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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – I – STRUCTURAL CONNECTIONS– SCIA5203

UNIT-I

STRUCTURAL CONNECTIONS

1.1 LIMIT STATE

Acceptable limit for the safety and serviceability requirements before failure occurs is called a Limit state

IS : 800 - 1984

Working stress method

- Factor of safely for yield stress, allowable stresses are less than ‗f_y‘.
- Pure elastic approach for analysis of structures under working loads.
- Yielding or buckling never occurs at working loads
- Deformations are evaluated at

working loads. IS : 800 – 2007

Limit State Method

- Partial safety factor for material ($ç_m$) for yield and ultimate stress.
- Working loads are factored (increased) as per partial safely factor ($ç_f$) causing Limit State of strength.
- Post buckling and post yielding plays important role in estimating capacity of structural elements at Limit State.
- Deformations are evaluated at working loads.

1.2 LIMIT STATES FOR DESIGN PURPOSES

- Ultimate Limit State is related to the maximum design load capacity under extreme conditions. The partial load factors are chosen to reflect the probability of extreme conditions, when loads act alone or in combination.
- Serviceability Limit State is related to the criteria governing normal use. Service loads are used to check the adequacy of the structure.
- Fatigue Limit State is important where distress to the structure by

repeated loading is a possibility.

1.3 CONNECTIONS

Connections form an important part of any structure and are designed more conservatively than members. This is because, connections are more complex than members to analyse, and the discrepancy between analysis and actual behaviour is large. Further, in case of overloading, we prefer the failure confined to an individual member rather than in connections, which could affect many members.

Connections account for more than half the cost of structural steelwork and so their design and detailing are of primary importance for the economy of the structure.The connections provided in steel structures can be classified as 1) riveted 2) bolted and 3) welded connections. Riveted connections were once very popular and are still used in some cases but will gradually be replaced by bolted connections. This is due to the low strength of rivets, higher installation costs and the inherent inefficiency of the connection. Welded connections have the advantage that no holes need to be drilled in the member and consequently have higher efficiencies. However, welding in the field may be difficult, costly, and time consuming. Welded connections are also susceptible to failure by cracking under repeated cyclic loads due to fatigue which may be due to working loads such as trains passing over a bridge (high-cycle fatigue) or earthquakes (low- cycle fatigue). A special type of bolted connection using High Strength Friction Grip (HSFG) bolts has been found to perform better under such conditions than the conventional black bolts used to resist predominantly static loading. Bolted connections are also easy to inspect and replace. The choice of using a particular type of connection is entirely that of the designer and he should take his decision based on a good understanding of the connection behaviour, economy and speed of construction.

Bolts used in steel structures are of three types: 1) Black Bolts, 2) Turned and Fitted Bolts and 3) High Strength Friction Grip (HSFG) Bolts.

1.4 FAILURE OF CONNECTIONS

1.4.1 Connections in shear

The failure of connections with bearing bolts in shear involves either bolt failure or the failure of the connected plates. In the case of HSFG bolts, however, it may simply be a bolted slip between the connected plates. In this section, the failure modes are described along with the codal provisions for design and detailing shear connections.

1.4.2 Bearing bolts

In connections made with bearing type of bolts, the behaviour is linear until i) yielding takes place at the net section of the plate under combined tension and flexure or ii) shearing takes place at the bolt shear plane or iii) failure of bolt takes place in bearing, iv) failure of plate takes place in bearing and v) block shear failure occurs.

1. Shear capacity of Bolts as per IS 800: The design strength of the bolt, Vdsb, as governed shear strength is given by

Vdsb = Vnsb / ɣmb

Vnsb = nominal shear capacity of a bolt, calculated as follows: Vnsb = $fu/\sqrt{\gamma}(n_n A_{nb}+n_s A_{sb})$ where

fu = ultimate tensile strength of a bolt

nn = number of shear planes with threads intercepting the shear plane

ns = number of shear planes without threads intercepting the shear plane Asb = nominal plain shank area of the bolt

Anb = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter at the thread ɣmb =Partial safety factor for bolted connection with bearing type bolts

The underlying assumption behind the design of bolted connections, namely that all bolts carry equal load is not true in some cases as mentioned below.

Long Joints:

When the length of the joint, lj, of a splice or end connection in a compression or tension element containing more than two bolts (i.e. the distance between the first and last rows of bolts in the joint, measured in the direction of the load transfer) exceeds 15d in the direction of load, the nominal shear capacity Vns, shall be reduced by the factor, þij, given by þij = 1.075 – lj / (β00 d) but 0.75 < þij < 1.0

= 1.075 – 0.005(lj / d)

d= nominal diameter of the fastener

It shall be kept in mind that this provision does not apply when the distribution of shear over the length of joint is uniform as in the connection of web of a section to the flanges.

2. Bearing strength of Bolts as per IS 800: The design bearing strength of a bolt on any plate, Vdpb, as governed by bearing is given by

Vdpb = Vnpb / ɣmb

Vnpb = nominal bearing strength of a bolt, calculated as follows:

Vnpb = β.5 kb d t f'u

kb is smaller of 3/3do, $(p/3d_0)$-0.25, fub/fu, 1

e, p = end and pitch distances of the fastener along bearing direction d0 = diameter of the hole

f'u = smaller of fub, fu

fub, fu = ultimate tensile stress of the bolt and the ultimate tensile stress of the plate, respectively

d = nominal diameter of the bolt

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or, if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking

In the direction normal to the slots in slotted holes the bearing resistance of bolts in holes other than standard clearance holes is reduced by multiplying the bearing resistance obtained (i.e., Vnpb), by 0.7 for over size & short slotted holes or 0.5 for long slotted holes.

3. Tensile capacity of Bolts as per IS 800: A bolt subjected to a factored tensile force (Tb) shall satisfy

Tb < Tdb Where, Tdb = Tnb / ɣmb and Tnb is the nominal tensile capacity of the bolt, given as: Tnb = 0.90 fub An < fyb Asb (ɣmb / ɣm0)

fub is the ultimate tensile stress of the bolt, fyb is the yield stress of the bolt, An is the net tensile stress area. For bolts where the tensile stress area is not defined, An is taken as the area at the bottom of the threads and Asb is the shank area of the bolt

4. Bolt Subjected to Combined Shear and Tension : A bolt required to resist both design shear force (Vsd) and design tensile force (Tb) at the same time shall satisfy $(Vsb/Vdb)^2+(Tb/Tdb)^2 \leq 1.0$

Vsb is the factored shear force acting on the bolt and Vdb is the design shear capacity Similarly, Tb is the factored tensile force acting on the bolt and Tdb is the

design tension capacity

1.5 HSFG bolts

HSFG bolts will come into bearing only after slip takes place. Therefore if slip is critical (i.e. if slip cannot be allowed) then one has to calculate the slip resistance, which will govern the design. However, if slip is not critical, and limit state method is used then bearing failure can occur at the Limit State of collapse and needs to be checked. Even in the Limit State method, since HSFG bolts are designed to withstand working loads without slipping, the slip resistance needs to be checked anyway as a Serviceability Limit State.

1. Slip Resistance as per IS: 800: Slip resistance per bolt is given by $V_{sf} < V_{dsf}$ $V_{dsf} = V_{nsf} / \gamma_{mf}$

V_{nsf} = nominal shear capacity of a bolt as governed by slip for friction type connection, and is given as:

$V_{nsf} = \mu_f . n_e . K_h . F_o$

μ_f is the coefficient of friction (slip factor) as specified in Table 4 ($\mu_f < 0.55$), n_e is the number of effective interfaces offering frictional resistance to slip, K_h is equal to 1.0 for fasteners in clearance holes, 0.85 for fasteners in oversized short slotted holes & for fasteners in long slotted holes loaded perpendicular to the slot and 0.7 for fasteners in long slotted holes loaded parallel to the slot γ_{mf} is equal to 1.10 (if slip resistance is designed at service load), 1.25 (if slip resistance is designed at ultimate load), F_o is the minimum bolt tension (proof load) at installation and may be taken as $0.8 A_{sb} . f_o$, where A_{sb} is the shank area of the bolt in tension and f_o is the proof stress (= 0.70 f_{ub}) Average Values of Coefficient of Friction (μ_f)I

Treatment of surface Coefficient of friction (μ_f)Surfaces not treated 0.20
Surfaces blasted with short or grit with any loose rust removed, no pitting0.50
Surfaces blasted with shot or grit and hot-dip galvanized 0.10
Surfaces blasted with shot or grit and spray-metallized with zinc (thickness 50-70Pm) 0.25 Surface s blasted with shot or grit and painted with ethyl zinc silicate coat (thickness 30-60 Pm)0.30
Sand blasted surface, after light rusting 0.52
Surfaces blasted with shot or grit and painted with ethyl zinc silicate coat (thickness 60-80 Pm)0.30
Surfaces blasted with shot or grit and painted with alkali zinc silicate coat 0.30
Surface blasted with shot or grit and spray metallized with aluminium (thickness > 50 Pm)0.50 Clean mill scale 0.33

Sand blasted surface 0.48 Red lead painted surface 0.10 Tension Failure

In a tension or hanger connection, the applied load produces tension in the bolts. If the attached plate is allowed to deform, additional tensile forces called prying forces are developed in the bolts. The prying forces can be kept small by using a

thick plate or by limiting the distance between the bolt and the plate edge. Black bolts and turned and fitted bolts have sufficient ductility which takes care of prying forces simply by an increase in the bolt strain under constant yield stress. Tensile stresses recommended by BS 5950 for grade 4.6 and grade 8.8 bolts are 195 and 450 N/mm2 respectively. However,

HSFG bolts which are pre-tensioned and which have less ductility

are susceptible to failure and so are normally designed to take only 0.9 times their proof load. A friction bolt subjected to a factored tension force (Tf) shall satisfy Tf < Tdf where, Tdf = Tnf /ɣmb

Tnf is the nominal tensile strength of the friction bolt, calculated as follows:

Tnf==0.9fubAnb<fybAsb

ɣm1/ɣm0)

fub is the ultimate tensile stress of the bolt, An is the net tensile stress area as specified in IS: 1367. For bolts where the tensile stress area is not defined, An shall be taken as the area at the root of the threads and Asb is the shank area of the bolt.

Combined Shear and tension failure

In the case of black bolts subjected to combined action of shear and tension the following relation has to be satisfied. $(Vsf/Vdf)^2+(Tf/Tdf)^2 \leq 1.0$

Vsf is the applied factored shear at design load, Vdf is the design shear strength, Tf is the externally applied factored tension at design load and Tdf is the design tension strength Block shear

Failure by block shear occurs when a portion of the member tears out in a combination of tension and shear. The equations given for block shear in the chapter on Tension Members are repeated here. The strength as governed by block shear is the minimum of Tdb = (Avg fy

/(3 ɣm0) +

0.9Atn fu

/ɣm1)or

Tdb = (0.9Avn fu /(3ɣm1) + Atg fy /ɣm0)

Avg, and Avn are the minimum gross and net area respectively in shear along bolt line parallel to external force, respectively

Atg, and Atn are the minimum gross and net area respectively in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force, respectively fu and fy are the ultimate and yield stress of the material, respectively.

1.6 WELDED CONNECTIONS

Welding is the process to unite various pieces of metal by creating a strong metallurgical bond. Bond is achieved by heat or pressure or both. Welding is the most efficient and direct way of connecting the metal pieces. Over many decades, different welding techniques have been developed to join metals.

1.6.1 TYPESOF WELDS

There are four types of welds. They are:

1. Fillet welds
2. Groove welds
3. Slot welds, and
4. Plug welds

Of these welds, fillet is used to a large extent. Groove welds are used to a lesser extent. However, slot and plug welds are rarely used.

1.6.2 DESIGN STRESSES IN WELDS

Shop welds

Fillet welds − The design strength of a fillet weld, fwd, shall be based on its throat area. fwd

= fwn / çmw in which fwn = fu / √ γ

where fu = smaller of the ultimate stress of the weld and

the parent metal çmw = 1.5 = partial safety factor

Butt welds − Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed those permitted in the parent metal.

Site Welds − The design strength in shear and tension for site welds made during erection of structural members shall be calculated as per 10.5.7.1 but using a partial safety factor çmw of 1.5.

Stresses Due to Individual forces :

When subjected to either compressive or tensile or shear force alone, the stress in the weld is given by:

fa or q = P/tt*lw where

fa = calculated normal stress due to axial force in N/mm2 q = shear stress in N/mm2

P = force transmitted (axial force N or the shear force Q) tt = effective throat thickness of weld in mm

lw= effective length of weld in mm Combination

of stresses: Fillet Welds

When subjected to a combination of normal and shear stress, the equivalent stress fe shall satisfy the following

$f\,e = (fa^2+3q^2)^{1/2} < fu/\sqrt{\gamma}\gamma mw$

fe = normal stresses, compression or tension, due to axial force or

bending moment and q = shear stress due to shear force or tension

Combined bearing, bending and shear - Where bearing stress, fbr is combined with bending (tensile or compressive) and shear stresses under the most unfavorable conditions of loading, the equivalent stress fe is obtained from the following formulae:

fe =(

$fb^2+fbr^2+fbfbr+3q^2$

$)^{1/2}$ where fe =

equivalent stress

fb = calculated stress due to bending in N/mm^2

fbr =calculated stress due to bearing in N/mm^2 q = shear stress in N/mm

1.7 ANALYSIS OF A BOLT/WELD GROUP

Bolt/Weld Group Subject to In-plane Loading :

General Method of Analysis − The design force in a bolt/weld in a bolt/weld group or design force per unit length in a bolt/weld group subject to in-plane loading shall be determined in accordance with the following:

a) The connection plates shall be considered to be rigid and to rotate relative to each other about a point known as the instantaneous centre of rotation of the group.

b) In the case of a group subject to a pure couple only the instantaneous centre of rotation coincides with the group centroid. In the case of in-plane shear force applied at the group centroid the instantaneous centre of the rotation is at infinity and the design force is uniformly distributed throughout the group. In all other cases either the results of independent analyses for a pure couple alone and for an in-plane shear force applied at the group centroid shall be superposed, or a recognized method of analysis shall be used.

c) The design force in a bolt or design force per unit length at any point in the group shall be assumed to act at right angles to the radius from that point to the instantaneous centre, and shall be taken as proportional to that radius.

Bolt/Weld group Subject to Out-of-Plane Loading :

General Method of Analysis −The design force of a bolt in bolt group or design force per unit length in the fillet weld group subject to out-of-plane loading shall be determined in accordance with the following:

a) The design force in the bolts per unit length in the fillet weld group resulting from any shear force or axial force shall be considered to be equally shared by all bolts in the group or uniformly distributed over the length of the fillet weld group.

b) The design force resulting from a design bending moment shall be considered to vary linearly with the distance from the relevant centroidal axes.

i) In bearing type of bolt group plates in the compression side of the neutral axis and only bolts in the tension side of the neutral axis may be considered for calculating the neutral axis and second moment of area. ii) In the friction grip bolt group only the bolts shall be considered in the calculation of neutral axis and second moment of area. iii) The fillet weld group shall be considered in isolation from the connected element; for the calculation of centroid and second moment of the weld length.

Alternative Analysis − The design force per unit length in a fillet weld/bolt group may alternatively be determined by considering the fillet weld group as an extension of the connected member and distributing the design forces among the welds of the fillet weld group so as to satisfy equilibrium between the fillet weld group and the elements of the connected member

1.8 BOLTED BEAM CONNECTIONS

Types are

1. Simple connection or flexible connection: In this type of connections no restraint is imposed for rotation.
2. Moment resistant or rigid connection: In this the joint is designed to resist end shear as well as moment

3. Semirigid connection: In this type of rotation of end is partially restrained.

Simple beam connections may be further classified as framed, unstiffened and
stiffened and is shown in fig2

1. Framed connection: when the end shear to be transferred is less, it is possible to connect the beam to main beam or to the column using cleat angles
2. Unstiffened seated connection : when shear force is larger the depth of cleat angle required for framed connection may be more than that can be provided in the available space
3. Stiffened seated connection :If shear force to be transferred in the beam is still large, the seat angle may fail. To strengthen it a stiffener angle may be provided

Fig 1.1 bolted unstiffened seated connection

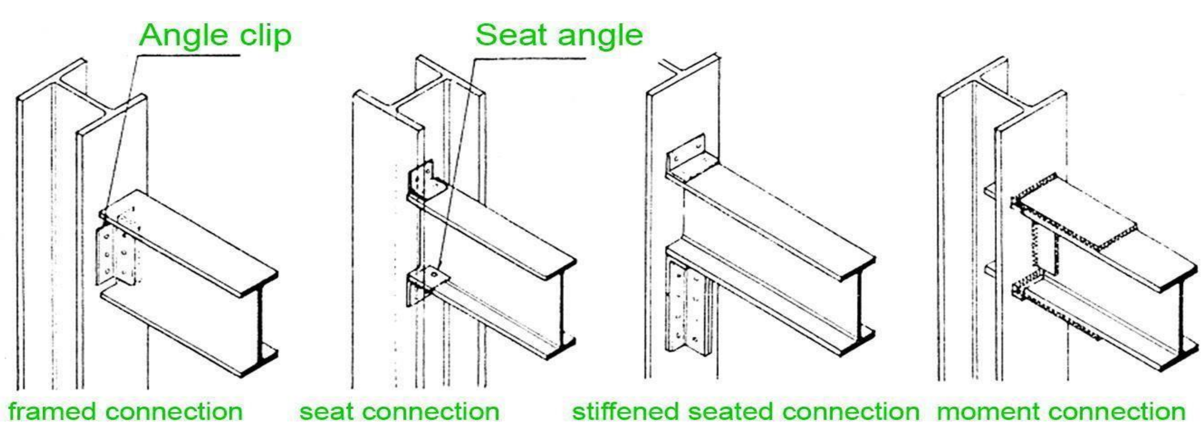

Fig 1.2 bolted connections

1.9 WELDED BEAM CONNECTIONS

Beams may be connected to the supporting beam or to the supporting column by welding.

They are

Framed connections: Two types are double plated framed connection and double angle framed connection

Unstiffened seated connection Stiffened seated connection

Descriptive Questions

1. Explain HSFG bolt with neat sketch.
2. Differentiate between stiffened and stiffened seat connection.
3. Explain the design parameters involved in framed connection.
4. Explain the design procedure of bolted seat connection.
5. Explain the design procedure of welded seat connection.

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UNIT – II – PRE ENGINEERED INDUSTRIAL BUILDING – SCIA5203

UNIT-II

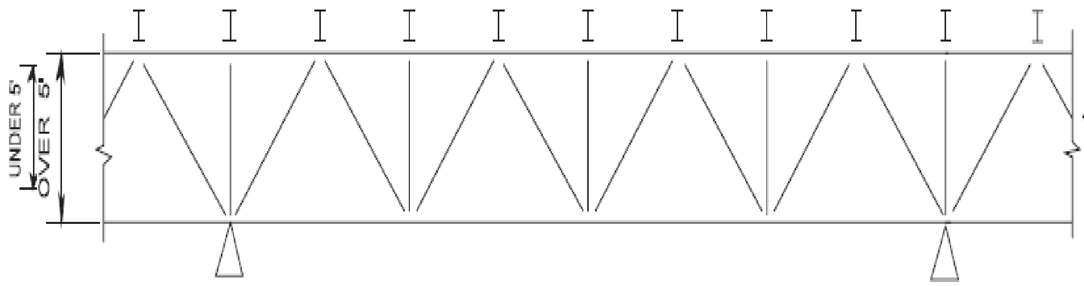
PRE ENGINEERED INDUSTRIAL BUILDING

2.1 INTRODUCTION TO INDUSTRIAL BUILDINGS

Most industrial buildings primarily serve as an enclosure for production and/or storage. The design of industrial buildings may seem logically the province of the structural engineer. It is essential to realize that most industrial buildings involve much more than structural design. The designer may assume an expanded role and may be responsible for site planning, establishing grades, handling surface drainage, parking, on-site traffic, building aesthetics, and, perhaps, landscaping. The design of industrial buildings is governed mainly by functional requirements and the need for economy of construction. In cross section these buildings will range from single or multi-bay structures of large span when intended for use as warehouses or aircraft hangars to smaller span buildings as required for factories, assembly plants, maintenance facilities, packing plants, etc. The main dimensions will nearly always be dictated by the particular operational activities involved, but the structural designer's input on optimum spans and the selection of suitable cross-section profiles can have an important bearing on achieving overall economy. Discussions between the owner or his architect and the engineer at an early stage of planning can help to secure a good balance between operational and structural considerations. An aspect where the structural designer can make a more direct contribution is in the lengthwise dimensions, i.e. the bay lengths of the building. Here a balance must be struck between large bays involving fewer, heavier main components such as columns, trusses, purlins, crane beams, etc, and smaller bays with a larger number of these items at lower unit mass.An important consideration in this regard is the cost of foundations, since a reduction in the number of columns will always result in lower foundation costs.In large industrial buildings with heavy overhead cranes, an economical dimension for the centres of the main columns (i.e. the span of the crane girders) is 15.0 m. Another rule of thumb is to make this dimension equal to the height of the rails above floor level. In the simpler type of building an optimum purlin span can have a bearing on bay length. here crane girders are not required and the structure comprises mainly of columns, trusses (or rafters), purlins and girts, the spacing of roof principals at large intervals will nearly always be more economical. The use of continuous cold-formed purlins – sleeved or lapped at the joints – will permit truss spacing of 9.0 m or more, greatly reducing the total steelwork mass.

2.2 GENERAL DESIGN AND ECONOMIC CONSIDERATIONS

No absolute statements can be made about what truss configuration will provide the most economical solution. For a particular situation, however, the following statements can be made regarding truss design:Span-to-depth ratios of 15 to 20 generally prove to be economical; however, shipping depth limitations should be considered so that shop fabrication can be maximized. The maximum depth for shipping is conservatively 14 ft (4.27m). Greater depths will require the web members to be field bolted, which will increase erection costs.The length between splice points is also limited by shipping lengths. The maximum shippable length varies according to the destination of the trusses, but lengths of 40 ft (12.2m) are generally shippable and 50 ft (15.25) is often possible. Because maximum available mill length is approximately 70 ft (21.35m), the distance between splice points is normally set at a maximum of 70 ft (21.35m). Greater distances between splice points will generally require truss chords to be shop spliced. In general, the rule –**deeper is cheaper**‖ is true; however, the costs of additional lateral bracing for more flexible truss chords must be carefully



examined relative to the cost of larger chords which may require less lateral bracing. The lateral bracing requirements for the top and bottom chords should be considered interactively while selecting chord sizes and types. Particular attention should be paid to loads that produce compression in the bottom chord. In this condition additional chord bracing will most likely be necessary. If possible, select truss depths so that tees can be used for the chords rather than wide flange shapes. Tees can eliminate (or reduce) the need for gusset plates.Higher strength steels (Fy =300 N/mm^2 or more) usually results in more efficient truss members. Illustrated in Figures 1.1 are web arrangements that generally provide economical web systems

Economical truss web arrangement Utilize only a few web angle sizes, and makeuse of efficient long leg angles for greater resistance to buckling. Differences in angle sizes should be recognizable. For instance avoid using an angle 4×3×¼ and an angle 4×3×5/16 in the same truss. HSS, wide flange or pipe sections may prove to be more effective web members at some web locations, especially where subsystems are to be supported by web members. Designs using the Indian standard General construction in Steel – code of practice (IS 800-2007) will often lead to truss savings when heavy long span trusses are required. This is due to the LIMIT STATE approach. The weight of gusset plates, shim plates and bolts can be significant in large trusses. This weight must be considered in the design since it often approaches 10 to 15 percent of the truss weight.Repetition is beneficial and economical. Use as few different truss depths as possible. It is cheaper to vary the chord size as compared to the truss depth.

LOADING CONDITIONS AND LOADING COMBINATIONS

Loading conditions and load combinations for industrial buildings without cranes are well established by building codes. *Dead load:* This load represents the weight of the structure and its components, and is usually expressed in kN/m. In an industrial building, the building use and industrial process usually involve permanent equipment that is supported by the structure. This equipment can sometimes be represented by a uniform load (known as a collateral load), but the points of attachment are usually subjected to concentrated loads that require a separate analysis to account for the localized effects.

Live load: This load represents the force imposed on the structure by the occupancy and use of the building. Building codes give minimum design live loads in kN/m^2, which vary with the classification of occupancy and use. While live loads are expressed as uniform, as a practical matter any occupancy loading is inevitably non-uniform. The degree of non-uniformity that is acceptable is a matter of engineering judgment. Some building codes deal with non-uniformity of loading by specifying concentrated loads in addition to uniform loading for some occupancies. In an industrial building, often the use of the building may require a live load in excess of the code stated minimum. Often this value is specified by the owner or calculated by the engineer. Also, the loading may be in the form of significant concentrated loads as in the case of storage racks or machinery.

Snow loads: Most codes differentiate between roof live and snow loads. Snow loads are a function of local climate, roof slope, roof type, terrain, building internal temperature, and building geometry. These factors may be treated differently by various codes.

Rain loads: These loads are now recognized as a separate loading condition. In the past, rain was accounted for in live load. However, some codes have a more refined standard. Rain loading can be a function of storm intensity, roof slope, and roof drainage. There is also the potential for rain on snow in certain regions.

Wind loads: These are well codified, and are a function of local climate conditions, building height, building geometry and exposure as determined by the surrounding environment and terrain. Typically, they‘re based on a 50-year recurrence interval— maximum three- second gust. Building codes account for increases in local pressure at edges and corners, and often have stricter standards for individual components than for the gross building. Wind can apply both inward and outward forces to various surfaces on the building exterior and can be affected by size of wall openings. Where wind forces produce overturning or net upward forces, there must be an adequate counterbalancing structural dead weight or the structure must be anchored to an adequate foundation.

Earthquake loads: Seismicloads are established by building codes and are based on:

- The degree of seismic risk
- The degree of potential damage
- The possibility of totalcollapse
- The feasibility of meeting a given level of protection

Earthquake loads in building codes are usually equivalent static loads.Seismic loads are generally a function of:

The geographical and geological location of the building

- The use of the building
- The nature of the building structural system
- The dynamic properties of the building
- The dynamic properties of the site
- The weight of the building and the distribution of the weight.

Load combinations are formed by adding the effects of loads from each of the load sources cited above. Codes or industry standards often give specific load combinations that must be satisfied. It is not always necessary to consider all loads at full intensity.

Also, certain loads are not required to be combined at all. For example, wind need not be combined with seismic. In some cases only a portion of a load must be combined with other loads. When a combination does not include loads at full intensity it represents a judgment as to the probability of simultaneous occurrence with regard to time and intensity.

2.3 DESIGN PROCEDURES

In an effort to optimize design time, the following procedural outline has been developed for the designer.

- Determine the best geometrical layout for the building in question.
- Design the crane girders and determine column and frame forces from the crane loadings.
- Perform preliminary design of the crane columns.
- the roof trusses or roof beams for dead loads and live loads.
- Determine all loading conditions for which the entire frame must be analyzed.
- Analyze the frame in question for dead, live, wind and seismic loadings.
- This analysis should be performed without load sharing from the adjacent frames. Also determine the lateral stiffness of the frame.
- Analyze the frame (considering load sharing) for crane loadings.
- Combine moments and forces from the two analyses for subsequent design.
- Perform the final design of columns, trusses, braces and details.

2.4 EXPANSION JOINTS

Although industrial buildings are often constructed of flexible materials, roof and structural expansion joints are required when horizontal dimensions are large.It is not possible to state exact requirements relative to distances between expansion joints because of the many variables involved, such as ambient temperature during construction and the expected temperature range during the life of the buildings.

Expansion joint recommendation as per IS 800-2007:Structures in which marked changes in plan dimensions take place abruptly, shall be provided with expansion joints at the section where such changes occur. Expansion joints shall be so provided that the necessary movement occurs with minimum resistance at the joint. The gap at the expansion joint should be such that: It accommodates the expected expansion contraction due to seasonal and diurnal variation of temperature, and It avoids pounding of adjacent units under earthquake. The structure adjacent to the joint b) It avoids pounding of adjacent units under earthquake. The structure adjacent to the joint should preferably be supported on separate columns but not necessarily on separate foundations.one bay of longitudinal bracing is provided at the centre of the building or building section, the length of the building section may be restricted to 180 m in case of covered buildings and 120 m in case of open gantries (Fig. 2.2).

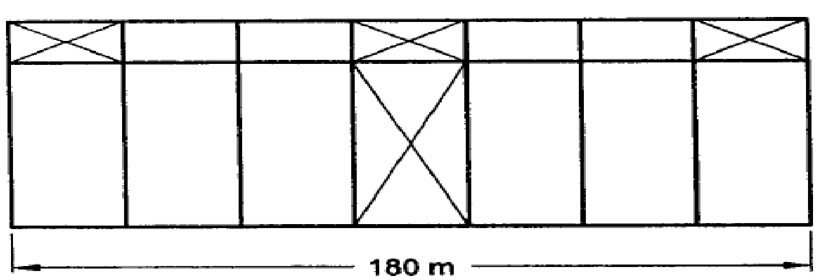


Fig 2.2 Maximum length of building with one bay of bracing.

If more than one bay of longitudinal bracing is provided near the centre of the building /section, the maximum centre line distance between the two lines of bracing may be restricted to 50 m for covered buildings (and 30 m for open gantries) and the maximum distance between the centre of the bracing to the nearest expansion jointiend of building or section may be restricted to 90 m (60 m in case of open gantries).The maximum length of the building section thus may be restricted to 230 m for covered buildings (150 m for open gantries). Beyond this, suitable expansion joints shall be provided (Fig.2.3).

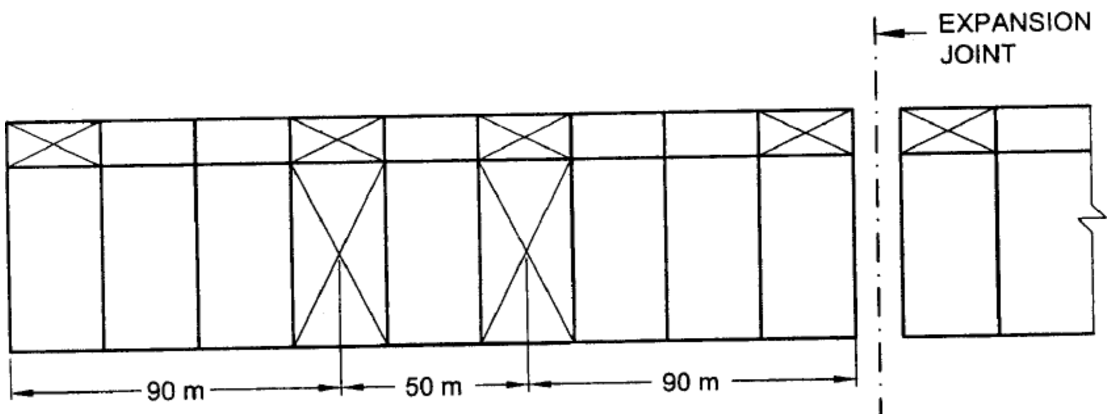


Fig 2.3.Maximum length of building/section with two bays of bracing

Expansion joint along the width of the building. The maximum width of the covered building section should preferably be restricted to 150 m beyond which suitable provisions for the expansion joint may be made. Temperature stress analysis When the provisions of these sections are met for a building or open structure, the stress analysis due to temperature is not required.

Regarding the type of structural expansion joint, most engineers agree that the best method is to use a line of double columns to provide a complete separation at the joints. When joints other than the double column type are employed, low friction sliding elements, such as shown in Figure 4, are generally used. Slip connections may induce some level of inherent restraint to movement due to binding or debris build- up.

2.5 BRACING SYSTEMS

Roof Bracing (Horizontal or wind bracing) Roof bracing is very important in the design of an industrial building.The roof bracing allows the lateral forces induced due to wind and crane to be shared by adjacent bents.

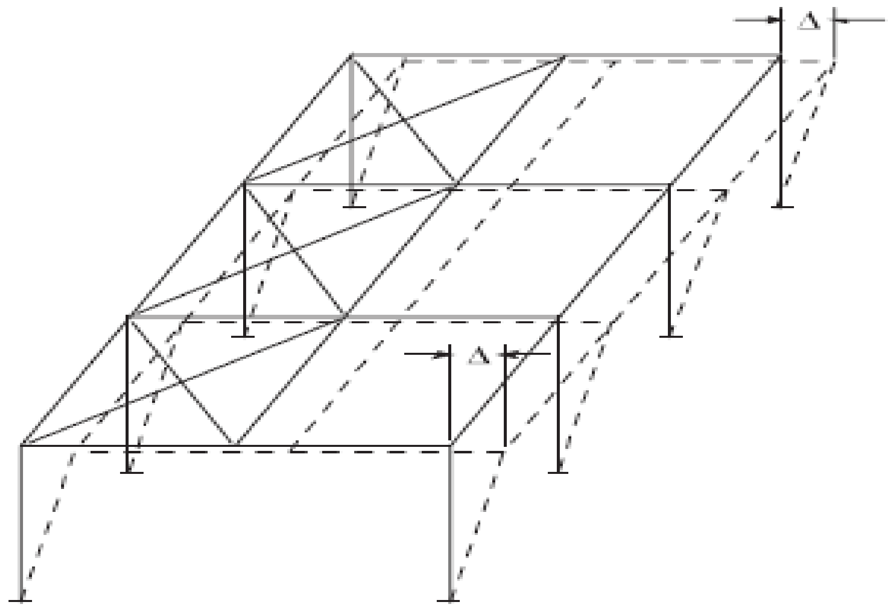


Fig 2.4 Uniform Displacement due to wind

This sharing of lateral load reduces the column moments in the loaded bents. This is true for all framing schemes (in other words, rigid frames of shapes, plates, trusses, or braced frames).It should be noted, however, that in the case of rigid frame structures the moments in the frame cannot be reduced to less than the wind induced moments. Figures 2.4, 2.5 and 2.6 graphically illustrate the concept of using roof bracing to induce sharing of lateral loads in the columns. For wind loading all frames and columns are displaced uniformly as shown in Figure. For a crane building without roof bracing the lateral crane loads are transmitted to one frame line (Figure 2.5) causing significant differential displacement between frames. The addition of roof bracing will create load sharing. Columns adjacent to the loaded frame will share in the load thus reducing differential and overall displacement. (Figure 2.6). Angles or tees will normally provide the required stiffness for this system.

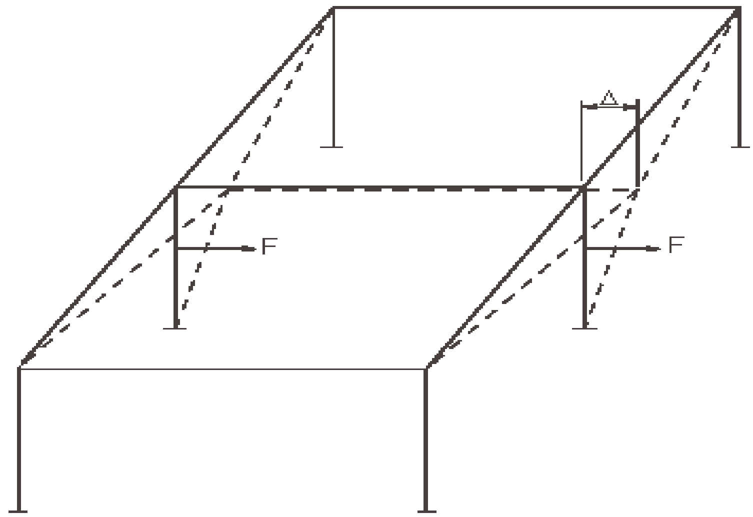


Fig. 2.5 Displacement of Un-braced Frames due to Crane Lateral Load

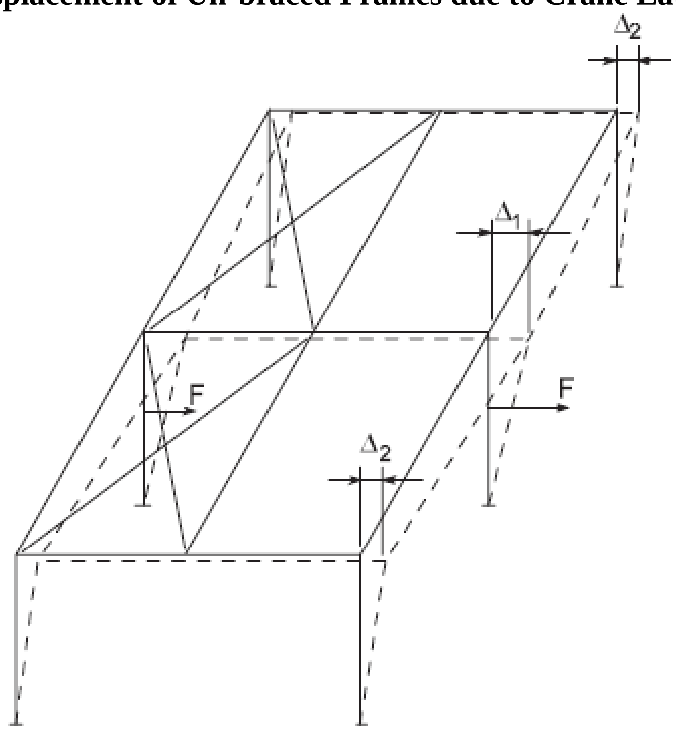


Fig.2.6 Displacement of bracedFrames due to Crane Lateral Load

2.6 WALL BRACING (Vertical bracing)

It is important to trace the longitudinal crane forces through the structure in order to insure proper wall and crane bracing (wall bracing for wind and crane bracing may in fact be the same braces).

For lightly loaded cranes, wind bracing in the plane of the wall may be adequate for resisting longitudinal crane forces. While for very large longitudinal forces, the bracing will most likely be required to be located in the plane of the crane rails. The crane column may tend to twist if the horizontal truss is not provided. Such twisting will induce additional stresses in the column. The designer should calculate the stresses due to the effects of the twisting and add these stresses to the column axial and flexural stresses. A torsional analysis can be made to determine the stresses caused by twist, or as a conservative approximation the stresses can be determined by assuming that the twist is resolved into a force couple in the column flanges as shown in Figure 6.6.The following criteria will normally define the longitudinal crane force transfer: For small longitudinal loads (up to 18kN) use of wind bracing is generally efficient, where columns are designed for the induced eccentric load. For medium longitudinal loads (18kN -36 kN) a horizontal truss is usually required to transfer the force to the plane of X-bracing. For large longitudinal loads (more than 36 kN) bracing in the plane of the longitudinal force is generally the most effective method of bracing. Separate wind X-bracing on braced frames may be required due to eccentricities. Normally the X-bracing schemes resisting these horizontal crane forces are best provided by angles or tees rather than rods. In cases where aisles must remain open, portal type bracing may be required in lieu of designing the column for weak axis bending. It should be noted that portal bracing will necessitate a special design for the horizontal (girder) member, and that the diagonals will take a large percentage of the vertical crane forces. This system should only be used for lightly loaded, low fatigue situations. The following summary of crane girder selection criteria may prove helpful Light cranes and short spans—use a wide flange beam. Medium cranes and moderate spans use a wide flange beam, and if required reinforce the top flange with a channel. Heavy cranes and longer spans—use a plate girder, with a horizontal truss or solid plate atthe top flange.

Limit deflections under crane loads as follows:

2.6 TRUSSES - HOW THEYWORK

The Romans found that if they leant stones against one-another in the shape of an arch, (Fig-1) they could span greater distances than by using the stone as simple lintels or beams. In an arch the stones are in compression. The arch will perform as long as the supports or buttresses at each end of the arch provide restraint, and do not spread apart. Timber beams can also be propped against one-another to form arches. The timber members will be in compression and will also act as simple beams. To turn the arch into a truss all that is required is to provide a tie between the two buttresses to stop them from being pushed apart by the arch. The arch, beam, tie combinations is self- supporting – we call this structure a truss.

Type of Building	Deflection	Design Load	Member	Supporting	Maximum Deflection
(1)	(2)	(3)	(4)	(5)	(6)
Industrial Buildings		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50 t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50 t)	Gantry	Crane	Span/1 000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm
		Crane+ wind	Column/frame	Gantry (Elastic cladding; pendent operated)	Height/200
				Gantry (Brittle cladding; cab operated)	Height/400

2.7 SELECTION OF ROOF TRUSSES

Architectural style, types of roofing material, methods of support of column framing, and relative economy are the principal factors influencing a choice among the three basic types of trusses bowstring, pitched, and flat.In addition, side- and end-wall height and type, roof shape, and bracing requirements must be considered. Other factors being equal, economy is the prime consideration. Economy is dependent upon efficiency in use of material relative to truss type and proportions and to fabrication labour. Theoretically, the three basic types in order of relative efficiency are bowstring, pitched, and flat. The function of a truss is to transfer load from point of application to the supports as directly as possible. Thus for a concentrated load at the centre line of a span, a simple –A‖ frame is the most efficient. Like-wise, if only two equal and symmetrically placed concentrated loads are involved, a truss similar to the queen-post type is the most efficient. In both trusses, the load is transferred to the support directly through the sloping top- chord members without the need for web members. Economic factors. The maximum economical span of any given type of truss will vary with The material available Loading conditions Spacing Type of truss Ratio of labour to material cost &Fabrication methods.

TRUSS PROPORTIONS It is desirable to use as few truss panels as the use of reasonable member sizes will allow. This practice will mean fewer members to handle, fewer joints to fabricate and assemble, and theoretically improved performance. The number of panels usually should be determined by slenderness ratio of top-chord sizes rather than by any fixed formula. Depth-to-span ratio. Certain ratios of effective depth to span are recommended as being satisfactory on the basis of experience. The larger the span, the more desirable it is to use deeper trusses. Although trusses of less depth than these may be acceptable, special attention should then be given to the possibility of greater deflection and secondary stresses. Deflection in trusses of less- than-average depth may be held to a minimum by the following practices: Conservative design the use of low or intermediate grades of material the use of a minimum number of chord splices (by employing the longest available lengths)the use of fastenings with the smallest deformation &the use of as few panels as possible. Stiffer members are also obtained, and therefore less deflection for a given load. It is recommended that the top chord of a bowstring truss be fabricated with a radius about equal to the span. The suggested effective depth-to-span ratio is between 1:6 and 1:8. A radius equal to the span will give a ratio slightly larger than the suggested minimum. For pitched trusses, an effective depth- to-span ratio between 1:5 and 1:6 is recommended, and a minimum of not less than 1:7 unless special consideration is given to deflection. Much deeper trusses may be used for the sake of appearance, such as for the steeply pitched roofs popular in churches. For flat trusses, a minimum depth-to-span ratio between 1:8 and 1:10 is recommended, the deeper trusses being preferred for the longer spans. Roofs should have a minimum slope of ¼ in. per ft for proper drainage, although steeper slopes are often desirable. Flat roofs with no slope for drainage are not recommended unless provision is made in the design for possible accumulation of water due to a stopped drain or natural deflection. Drains on flat roofs should be located at the low points. These are at the center of the span if the truss is built flat. In longer spans, secondary deflection stresses are probably more important. As these stresses are not capable of exact computation, the larger depth-to-span ratios should be used for trusses employing such spans. Deflection of free-span trusses is usually well within acceptable limits, even that for plaster, but care should be taken to see that the natural deflection does not interfere with auxiliary framing. Suspended ceilings are often desirable. Ample clearance should be provided between trusses and so- called non healing partitions or plate glass windows. Provision should also be made for adjustment in the level of the hinges if there is a possibility that deflection may interfere with the proper operation of truss - suspended doors or machinery.

Number of panels. It is desirable to use as few truss panels as the use of reasonable member sizes will allow. This practice will mean fewer members to handle, fewer joints to fabricate and assemble, and theoretically improved performance. The number of panels usually should be determined by slenderness ratio of top-chord sizes rather than by any fixed formula.

2.8 ROOF CONSTRUCTION SYSTEMS

Provision should also be made for adjustment in the level of the hinges if there is a possibility that deflection may interfere with the proper operation of truss - suspended doors or machinery. Only two basic systems of roof construction need be considered in truss design. One applies roof loads to the truss only at the panel points; the other applies UDL over the top. The former system produces only direct stress in the chord member; the latter introduces bending as well as direct stress.

2.9 ROOF- TRUSS SPACING

There are no fixed rules for spacing trusses in buildings. Spacing may be affected byroof framing wall construction size of material available loading conditions &the column spacing desired for material handling or traffic. In general, greater the bay spacing, lesser be the economical. Spacing limits are set by the purlin or girt sizes available for framing between trusses.

2.10 ROOF- TRUSS BRACING AND ANCHORAGE

Bracing and anchorage is necessary to hold trusses and truss members in proper position so that they can resist vertical loads as well as lateral loads such as wind, impact, or earthquake. Although roof framing will usually serve as lateral bracing for the top-chord members, it is important that adequate lateral supports be provided for the bottom- chord members (Fig 7), and also that consideration be given to the possible need for vertical- sway bracing between top and bottom chords of adjacent trusses (Fig.8).Horizontal cross-bracing is sometimes required in the plane of either the top and bottom chord, particularly in long buildings in which the diaphragm action of the roof framing is not adequate for end-wall forces, or in which side-wall loads are resisted by end walls or truss and its support are not designed as a bent to resist the lateral load. Trusses must be securely anchored to properly designed walls or columns and columns in turn anchored to foundations. Unless some other provision made for lateral loads on the side walls and on the vertical projection of the roof-such as for diaphragm action in walls and roof sheathing- lateral resistance should be provided in the column members by means of knee braces or fixity at the column base. The bracing should be designed and detailed with the same care as the truss itself and not left to the judgment of the contractor. The bracing requirements here suggested are minimums, and are not dependent on actual lateral- load analysis or on local code requirements. Vertical cross- bracing should be installed at bottom chord at the location of the vertical bracing and be continuous from end.

Design conditions vertical sway bracing is to be used in end section as a minimum, possibly two sections at each end and near mid span for long buildings. Column-and-wall bracing should be used where possible; it may consist of diagonal sheathing with studs or girt or cross-bracing.

2.11 STEPS INVOLVED IN THE ANALYSIS AND DESIGN OF ROOF TRUSS

2.11.1 Load calculations and Load combinations

Normally trusses are analyzed for Dead load, Live load, Wind load, Snow load, Seismic load and for different load combinations. All trusses in a roof structure are designed for the worst possible combination of dead, live and wind loads. The individual truss members are designed to restrain the corresponding forces i.e.., tension or compression, or a combination of bending with either the tension or compression force. Analysis. The truss was analyzed using STAAD pro software as plane frame and care is taken to do the proper analysis for Member release and Member truss options. All the loads are calculated and applied as UDL. Design can be done either by manual or using software.

ROOF TRUSS – LOAD CALCULATION DATA

Plan area (a/c sheet) =23.32 × 11.37 m

Span of truss =11.42 m c/c

Spacing of truss = 4.632 m

DEAD LOAD:Length of sloping roof =Weight of A/C sheet = 0.17×4.632 = 0.79kN/m No of purlins on each slope = 6 Nos.

Total weight of purlins per slope = 6 × 0.127 × 4.632= 3.53kN

Weight of purlins distributedas UDL on rafter = .53/6.08=0.58 kN/m Total D.L (excluding wt of truss) =0.79 + 0.58= 1.37 kN/m Say =1.40 kN/m

LIVE LOAD (Ref IS: 875 – Part II)

Slope of the Truss

Live Load = 0.75 – 0.02(20.2 – 10)= 0.55 kN/m

Actual Live Load =2/3 × 0.55 = 0.37 kN/m (For truss & Columns except purlin)

Live load per m run on Rafter = 0.37 × 4.632= 1.71 kN/mSay =1.80 kN/m

WIND LOAD (Ref IS: 875 – PartIII)

Design Wind Speed =Vz = K1×K2 ×K3. Vb

Where, Vb = Basic Wind Speed=50 m/s (For Chidambaram)

K1 = Risk Coefficient = 1.00

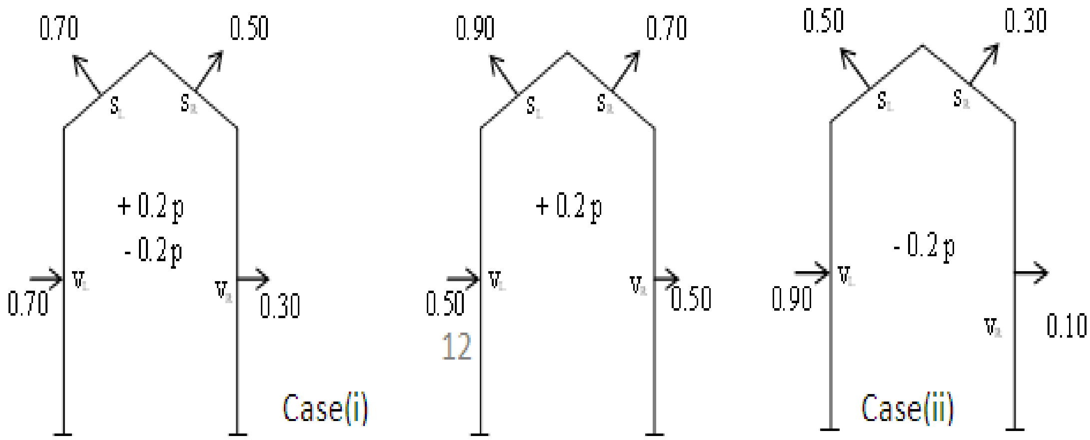
K2 = Terrain Factor = 0.98(Assumed for category 2 & class B)

(Total Height of Building = 9.70m) (Length of Building = 11.37m)

K3 = Topography Factor = 1.00(Plain Ground) Design Wind Speed Vz = 1×1.0×0.98×50 = 49 m/s

= 0.60 Vz = 0.6 ×49 = 1441N/m = 1.44kN/m Say = 1.50 kN/m

Location of building =Purlins (assuming ISMC 125) = 12.7 kg/m= 0.127 kN/m



Design Wind Pressure

DETERMINATION OF WIND COEFFICIENTS:

PITCHED ROOF (REF TABLE 5/IS 875 – P16) h = 9.70m

w =11.37 m

h/w = 9.70/11.37 = 0.85(This is the case of ½< h/w<3/2)

Referring to Table 5 / IS: 875 – P16,External coefficients are found and marked in the sketches shown on fig.

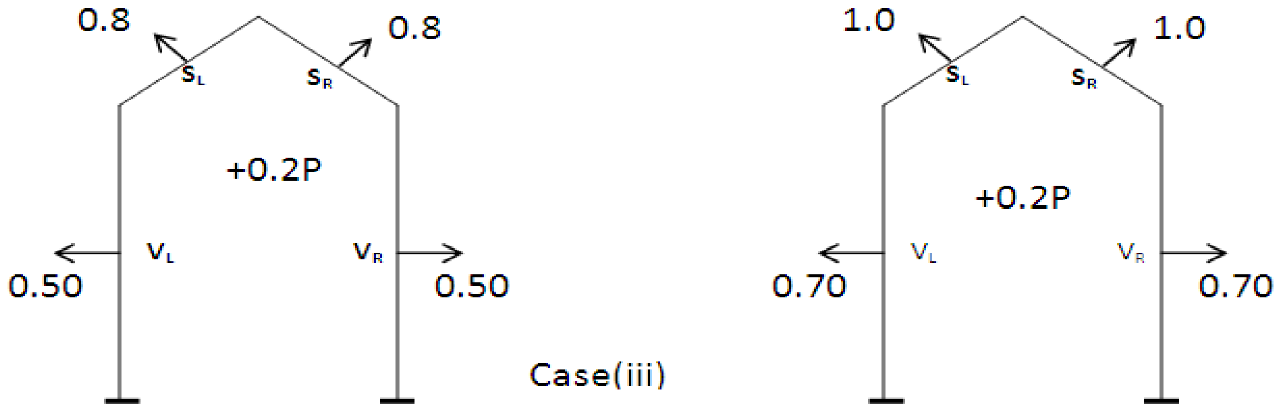
WIND LOAD ON COLUMNS (REF TABLE, 4/IS 875 – P14)CONDITION (i)h/w = 9.70/11.37 = 0.85 (i.e., ½<h/w<3/2)CONDITION (ii)l = 30.5 m w = 24.4 ml/w = 23.32 / 11.37 = 2.05 (i.e.,3/2 <
h/w < 4)

External coefficients are found and marked in the sketches shown on fig.AN1,INTERNAL PRESSURE COEFFICIENT:(REF P27 / IS – 875 Cl: 6.2.3)

Assume low permeability Cpi = ± 0.2

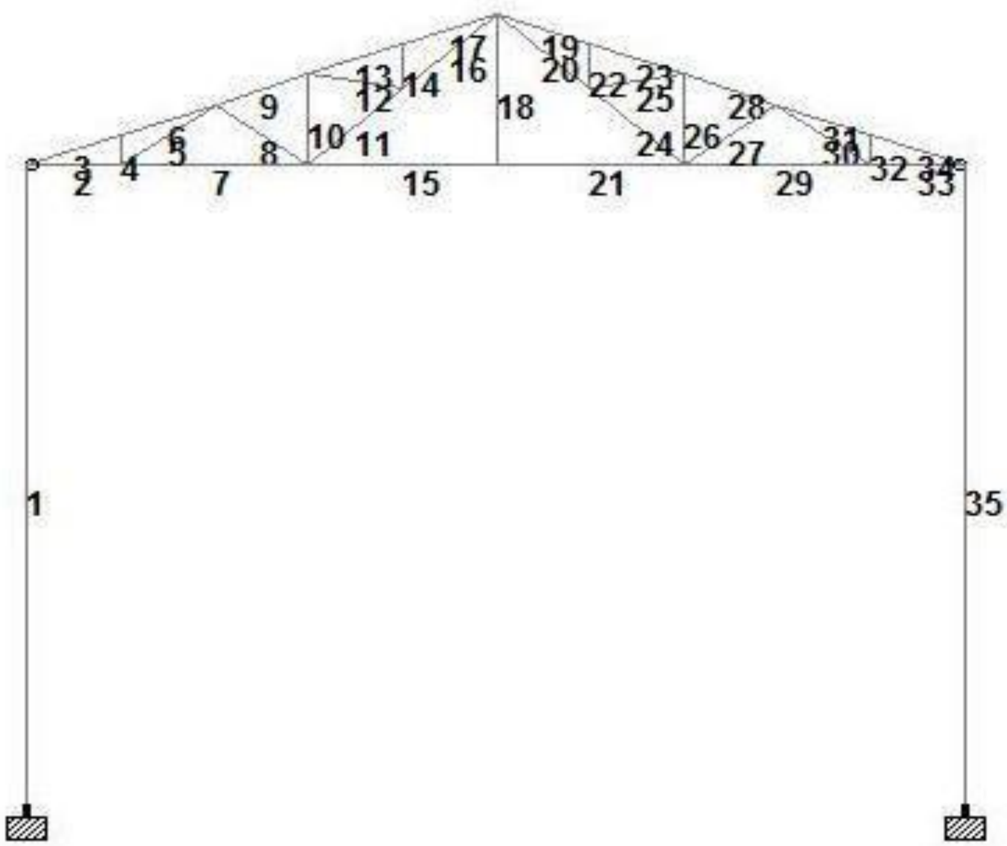
WIND COEFFICIENTS

Wind angle 0º/-



Case(iii)

Fig .AN1



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2.12 COLUMN BASES

Column bases transmit the column load to concrete or masonry foundation blocks. Assist in reducing the intensity of loading and distributing it over the foundations. Safety of a column depends upon the stability of the foundations and consequently on the bases in the case of steel columns.

2.12.1 TYPES OF COLUMN BASES

1. Slab Base
2. Gusseted Base
3. Moment Resisting Base
4. Pocket bases

Slab Base:

Used in columns carrying small load. Column is directly connected to the base plate through cleat angles.The load is transferred to the base plate through bearing.

Gusseted Base:

For columns carrying heavy loads gusseted bases are used. The column is connected to base plate through gussets. The load is transferred to the base plate partly through bearing and partly through gussets.

Moment Resisting Base

Transmit forces and moments

Forces can be axial and shear

Moment can be uniaxial or biaxial

Two cases

Compression over the whole base

Compression over part of base and tension in holding own bolts

Pocket Bases

Column is grouted into a pocket in the concrete foundation. Erection is difficult as the column is not stable before the concrete hardens.

Loads: Compression over the whole or part of the base.

Tension in the holding down bolts.

Horizontal loads are restricted by shear in the weld between column and base plates & friction and bond between the base and the concrete. Suggested base plate details.

Methods for Column Erection

Levelling Plates (for small to medium sized baseplates, say 560mm)Leveling Nuts Pre-set base Plates Grout Thickness 20 to 40mmSpace between bottom of steel base plate and top of concrete – min 40mmNon-shrink grout Anchor bolt dislocation Small dislocation – tolerated Tilted bolts can be straightened using rod bending device Upto 20mm tilt, the concrete may be chipped to a certain depth and the bolt straightened Beyond 20mm base plate can be slotted

Design of Moment Resisting Bases

Case 1: Compression over the whole base plateIf Moment/axial load< L/6, then positive pressure exists over the whole base. (L – length of base plate)P_{max} = (P/A) + (M_Z/Z_z) +

(M_y/Z_y) ≤

0.45 f_{ck}A = L x B; $Z_z = BL^2/6$; $Z_y = LB^2/$

Design of Moment Resisting Bases

Case 2: Compression over part of the base plate and Tension in holding down

Elastic Method: Linear variation of pressure from zero at NA to p_{max} at one edge of base plate is assumed

Plastic Method: Pressure assumed to be constant from NA to one edge of base plate

Effect of Shear Force :

Friction between end plate and the concrete is sufficient to transmit the shear force from the column to the foundation Friction co-efficient of 0.3 is adopted Base plates subject to heavy bending moments – Advantages

Base plate is very stiff and may be assumed as fixed

Tension in anchor bolt is more uniform due to stiffness of the collar

Total elongation of anchor bolt before rupture is large as their free length is large

Tightening of nuts is easy

Pocket Bases- Design Assumptions Axial load is resisted by direct bearing and bond between steel and concrete Moment is resisted by compression forces in the concrete acting on the flanges of the column. Upper part (50mm) of concrete foundation cannot support any pressure due to local cracks and can be ignored Pocket Bases- Design Assumption. Due to flexibility of flanges, the effective width of flange is limited to 20 times the thickness Length of column embedded in concrete should be atleast 1.5 times the depth of the column section

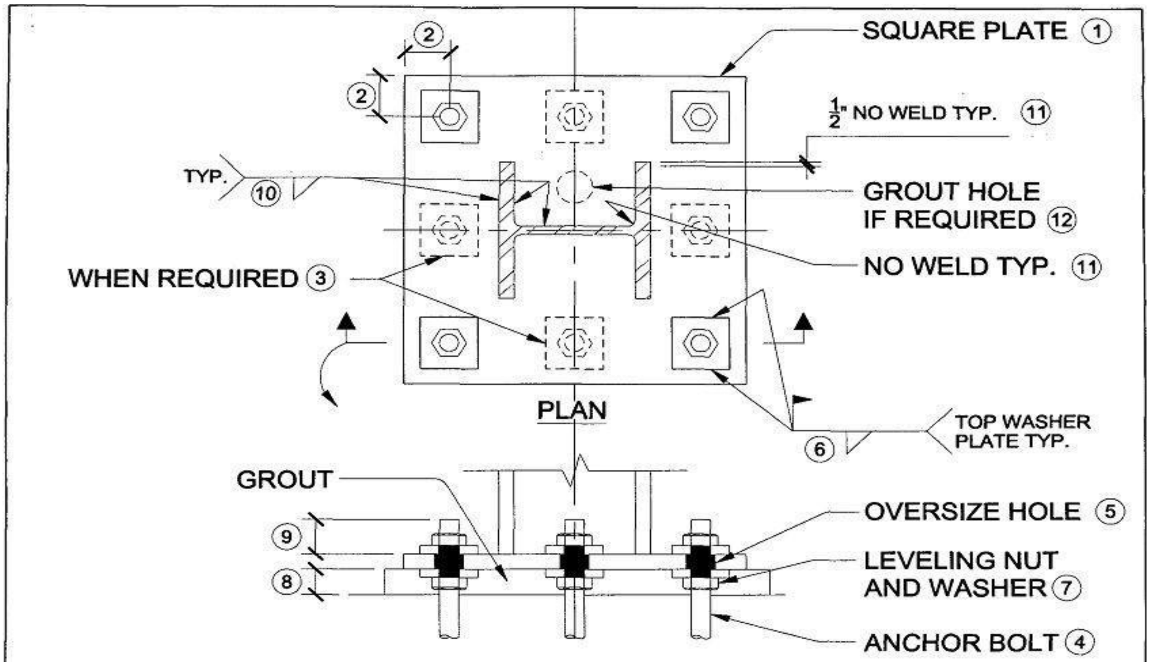

Fig.2.8 Column Base

Descriptive Questions

1. Design a channel purlin for a roof shed made of fan type truss with slope 30º. The span of truss is 9 m and spaced at 4 m. The site is located at Chennai and the wind pressure on truss is found to be 756 N/m2. Use Fe410 steel.

2. Analyze the point loadsacting on fan roof truss having 30º slope for an industrial shed of area 24 m x 9 m. The columns are spaced at 4 m intervals. The height of eaves is 6 m. The shed is to be located at Kancheepuram. The site is surrounded with closely spaced buildings of height less than 10 m and length less than 25 m. Use Fe410 steel.

3. Assess the following for a gantry girder of span 8 m supporting an EOT crane. It is subjected to maximum static wheel load and maximum shear force of 240 kN and 420 kN respectively. The wheel base is 3.5 m.Use Fe410 steel.
   a. Shear capacity of web
   b. Deflection
   c. Strength of weld.

4. Evaluate the elastic moment capacity of a gantry girder made up of ISWB 600 and ISMC 300. Use Fe410 steel.

5. Determine the plastic moment capacity of a gantry girder made up of ISWB 500 and ISMC 300. Use Fe410 steel.

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DEPARTMENT OF CIVIL ENGINEERING

UNIT – III – PLASTIC ANALYSIS OF STRUCTURES – SCIA5203

UNIT III

PLASTIC ANALYSIS OF STRUCTURES

3.1 INTRODUCTION

The elastic design method, also termed as *allowable stress method (or Working stress method),* is a conventional method of design based on the elastic properties of steel. This method of design limits the structural usefulness of the material upto a certain allowable stress, which is well below the elastic limit. The stresses due to working loads do not exceed the specified allowable stresses, which are obtained by applying an adequate factor of safety to the yield stress of steel. The elastic design does not take into account the strength of the material beyond the elastic stress. Therefore the structure designed according to this method will be heavier than that designed by plastic methods, but in many cases, elastic design will also require less stability bracing.

In the method of plastic design of a structure, the ultimate load rather than the yield stress 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 also known as *method of load factor design* or *ultimate load design*. The strength of steel beyond the yield stress is fully utilised in this method. This method is rapid and provides a rational approach for the analysis of the structure. This method also provides striking economy as regards the weight of steel since the sections designed by this method are smaller in size than those designed by the method of elastic design. Plastic design method has its main application in the analysis and design of statically indeterminate framed structures.

3.2 PLASTIC THEORY

3.2.1 Ductility of Steel

Structural steel is characterised by its capacity to withstand considerable deformation beyond first yield, without fracture. During the process of 'yielding' the steel deforms under a constant and uniform stress known as 'yield stress'. This property of steel, known as *ductility*, is utilised in plastic design methods.

Fig. 3.1 shows the idealised stress-strain relationship for structural mild steel when it is subjected to direct tension. Elastic straining of the material is represented by line *OA*. *AB* represents yielding of the material when the stress remains constant, and is equal to the yield stress, f_y. The strain occurring in the material during yielding remains after the load has been removed and is called the plastic strain and this strain is at least ten times as large as the elastic strain, $_{y}$ at yield point.

When subjected to compression, the stress-strain characteristics of various grades of structural steel are largely similar to Fig. 1 and display the same property of yield. The major difference is in the strain hardening range where there is no drop in stress after a peak value. This characteristic is known as ductility of steel.

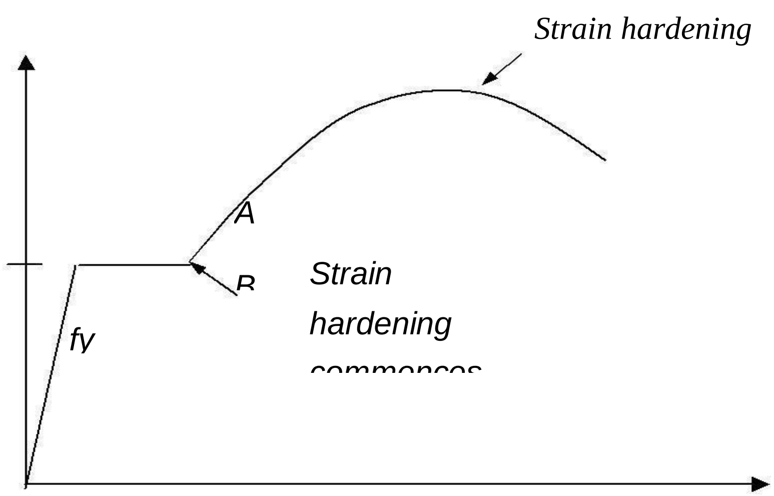


Fig 3.1 Idealised Stress Strain curve

3.2.2 Theoretical Basis

As an incremental load is applied to a beam, the cross-section with greatest bending moment will eventually reach the yield moment. Elsewhere the structure is elastic and the 'peak' moment values are less than yield. As load is incremented, a zone of yielding develops at the first critical section, but due to ductility of steel, the moment at that section remains about constant. The structure, therefore, calls upon its less heavily stressed portions to carry the increase in load. Eventually the zones of yielding are formed at other sections until the moment capacity has been exhausted at all necessary critical sections. After reaching the maximum load value, the structure would simply deform at constant load. Thus it is a design based upon the ultimate load-carrying capacity (maximum strength) of the structure. This ultimate load is computed from a knowledge of the strength of steel in the plastic range and hence the name 'plastic'.

3.2.3 Perfectly Plastic Materials

The stress-strain curve for a perfectly plastic material upto strain hardening is shown in Fig. 3.2. Perfectly plastic materials follow Hook's law upto the limit of proportionality. The slopes of stress-strain diagrams in compression and tension i.e. the values of Young's modulus of elasticity of the material, are equal. Also the values of yield stresses in tension and compression are equal. The strains upto the strain hardening in tension and compression are also equal. The stress strain curves show horizontal plateau both in tension and compression. Such materials are known as perfectly plastic materials.

3.2.4 Fully Plastic Moment of a Section

The fully plastic moment M_p, of a section is defined as the maximum moment of resistance of a fully plasticized or yielded cross-section. The assumptions used for finding the plastic moment of a section are:

(i) The material obeys Hooke's law until the stress reaches the upper yield value; on further straining, the stress drops to the lower yield value and thereafter remains constant.

(ii) The yield stresses and the modulus of elasticity have the same value in compression as in tension.

(iii) The material is homogeneous and isotropic in both the elastic and plastic states.

(iv) The plane transverse sections (the sections perpendicular to the longitudinal axis of the beam) remain plane and normal to the longitudinal axis after bending, the effect of shear being neglected.

(v) There is no resultant axial force on the beam.

(vi) The cross section of the beam is symmetrical about an axis through its centroid parallel to plane of bending.

(vii) Every layer of the material is free to expand and contract longitudinally and laterally under the stress as if separated from the other layers.

In order to find out the fully plastic moment of a yielded section of a beam as shown in Fig. 3.3, we employ the force equilibrium equation, namely the total force in compression and the total force in tension over that section are equal.

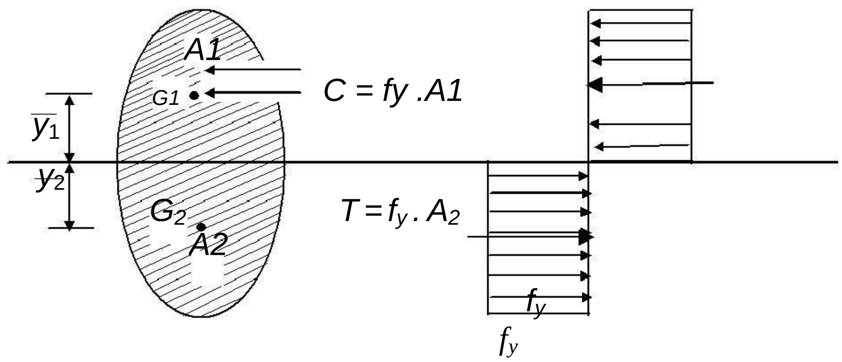


Fig. 3.3 Fully plastic moment

Total compression , C

= Total tension , $T\, f_y . A_1$

$= f_y . A_2$

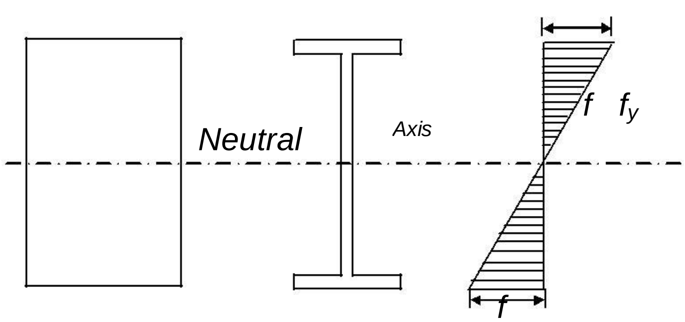
$A1 \quad = A_2$

The plastic modulus of a completely yielded section is defined as the combined statical moment of the cross-sectional areas above and below the neutral axis or equal area axis. It is the resisting modulus of a completely plasticised section.

3.3 BENDING OF BEAMS SYMMETRICAL ABOUT BOTH AXES

The bending of a symmetrical beam subjected to a gradually increasing moment is considered first. The fibres of the beam across the cross section are stressed in tension or compression according to their position relative to the neutral axis and are strained.

Fig. 3.4 Elastic stresses in beams



While the beam remains entirely elastic the stress in every fibre is proportional to its strain and to its distance from the neutral axis. The stress *(f)* in the extreme fibres cannot exceed f_y. as in Fig.3.4

When the beam is subjected to a moment slightly greater than that, which first produces yield in the extreme fibres, it does not fail. Instead the outer fibres yield at constant stress (f_y) while the fibres nearer to the neutral axis sustain increased elastic stresses. Fig. 3.5 shows the stress distribution for beams subjected to such moments.

Such beams are said to be 'partially plastic' and those portions of their cross- sections, which have reached the yield stress, are described as 'plastic zones'.

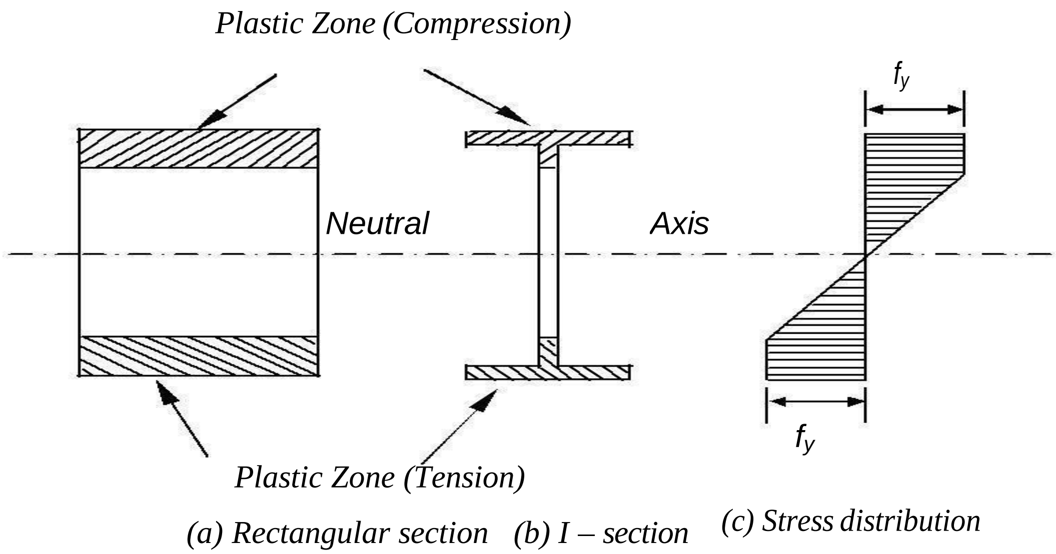


Fig. 3.5 Stresses in partially plastic beams

The depths of the plastic zones depend upon the magnitude of the applied moment. As the moment is increased, the plastic zones increase in depth, and, it is assumed that plastic yielding can occur at yield stress (f_y) resulting in two stress blocks, one zone yielding in tension and one in compression. Fig. 3.6 represents the stress distribution in beams stressed to this stage. The plastic zones occupy the whole of the cross section, and are described as being 'fully plastic'. When the cross section of a member is fully plastic under a bending moment, any attempt to increase this moment will cause the member to act as if hinged at the neutral axis. This is referred to as a plastic hinge.

The bending moment producing a plastic hinge is called the full plastic moment and is denoted by 'M_p'.

Note that a plastic hinge carries a constant moment, $M_{P.}$

3.4 REQUIREMENTS FOR PLASTIC DESIGN

IS 800 allow the use of plastic design only where loading is predominantly static and fatigue is not a design criterion.

3.4.1 Shape Factor

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

Here the plastic moment M_p is about 1.5 times greater than the elastic moment capacity. In developing this moment, there is a large straining in the extreme fibres together with large rotations and deflection.

The ratio of the plastic modulus *(Z_p)* to the elastic modulus *(Z)* is known as the shape factor *(S)* and will govern the point in the moment-rotation curve when non-linearity starts. For the theoretically ideal section in bending i.e. two flange plates connected by a web of insignificant thickness, this will have a value of 1. When the material at the centre of the section is increased, the value of S increases. For a universal beam the value is about 1.15 increasing to 1.5 for a rectangle.

3.5 PLASTIC HINGES

At the plastic hinge an infinitely large rotation can occur under a constant moment equal to the plastic moment of the section. Plastic hinge is defined as a yielded zone due to bending in a structural member at which an infinite rotation can take place at a constant plastic moment M_p of the section. The number of hinges necessary for failure does not vary for a particular structure subject to a given loading condition, although a part of a structure may fail independently by the formation of a smaller number of hinges. The member or structure behaves in the manner of a hinged mechanism and in doing so adjacent hinges rotate in opposite directions.

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.

The values of moment, at the adjacent section of the yield zone are more than the yield moment upto a certain length *L*, of the structural member. This length *L*, 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**.**In a simply supported beam 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* (= *L)* be the length of plasticity zone.

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

3.5.1 FUNDAMENTAL CONDITIONS FOR PLASTIC ANALYSIS

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.*

Equilibrium condition *:* $F_x = 0, F_y = 0, M_{xy} = 0$

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

3.5.2 Mechanism

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.*

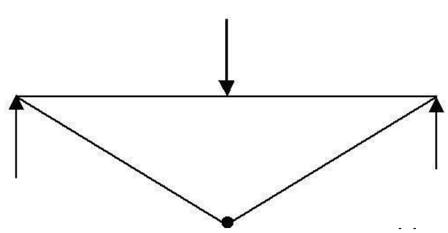
Various types of independent mechanisms are

3.5.3 Beam Mechanism

Fig. 3.8 sketches three simple structures and the corresponding mechanisms.

(a) A simply supported beam has to form one plastic hinge

Redundancy, $r = 0$



(b) A propped cantilever requires two hinges to form a mechanism. Redundancy, $r = 1$

No. of plastic hinges formed,

$= r + 1 = 2$

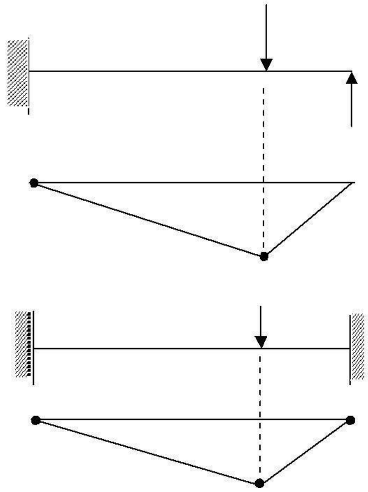


Fig. 3.8 Beam Mechanism

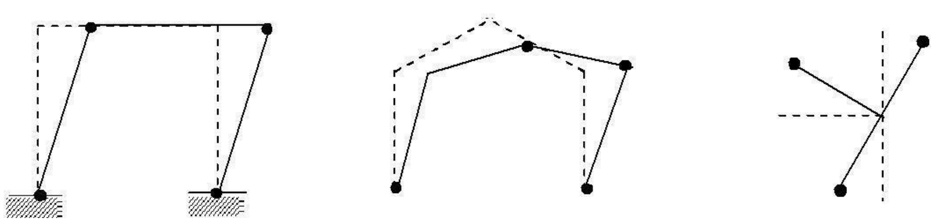
(c) A fixed beam requires three hinges to form a mechanism. Redundancy, $r = 2$

No. of plastic hinges $= 2 + 1 = 3$

Therefore the number of hinges needed to form a mechanism equals the static redundancy of the structure
plus one.

Panel or Sway Mechanism

Fig. 3.9 (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. 3.9 Gable 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. Fig.10 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 denoted by the right hinge being shown on the beam, in a position slightly away from the joint.

Two hinges developed on the beam

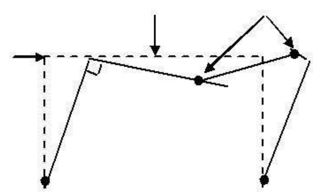


Fig. 3.10 Combined Mechanism

3.6 LOAD FACTOR AND THEOREMS OF PLASTIC COLLAPSE

Plastic analysis of structures is governed by three theorems,

The load factor at rigid plastic collapse *($_p$)* 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 $_p$ 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.

$$n = h - r$$

The three theorems of plastic collapse are given below for reference.

3.6.1 Lower Bound or Static Theorem A load factor (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, (p).

p is the highest value of s which can be found.

3.6.2 Upper Bound or Kinematic Theorem

A load factor (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 ($_p$)

p is the lowest value of $_k$ 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 ($_p$).

3.7 RIGID PLASTIC ANALYSIS

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
Workdone at the plastic hinge = $M_p 2\theta$
Work done by the displacement of
the load =$W(l/2)\theta$ At collapse, these
two must be equal, Mp = WL/4

3.7.1 Mechanism Method

In the mechanism or kinematic 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.

3.7.2 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.

3.8 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 as in Fig 3.12. This means that the neutral axis moves away from theequal area axis providing an additional area in tension or compression depending on the type of axial load.

Considering a rectangular member of width b and depth d subjected to an axial compressive force P together with a moment M in the vertical plane .
The values of M and P are increased at a constant value of M/P until the fully plastic stage is attained, then the values of M and P become:

$Mpa = 0.25\, f_y\, b\, (d^2 – 4y^2)$

$P = 2y^*\, b\, f_y$ where f_y = yield stress

y = distance from the neutral axis to the stress change for M_p without axial force, $M_p = f_y bd^2/4$ If axial force acts alone $-P_y = f_y bd$ at the fully plastic state.

From the above equations the interaction equation can be obtained:

$$M_x/M_p = 1 – P^2/ P_y$$

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

3.9 CONNECTIONS

Connections play a key role in determining whether or not a structure will reach its computed ultimate load, because plastic hinges usually form at the intersection of two or more members.

There are four principal requirements, in design of a connection

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.

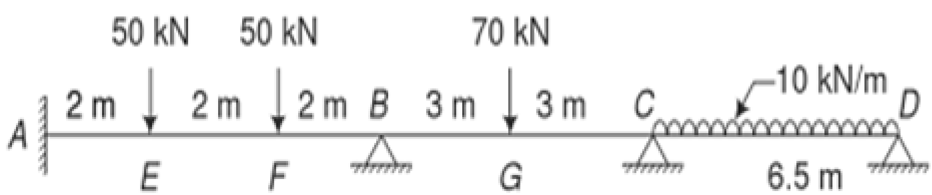
Stiffness – Average unit rotation of the connecting region should not exceed that of an equivalent length of the beam being joined.

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.

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

Descriptive Questions

1. Determine the plastic modulus for the I section having flange width = 180mm, flange thickness = 17.2mm, overall depth = 500mm and thickness of web = 11.2mm
2. Analyze the collapse load for a propped cantilever beam of span L subjected to an uniformly distributed load of W / m on entire span.
3. A beam fixed at both ends is subjected to a uniformly distributed load ‘w’ on its right half portion. Determine the collapse load if the beam has uniform cross section.
4. Design the continuous beam with the service load as shown in the fig. The load factor may be assumed as 1.7. Provide a uniform cross section throughout the beam.



SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – IV – ANALYSIS AND DESIGN OF SPECIAL STRUCTURES – SCIA5203

UNIT 4 ANALYSIS AND DESIGN OF SPECIAL STRUCTURES

4.1 CHIMNEYS

4.1.1 Introduction

Chimneys or stacks are very important industrial structures for emission of poisonous gases or smoke from a boiler, stove, furnace or fireplace to a higher elevation such that the gases do not contaminate surrounding atmosphere. These structures are tall, slender and generally with circular cross-sections. Different construction materials, such as concrete, steel or masonry, are used to build chimneys. Steel chimneys are ideally suited for process work where a short heat- up period and low thermal capacity are required. Also, steel chimneys are economical for height up to 45m. They are typically almost vertical to ensure that the hot gases flow smoothly, drawing air into the combustion through the chimney effect. Chimneys are tall to increase their draw of air for combustion and to disperse the pollutants in flue gases over a greater area in order to reduce the pollutant concentrations in compliance with regulatory or other limits.

Types and Design of steel Chimney

The design of steel chimney can be done as two types:

- Self-supporting steel chimneys
- **Guyed steel chimneys.**

4.2 Self-supporting steel chimneys: When the lateral forces (wind or seismic forces) are transmitted to the foundation by the cantilever action of the chimney, then the chimney is known as self-supporting chimney. The self-supporting chimney together with the foundation remains stable under all working conditions without any additional support. A self-supporting chimney is shown in Fig 4.1. The self-supporting chimneys are made upto 10 m diameter and from 50 m to 100m in height.

4.3 Guyed steel chimneys: In high steel chimneys, the mild steel wire ropes or guys are attached to transmit the lateral forces. Such steel chimneys are known as *guyed steel chimneys.* In guyed steel chimneys, all the externally applied loads (wind, seismic force, etc.) are not totally carried by the chimney shell. These attached guys or stays do share these applied loads. These guys or stays ensure the stability of the guyed steel chimney. These steel chimneys may be provided with one, two or three sets of guys. In each set of guys, three or four or sometimes six wires are attached to the collars. When one set of guy is used, then the guys are attached to a collar at one-third or one-fourth of the height from the top. When- more than one set of guys are used, then these are used at various heights. A guyed chimney is shown in Fig 4.2.

A particular type of steel chimney is selected depending on the advantage and disadvantages with reference to economy. A choice between self-supporting and guyed steel chimney is made by considering some of the important factors, number of units, type of equipment and the type of fuel to be used are considered. In case the chimney is to be used for boilers, the surface area, output efficiency, draft requirements etc. are taken into account. The mode of operation of the equipment shall also be considered.

The temperature of the flue gases before entering the chimney and its likely variation, are studied. The type of lining is decided knowing the composition of the flue gases. The specific weight, the quantity of dust and data about the aggressiveness of the flue gases must be known. The local statutory regulations, relating to height, dispersion of ash, provision for earthing aviation warning lamp, health etc. are the factors which should be considered for selecting a type of steel chimney. The mode of erection is also considered.

4.3.1 Steel -plates for Chimney

The width of steel plates required for the steel chimney varies from 0.9 m to 2.5 m. The steel plates of 1.50m width are most commonly used. The thickness of steel plates should not be less than 6 mm. The thickness of steel plates in the two upper sections of the chimney should not be less than 8 mm to resist more corrosion likely at the top of chimney. The thickness of steel plate in the flared portion should not be less than the thickness at the lowest section of the cylindrical portion. The steel plates are available in thickness of 5, 6, 8, 10, 12, 14, 16, 18, 20, 22, 25, 28, 32, 36, 40, 45, 50, 56 and 63 mm. For the ease in construction, the upper diameter of plates forming the side of chimney is kept less than the lower diameter. Each course fits telescopic over the lower course.

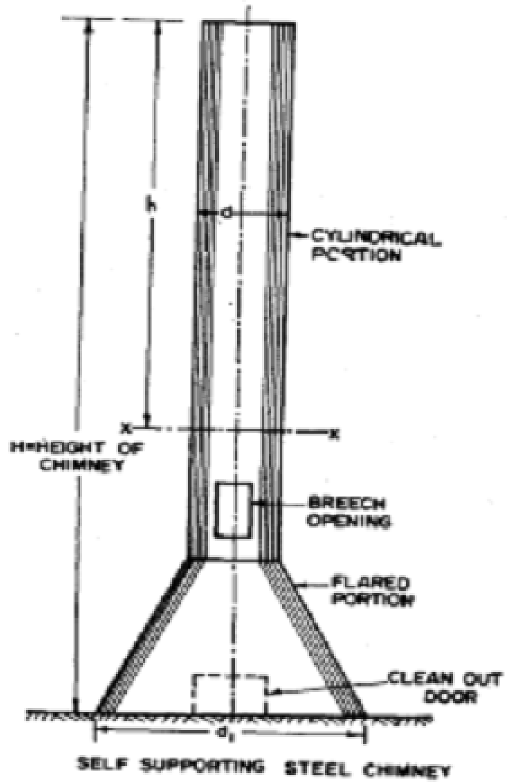


Fig 4.1 Self supporting chimney

4.3.2 Breech Opening

The breech opening is also known as *flue opening*, The flue opening is provided for the entrance of flue gases. The flue gases come from furnaces of the boilers. A breech opening is provided in the steel chimney as shown in Fig 4.3. The area of breech opening is kept about 20 percent larger than the internal cross-sectional area of the chimney. The maximum width of the breech opening should not be greater than two-thirds of the diameter. In, order to compensate the removed material. The reinforcement should be provided all around the breech opening. The vertical reinforcement provided should be 20 percent larger than the material removed in the ratio of diameter to the long chord perpendicular to the face of the opening. The horizontal reinforcement provided at the top and bottom of the opening is kept equal to the vertical reinforcement. The reinforcing material provides sufficient vertical stiffness. In order to transfer distribute the stress into the steel of the chimney, the reinforcing material should be extended above and below the opening. In the self-supporting steel chimney the breech opening is kept well above the flared base, so that it does not extend into the flared base.

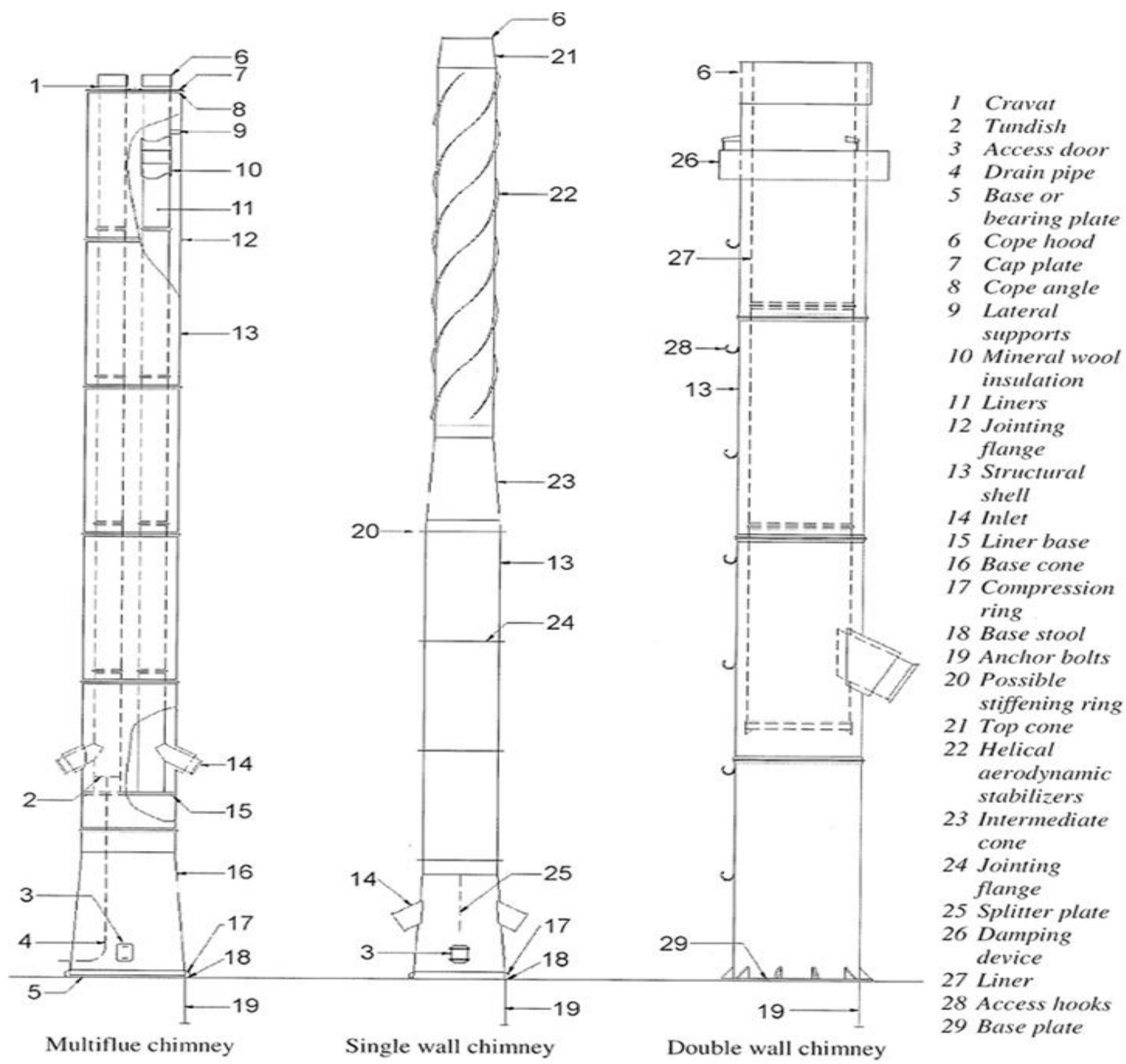


Fig 4.2 Guyed Steel chimney

The steel chimneys may have one breech opening, two breech openings in the same direction two breech openings at right angles and three breech openings as shown in Fig.4.2. The number of flue-openings maybe one, two, three or four depending on the requirement. It is suggested that a maximum of two flue- openings may be provided at one level so that the chimney remains enough to resist the applied forces at the plane of the openings. However, it is possible to provide three openings in one plane. This is done only when the number of flue openings is three only. The width of opening does not exceed one third of the diameter of the chimney at that plane.

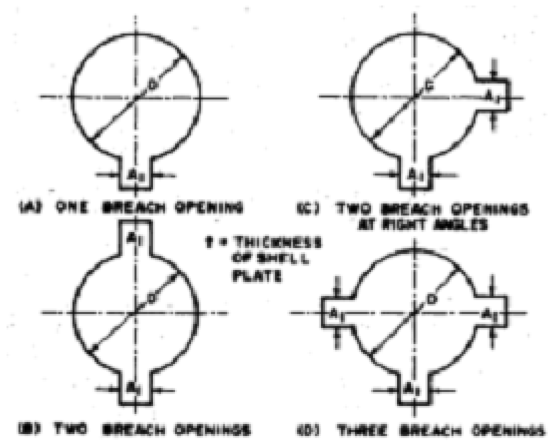


Fig 4.3 Breech Opening

Clean Out door

A clear out door as shown by dotted lines in Fig. 1 is provided preferably on the opposite side of the breech opening near the base. The minimum size of Cleanout door shall be 500 mm x 800 mm clear. The cleanout doors are also properly reinforced.

Forces acting on Steel Chimney

The various forces acting on the self-supporting steel chimney are as follows:

1. Self-weight of the steel chimney
2. Weight of lining
3. Wind pressure
4. Seismic forces

1. **Self-weight of the chimney.**

The self-weight of steel chimney, *Ws* acts vertically.

Consider a horizontal section XX as shown in Fig1. The thickness of steel plates of chimney above the section *XX*, may be assumed constant. The self-weight of chimney is given by

$$w_s = \boldsymbol{\rho} \cdot (\boldsymbol{\pi} d) \cdot t \cdot h$$

where ρ= Unit weight of steel = 79 kN/m^3 d = Diameter of chimney in meters
t = Thickness of steel plates in meters

h = *Height* of steel chimney above the section XX in meters

$$w_s = 79 \cdot (\pi d) \cdot t \cdot h$$

The compressive stress in the steel plates at the section XX due to the self weight of chimney is, given by

$$f_{s1} = \frac{w_s}{\pi dt} = \frac{\rho \cdot (\pi d) \cdot t \cdot h}{\pi dt} = 0.079h\,(N/mm^2)$$

2. Weight of lining.

The weight of the lining in the steel chimney W_L, also acts vertically. The thickness of brick lining may be assumed as 100 mm.

The weight of brick lining,

$$w_L = \rho_1 \cdot (\pi d) \cdot (0.1) \cdot h$$

ρ_1=Unit weight of brick lining = 20 kN/m^3

The compressive stress in the steel plates at the section *XX* due to the weight of lining

$$f_{s2} = \frac{w_L}{\pi dt} = \frac{20 \cdot (\pi d) \cdot (0.1) \cdot h}{\pi dt} = 0.002\left(\frac{h}{t}\right)$$

3. Wind pressure.

The wind pressure acts horizontally. The wind pressure acting on a structure depends on the shape of the structure, the width of the structure, the height of the structure, the location of the structure, and the climatic condition. The wind pressure per unit area increases with the height of the structure above the ground level. In order to simplify the· design, the steel chimney is divided into number of segments of equal height. Each segment may be kept equal upto 10 m. The intensity of wind pressure in throughout the area of each segment may be assumed as uniform. The intensity of wind pressure corresponding to the mid-height of each segment may be noted from IS: 875-1987. The wind pressure on the flared portion may be found by using average diameter. The wind pressure is assumed to act at the mid-height of each segment and as also in the flared portion. It has also been practice *to* take uniform wind pressure over the full height of chimney. The wind pressure $P_z = 0.6V^2$

Design wind speed $V_z = V_b k_1 k_2 k_3$

V_b = Basic wind speed

k_1 = Probability factor or risk coefficient k_2 =

Terrain and height factor k_3 = Topography factor

IS 875 (Part 3) gives the basic wind speeds having a return period of 50 years and at a height of 10 m above ground level. Entire country is divided into six wind zones.

Basic wind speed is in m/s (Based on 50yr return period). For some important cities, basic wind speed is given in Appendix A of the code

4. Seismic forces

The seismic forces also act horizontally. The seismic forces act on a structure, when the structures are located in the seismic areas.

The total design lateral force or design base shear along any principal direction shall be determined by this expression:

$V_b = A_h W$ where,

A_h = design horizontal seismic coefficient for a structure W= seismic weight of building.

The design horizontal seismic coefficient for a structure A_h is given by: $A_h = ZIS_a/2R_g$

Z is the zone factor given in Table 2 of IS 1893:2002 (part 1) for the maximum considered earthquake (MCE) and service life of a structure in a zone. The minimum values of importance factor are given in table 6 of IS 1893:2002. R is the response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. The values of R are given in Table 7 of IS 1893:2002 (part 1).

Sa/g is the average response acceleration coefficient for rock and soil sites as given in figure 2 of IS 1893:2002 (part 1). The values are given for 5 % of damping of the structure.

The fundamental natural period for buildings is calculated as per Clause 7.6 of IS 893:2002 (part 1). As

$$T_a = 0.085h^{0.75}$$

h is the height of the building in m The horizontal reinforcement provided at the top and bottom of the opening is kept equal to the vertical reinforcement. The reinforcing material provides sufficient vertical stiffness. In order to transfer distribute the stress into the steel of the chimney, the reinforcing material should be extended above and below the opening. In the self-supporting steel chimney the breech opening is kept well above the flared base, so that it does not extend into the flared base.

After the total design base shear is calculated, it is distributed along the height of the building. The base shear at any floor or level depends on the mass of the level and deformed shape of the structure. Earthquake forces can deflect a building into a number of shapes, the natural mode shapes of the building which in turn depend upon the degree of freedom of the building. A building can have infinite degree of freedoms but we convert it to finite degree of freedom by idealizing a multi storeyed building into a lumped mass model by assuming the mass of the building lumped at each floor level with one degree of freedom in the direction of lateral displacement in which the structure is being analyzed per floor, resulting in degrees of freedom equal to the number of floors.

The magnitude of the lateral force at a floor (node) depends on:

- Mass of that floor
- Distribution of stiffness over the height of structure
- Nodal displacements in a given mode

IS 1893:2002 (part 1) uses a parabolic distribution of lateral force along the height of the building. Distribution of base shear along the height is done according to this equation:

$Q_i = W_i h_i 2 / \sum W_j h_j^2$ where

Q_i = design lateral force at floor i

W_i = seismic weight of floor i

h_i = height of floor I measured from foundation

n = number of stories in the building or the number of levels at which masses are located

Bending Moment

The wind force acts as uniformly distributed load on the self-supporting steel chimney.

For the purpose of determining bending moment at any section *XX* Fig. 1, the wind force is assumed to act at the middle height above the section. The bending moment due to wind at section *XX*, *h* metres below the top,

$Mw = (p \times h/2)$ where P = Total wind force

Bending Stress on Steel chimney due to wind

$$F_w = \frac{M_w \cdot d}{2I}$$

The bending stress, fw at the extreme fibre of steel chimney due to overturning moment Mw is Z =0.77 $d^{2*}t$

Permissible Stresses

The windward side of steel chimney is subjected to tensile stress due to the combined effect of the wind and weight of steel chimney. The leeward side of steel chimney is subjected to compressive stress due to the combined effect of the wind, weight of steel chimney and the weight of lining. On the compressive side the efficiency of the joint depends on the strength of rivet in shear and in bearing and does not depend on the tensile strength of plate. The efficiency of joint on compression side is 100 percent. The efficiency of joint on the tension side is 70 percent.

In order to prevent the flattening of the steel plates on the tension or windward side, and buckling of the steel plates on compression or leeward side, the permissible stress in compression on gross-sectional area is adopted less than the permissible stress in tension on the net sectional area. The permissible stresses in steel chimney in axial tension, shear and bearing shall be adopted as specified in IS: 800-1984. The allowable stresses in axial compression and in bending from the table of IS 6533.

4.4 TOWERS AND MASTS

4.4.1 Introduction

- Towers and Masts are typically tall constructions specially designed to support the phase conductors and shield wires of transmission lines, antennas for radio communication (television, radio, GSM and Internet traffic), floodlight projectors, wind turbines or platforms for inspection.
- Towers and masts may also be required to raise antennas above tree lines and roof tops for line of sight connections. The characteristic dimension of a tower is its

height.

- The terms –tower‖ and –mast‖ are often used for the same type of structure. In order to avoid confusion, a tower will be considered as a self supporting structure while a mast is supported by stays or guys.
- There are three most common types of towers/masts that are used today: • Monopoles • Self supporting towers • Guyed masts.
- Selecting one solution or another is based in general on four major considerations: load, footprint, height and budget. Each factor can be critical in the selection of the type of structure.

4.4.2 Load on tower

- The loading capability of a tower depends on the structure of the tower. The more surface area of equipments (eg. antennas), coaxial cables, brackets and other equipment mounted on the tower and exposed to the wind, the more robust tower is required.
- The wind load is proportional to the area of the exposed structure and distance from the attachment to the ground. Curved and perforated shapes (grids and trusses) offer less wind resistance and are therefore preferred to achieve a low wind load. Solid dishes are quite vulnerable to wind load and should be avoided in windy environments.
- When it is considered necessary, wind tunnel tests may be performed to evaluate the wind action.

4.4.3 Tower footprint

- The footprint of a tower is the amount of free space on the ground that is required
- Depending on the structure of the tower, it requires more or less space for installation.
- For tall guyed masts (> 30m), each guy anchor is typically 10-15m from the base of the mast. For a mast with 3 guy wires per level, that results in a footprint of approx. 90 – 200 m^2.

Height of tower

- Adding guys cables to a structure will allow higher height.
- –The smaller the tower base, the more costly to purchase and install the tower‖
- Monopoles have the smallest footprint of all towers, and are hence the most

expensive towers. It is followed by self supported towers and then guyed masts which require the largest footprints. Depending on the tower type, certain tools, machinery and cranes are needed to assemble the tower which must be taken into consideration in the final budget.

Other factors to be considered in design

- mean aerial height for each aerial system
- directions for the various directional antennas
- wind drag on each element of the array and dependent on wind direction, size, weight and disposition of all feeders and cables,
- the permitted angular rotations in azimuth and elevation of each aerial above which the broadcast signal is significantly reduced,
- the need for all-weather access to some of the aerials
- besides the known antenna and aerial configuration the possible future extension should be defined,
  - atmospheric ice formation on the structure and aerials and its likelihood to occur with high wind,
- wind drag of the structure itself without ice and with ice if feasible,
- the degree of security required,
- the available ground area and access to the site,
- the geological nature of the site,
- the overall cost of land, foundations and structure,
- the cost and implications of future maintenance or structural replacement,
- any special planning considerations imposed by statutory bodies,
- the aesthetic appearance of the structure.

4.5 Towers

- A self supporting tower (freestanding tower) is constructed without guy wires.
- Self supporting towers have a larger footprint than monopoles, but still require a much smaller area than guyed masts.
- Self supporting towers can be built with three or four sided structures.
- They are assembled in sections with a lattice work of cross braces bolted to three - four sloping vertical tower legs. They can be used for power transmission lines, lighting, wind turbines, communications, etc.

Power transmission line Wind Turbine - Lattice Tower Radio Lattice Tower

Fig.4.4 Types of towers

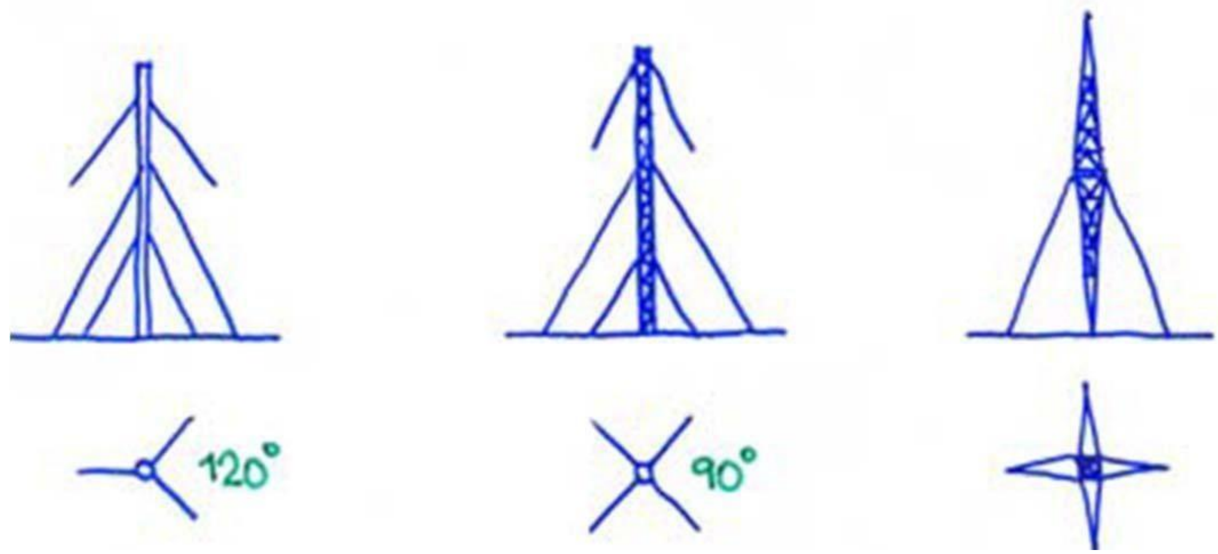
- An overhead transmission line connects two nodes of the power supply grid.
- The route of the line has as few changes in direction as possible, based on the line route and the type of terrain it crosses. Depending on their position in the line various types of towers occur, such as:

✓ suspension towers
✓ angle suspension towers
✓ angle towers
✓ tension towers
✓ terminal towers (dead-end type).

4.6 Guyed masts

- A guyed mast is one of the most complicated structures an engineer may be faced with

✓ The number of masts collapses is relatively far greater than for other types of structurescarry aerials of TV, radio etc., or may be themselves emitters (then isolation - separation from ground is necessary).



solid structure truss structure variable section

The component parts of a guyed mast are:

- the foundation
- the steel mast, which generally has a pinned
- foot
- the guy cables
- the structural accessories
- the quipment (antenna)

Design of steel mast

- Steel mast anchored at different heights, is loaded in bending due to wind, and compression from vertical loads (dead load, live load, pretension forces in the cables.
- Pretension force in cables is determined so that in case the cable is unloaded due to wind action, the cable should still be subjected to a tension force.
- Value of pretension force is about 0.10 – 0.15 fu

After the internal efforts in members is determined (legs, diagonals), they are designed using relevant verifications.

- For one segment of the mast, members of the same type have similar sections

• Steel mast anchored at different heights, is loaded in bending due to wind, and compression from vertical loads (dead load, live load, pretension forces in the cables.

• Pretension force in cables is determined so that in case the cable is unloaded due to wind action, the cable should still be subjected to a tension force.

• As a rule, if the guy is attached in the top of the tower (100%), the tension should be 8% of the tensile strength.

• For 80% of the tower's height, 10% tension should be applied.

• If the anchor point is at 65% of tower height, 15% tension can be applied as you loose a lot of wind load in this last type of installation.

• The breaking strength will improve the control of the flexibility and still not cut down on the cable strength. Normally, a tower has 2 – 3 levels of guys (depending on the height of the tower/mast) and three guys on each level. It is recommendable to use turnbuckles as it will allow you to fine tune your adjustments later on.

• After the internal efforts in members is determined (legs, diagonals), they are designed using relevant verifications.

• For one segment of the mast, members of the same type have similar sections.

Typical design problems are:

- establishment of load requirements.
- consistency between loads and tower design.
- establishment of overall design, including choice of number of tower legs.
- consistency between overall design and detailing.
- detailing with or without node eccentricities.
- sectioning of structure for transport and erection.

The loads acting on a transmission tower are:

- dead load of tower.
- dead load from conductors and other equipment.
- load from ice, rime or wet snow on conductors and equipment.
- ice load, etc. on the tower itself

- erection and maintenance loads.
- wind load on tower.
- wind load on conductors and equipment.
- loads from conductor tensile forces.
- damage forces
- earthquake forces.

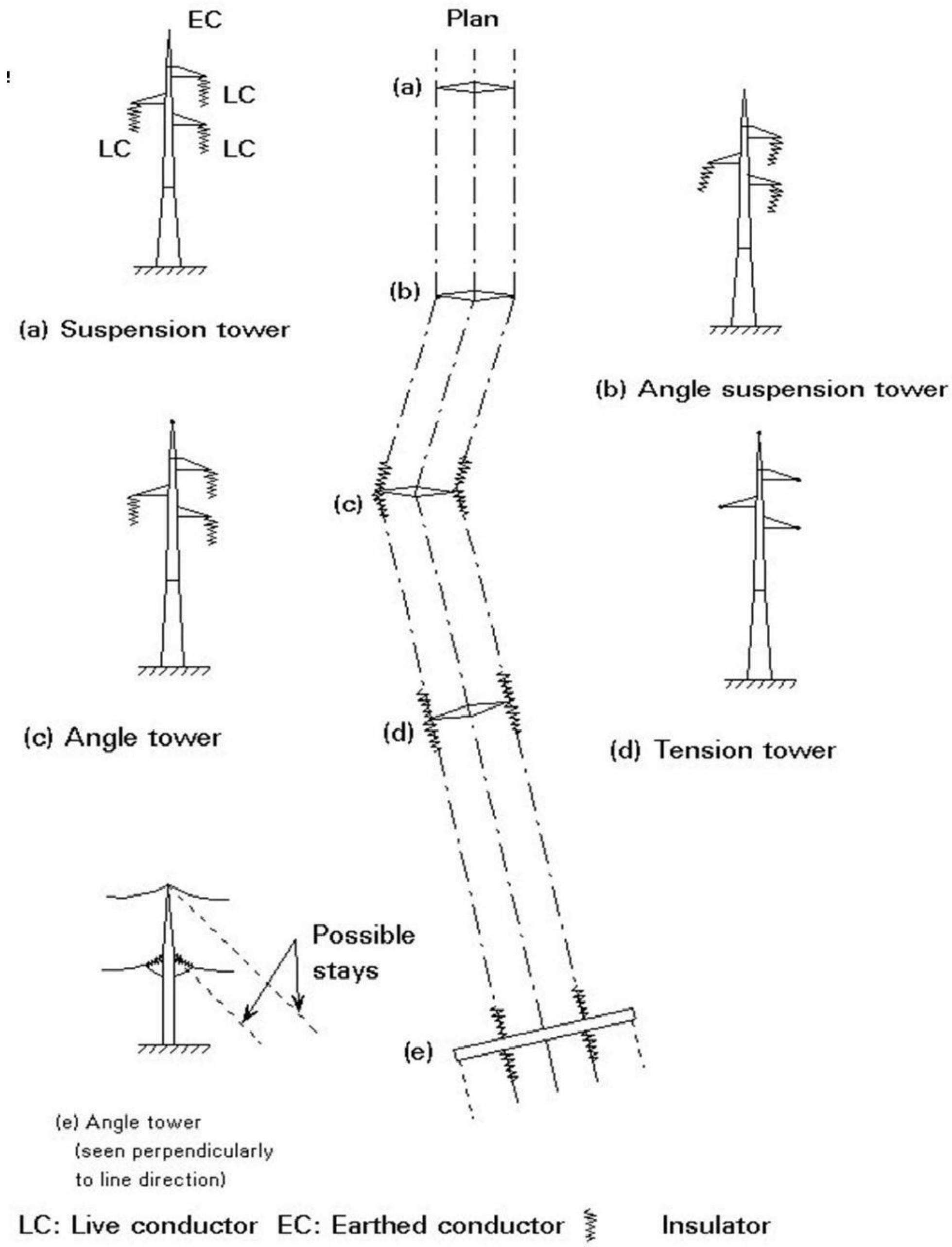


Fig.4.5 Types of towers along transmission

Descriptive Questions

1. Based on tension, design the thickness of a self-supported steel stack of height 80 m and diameter 3 m located in the outskirts of Bhopal. Assume II terrain category and flat topography at site.
2. Based on compression, design the thickness of a self-supported steel stack of height 80 m and diameter 3 m located in the outskirts of Bhopal. Assume II terrain category and flat topography at site.
3. Design the following for a self-supported steel chimneyof height 80 m, diameter 3 m and average thickness of 10 mm subjected to a base moment of 7800 kNm
   a) Anchor bolts
   b) Base plate
   c) Foundation
4. Explain the types of load acting on transmission towers.
5. Design a guyed steel stack of 21m height with one set of guy wires attached at 7m from the top. The diameter of the shaft is 1.2m. Its shape factor is 0.7 and wind pressure is 1.2 kN/m2. Assume initial tension in guy wire as 35 N/m2.

6. Estimate the critical parameters for designing transmission line towers.

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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – V – DESIGN OF COLD FORMED STEEL STRUCTURES – SCIA5203

UNIT 5 DESIGN OF COLD FORMED STEEL STRUCTURES

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. 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 are required and press breaking is used when small volume 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.

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. Table 5.1 shows the Comparison of Hot Rolled and Cold formed sections.

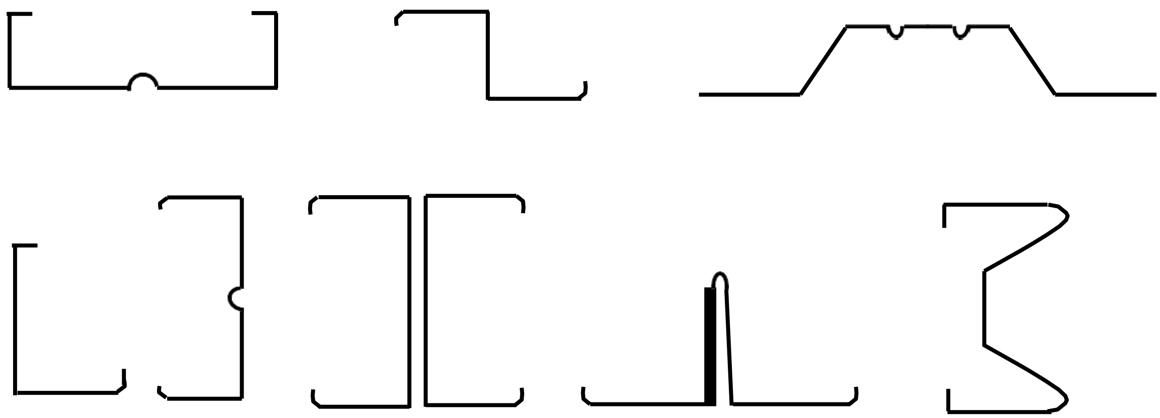
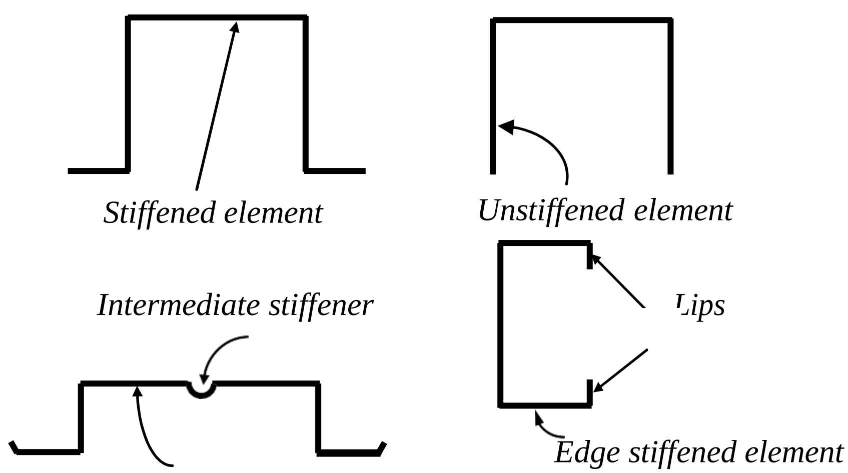


Fig. 5.1 Typical Cold Formed Steel Profiles

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.

5.3 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. 2.



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.

5.4 **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.

5.4.1 Elastic Buckling of Thin Plates

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 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.

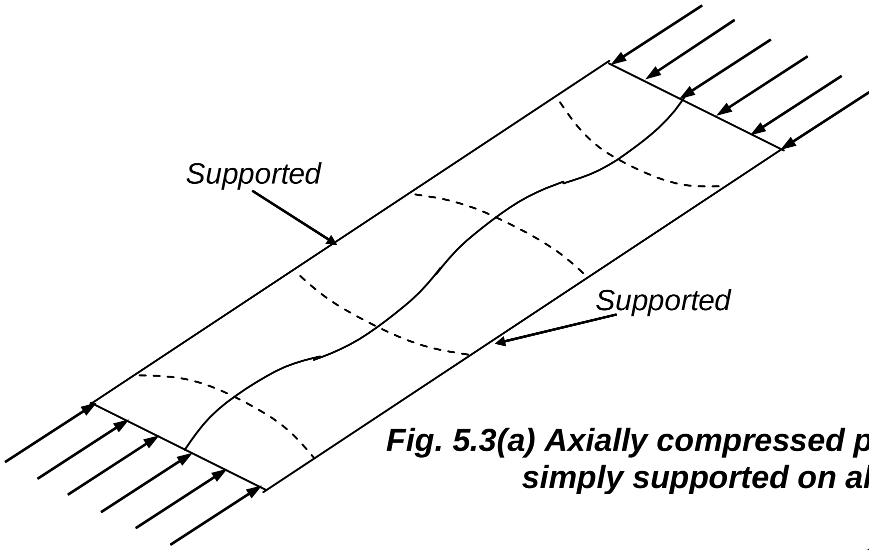
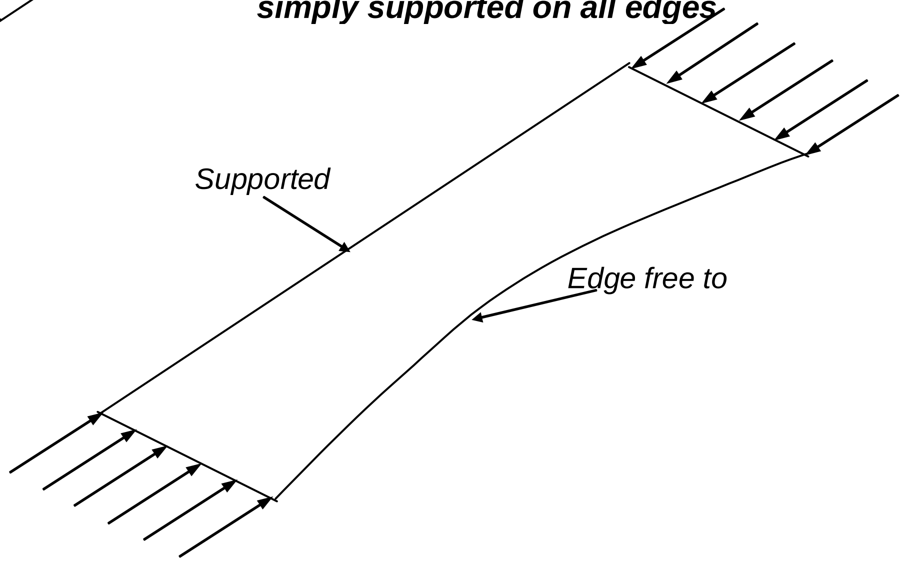


Fig. 5.3(a) Axially compressed plate simply supported on all edges



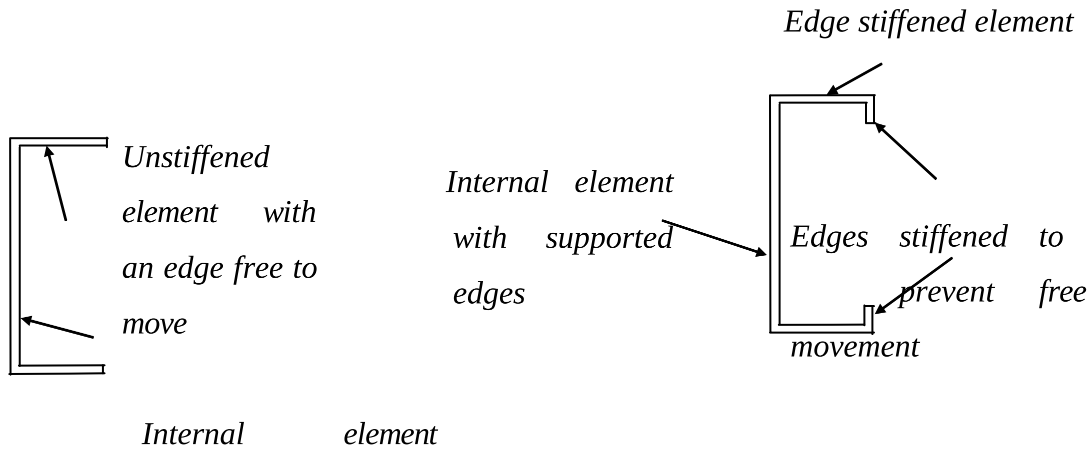
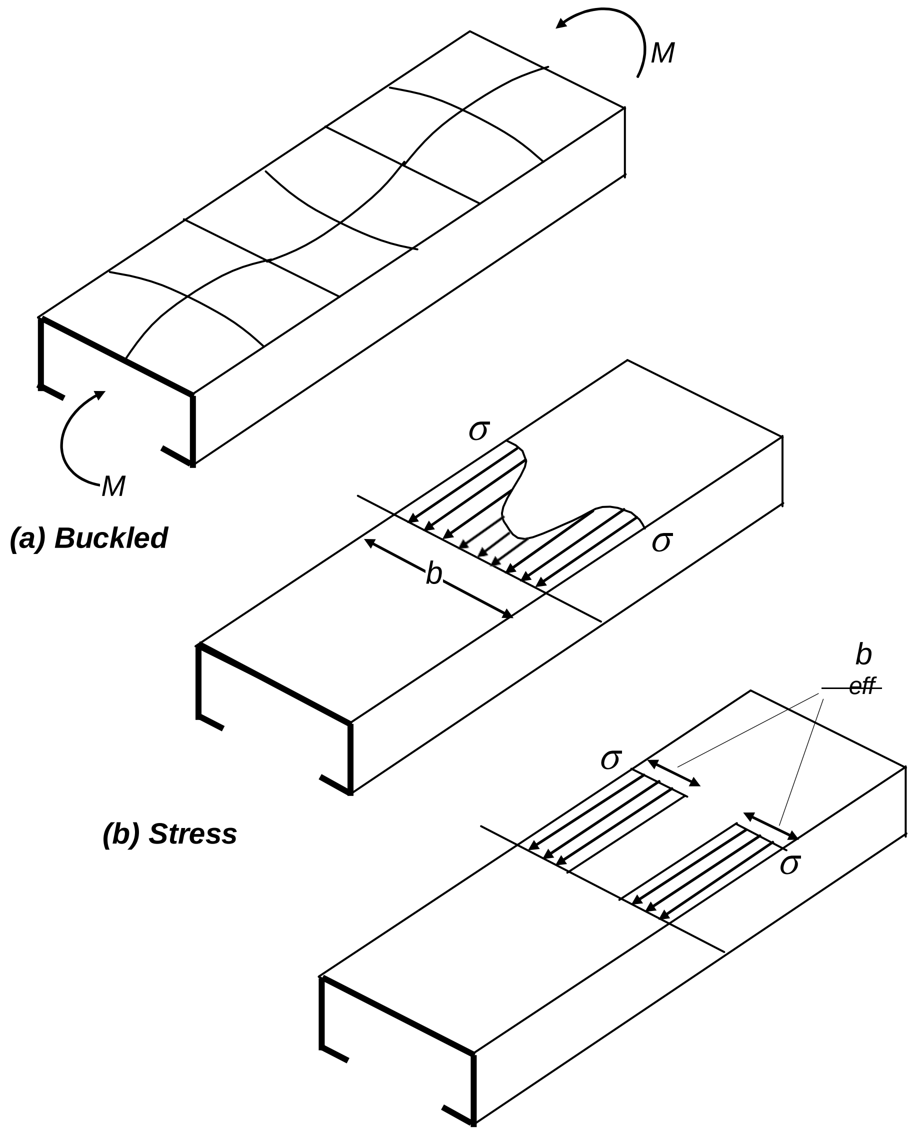


Fig. 5.4 The technique of stiffening the element

5.5 Post Critical Behaviour:

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.



***(d)* Effective width**

Fig. 5.5 Local Buckling Effects

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.

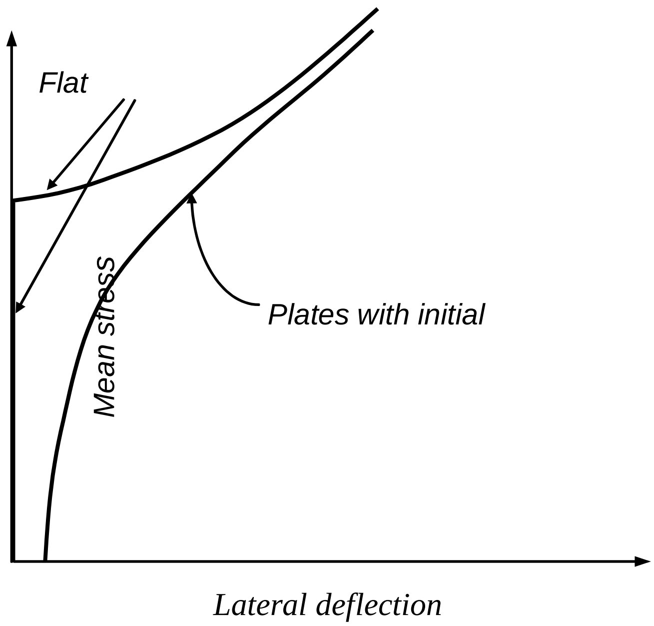


Fig. 5.6 Mean stress Vs Lateral deflection relation

5.6 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 modelled 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 (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.

5.7 Axially Compressed Columns

Local buckling under compressive loading is an extremely important feature of thin walled sections.

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

$P_{cs} = A_{eff} . f_{yd} \qquad = Q. A. f_{yd}$

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

5.7.1 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.

F_c = applied axial load

P_{cs} = short strut capacity defined by $A_{eff}.P_{yd}$

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.

Descriptive Questions

1. A hat section of 100 mm x 80 mm x 25 mm x 4 mm is to be used a concentrically loaded column of 3.1 m effective length. Determine the allowable load. Take fy of light gauge steel as 235 MPa.
2. Two channels without bent lips 200 x 50 x 4 mm are connected with webs to act as a simply supported beam having effective span of 4 m. Determine the maximum uniformly distributed load inclusive of self-weight that can be imposed on it. The beam is laterally supported throughout its length. Take fy of light gauge steel as 230 MPa.
3. Explain the effects of local buckling of thin elements in cold formed light gauge sections.
4. Explain the effects of post buckling of thin elements in cold formed light gauge sections.

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