for uplift resistance

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

Referring to the figure 8.8 the ultimate resistance to up lift is given by:

$$U_p = W_s + W_f$$

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

W_f is the buoyant weight /overload of the foundation.

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

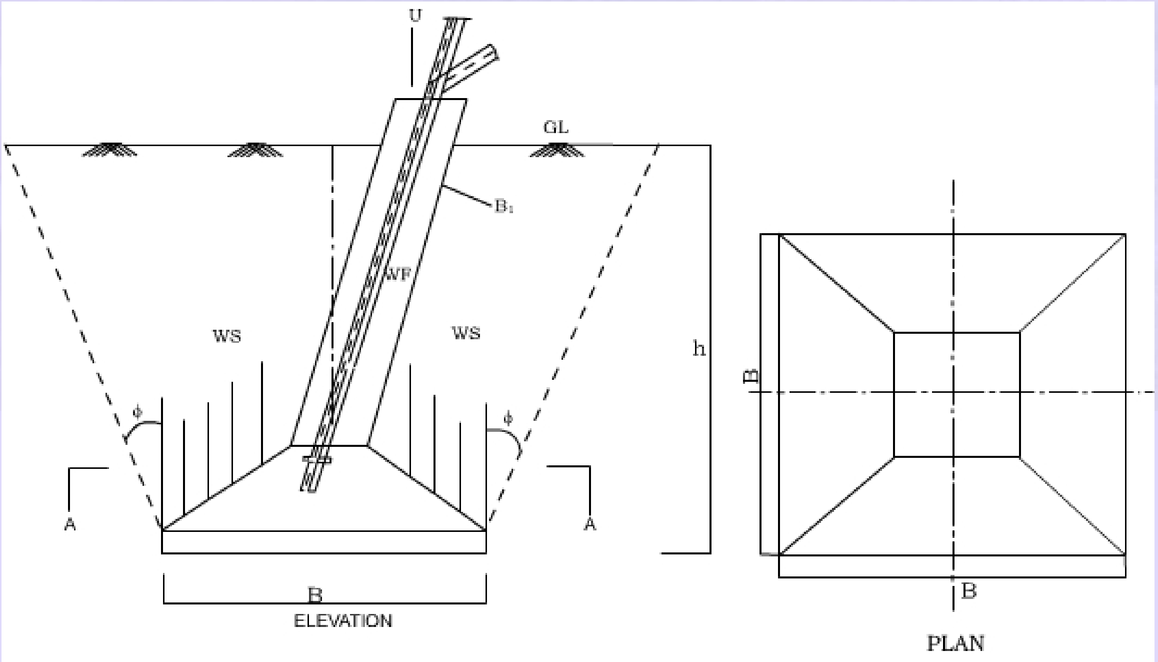


Figure 8.8

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

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

Check for side thrust

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

When acted upon by a lateral load, the chimney will act as a cantilever beam free at the top and fixed at the base and supported by the soil along its height. Analysis of such foundations and design of the chimney for bending moments combined with down thrust uplift is very important. Stability of a footing under a lateral load depends on the amount of passive pressure mobilized in the adjoining soil as well as the structural strength of the footing in transmitting the load to the soil. (Refer figure 8.9)

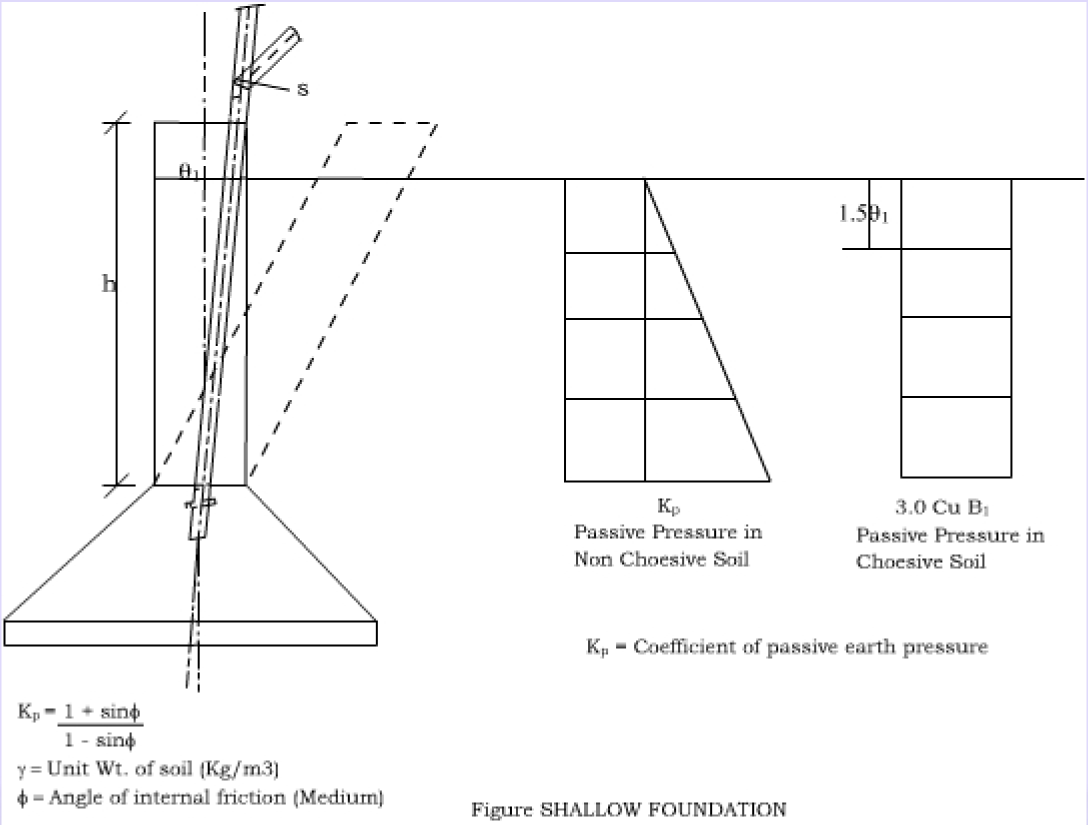


Figure 8.9

Check for over-turning

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

i The foundation over-turns at the toe

ii The weight of the footing acts at the center of the base and

iii Mainly that part of the earth cone which stands over the heel causes the stabilizing moment. However, for design purposes this may be taken equal to the half of the cone of earth acting on the base. It is assumed to act through the tip of the heel.

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

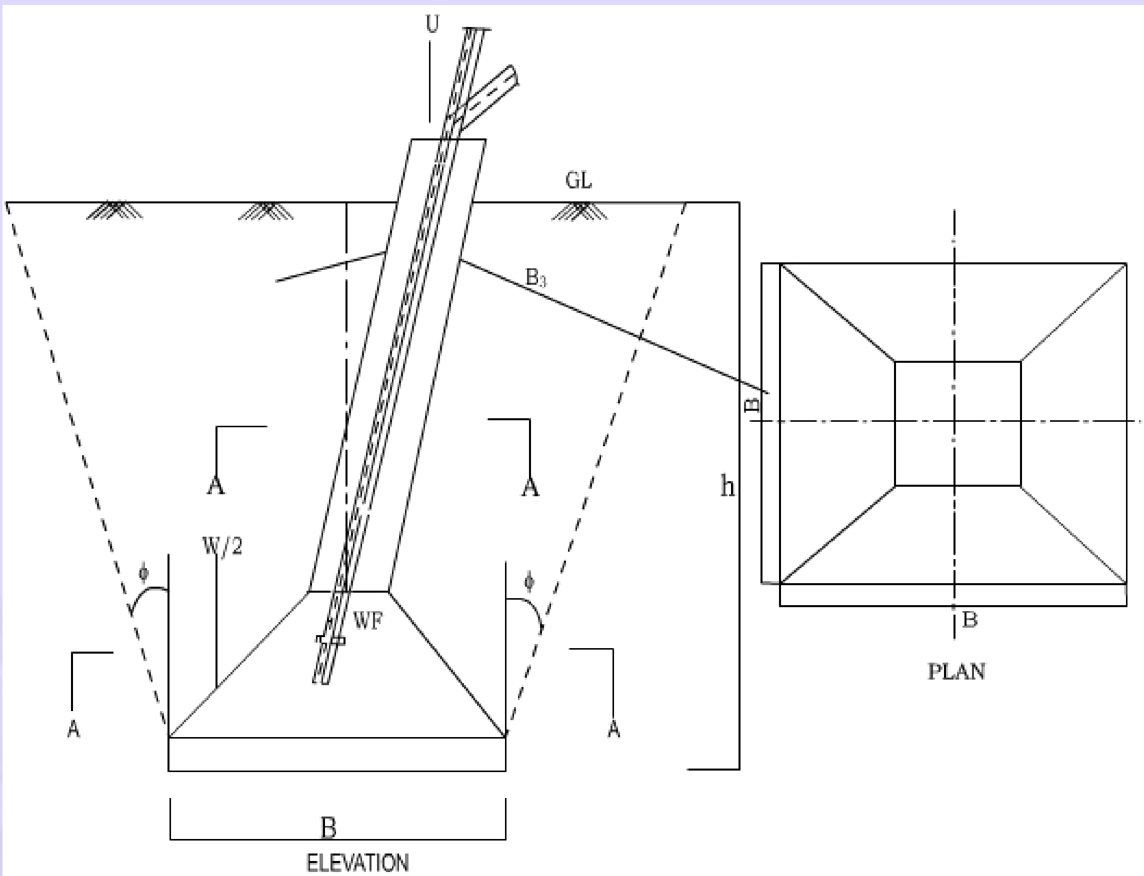


Figure 8.10

Check for sliding

In the foundation of towers, the horizontal shear is comparatively small and possibility of sliding is generally negligible. However, resistance to sliding is evaluated assuming that passive earth pressure conditions are developed on vertical projections above the toe of foundations. The friction between bottom of the footing and soil also resist the sliding of footing and can be considered in the stability of foundation against sliding. The coefficient of friction between concrete and soil can be considered between 0.2 and 0.3. However, the frictional force is directly proportional to vertical downward load and as such may not exist under uplift condition. For cohesive soil the following formula can be applied for calculating the passive pressure to resist sliding.

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

Where C = Cohesion

$\theta = 45^o + 1/2$ of angle of earth frustum

H = height of foundation

γ = unit weight of soil

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

8.7.2 Structural design of foundation

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

Structural design of chimney

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

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

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

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

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

$$\frac{P_u}{f_{ck}\,B_1^2} = 0.36k + \sum\left[\left(P_i/100\right)\left(f_{si}-f_{ci}\right)\right]/f_{ck} \qquad \textbf{\textit{(8.1)}}$$

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

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

Where

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

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

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

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

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

n = Number of rows of reinforcement

f_{ss} = stress in stubs

f_{cs} = stress in concrete

f_{ck} = characteristic compressive strength of concrete

m = modular ratio

σ_{Cbc} = permissible bending compressive stress in concrete

σ_{St} = permissible tensile stress in steel

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

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

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

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

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

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

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

Where,

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

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

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

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

In the above equation

A_c is the area of concrete

A_s is the area of reinforcement steel

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

f_y is the yield stress of reinforcement steel and

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

P_u / P_{uz}	N
0.2 0.3	1.0 2.0

For intermediate values, linear interpolation may be done.

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

Important codal provision F

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

(a) In any chimney that has a larger cross sectional area than that required to support the load the minimum percentage of steel shall be based on the area of concrete required to resist the direct stress and not on the actual area.

(b) The minimum number longitudinal bars provided in a column shall be four in square chimney and six in a circular chimney.

(c) The bars shall not be less than 12mm in diameter.

(d) In case of a chimney in which the longitudinal reinforcement is not required in strength calculation, nominal longitudinal reinforcement not less than 0.15% of the cross sectional area shall be provided.

(e) The spacing of stirrups/ lateral ties shall be not more than the least of the following distances:

i. The least lateral dimension of the chimney

ii. Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied.

iii. Forty-eight times the diameter of the transverse stirrups / lateral ties.

(f) The diameter of the polygonal links or lateral ties shall be not less than one-fourth of the diameter of the largest longitudinal bar and in no case less 6mm.

(g) Structural design of base slab

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

Important codal stipulations for R.C.C foundations

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

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

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

(c) Bending moment

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

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

a. At the face of the chimney.

b. At the sections where width / thickness of the footing changes.

(d) Shear and bond

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

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

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

(e) Critical section

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

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

8.7.3 Concrete technology for tower foundation designs

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

Properties of concrete

The grade of the structural concrete used for tower foundations should not be leaner than M15 having a 28-day cube strength of not less than 15 N/mm^2 and concrete shall conform to IS: 456, for special foundations like pile foundations richer concrete of grade of M20 having a 28-day cube strength of not less than 20 N/mm^2 should be used. M15 grade concrete shall have the nominal strength of not less than 15 N/mm^2 at the end of 28 days as ascertained from the cube test. Such strength at the end of 7 days shall not be less than 10 N/mm^2.

The density of the concrete will be 2300 kg/m^3 for plain concrete and 2400 kg/m^3. For R.C.C other properties of concrete shall be as given in IS: 456-2000.

Properties of steel

The high yield stress cold deformed reinforcement bars used in the R.C.C shall conform to IS: 1786-1979 and shall have yield stress of not less than 415 N/mm^2. When mild steel reinforcement bars are used in R.C.C., they shall conform to IS:432 (part-I) and shall have yield stress of not less than 26 N/mm^2 for bars of size up to 20mm diameter and 24 N/mm^2 for bars above 20mm diameter.

Pull-out tests on tower foundation

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

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

Selection of site

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

Design of foundation for pull-out test

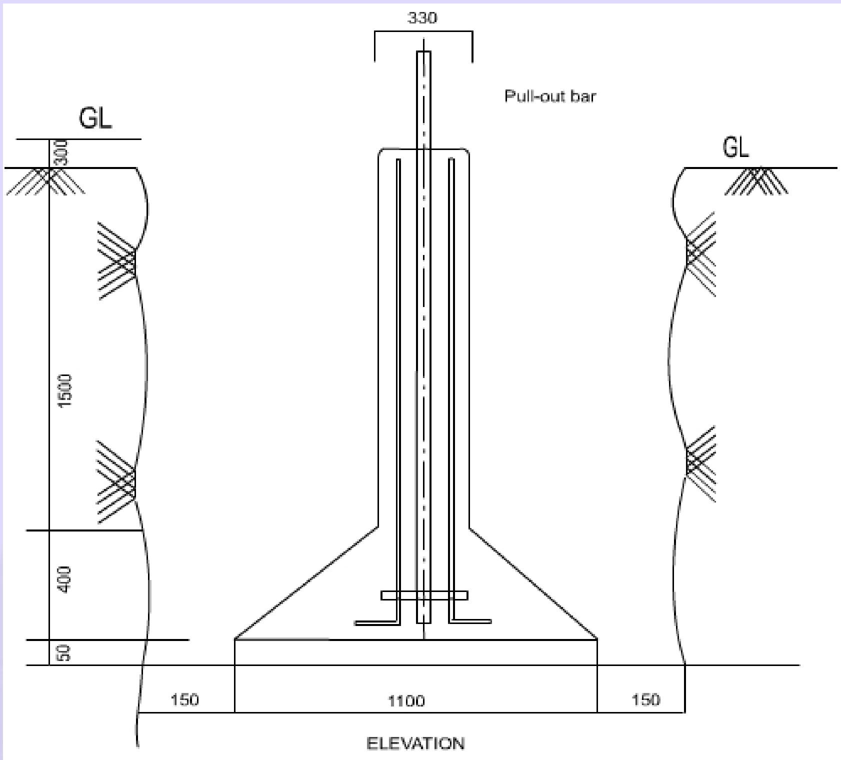


Figure SETUP FOR PULLOUT TEST

Figure 8.11

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

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

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

Casting of foundation

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

Investigation of foundation towers

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

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

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

Repair of foundation of a tower

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

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

Foundation defects and their repairs

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

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

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

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

Foundation for roof top communication towers

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

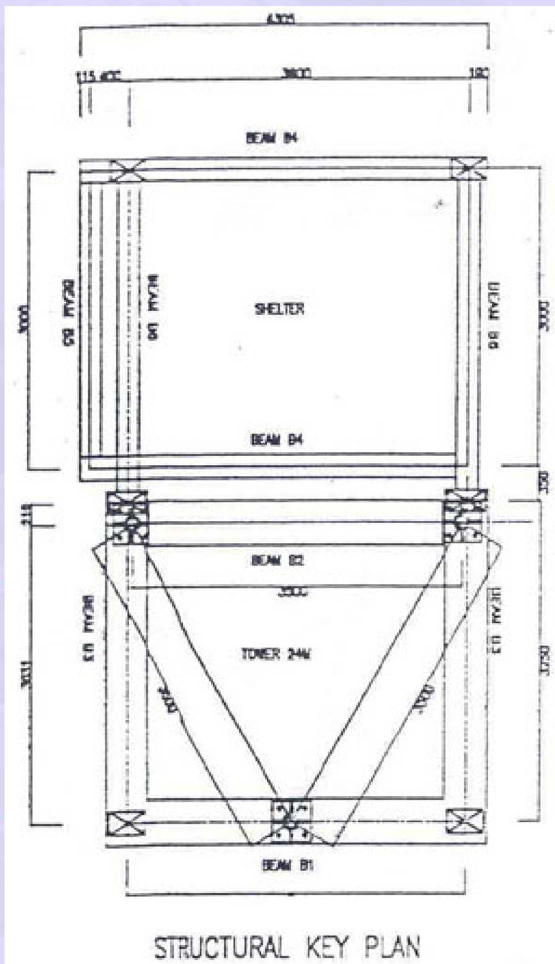


Figure 8.12

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

(a) The column rods of the building are exposed and the reinforcement required for the tower pedestal is welded to the exposed rods.

(b) Where the exposing of the rods is not possible the pedestal rods can be anchored by drilling holes vertically on top of the columns and grouting them with the chemical grouts.

(c) Welding the pedestal rods to the beam rods at the column beam junctions.

Typical details are shown in the figure 8.12

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

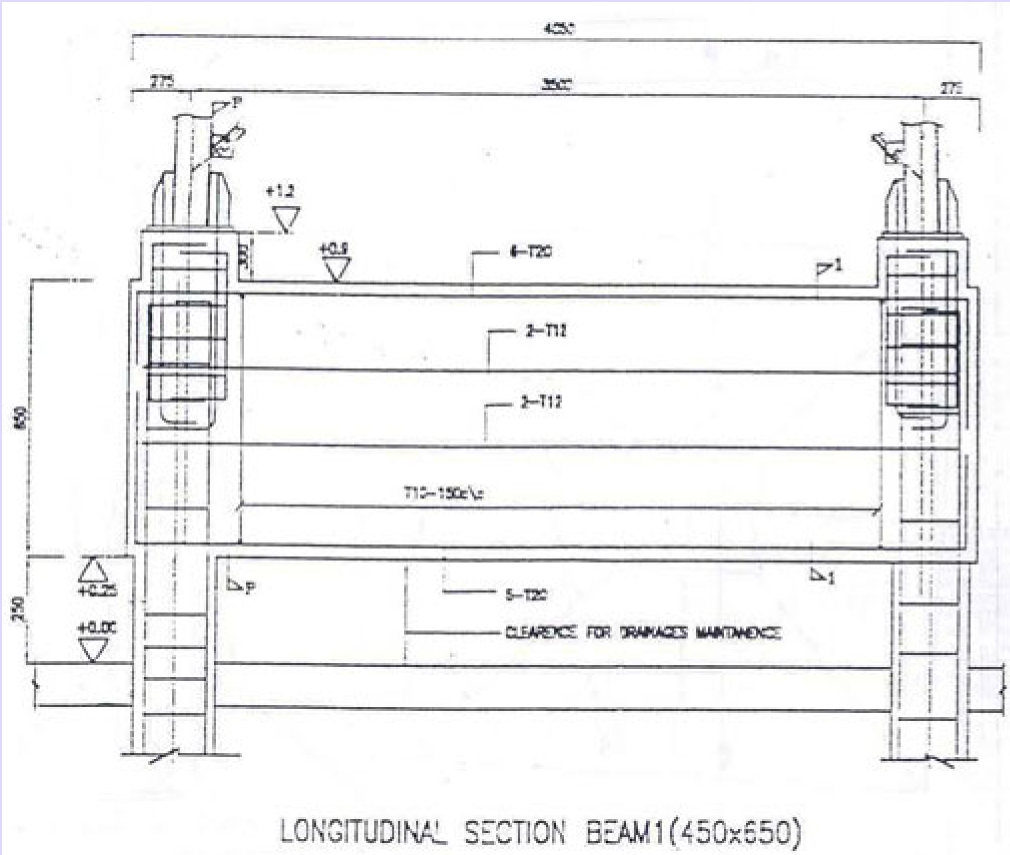


Figure 8.13

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

Example 1

Design forces on tower leg

Ultimate compression: 81,400 kg

Ultimate uplift : 58,250 kg

Ultimate shear : 2,250 kg

Tower data

Base width = 4m

Height = 36m

Soil data

Soil type : medium dense sand

Site location : Manali (Madras)

Average SPT value: 12

In-situ density γ = 1.79 t/m^3

γ_{sub} = 1.0 t/m^3

ϕ = 32^o

Water table at 1.5m below ground level.

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

N_q = 23, N_γ = 30

Type of foundation

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

Check for uplift

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

1. From equation (9.7) of Mayerhof,

$$T_u = \pi CBD + S_J \frac{\pi}{2} B \gamma D^2 K_u \tan\varphi + W \qquad \textbf{\textit{(1)}}$$
$$= 58.32\,t$$

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

T_u = 44.39 t (weight of frudtum of earth + weight of concrete as show Figure) ***(2)***

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

$$Q_{\phi r} = 2\pi\left(C + K_{\phi}\,\gamma \tan j\right)$$
$$\left[\frac{R_e\,x^2}{2} + D\tan\alpha\frac{x^2}{2} - \frac{x^3}{3}\tan\alpha\right]_0^D$$
$$= 30.12\,\text{t}$$

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

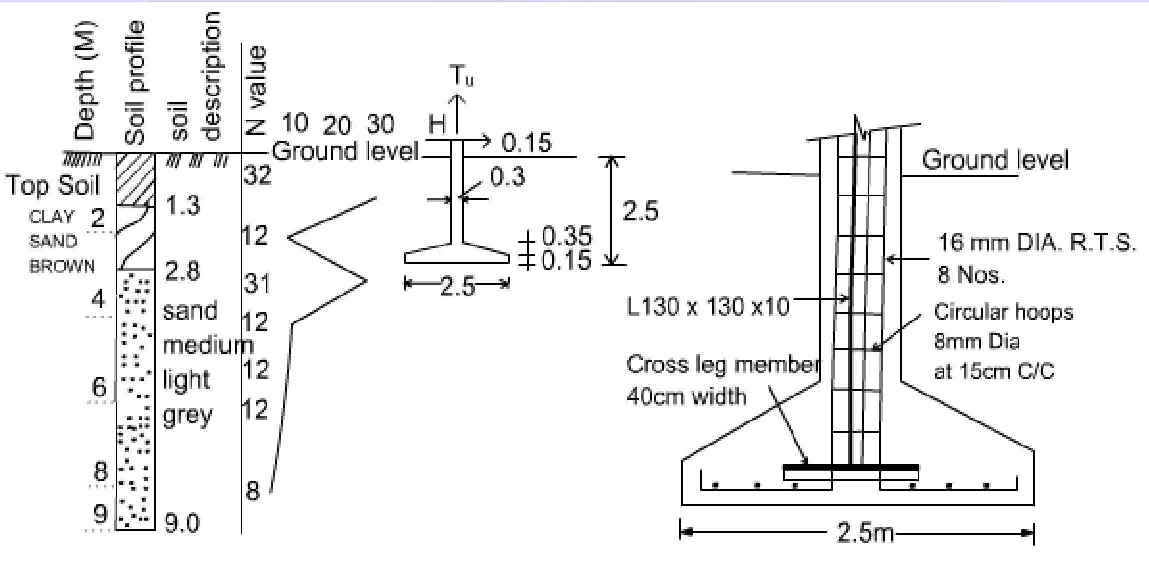


Figure 1 Pad type foundation

Q_s allowable = 30.12 / 2 = 15.06 t ***(3)***

Adding equation 2 and 3, the total uplift resistance,

T_u = 44.39 + 15.06 = 59.45 t ***(4)***

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

Design uplift force = 58.25 t, hence, safe.

Check for bearing capacity

From equation 9.1,

$$Q_{uo} = CN_c s_c d_c i_c + q\,(N_q - 1) s_q d_q i_q + \frac{1}{2} B\gamma N_r s_r d_r i_r w$$
$$= 94.8\ t/m^2$$

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

Area of footing = 2.5 x 2.5 = 6.25 m^2

Ultimate bearing capacity of footing = 6.25 x 94.8

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

Generally, bearing capacity is not the governing criterion.

Check for settlement

From equation (9.20) instantaneous settlement

$$s_i = I_P qB\left(\frac{1-v^2}{E_\theta}\right)$$

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

s_i = 0.313cm

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

Base width of tower = 4m

Maximum possible rotation

θ = 0.313 / 400 = 0.00078

Check for lateral capacity by Reese and Matlock Method

From Table 9.5, η_n = 1.5

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

Z_{max} = L/T = 2.78

Z = x / T at x = 0, Z = 0;

From Figure,

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

$H = H_u / 2 = 1,125$ kg

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

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

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

$M_{max} = A_m HT + B_m M_t$

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

Structural design

Deflection at ultimate load = 2 x 0.31 = 0.62cm.

Check for compression and bending

Moment due to eccentricity = 81,400 x 0.62

= 50,468 kg cm

Design moment = 154,856 + 50,468

= 205,324 kg cm

Design compressive load = 81,400 kg

Use M 150 concrete and Fe 415 steel.

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

Let $d' / D = 0.1$

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

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

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

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

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

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

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

Check for tension

Design uplift = 58,250 kg

Let the leg section be L 130 x 130 x 10

Perimeter = 52

Design bond stress 1.0 N/mm^2 (IS: 456 - 1978)

Development length required

$$= \frac{\text{load}}{\text{perimeter x stress}}$$

$$= \frac{58,250}{52 \text{ x } 10} = 112$$

Hence safe.

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

7. BRIDGES

7.1 Introduction

As discussed in earlier chapters the main advantages of structural steel over other construction materials are its strength and ductility. It has a higher strength to cost ratio in tension and a slightly lower strength to cost ratio in compression when compared with concrete. The stiffness to weight ratio of steel is much higher than that of concrete. Thus, structural steel is an efficient and economic material in bridges. Structural steel has been the natural solution for long span bridges since 1890, when the Firth of Forth cantilever bridge, the world's major steel bridge at that time was completed. Steel is indeed suitable for most span ranges, but particularly for longer spans. Howrah Bridge, also known as Rabindra Setu, is to be looked at as an early classical steel bridge in India. This cantilever bridge was built in 1943. It is 97 m high and 705 m long. This engineering marvel is still serving the nation, deriding all the myths that people have about steel. [See Fig.7.1]

Fig.7.1 Howrah bridge

The following are some of the advantages of steel bridges that have contributed to their popularity in Europe and in many other developed countries.

· They could carry heavier loads over longer spans with minimum dead weight, leading to smaller foundations.

· Steel has the advantage where speed of construction is vital, as many elements can be prefabricated and erected at site.

· In urban environment with traffic congestion and limited working space, steel bridges can be constructed with minimum disruption to the community.

· Greater efficiency than concrete structures is invariably achieved in resisting seismic forces and blast loading.

· The life of steel bridges is longer than that of concrete bridges.

· Due to shallow construction depth, steel bridges offer slender appearance, which make them aesthetically attractive. The reduced depth also contributes to the reduced cost of embankments.

· All these frequently leads to low life cycle costs in steel bridges

In India there are many engineers who feel that corrosion is a problem in steel bridges, but in reality it is not so. Corrosion in steel bridges can be effectively minimised by employing newly developed paints and special types of steel. These techniques are followed in Europe and other developed countries. These have been discussed in chapter 2.

7.2 Steel used in bridges

Steel used for bridges may be grouped into the following three categories:

(i) **Carbon steel:** This is the cheapest steel available for structural users where stiffness is more important than the strength. Indian steels have yield stress values up to 250 N/mm2 and can be easily welded. The steel conforming to IS: 2062 - 1969, the American ASTM A36, the British grades 40 and Euronorm 25 grades 235 and 275 steels belong to this category.

(ii) **High strength steels:** They derive their higher strength and other required properties from the addition of alloying elements. The steel conforming to IS: 961 - 1975, British grade 50, American ASTM A572 and Euronorm 155 grade 360 steels belong to this category. Another variety of steel in this category is produced with enhanced resistance to atmospheric corrosion. These are called 'weathering' steels in Europe, in America they conform to ASTM A588 and have various trade names like ' cor-ten'.

(iii) **Heat-treated carbon steels:** These are steels with the highest strength. They derive their enhanced strength from some form of heat-treatment after rolling namely normalisation or quenching and tempering.

The physical properties of structural steel such as strength, ductility, brittle fracture, weldability, weather resistance etc., are important factors for its use in bridge construction. These properties depend on the alloying elements, the amount of carbon, cooling rate of the steel and the mechanical deformation of the steel. The detailed discussion of physical properties of structural steel is presented in earlier chapter.

7.3 Classification of steel bridges

Steel bridges are classified according to

· the type of traffic carried

· the type of main structural system

· the position of the carriage way relative to the main structural system

These are briefly discussed in this section.

7.3.1 Classification based on type of traffic carried

Bridges are classified as

· Highway or road bridges

· Railway or rail bridges

· Road - cum - rail bridges

7.3.2 Classification based on the main structural system

Many different types of structural systems are used in bridges depending upon the span, carriageway width and types of traffic. Classification, according to make up of main load carrying system, is as follows:

(i) Girder bridges - Flexure or bending between vertical supports is the main structural action in this type. Girder bridges may be either solid web girders or truss girders or box girders. Plate girder bridges are adopted for simply supported spans less than 50 m and box girders for continuous spans upto 250 m. Cross sections of a typical plate girder and box girder bridges are shown in Fig.7.2 (a) and Fig. 7.2(b) respectively. Truss bridges [See Fig.7. 2(c)] are suitable for the span range of 30 m to 375 m. Cantilever bridges have been built with success with main spans of 300 m to 550 m. In the next chapter girder bridges are discussed in detail. They may be further, sub-divided into simple spans, continuous spans and suspended-and-cantilevered spans, as illustrated in Fig.7. 3.

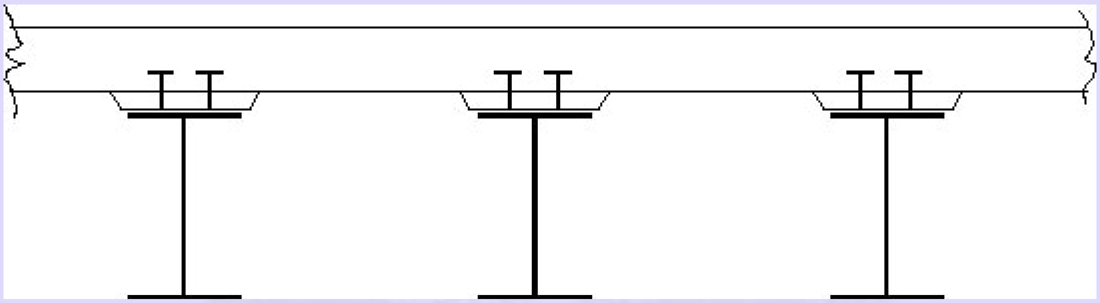


Fig.7.2 (a) Plate girder bridge section

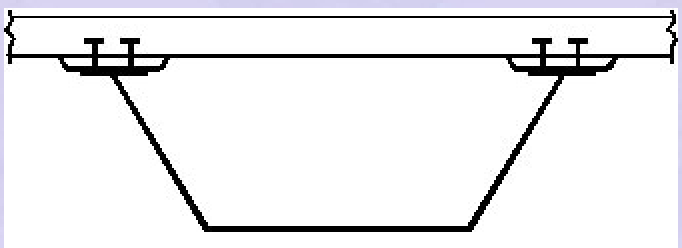


Fig.7.2 (b) Box girder bridge section

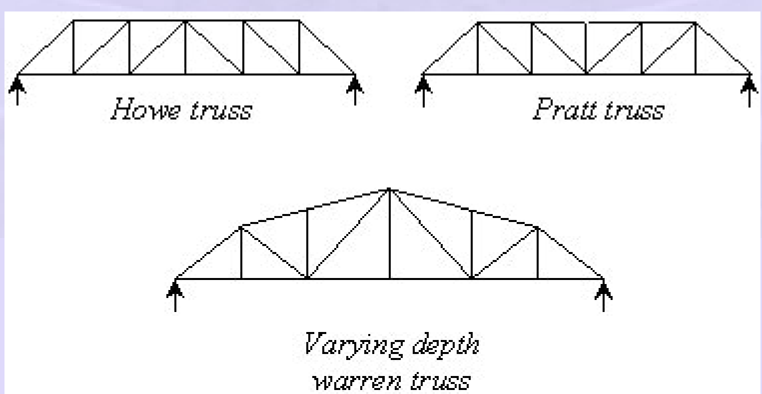


Fig.7.2 (c) Some of the trusses used in steel bridges

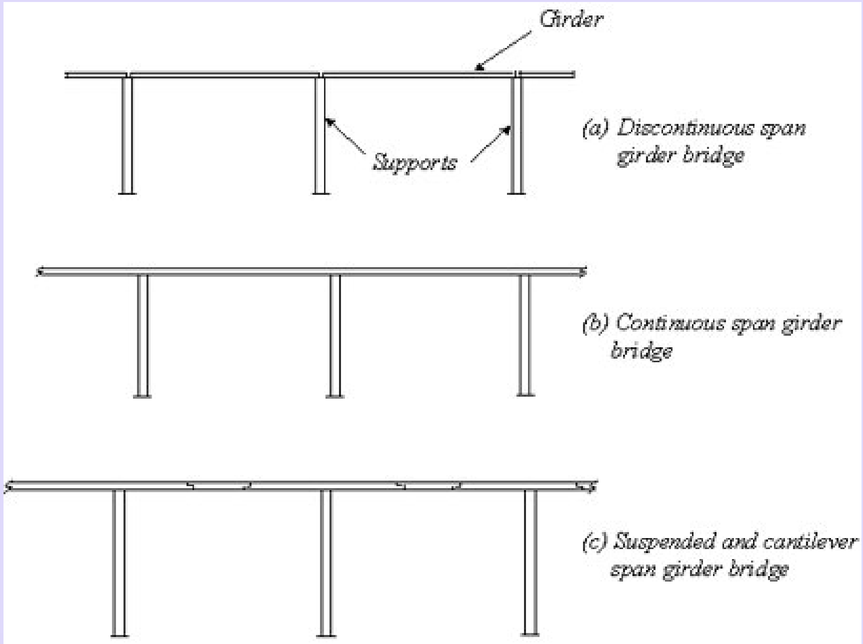


Fig.7.3 Typical girder bridges

(ii) Rigid frame bridges - In this type, the longitudinal girders are made structurally continuous with the vertical or inclined supporting member by means of moment carrying joints [Fig.7.4]. Flexure with some axial force is the main forces in the members in this type. Rigid frame bridges are suitable in the span range of 25 m to 200 m.

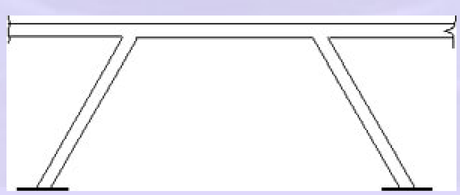


Fig.7.4 Typical rigid frame bridge

(iii) Arch bridges

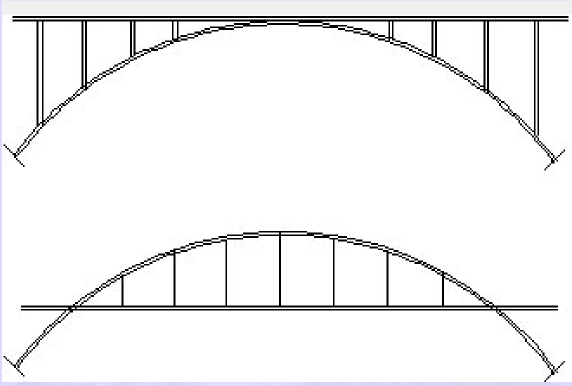


Fig.7.5 Typical arch bridges

The loads are transferred to the foundations by arches acting as the main structural element. Axial compression in arch rib is the main force, combined with some bending. Arch bridges are competitive in span range of 200 m to 500 m. Examples of arch bridges are shown in Fig. 7.5.

(iv) Cable stayed bridges - Cables in the vertical or near vertical planes support the main longitudinal girders. These cables are hung from one or more tall towers, and are usually anchored at the bottom to the girders. Cable stayed bridges are economical when the span is about 150 m to 700 m. Layout of cable stayed bridges are shown in Fig. 7.6.

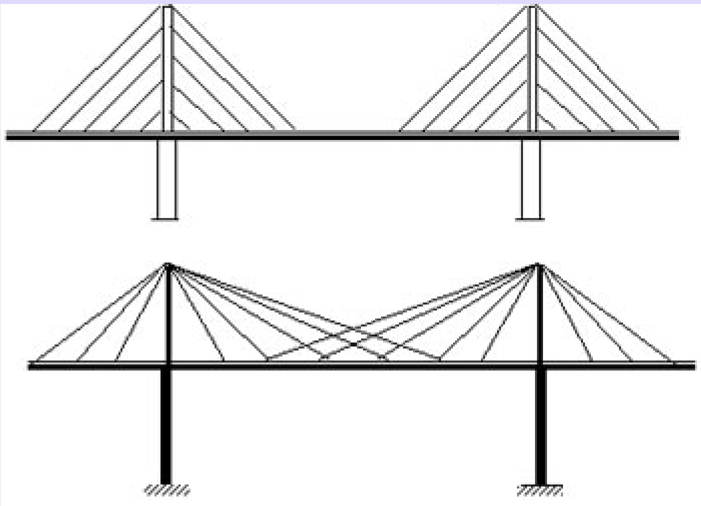


Fig.7.6 Layout of cable stayed bridges

(v) Suspension bridges - The bridge deck is suspended from cables stretched over the gap to be bridged, anchored to the ground at two ends and passing over tall towers erected at or near the two edges of the gap. Currently, the suspension bridge is best solution for long span bridges. Fig. 7.7 shows a typical suspension bridge. Fig. 7.8 shows normal span range of different bridge types.

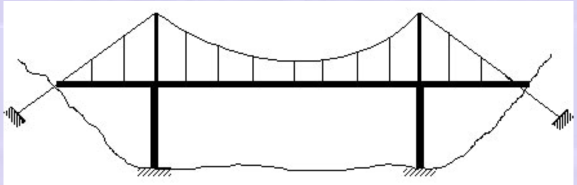


Fig.7.7 Suspension bridge

7.3.3 Classification based on the position of carriageway

The bridges may be of the "deck type", "through type" or "semi-through type". These are described below with respect to truss bridges:

(i) Deck type bridge - The carriageway rests on the top of the main load carrying members. In the deck type plate girder bridge, the roadway or railway is placed on the top flanges. In the deck type truss girder bridge, the roadway or railway is placed at the top chord level as shown in Fig. 7.9(a).

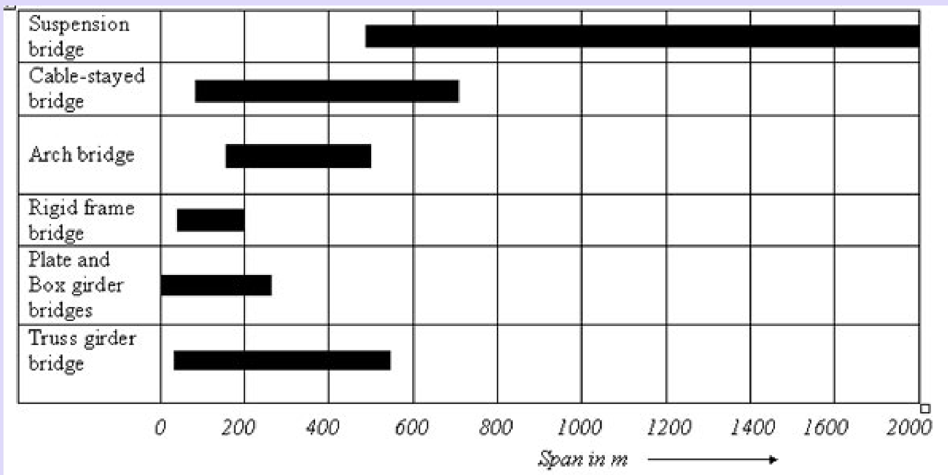


Fig.7.8 Normal span ranges of bridge system

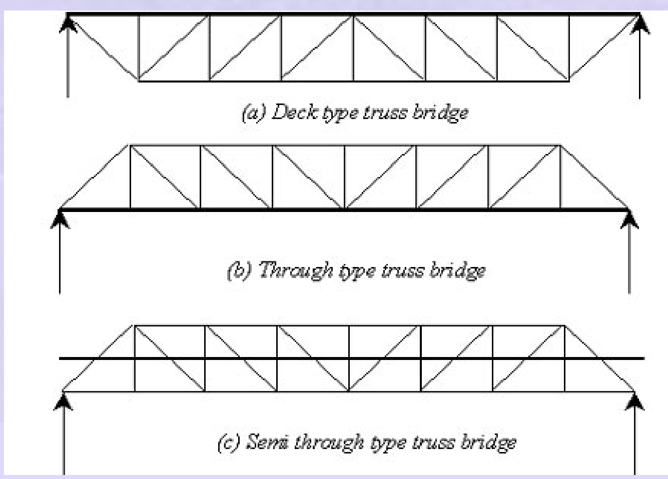


Fig.7.9 Typical deck, through and semi-through type truss bridges

(ii) Through type bridge - The carriageway rests at the bottom level of the main load carrying members [Fig. 7.9(b)]. In the through type plate girder bridge, the roadway or railway is placed at the level of bottom flanges. In the through type truss girder bridge, the roadway or railway is placed at the bottom chord level. The bracing of the top flange or lateral support of the top chord under compression is also required.

(iii) Semi through type bridge - The deck lies in between the top and the bottom of the main load carrying members. The bracing of the top flange or top chord under compression is not done and part of the load carrying system project above the floor level as shown in Fig. 7.9(c). The lateral restraint in the system is obtained usually by the U-frame action of the verticals and cross beam acting together.

7.4 Loads and Load combinations

7.4.1 Loads on bridges

The following are the various loads to be considered for the purpose of computing stresses, wherever they are applicable.

- Dead load
- Live load
- Impact load
- Longitudinal force
- Thermal force
- Wind load
- Seismic load
- Racking force
- Forces due to curvature.
- Forces on parapets
- Frictional resistance of expansion bearings
- Erection forces

Dead load – The dead load is the weight of the structure and any permanent load fixed thereon. The dead load is initially assumed and checked after design is completed.

Live load – Bridge design standards specify the design loads, which are meant to reflect the worst loading that can be caused on the bridge by traffic, permitted and expected to pass over it. In India, the Railway Board specifies the standard design loadings for railway bridges in bridge rules. For the highway bridges, the Indian Road Congress has specified standard design loadings in

IRC section II. The following few pages brief about the loadings to be considered. For more details, the reader is referred to the particular standard.

Railway bridges: Railway bridges including combined rail and road bridges are to be designed for railway standard loading given in bridge rules. The standards of loading are given for:

- **Broad gauge** - Main line and branch line
- **Metre gauge** - Main line, branch line and Standard C
- **Narrow gauge** - H class, A class main line and B class branch line

The actual loads consist of axle load from engine and bogies. The actual standard loads have been expressed in bridge rules as equivalent uniformly distributed loads (EUDL) in tables to simplify the analysis. These equivalent UDL values depend upon the span length. However, in case of rigid frame, cantilever and suspension bridges, it is necessary for the designer to proceed from the basic wheel loads. In order to have a uniform gauge throughout the country, it is advantageous to design railway bridges to Broad gauge main line standard loading. The EUDLs for bending moment and shear force for broad gauge main line loading can be obtained by the following formulae, which have been obtained from regression analysis:

For bending moment:

$$\text{EUDL in kN} = 317.97 + 70.83l + 0.0188l2 \geq 449.2 \text{ kN} \qquad \textbf{\textit{(7.1)}}$$

For shear force:

$$\text{EUDL in kN} = 435.58 + 75.15l + 0.0002l2 \geq 449.2 \text{ kN} \qquad \textbf{\textit{(7.2)}}$$

Note that, l is the effective span for bending moment and the loaded length for the maximum effect in the member under consideration for shear. 'l ' should be in metres. The formulae given here are not applicable for spans less than or equal to 8 m with ballast cushion. For the other standard design loading the reader can refer to Bridge rules.

Highway bridges: In India, highway bridges are designed in accordance with IRC bridge code. IRC: 6 - 1966 – Section II gives the specifications for the various loads and stresses to be considered in bridge design. There are three types of standard loadings for which the bridges are designed namely, IRC class AA loading, IRC class A loading and IRC class B loading.

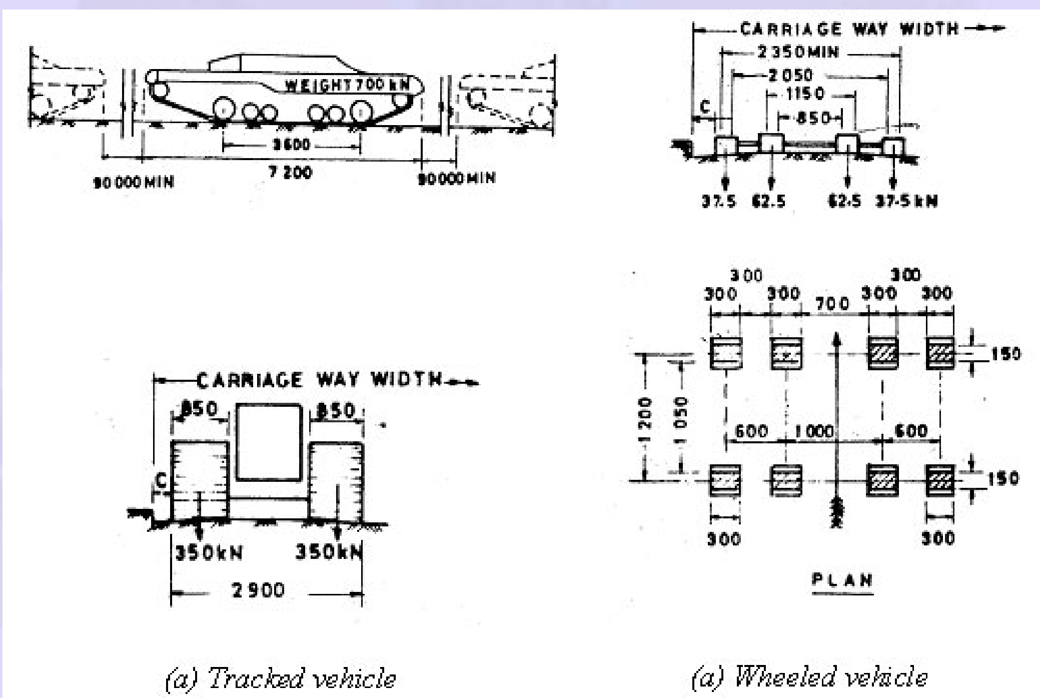


Fig.7.10 IRC AA loading

IRC class AA loading consists of either a tracked vehicle of 70 tonnes or a wheeled vehicle of 40 tonnes with dimensions as shown in Fig. 7.10. The units in the figure are mm for length and tonnes for load. Normally, bridges on national highways and state highways are designed for these loadings. Bridges designed for class AA should be checked for IRC class A loading also, since under certain conditions, larger stresses may be obtained under class A loading. Sometimes class 70 R loading given in the Appendix - I of IRC: 6 - 1966 - Section II can be used for IRC class AA loading. Class 70R loading is not discussed further here.

Class A loading consists of a wheel load train composed of a driving vehicle and two trailers of specified axle spacings. This loading is normally adopted on all roads on which permanent bridges are constructed. Class B loading is adopted for temporary structures and for bridges in specified areas. For class A and class B loadings, reader is referred to IRC: 6 - 1966 – Section II.

Foot Bridges and Footpath on Bridges – The live load due to pedestrian traffic should be treated as uniformly distributed over the pathway. For the design of footbridges or footpaths on railway bridges, the live load including dynamic effects should be taken as 5.0 kN/m^2 of the footpath area. For the design of foot-path on a road bridges or road-rail bridges, the live load including dynamic effects may be taken as 4.25 kN/m^2 except that, where crowd loading is likely, this may be increased to 5.0 kN/m^2.

The live load on footpath for the purpose of designing the main girders has to be taken as follows according to bridge rules:

(i) For effective spans of 7.5 m or less - 4.25 kN/m^2

(ii) The intensity of load is reduced linearly from 4.25 kN/m^2 for a span of 7.5 m to 3.0 kN/m^2 for a span of 30m

(iii) For effective spans over 30 m, the UDL may be calculated as given below:

$$P = \frac{1}{100}\left(13.3 + \frac{400}{l}\right)\left(\frac{17 - W}{1.4}\right) kN/m^2 \qquad \textbf{\textit{(7.3)}}$$

Where, P = Live load in kN/m^2

l = Effective span of the bridge in m.

W = Width of the foot path in m.

Where foot-paths are provided on a combined rail-road bridge, the load on foot-path for purpose of designing the main girders should be taken as 2.0 kN/m^2.

Impact load

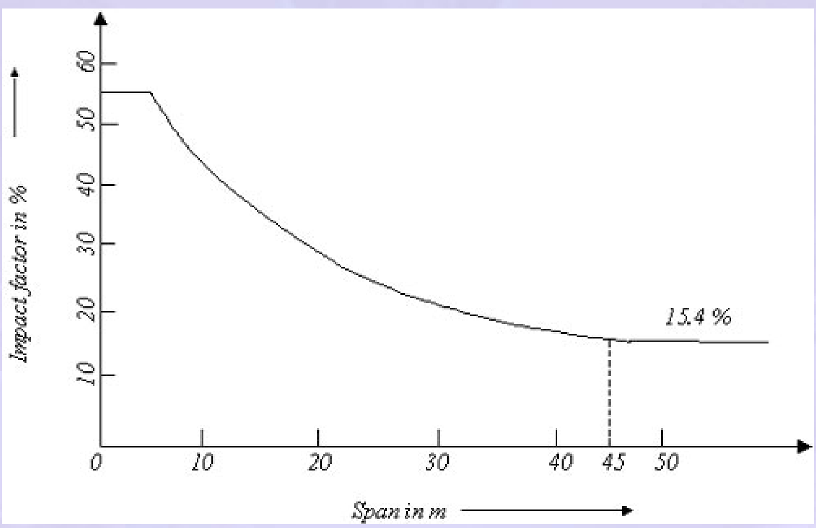


Fig 7.11 Impact percentage curve for highway bridges for IRC class A and IRC class B loadings

The dynamic effect caused due to vertical oscillation and periodical shifting of the live load from one wheel to another when the locomotive is moving is known as impact load. The impact load is determined as a product of impact factor, I, and the live load. The impact factors are specified by different authorities for different types of bridges. The impact factors for different bridges for different types of moving loads are given in the table 7.1. Fig.7.11 shows impact percentage curve for highway bridges for class AA loading. Note that, in the above table l is loaded length in m and B is spacing of main girders in m.

Longitudinal forces – Longitudinal forces are set up between vehicles and bridge deck when the former accelerate or brake. The magnitude of the force F, is given by

$$F = \frac{W}{g} \frac{\delta V}{\delta t} \qquad \textbf{\textit{(7.4)}}$$

Where, W – weight of the vehicle

g – acceleration due to gravity

δV – change in velocity in time d t

This loading is taken to act at a level 1.20 m above the road surface. No increase in vertical force for dynamic effect should be made along with longitudinal forces. The possibility of more than one vehicle braking at the same time on a multi-lane bridge should also be considered.

Table 7.1: Impact factors for different bridges

BRIDGE	LOADING		IMPACT FACTOR (I)
Railway bridgesaccording to bridge rules	Broad gauge and Meter gauge	(a) Single track	$\frac{20}{14+\ell} \le 1.0$
		(b) Main girder of double track with two girders	$0.72 \times \frac{20}{14+\ell} \le 0.72$
		(c) Intermediate main girder of multiple track spans	$0.60 \times \frac{20}{14+\ell} \le 0.60$
		(d) Outside main girders of multiple track spans	Specified in (a) or (b) whichever applies
		(e) Cross girders carrying two or more tracks	$0.72 \times \frac{20}{14+\ell} \le 0.72$

	Broad gauge	Rails with ordinary fish plate joints and supported directly on sleepers or transverse steel troughing	$\frac{7.32}{B+5.49}$
	Meter gauge		$\frac{5.49}{B+4.27}$
	Narrow gauge		$\frac{9.5}{91.5+\ell}$
Highway bridges according to IRC regulations	IRC class AA loading	(i) Spans less than 9 m. (a) Tracked vehicle (b) Wheeled vehicle	0.25 for spans up to 5 mand linearly reducing to 0.10 to spans of 9m 0.25
		(ii) Spans 9 m or more (a) Tracked vehicle (b) Wheeled vehicle	0.10 0.25 for spans up to 23 mand in accordance with the curve indicated inFig .11 for spans in excess of 23 m
	IRC class A loading and IRC class B loading	Spans between 3 m and45 m	$\frac{9}{13.5+\ell}$ In accordance with the curve indicated inFig .11 for all spans
Foot bridges			No separate impact allowance is made

Thermal forces – The free expansion or contraction of a structure due to changes in temperature may be restrained by its form of construction. Where any portion of the structure is not free to expand or contract under the variation of temperature, allowance should be made for the stresses resulting from this condition. The coefficient of thermal expansion or contraction for steel is 11.7 x 10-6 / ^{o}C

Wind load – Wind load on a bridge may act

· Horizontally, transverse to the direction of span

· Horizontally, along the direction of span

· Vertically upwards, causing uplift

· Wind load on vehicles

Wind load effect is not generally significant in short-span bridges; for medium spans, the design of sub-structure is affected by wind loading; the super structure design is affected by wind only in long spans. For the purpose of the design, wind loadings are adopted from the maps and tables given in IS: 875 (Part III). A wind load of 2.40 kN/m^2 is adopted for the unloaded span of the railway, highway and footbridges. In case of structures with opening the effect of drag around edges of members has to be considered.

Racking force – This is a lateral force produced due to the lateral movement of rolling stocks in railway bridges. Lateral bracing of the loaded deck of railway spans shall be designed to resist, in addition to the wind and centrifugal loads, a lateral load due to racking force of 6.0 kN/m treated as moving load. This lateral load need not be taken into account when calculating stresses in chords or flanges of main girders.

Forces on parapets - Railings or parapets shall have a minimum height above the adjacent roadway or footway surface of 1.0 m less one half the horizontal width of the top rail or top of the parapet. They shall be designed to resist a lateral horizontal force and a vertical force each of 1.50 kN/m applied simultaneously at the top of the railing or parapet.

Seismic load – If a bridge is situated in an earthquake prone region, the earthquake or seismic forces are given due consideration in structural design. Earthquakes cause vertical and horizontal forces in the structure that will be proportional to the weight of the structure. Both horizontal and vertical components have to be taken into account for design of bridge structures. IS:1893 – 1984 may be referred to for the actual design loads.

Forces due to curvature - When a track or traffic lane on a bridge is curved allowance for centrifugal action of the moving load should be made in designing the members of the bridge. All the tracks and lanes on the structure being considered are assumed as occupied by the moving load.

This force is given by the following formula:

$$C = \frac{W V^2}{12.7 R} \qquad \textit{(7.5)}$$

Where, C - Centrifugal force in kN/m

W - Equivalent distributed live load in kN/m

V - Maximum speed in km/hour

R - Radius of curvature in m

Erection forces – There are different techniques that are used for construction of railway bridges, such as launching, pushing, cantilever method, lift and place. In composite construction the composite action is mobilised only after concrete hardens and prior to that steel section has to carry dead and construction live loads. Depending upon the technique adopted the stresses in the members of the bridge structure would vary. Such erection stresses should be accounted for in design. This may be critical, especially in the case of erection technologies used in large span bridges.

7.4.2 Load combinations

Stresses for design should be calculated for the most sever combinations of loads and forces. Four load combinations are generally considered important for checking for adequacy of the bridge. These are given in Table 7.2 and are also specified in IS 1915 - 1961.

Table 7.2: Load combinations

SNo	Load combination	Loads
1	Stresses due to normal loads	Dead load, live load, impact load and centrifugal force
2	Stresses due to normal loads + occasional loads	Normal load as in (1) + wind load, other lateral loads, longitudinal forces and temperature stresses
3	Stresses due to loads during erection	-
4	Stresses due to normal loads + occasional loads + Extra-ordinary loads like seismic excluding wind load	Loads as in (2) + with seismic load instead of wind

7.5 Analysis of girder bridges

As discussed above, bridge decks are required to support both static and moving loads. Each element of a bridge must be designed for the most severe conditions that can possibly be developed in that member. Live loads should be placed in such a way that they will produce the most severe conditions. The critical positions of live loads will not be the same for every member. A useful method for determining the most severe condition of loading is by using “influence lines”.

An influence line represents some internal force such as shear force, bending moment etc. at a particular section or in a given member of girder, as a unit load moves over the span. The ordinate of influence line represents the value of that function when the unit load is at that particular point on the structure. Influence lines provide a systematic procedure for determining how the force (or a moment or shear) in a given part of a structure varies as the applied load moves about on the structure. Influence lines of responses of statically determinate structures consist only of straight lines whereas this is not true of indeterminate structures. It may be noted that a shear or bending moment diagram shows the variation of shear or moment across an entire structure for loads fixed in one position. On the other hand an influence line for shear or moment shows the variation of that response at one particular section in the structure caused by the movement of a unit load from one end of the structure to the other. In the following section, influence lines only for statically determinate structures are discussed.

7.5.1 Influence lines for beams and plate girders

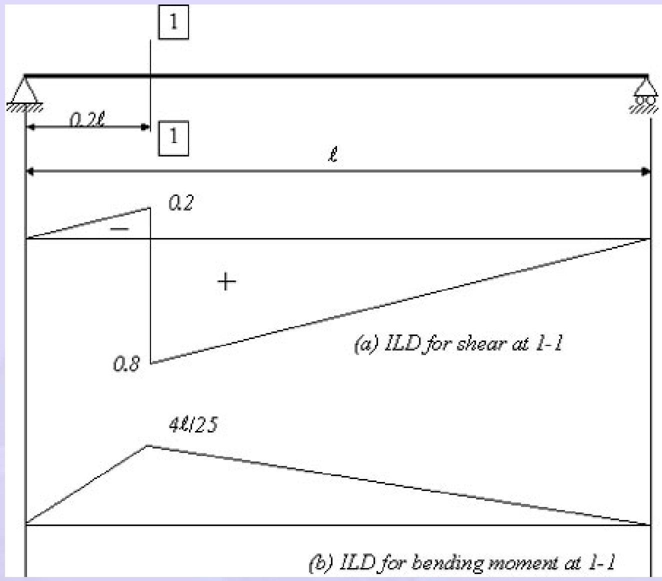


Fig. 7.12 Influence lines for shear and bending moment

Fig. 7.12(a) shows the influence line for shear at a section in a simply supported beam. It is assumed that positive shear occurs when the sum of the transverse forces to the left of a section is in the upward direction or when the sum of the forces to the right of the section is downward.

A unit force is placed at various locations and the shear force at sections 1-1 is obtained for each position of the unit load. These values give the ordinates of influence line with which the influence line diagram for shear force at sections 1-1 can be constructed. Note that the slope of the influence line for shear on the left of the section is equal to the slope of the influence line on the right of the section. This information is useful in drawing shear force influence line in all cases.

Influence line for bending moment at the same section 1-1 of the simple beam is shown in Fig. 7.12(b). For a section, when the sum of the moments of all the forces to the left is clockwise or when the sum to the right is counter-clockwise, the moment is taken as positive. The values of bending moment at sections 1-1 are obtained for various positions of unit load and influence line is plotted. The detailed calculation of ordinates of influence lines is illustrated for members of the truss girder in the following section.

7.5.2 Influence lines for truss girders

Influence lines for support reactions and member forces for truss may be constructed in the same manner as those for beams. They are useful to determine the maximum force that may act on the truss members. The truss shown in Fig.7.13 is considered for illustrating the construction of influence lines for trusses.

The member forces in U_3U_4, U_3L_4 and L_3L_4 are determined by passing a section X-X and considering the equilibrium of the free body diagram of one of the truss segments.

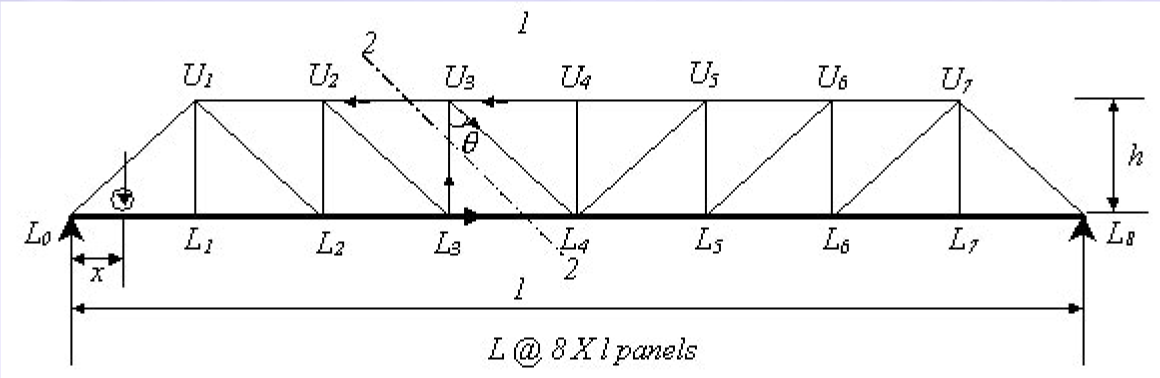


Fig.7.13 A typical truss

Consider a section 1-1 and assume unit-rolling load is at a distance x from L_0. Then, from equilibrium considerations reactions at L_8 and L_0 are determined. The reactions are:

$$\text{Reaction at } L_8 = \left(\frac{x}{L}\right)$$

$$\text{Reaction at } L_0 = \left(1-\frac{x}{L}\right)$$

Consider the left-hand side of the section and take moments about L_4 by assuming appropriate directions for the forces in the members.

When unit load is in between L_0 and L_4:

$$\sum M_{L4} = 0$$

$$U_3 U_4 \text{ x } h - \left(\frac{x}{L}\right) \text{x } 4l = 0$$

$$U_3 U_4 = \frac{x}{h}\frac{4l}{L} = 0.5\frac{x}{h}$$

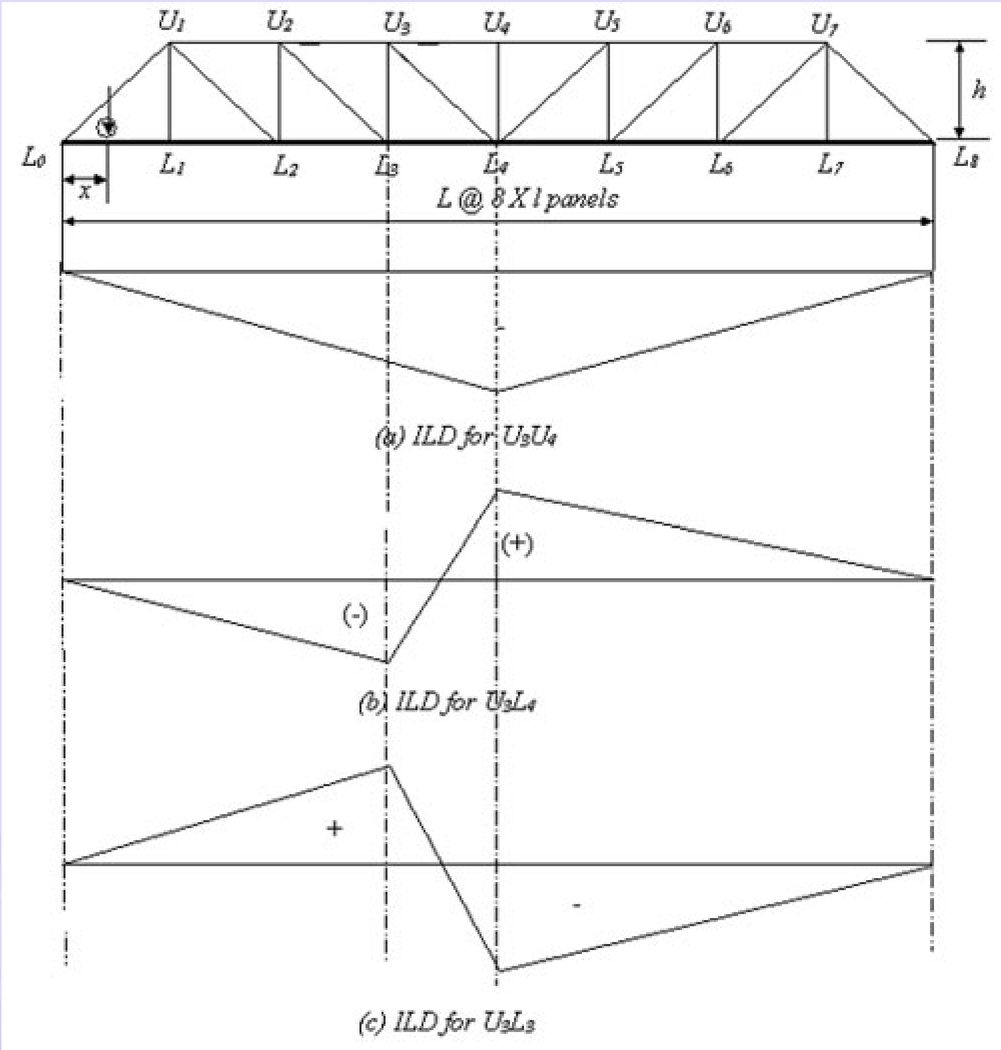
When unit load is in between L_4 and L_8:

Then, there will not be rolling unit load in the left-hand side section.

$$U_3 U_4 = \frac{4l}{h}\left(1-\frac{x}{L}\right)$$

Note that the influence diagram gives force in the member U_3U_4 directly, due to the unit load.

7.5.2.2 Influence line diagram for member U_3L_4 (Inclined member) [Fig 7.14(b)]



(a) ILD for U_3U_4

(b) ILD for U_3L_4

(c) ILD for U_3L_3

Fig.7.14 Typical shapes of influence lines

Again consider the left-hand side of the section 1-1, and use the equilibrium equation for vertical forces i.e

$\sum V = 0$ where, V represents the vertical force.

When unit load is in between L_0 and L_3:

$$\frac{x}{L} + U_3 L_4 \cos\theta = 0$$

$$\Rightarrow U_3 L_4 = \frac{-x}{L\cos\theta}$$

$$\text{Where, } \theta = \tan^{-1}\left(\frac{l}{h}\right)$$

When unit load is in between L_4 and L_8:

$$U_3 U_4 = \frac{1}{\cos\theta}\left(1 - \frac{x}{L}\right)$$

When unit load is in between L_3 and L_4:

Since the variation of force in member U_3L_4 is linear as the unit load moves from L_3 to L_4 joining the ordinates of influence line at L_3 and L_4 by a straight line gives the influence line diagram in that zone. Note that, U_3L_4 represents the force in that member.

7.5.2.3 Influence line diagram for U_3L_3 (Vertical member) [Fig. 7.14(c)]

Consider the left-hand side of the section 2-2 shown in Fig.7.13 for illustrating the construction of influence line for vertical member.

When unit load is in between L_0 and L_3:

By considering the equilibrium equation on the section left hand side of axis 2-2.

$$U_3 L_4 - \frac{x}{L} = 0$$

$$\Rightarrow U_3 L_4 = \frac{x}{L}$$

When unit load is in between L_4 and L_8:

$$U_3 L_4 = -\left(1 - \frac{x}{L}\right)$$

When unit load is in between L_3 and L_4:

Joining the ordinates of influence line at L_3 and L_4 by a straight line gives the influence line diagram between L_3 and L_4. U_3L_3 represents the force in that member.

Similarly influence line diagrams can be drawn for all other members. Typical shapes of influence line diagrams for the members discussed are shown in Fig.7.14. The design force in the member is obtained in the following manner. In this chapter, compressive forces are considered negative and tensile forces are positive.

Case (1): If the loading is Railway loading (UDL)

· Influence line diagram for force is drawn for that member

· The algebraic sum of areas of influence line under loaded length multiplied by magnitude of uniformly distributed load gives the design force.

Case (2): If the loading is Highway loading (Concentrated loading)

· Influence line diagram for force is drawn for that member

· The algebraic sum of the respective ordinates of influence line at the concentrated load location multiplied by concentrated loads gives design load of that member

· The series of concentrated loads are arranged in such a way that the maximum value of the desired member force is obtained.

7.6 Plate girder bridges

Plate girders became popular in the late 1800's, when they were used in construction of railroad bridges. The plates were joined together using angles and rivets to obtain plate girders of desired size. By 1950's welded plate girders replaced riveted and bolted plate girders in developed world due to their better quality, aesthetics and economy. Fig.7.15 shows the cross sections of two common types of plate girder bridges. The use of plate girders rather than rolled beam sections for the two main girders gives the designer freedom to select the most economical girder for the structure.

If large embankment fills are required in the approaches to the bridge, in order to comply with the minimum head-room clearance required, the half through bridge is more appropriate [Fig.7.15 (a)]. This arrangement is commonly used in railway bridges where the maximum permissible approach gradient for the track is low. In this case the restraint to lateral buckling of compression flange is achieved by a moment resisting U-frame consisting of floor beam and vertical stiffness, which are connected together with a moment resisting joint. If the construction depth is not critical, then a deck-type bridge, as shown in Fig.7.15 (b) is a better solution, in which case the bracings provide restraint to compression flange against lateral buckling.

7.6.1 Main plate girders

The design criterion for main girders as used in buildings, was discussed in chapters on Plate Girders. In the following sections some additional aspects that are to be considered in the design of plate girders in bridges, are discussed.

Generally, the main girders require web stiffening (either transverse or both transverse and longitudinal) to increase efficiency. The functions of these web stiffeners are described in the chapters on plate girders. Sometimes variations of bending moments in main girders may require variations in flange thickness to obtain economical design. This may be accomplished either by welding additional cover plates or by using thicker flange plate in the region of larger moment. In very long continuous spans (span> 50 m) variable depth plate girders may be more economical.

Initial design of main plate girder is generally based on experience or thumb rules such as those given below. Such rules also give a good estimate of dead load of the bridge structure to be designed. For highway and railway bridges, indicative range of values for various overall dimension of the main girders are given below:

Overall depth, D: $l/18 \leq D \leq l/12$ (Highway bridges)

$l/10 \leq D \leq l/7$ (Railway bridges)

Flange width, 2b: $D/4 \leq 2b \leq D/3$

Flange thickness, T: $b/12 \leq T \leq b/5$

Web thickness, t: $t \approx D/125$

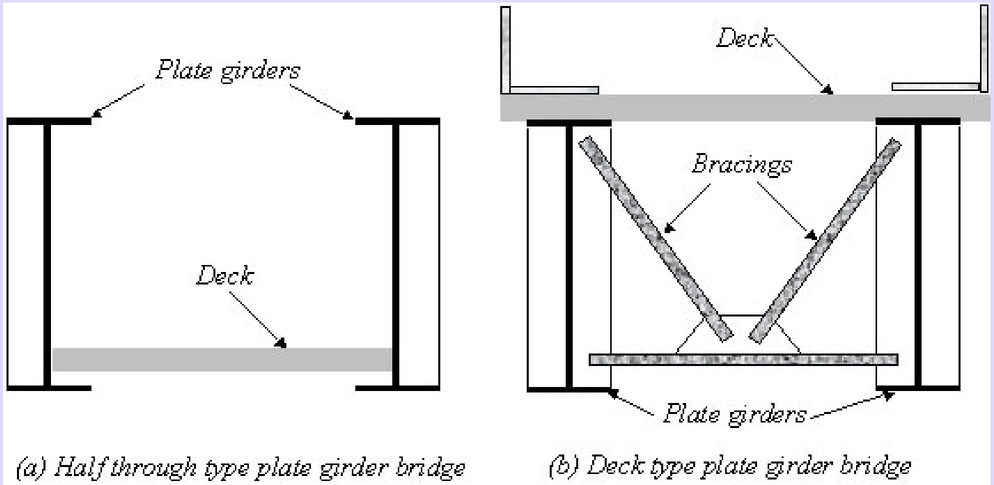


Fig.7.15 Common types of plate girder bridge

Here, l is the length between points of zero moment. The detailed design process to maximise girder efficiency satisfying strength, stability, stiffness, fatigue or dynamic criteria, as relevant, can be then carried out. Recent developments in optimum design methods allow direct design of girder bridges, considering minimisation of weight/cost.

7.6.1.1 Detailed design of main plate girders in bridges

The load effects (such as bending moment and shear force) are to be found using individual and un-factored load cases. Based on these, the summation of load effects due to different load combinations for various load factors are obtained. Since bridges are subjected to cyclic loading and hence are vulnerable to fatigue, redistribution of forces due to plastic mechanism formation is not permitted under BS 5400: Part - 3. The design is made based on Limit State of collapse for the material used considering the following:

· Shape limitation based on local buckling

· Lateral torsional buckling

· Web buckling

· Interaction of bending and shear

· Fatigue effect

Shape limitation based on local buckling

Depending on the type of cross section (compact or non-compact) the variation of stress over the depth at failure varies. A compact section can develop full plastic moment i.e. rectangular stress block as shown in Fig.7.16 (a). Before the development of this full plastic moment, local buckling of individual component plates should not occur. Thus the compact section should possess minimum thickness of elements on the compression zone such that they do not buckle locally before the entire compression zone yields in compression. The minimum thickness of elements for a typical compact section is shown in Fig.7.17, where f_y is to be substituted in SI units (MPa).

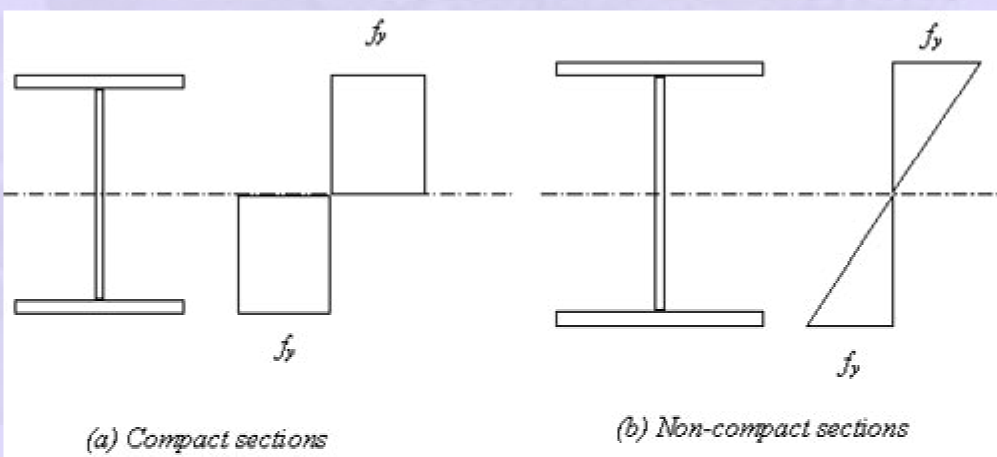


Fig.7.16 Design stresses

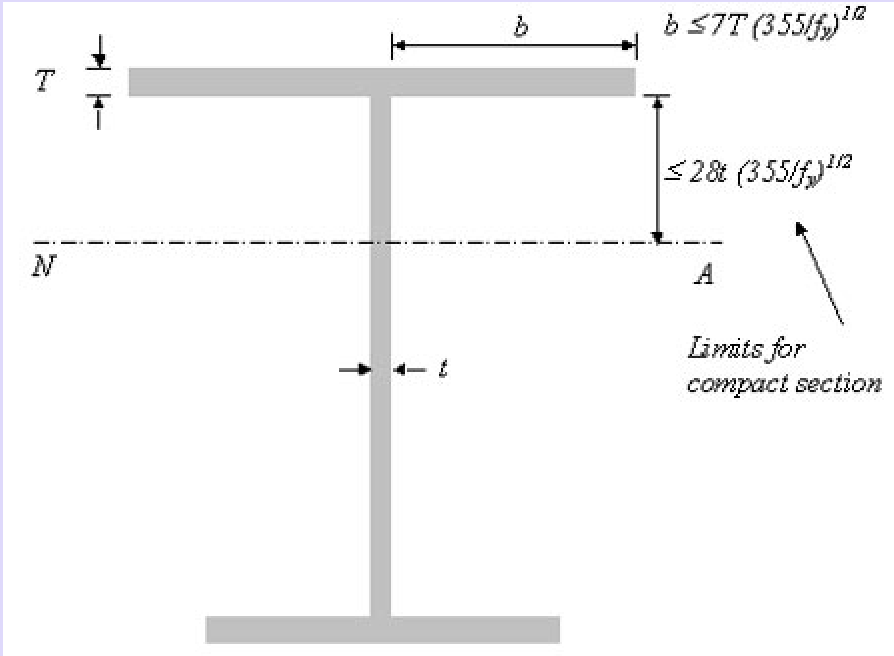


Fig.7.17 Shape limitations for plate girder

The section that does not fulfill the minimum thickness criterion of compact section is defined as non-compact section. A non-compact section may buckle locally before full section plastic capacity is reached. Therefore the design of such section is based on triangular stress block wherein yielding at the extreme fibre, as shown in Fig.7.16 (b), limit the design moment.

The moment capacity of the compact and non-compact cross sections can be evaluated by the following formulae:

$$M_u = Z_p f_y / \gamma_m \quad \text{for compact sections} \qquad \textbf{(7.6a)}$$

$$M_u = Z f_y / \gamma_m \quad \text{for non-compact sections} \qquad \textbf{(7.6b)}$$

Where, f_y - yield stress

Z_p - plastic modulus

Z - elastic modulus

γm - partial safety factor for material strength (1.15)

Even in the compact section, the use of plastic modulus does not imply that plastic analysis accounting for moment redistribution is applicable. BS 5400: Part - 3 precludes plastic analysis and does not allow any moment redistribution to be considered. This is to avoid repeated plastification under cyclic loading and the consequent low cycle fatigue failure. When non-compact sections are used the redistribution will not occur and hence plastic analysis is not applicable.

Lateral torsional buckling

A typical bridge girder with a portion of the span, over which the compression flange is laterally unrestrained, is shown in Fig. 7.18(a). Such a girder is susceptible to lateral torsional buckling. Fig. 7.18(b) shows a laterally buckled view of a portion of the span. The displacements at mid span, where the beam is laterally restrained, will be only vertical, as shown in Fig. 7.18(c). A part of the beam between restraints can translate downwards and sideways and rotate about shear centre [Fig. 7.18(d)]. Failure may then be governed by lateral torsional buckling. This type of failure depends on the unrestrained length of compression flange, the geometry of cross section, moment gradient etc. The procedure in detail for calculating the value of the limiting compressive stress is given in chapters on laterally unrestrained beams.

Web buckling

The web of plate girders resist the shear in the three modes, namely (i) pure shear, (ii) tension field action and (iii) that due to formation of collapse mechanism. These are discussed in detail in the chapters on plate girders. They are presented briefly below:

The elastic critical shear strength of a plate girder is given by

$$q_c = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{d}\right)^2 \qquad \textbf{\textit{(7.7)}}$$

Where,

$$k = 5.34 + 4\left(\frac{d}{a}\right)^2 \quad \text{when}\ \frac{a}{d} \geq 1.0$$

$$k = 4 + 5.34\left(\frac{d}{a}\right) \quad \text{when}\ \frac{a}{d} < 1.0$$

Where t, d and a are the web thickness, depth and distance between vertical stiffeners, respectively.

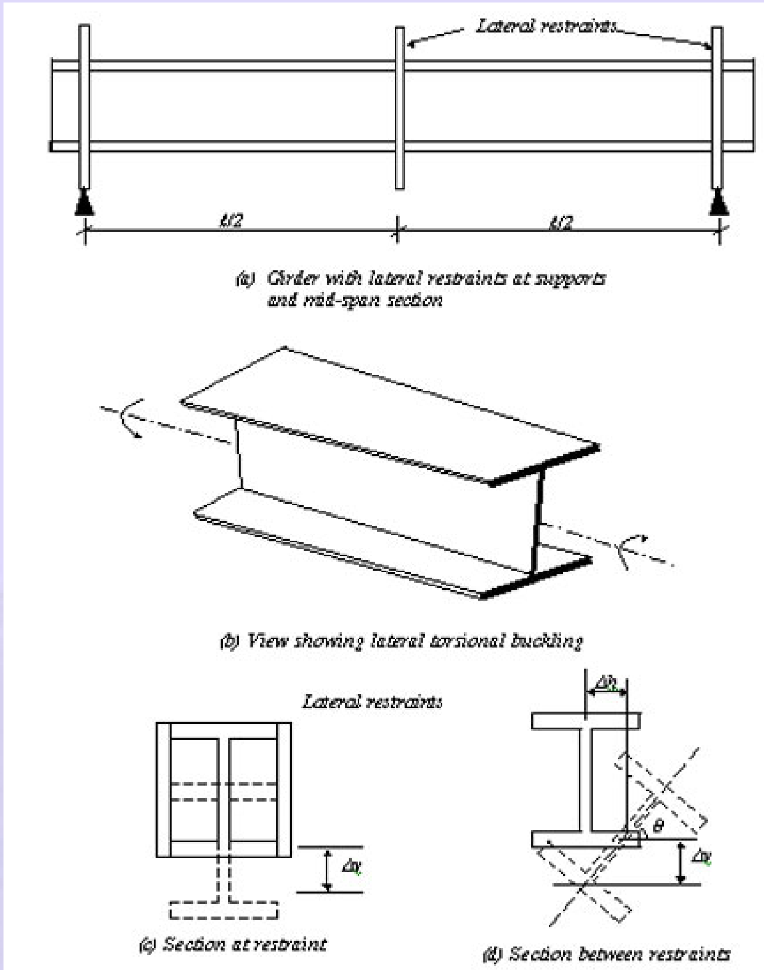


Fig.7.18 Distrosion caused by lateral torsional buckling

The elastic local buckling of the web in shear does not lead to collapse Limit State, since the web experiences stable post-buckling behaviour. In mode (ii), a tension field develops in the panel after shear buckling. In mode (iii) the maximum shear capacity is reached, when pure shear stress in mode (i) and the membrane stress, pt in mode (ii) cause yielding of the panel and plastic hinges in the flanges. This is discussed in detail in the chapters on plate girders.

The membrane tensile stress pt in terms of the assumed angle θ [= tan^{-1}(d/a)] of the tension field with respect to neutral axis (NA) and the first mode shear stress q, is given by,

Thus the resistance to shear in the three-modes put together is given by,

$$\frac{p_t}{q_y} = \left[3 + \left(2.25\,\mathrm{Sin}^2\theta - 3\right)\left(\frac{q_c}{q_y}\right)^2\right]^{1/2} - 1.5\frac{q_c}{q_y}\,\mathrm{Sin}^2\theta \qquad \textbf{\textit{(7.8)}}$$

$$\text{If} \quad m_{fw} \le \frac{1}{4\sqrt{3}}\left(\frac{a}{d}\right)^2 \frac{p_t}{q_y}\,\mathrm{Sin}^2\theta$$

$$\frac{q_u}{q_y} = \left[\frac{q_c}{q_y} + 5.264\,\mathrm{Sin}\theta\left(m_{fw}\frac{p_t}{f_y}\right)^{1/2} + \frac{p_t}{q_y}\left(\mathrm{Cot}\theta - \phi\right)\mathrm{Sin}^2\theta\right] \qquad \textbf{\textit{(7.9)}}$$

$$\text{If} \quad m_{fw} > \frac{1}{4\sqrt{3}}\left(\frac{a}{d}\right)^2 \frac{p_t}{q_y}\,\mathrm{Sin}^2\theta$$

$$\frac{q_u}{q_y} = \left[4\sqrt{3}m_{fw}\left(\frac{d}{a}\right) + \frac{p_t}{2q_y}\mathrm{Sin}^2\theta + \frac{q_c}{q_y}\right]$$

Where, m_{fw} is the non-dimensional representation of plastic moment resistance of the flange, given by

$$m_{fw} = \frac{M_p}{d^2\,t\,f_{yw}}$$

When tension field action is used, careful consideration must be given to the anchorage of the tension field forces created in the end panels by appropriate design of end stiffeners.

Shear-moment Interaction

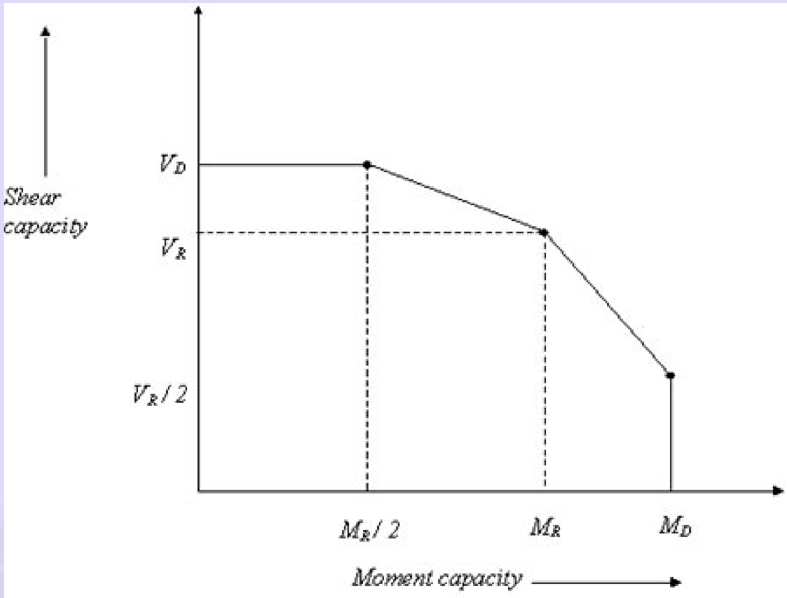


Fig.7.19 Shear-moment capacity interaction diagram

Bending and shear capacities of girders without longitudinal stiffeners can be calculated independently and then an interaction relationship as given in Fig. 7.19 is employed. In Fig. 7.19, M_D and M_R are the bending capacities of the whole section with and without considering contribution of the web, respectively. V_D and V_R are the shear capacities with tension field theory, considering flanges and ignoring the flanges, respectively. However, for girders with longitudinal stiffeners, combined effects of bending and shear is considered by comparing the stresses in the different web panels using the relevant critical buckling strengths of the panel.

Fatigue effect

Under cyclic load, experienced by bridges, flaws in tension zone lead to progressively increasing crack and finally failure, even though stresses are well within the static strength of the material. It may be low cycle fatigue, due to stress ranges beyond yielding or high cycle fatigue, at stresses below the elastic limit. IS: 1024 gives the guide line for evaluating fatigue strength of welded details, that may be used to evaluate the fatigue strength.

Stress concentration may lead to premature cracking near bracing stiffener and shear connector welds. Proper detailing of connections is needed to favourably increase design life of plate girders.

7.6.2 Lateral bracing for plate girders

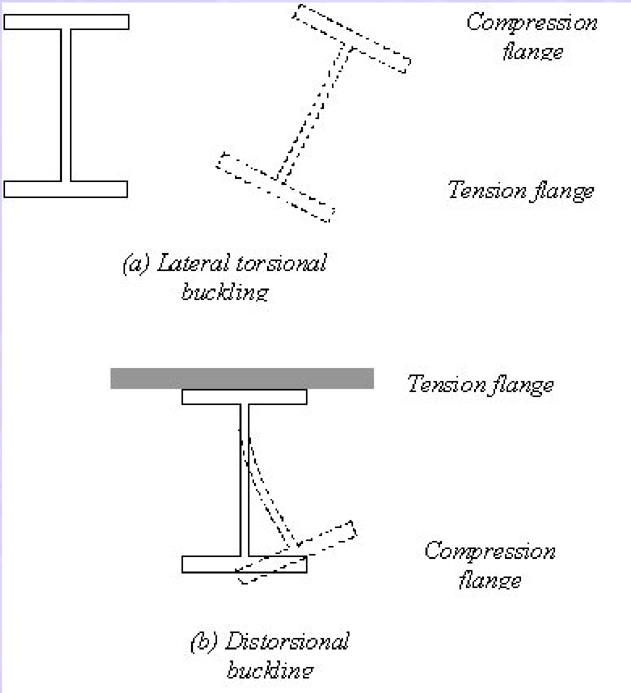


Fig.7.20 Modes of instability of plate girders

Plate girders have a very low torsional stiffness and a very high ratio of major axis to minor axis moment of inertia. Thus, when they bend about major axis, they are very prone to lateral-torsional instability as shown in Fig.7.20 (a). Adequate resistance to such instability has to be provided during construction. In the completed structure, the compression flange is usually stabilised by the deck.

If the unrestrained flange is in compression, distorsional buckling, Fig 7.20(b), is a possible mode of failure and such cases have to be adequately braced. Thus, lateral bracings are a system of cross frames and bracings located in the horizontal plane at the compression flange of the girder, in order to increase lateral stability.

Loads that act transverse on the plate girders also cause the lateral bending and the major contribution is from wind loads. Since plate girders can be very deep, increase in girder depth creates a larger surface area over which wind loads can act. This, in addition to causing lateral bending, contributes to instability of compression flange of the girder. Hence, design of lateral bracing should take account of this effect also.

Triangulated bracing as shown in Fig. 7.15(b) is provided for deck type of plate girder bridges to increase lateral stability of compression flange. But, it can not be adopted for the half-through or through girder bridges because it interferes with functions of the bridge. In these cases, the deck is designed as a horizontal beam providing restraint against translation at its level and the flange far away from the deck is stabilised by U-frame action as shown in Fig. 7.15(a). The degree of lateral restraint provided to the compression flange by U-frame action depends upon the transverse member, the two webs of the main girder (including any associated vertical stiffener) and their connections. In this case, the effective length of a compression flange is usually calculated similar to the theory of beams on elastic foundations, the elastic supports being the U-frames.

Plate grider bridges

7.7 Truss bridges

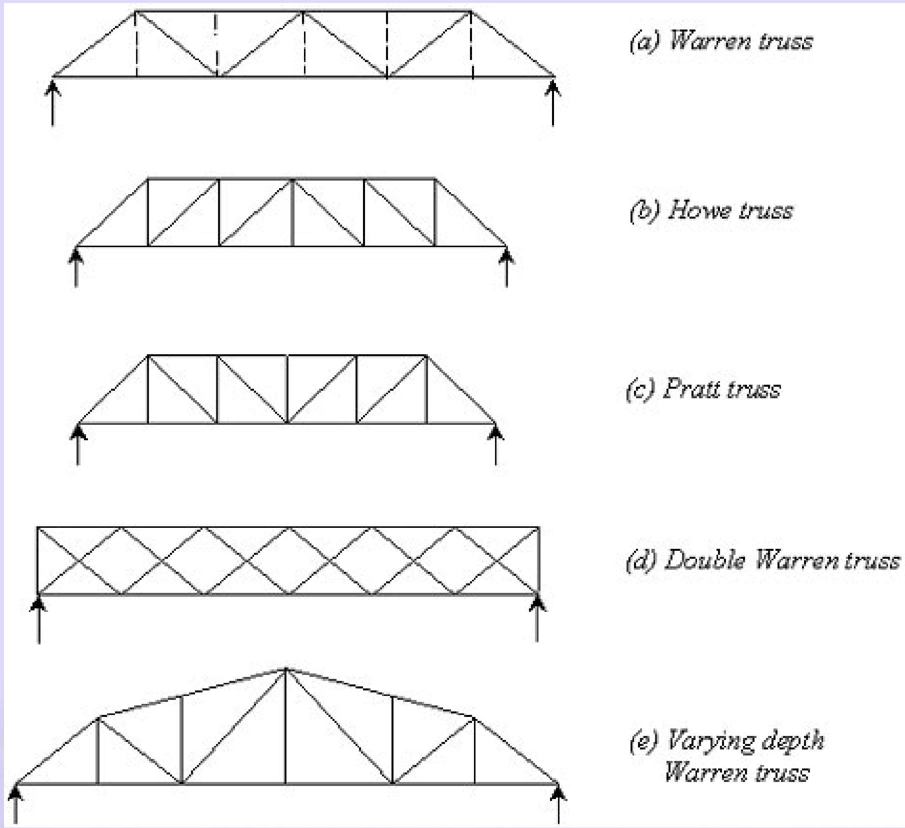


Fig. 7.21 some of the trusses that are used in steel bridges

Truss Girders, lattice girders or open web girders are efficient and economical structural systems, since the members experience essentially axial forces and hence the material is fully utilised. Members of the truss girder bridges can be classified as chord members and web members. Generally, the chord members resist overall bending moment in the form of direct tension and compression and web members carry the shear force in the form of direct tension or compression. Due to their efficiency, truss bridges are built over wide range of spans. Truss bridges compete against plate girders for shorter spans, against box girders for medium spans and cable-stayed bridges for long spans. Some of the most commonly used trusses suitable for both road and rail bridges are illustrated in Fig.7. 21.

For short and medium spans it is economical to use parallel chord trusses such as Warren truss, Pratt truss, Howe truss, etc. to minimise fabrication and erection costs. Especially for shorter spans the warren truss is more economical as it requires less material than either the Pratt or Howe trusses. However, for longer spans, a greater depth is required at the centre and variable depth trusses are adopted for economy. In case of truss bridges that are continuous over many supports, the depth of the truss is usually larger at the supports and smaller at midspan.

As far as configuration of trusses is concerned, an even number of bays should be chosen in Pratt and modified Warren trusses to avoid a central bay with crossed diagonals. The diagonals should be at an angle between 50^o and 60^o to the horizontal. Secondary stresses can be avoided by ensuring that the centroidal axes of all intersecting members meet at a single point, in both vertical and horizontal planes. However, this is not always possible, for example when cross girders are deeper than the bottom chord then bracing members can be attached to only one flange of the chords.

7.7.1 General design principles

7.7.1.1 Optimum depth of truss girder

The optimum value for span to depth ratio depends on the magnitude of the live load that has to be carried. The span to depth ratio of a truss girder bridge producing the greatest economy of material is that which makes the weight of chord members nearly equal to the weight of web members of truss. It will be in the region of 10, being greater for road traffic than for rail traffic. IS:

1915-1961, also prescribes same value for highway and railway bridges. As per bridge rules published by Railway board, the depth should not be greater than three times width between centres of main girders. The spacing between main truss depends upon the railway or road way clearances required.

7.7.1.2 Design of compression chord members

Generally, the effective length for the buckling of compression chord member in the plane of truss is not same as that for buckling out-of-plane of the truss i.e. the member is weak in one plane compared to the other. The ideal compression chord will be one that has a section with radii of gyration such that the slenderness value is same in both planes. In other words, the member is just likely to buckle in plane or out of plane. These members should be kept as short as possible and consideration is given to additional bracing, if economical.

The effective length factors for truss members in compression may be determined by stability analysis. In the absence of detailed analysis one can follow the recommendations given in respective codes. The depth of the member needs to be chosen so that the plate dimensions are reasonable. If they are too thick, the radius of gyration will be smaller than it would be if the same area of steel is used to form a larger member using thinner plates. The plates should be as thin as possible without losing too much area when the effective section is derived and without becoming vulnerable to local buckling.

Common cross sections used for chord members are shown in Fig. 7.22. Trusses with spans up to 100 m often have open section compression chords. In such cases it is desirable to arrange for the vertical posts and struts to enter inside the top chord member, thereby providing a natural diaphragm and also achieving direct connection between member thus minimising or avoiding the need for gussets. However, packing may be needed in this case. For trusses with spans greater than about 100 m, the chords will be usually the box shaped such that the ideal disposition of material to be made from both economic and maintenance view points. For shorter spans, rolled sections or rolled hollow sections may be used. For detailed design of compression chord members the reader is referred to the chapter on Design of axially compressed columns.

7.7.1.3 Design of tension chord members

Tension members should be as compact as possible, but depths have to be large enough to provide adequate space for bolts at the gusset positions and easily attach cross beam. The width out-of-plane of the truss should be the same as that of the verticals and diagonals so that simple lapping gussets can be provided without the need for packing. It should be possible to achieve a net section about 85% of the gross section by careful arrangement of the bolts in the splices. This means that fracture at the net section will not govern for common steel grades.

In this case also, box sections are preferable for ease of maintenance but open sections may well prove cheaper. For detailed design reader is referred to the chapter on Design of Tension members.

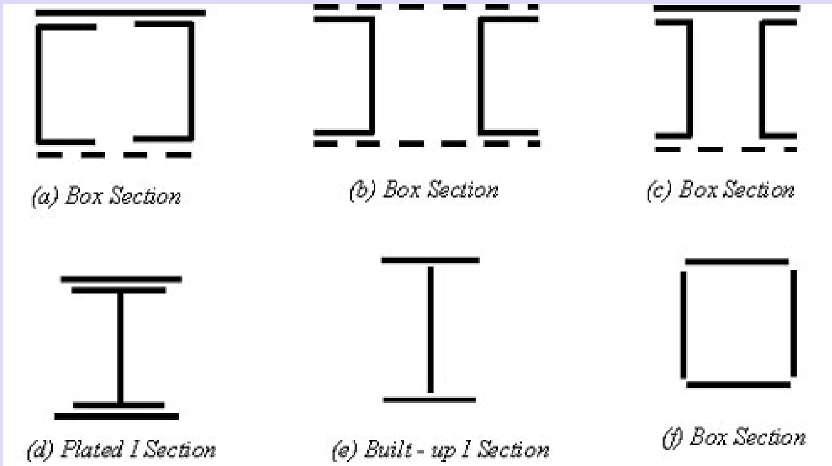


Fig. 7.22 Typical cross-section for truss members

7.7.1.4 Design of vertical and diagonal members

Diagonal and vertical members are often rolled sections, particularly for the lightly loaded members, but packing may be required for making up the rolling margins. This fact can make welded members more economical, particularly on the longer trusses where the packing operation might add significantly to the erection cost.

Aesthetically, it is desirable to keep all diagonals at the same angle, even if the chords are not parallel. This arrangement prevents the truss looking over-complex when viewed from an angle. In practice, however, this is usually overruled by the economies of the deck structure where a constant panel length is to be preferred. Typical cross sections used for members of the truss bridges are shown in Fig. 7.22.

7.7.2 Lateral bracing for truss bridges

Lateral bracing in truss bridges is provided for transmitting the longitudinal live loads and lateral loads to the bearings and also to prevent the compression chords from buckling. This is done by providing stringer bracing, braking girders and chord lateral bracing. In case of highway truss bridges, concrete deck, if provided, also acts as lateral bracing support system.

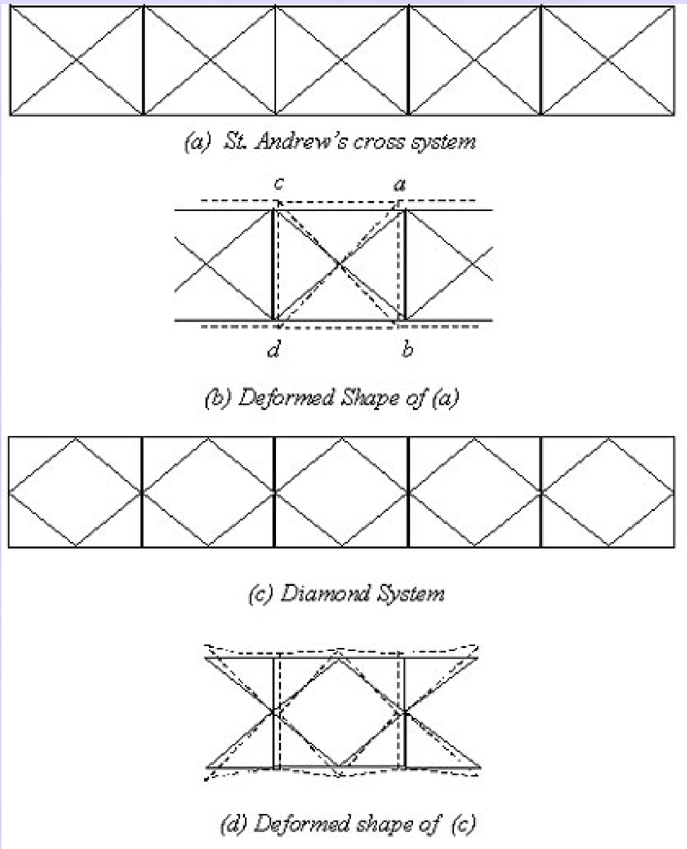


Fig.7.23 Lateral bracing systems

The nodes of the lateral system coincide with the nodes of the main trusses. Due to interaction between them the lateral system may cause as much as 6% of the total axial load in the chords. This should be taken into account.

Fig. 7.23 shows the two lateral systems in its original form and its distorted form after axial compressive loads are applied in the chords due to gravity loads. The rectangular panels deform as indicated by the dotted lines, causing compressive stresses in the diagonals and tensile stresses in the transverse members. The transverse bracing members are indispensable for the good performance of St. Andrew's cross bracing system.

In diamond type of lateral bracing system the nodes of the lateral system occur midway between the nodes of the main trusses [Fig.7. 23(c)]. They also significantly reduce the interaction with main trusses. With this arrangement, "scissors-action" occurs when the chords are stressed, and the chords deflect slightly laterally at the nodes of the lateral system. Hence, diamond system is more efficient than the St. Andrew's cross bracing system.

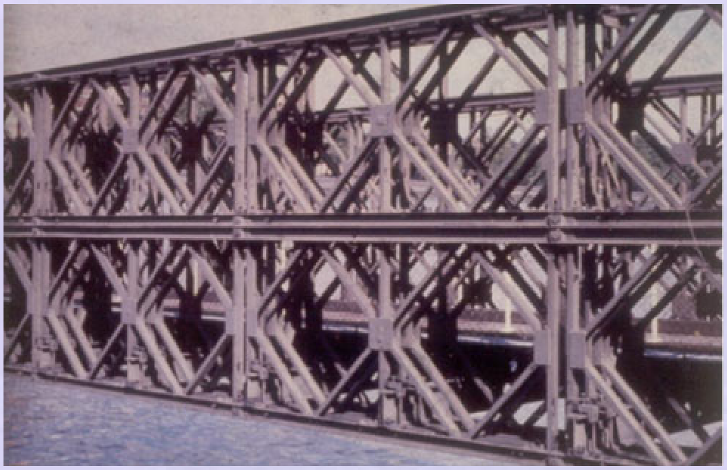
It is assumed that wind loading on diagonals and verticals of the trusses is equally shared between top and bottom lateral bracing systems. The end portals (either diagonals or verticals) will carry the load applied to the top chord down to the bottom chord. In cases, where only one lateral system exists (as in Semi-through trusses), then the single bracing system must carry the entire wind load.

Truss bridges

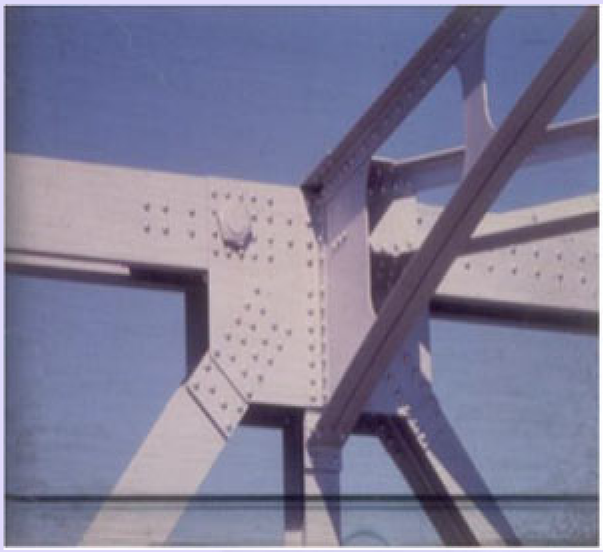
Suspended central span

Continuous span

Sloping chord



Bailey Bridge

Connection gusset

7.8.1 Examples:

Design a through type single lane truss bridge for broad gauge main line loading. The effective span length of the bridge is 50 m. Consider γ_m = 1.15.

(1)Truss arrangement [See Fig. E1]:

Effective Span of truss girder = 50 m.

Assume 10 panels @ 5 m interval.

Height and truss girder:

For economical considerations, height = $\frac{1}{8}$ to $\frac{1}{10}$ of span

Assume, height = 6m. ($\frac{1}{8.33}$ Of span) Hence, O.K.

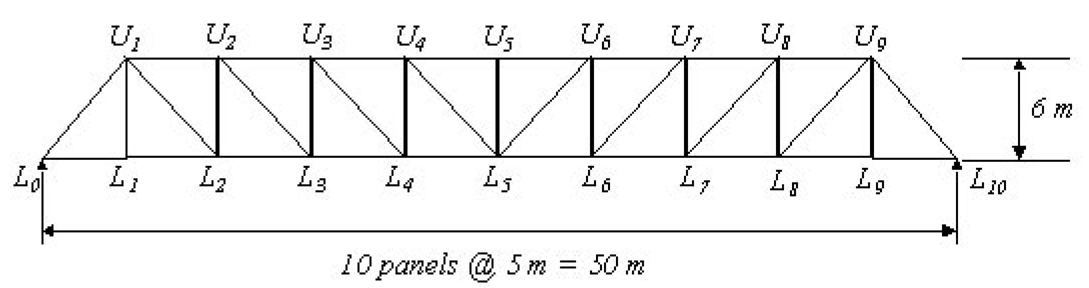


Fig. E1. Truss arrangement

(2) Influence line diagrams:

(i) ILD for L_0U_1 (Diagonal member):

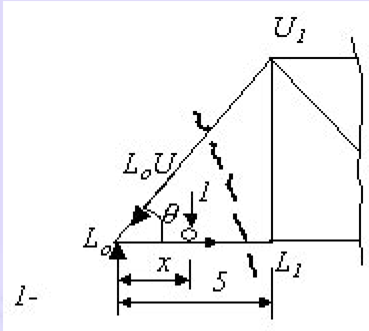


Fig. E2. Free body diagram

(a) If, unit load is in between L_1 and L_{10} (i.e. $5 \le x \le 50$)

$$\sum V = 0$$

$$L_0U_1 \sin\theta = 1 - (x/50) \Rightarrow L_0U_1 = \frac{1}{\text{son}\,\theta}\left(1 - \frac{x}{50}\right)$$

(b) If, unit load is in between L_0 and L_1 $(\text{i.e}\, 0 \le x \le 5)$

$$L_0U_1 = -\frac{1}{\sin\theta}\frac{9x}{50}$$

Then, we can get ILD as shown in Fig. E3.

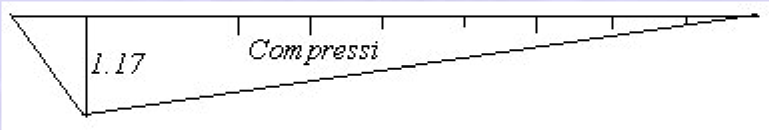


Fig. E3. ILD for *L_0U_1*

(ii) ILD for L_1U_1 (Vertical member): [See free body diagram Fig. E4]

(a) If, unit load is in between L_0 and L_1 (i.e. $5 \le x \le 50$)

$$\sum ML_0 = 0.$$

$$5L_1U_1 = x$$

$$L_1U_1 = x / 5$$

(b) If, unit load is in between L_2 and L_{10}

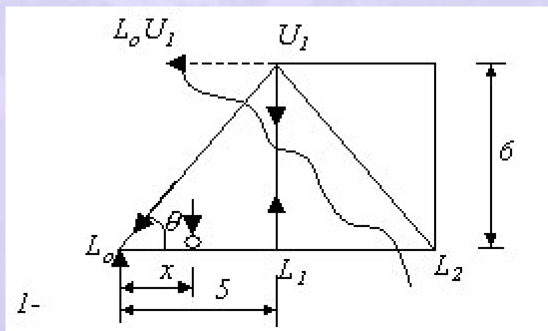


Fig. E4

$L_1U_1 = 0$

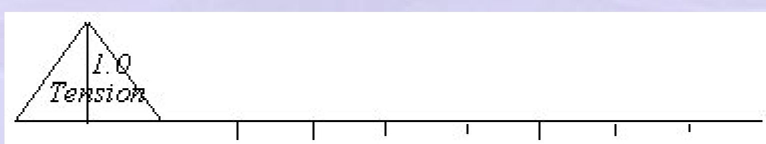


Fig. E5 ILD for L_1U_1

(iii) ILD for U_4U_5 and L_4L_5: (Top and Bottom chord members respectively)

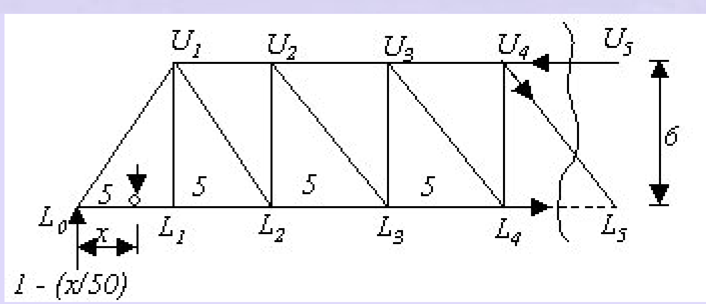


Fig. E6 Free body diagram

(a) If, the unit load is in between L_0 and L_4 (i.e. $0 \le x \le 20$)

$$\sum M_{L5} = 0$$

$$6U_4\,U_5 + (25 - x)*1 = 25*\left[1 - (x/50)\right]$$

$$U_4\,U_5 = \frac{1}{6}\left[\left(25\left(1 - \frac{x}{50}\right) - (25 - x)\right)\right]$$

$$\sum M_{u4} = 0$$

$$6L_4\,L_5 + (20 - x)*1 = 20*\left[1 - (x/50)\right]$$

$$L_4\,L_5 = \frac{1}{6}\left[\left(20\left(1 - \frac{x}{50}\right) - (20 - x)\right)\right]$$

(b) If, unit load is in between L_5 and L_{10} $(\text{i.e } 25 \le x \le 50)$

Then,

$$U_4\,U_5 = \frac{1}{6}\left[25\left(1 - \frac{x}{50}\right)\right]$$

$$L_4\,L_5 = \frac{1}{6}\left[20\left(1 - \frac{x}{50}\right)\right]$$

ILDs for U_4U_5 and L_4L_5 are shown in Fig. E7 and Fig. E8 respectively.

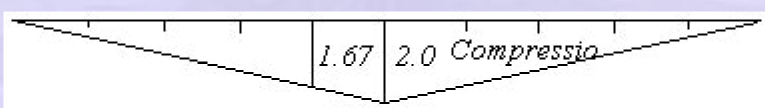


Fig. E7 ILD for U_4U_5

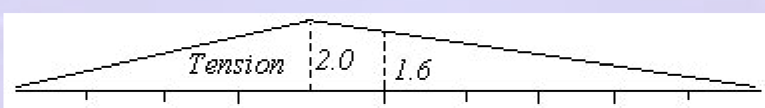


Fig. E8 ILD for L_4L_5

(3) Loads:

(i) Dead load - Dead loads acting on truss girder are as follows:

Weight of rails = 2 x 0.6 = 1.2 kN/m.

* Weight of sleepers = 0.25 x 0.25 x 7.5 / 0.4 = 2.34 kN/m.

Weight of fastenings (assumed) = 0.25 kN/m.

Weight of stringers (assumed) = 3.0 kN/m

Weight of cross girders (assumed) = 5.0 kN/m.

** Self-weight of truss by Fuller's Formula = 13.0 kN/m

Total dead load per track = 24.8 kN/m

Therefore, Total dead load per girder = 24.8 / 2 = 12.4 kN/m

*[Assume 250 mm 250 mm 2m wooden sleepers @ 400 mm apart and weight of 7.5 kN/m^3]

$$** \left[\text{Fuller's Formula} = \frac{15l + 550}{100} = \frac{15 \text{ x } 50 + 550}{100} = 13.0 \, \text{kN/m} \right]$$

(ii) Live load

(a) Areas of Influence line diagrams for truss members discussed:

Area of influence line for $L_0U_1 = \frac{1}{2} \text{x} \, 50 \, \text{x} \, 1.17 = -29.3$ (Compression)

Area of influence line for $L_1U_1 = \frac{1}{2} \text{x} \, 10 \, \text{x} \, 1.0 = +5.0$ (Tensile)

Area of influence line for $U_4U_5 = \frac{1}{2} \text{x} \, 50 \, \text{x} \, 2.08 = -52$ (Compression)

Area of influence line for $L_4L_5 = \frac{1}{2} \text{x} \, 50 \, \text{x} \, 2 = +50$ (Tensile)

(b) Live loads and impact loads from IRS Bridge Rules - 1982:

Live loads and impact factors for each loaded length are found from IRS Bridge Rules - 1982. For maximum forces in chord members, the whole of the span should be loaded and Live load is determined corresponding to maximum B.M. For other diagonal and vertical members, part of the span as indicated by influence line diagrams, should be loaded and the live load is determined corresponding to S.F. The impact factor is found corresponding to loaded length.

For maximum force in members L_4L_5 and U_4U_5:

Load length = 50 m

Live load for B.M. = 3895.2 kN

Impact factor = $0.15+\dfrac{8}{6+l}=0.15+\dfrac{8}{6+50}=0.293$

(LL+ IL) per m per girder = $\dfrac{3895.2\,\text{x}\,(1+0.293)}{2\,\text{x}\,50}=50.36\,\text{kN/m}$

For maximum force in members L_0 U_1 and L_1 U_1:

L_0 U_1

Load length = 50 m

Live load for B.M. = 4184.6 kN

Impact factor = $0.15+\dfrac{8}{6+l}=0.15+\dfrac{8}{6+50}=0.293$

(LL+ IL) per m per girder = $\dfrac{4184.6\,\text{x}\,(1+0.293)}{2\,\text{x}\,50}=54.1\,\text{kN/m}$

L_1 U_1:

Load length = 10 m

Live load for S.F. = 1227.8 kN

Impact factor = $0.15+\dfrac{8}{6+l}=0.15+\dfrac{8}{6+50}=0.293$

(LL+ IL) per m per girder = $\dfrac{1227.8\,\text{x}\left(1+0.65\right)}{2\,\text{x}\,10}=101.3\,\text{kN/m}$

(c) Longitudinal Loads from IRS Bridge Rules - 1982

Assume, there exist rail expansion joints in the bridge and prevent the transfer of longitudinal loads to approaches. It may be noted that for broad gauge bridges upto a loaded length of 44 m, the tractive effort is more than the braking force and for loaded lengths more than 44 m the braking force is more than the tractive effort.

Assume truss under consideration is simply supported by a hinge at L_0 and a roller at L_{10}. The longitudinal force in a member can be tensile or compressive depending on the direction of movement of train.

Panel L_4L_5:

Loaded length = 30 m

Tractive effort = 637.4 kN

Force per chord = 637.4/2 = $\pm 318.7\,\text{kN}$

Unfactored loads:

Member	Area of ILD	Load in kN/m		Forces in members (kN)		
		DL	LL+IL	DL	LL+IL	Long.L
L_0 U_1	- 29.3	12.4	54.1	-363.3	-1585.1	-
L_1 U_1	+5.0	12.4	101.3	+ 62	+ 506.5	-
U_4U_5	- 52.0	12.4	50.36	-644.8	-2618.7	-
L_4L_5	+ 50.0	12.4	50.36	+620	+2518	±318.7

Use following Partial safety factors for the loads:

$\gamma_{DL} = 1.35$; $\gamma_{LL} = 1.50$; $\gamma_{LongL} = 1.50$

Factored loads:

Member	Factored Forces in members (kN)			Total load (kN)	
	DL	LL+IL	Long.L		
$L_0 U_1$	-490.4	- 2377.6	-	- 2868.0	
$L_1 U_1$	+83.7	+ 759.8	-	+ 843.4	
$U_4 U_5$	-870.5	-3928	-	- 4798.5	
$L_4 L_5$	+ 837	+3777	±478	+ 5092	- 478

Note: Negative sign represents compression and positive sign represents tension.

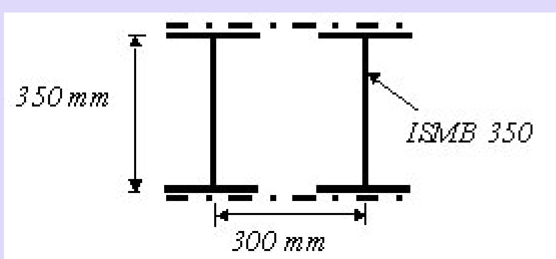
(4) Design for truss members:

(i) Design of diagonal member (L_0U_1): Note that in this illustration of this Member, the portal effect and fatigue are not considered.

Length of the chord, $L_0U_1 = l = 7810$ mm

Assume, effective length, $l_e = 0.7*l = 5467$mm

Try a built up member with two ISHB350 spaced @ 300 mm



A = 18442 mm^2

r_x = 146.5 mm

r_y = 158.8 mm

λ_x = 5467/146.5 = 37.3

Then, σ_c = 221.8 N/mm^2

[See chapter on axially compressed columns using curve c]

Axial capacity = (221.8/1.15)*18442/1000 = 3556.5 kN > 2868 kN

Hence, section is safe against axial compression

(ii) Design of vertical member (L_1U_1):

Maximum tensile force = 843.4 kN

Try ISMB 350 @ 0.524 kN/m shown.

A = 6671 mm^2

Axial tension capacity of the selected section = 6671* 250/1.15

= 1450 kN > 843.4 kN

Hence, section is safe in tension.

[Note: Welded connection assumed]

(iii) Design of top chord member (U_4U_5):

Member length, l = 5000 mm

Assume, effective length = 0.85l = 4250 mm

Try the section shown.

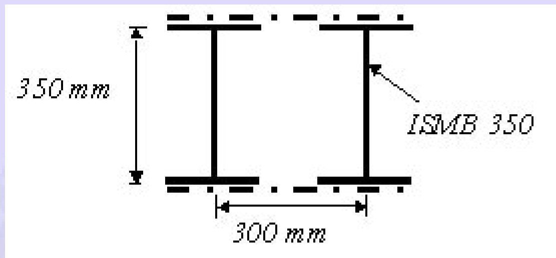
A = 25786 mm^2

r_x= 165.4 mm

r_y = 210 mm

λ_x = 4250/165.4 = 25.7

Then, σ_c = 239 N/mm^2



[See chapter on axially compressed columns using column curve c]

Axial capacity = (239/1.15)*25786/1000 = 5359 kN > 4798.5 kN

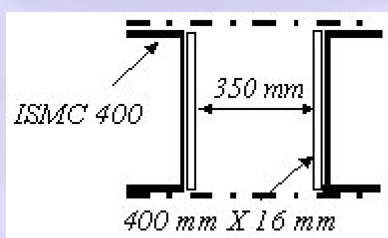
Hence, section is safe against axial compression

(iv) Bottom chord design (L_4L_5):

Maximum compressive force = 478 kN

Maximum tensile force = 5092 kN

Try the box section shown.



A = 25386 mm^2

r_x = 144 mm

r_y = 210 mm

Axial tension capacity of the selected section = 25386* 250/1.15

= 5518 kN > 5092 kN

Hence, section is safe in tension.

Maximum unrestrained length = l = 5000 mm

λ_x = 5000/144 = 34.7

Then, σ_c = 225 N/mm^2

Axial capacity = (225/1.15)* 25386/1000

= 4967 kN > 478 kN

Hence, section is safe against axial compression also.

The example is only an illustration. The following have to be taken into consideration:

- Design of lacings/batten
- Design of connections and effect of bolt holes on member strength
- Secondary bending effects
- Design for fatigue

7.9 Summary

After brief introduction, the steel used in bridges and its properties were discussed. The broad classification of bridges was mentioned and various loads to be considered in designing railway and highway bridges in India were discussed. Finally analysis of girder bridges was discussed using influence line diagrams.

This chapter deals with the design of steel bridges using Limit States approach. Various types of plate girder and truss girder bridges were covered. Basic considerations that are to be taken into account while designing the plate girder bridges are emphasised. Practical considerations in the design of truss members and lateral bracing systems are discussed briefly. A worked example on through type truss girder Railway Bridge is given in the appendix.

7.10 References

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