36 PORTAL FRAMES

1.0 INTRODUCTION

The basic structural form of portal frames was developed during the Second World War, driven by the need to achieve the low - cost building envelope. Now they are the most commonly used structural forms for single-storey industrial structures. They are constructed mainly using hot-rolled sections, supporting the roofing and side cladding via cold-formed purlins and sheeting rails. With a better understanding of the structural behaviour of slender plate elements under combined bending moment, axial load and shear force, many fabricators now offer a structural frame fabricated from plate elements. These frames are composed of tapered stanchions and rafters in order to provide an economic structural solution for single-storey buildings. Portal frames of lattice members made of angles or tubes are also common, especially in the case of longer spans.

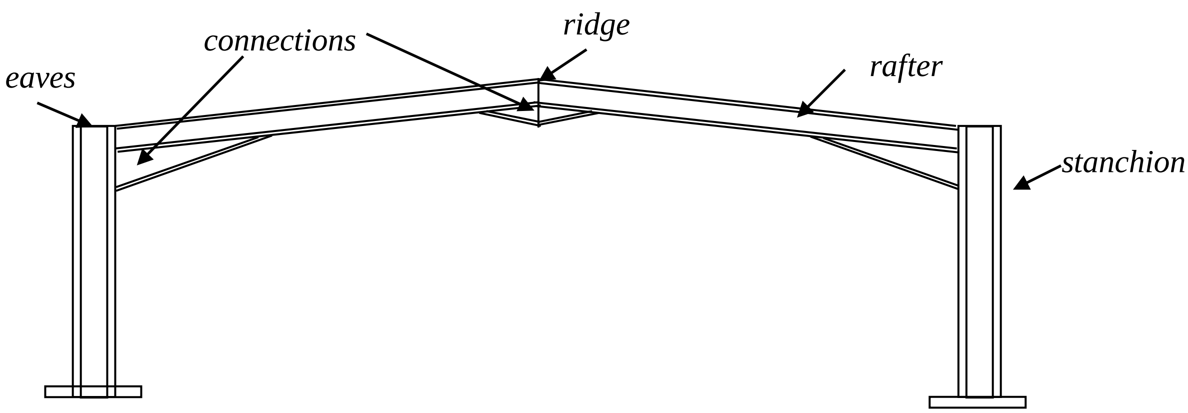
The slopes of rafters in the gable portal frames (Fig.1) vary in the range of *1* in *10* to *1* in *3* depending upon the type of sheeting and its seam impermeability. With the advent of new cladding systems, it is possible to achieve roof slopes as low as 1^0. But in such cases, frame deflections must be carefully controlled and the large horizontal thrusts that occur at the base should be accounted for. Generally, the centre-to-centre distance between frames is of the order *6 to 7.5 m,* with eaves height ranging from *6 -15 m*. Normally, larger spacing of frames is used in the case of taller buildings, from the point of economy. Moment-resisting connections are to be provided at the eaves and crown to resist lateral and gravity loadings. The stanchion bases may behave as either pinned or fixed, depending upon rotational restraint provided by the foundation and the connection detail between the stanchion and foundations. The foundation restraint depends on the type of foundation and modulus of the sub-grade. Frames with pinned bases are heavier than those having fixity at the bases. However, frames with fixed base may require a more expensive foundation.

For the design of portal frames, plastic methods of analysis are mainly used, which allows the engineer to analyse frames easily and design it economically. The basis of the plastic analysis method is the need to determine the load that can be applied to the frame so that the failure of the frame occurs as a mechanism by the formation of a number of plastic hinges within the frame. The various methods of plastic analysis are discussed in an earlier chapter. In describing the plastic methods of structural analysis, certain assumptions were made with regard to the effect of axial force, shear, buckling etc. Unless attention is given to such factors, the frame may fail prematurely due to local, or stanchion or rafter buckling, prior to plastic collapse.

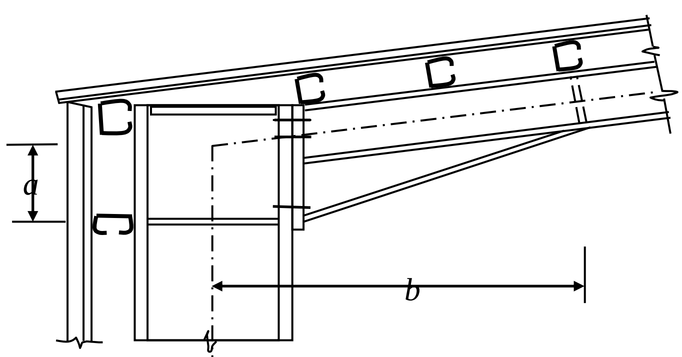
In the analysis, the problem is to find the ultimate load of a given structure with known plastic moment values of its members. But in design, the problem is reversed. Given a certain set of loads, the problem is to select suitable members.

2.0 HAUNCHED PORTAL FRAMES

The most common form of portal frame used in the construction industry is the pinned-base frame with different rafter and column member size and with haunches at both the eaves and apex connections (Fig.1). These two important design features of the modern portal frame have been developed over a number of years, from practical and economic considerations.



(a) Haunched portal frame



(b) Eaves detail

Fig. 1 Typical gable frame

Due to transportation requirements, field joints are introduced at suitable positions. As a result, connections are usually located at positions of high moment, i.e. at the interface of the column and rafter members (at the eaves) and also between the rafter members at the apex (ridge) (See Fig.1). It is very difficult to develop sufficient moment capacity at these connections by providing 'tension' bolts located solely within the small depth of the rafter section. Therefore the lever arm of the bolt group is usually increased by haunching the rafter members at the joints. This addition increases the section strength.

Although a short length of the haunch is enough to produce an adequate lever arm for the bolt group, haunch is usually extended along the rafter and column adequately to reduce the maximum moments in the uniform portion of the rafter and columns and hence reduce the size of these members. But due to this there will be a corresponding increase in the moment in the column and at the column-haunch-rafter interface. This allows the use of smaller rafter member compared to column member. The resulting solution usually proves to be economical, because the total length of the rafter is usually greater than the total length of the column members. The saving in weight is usually sufficient to offset the additional cost of haunch.

The haunched frame may be designed in a manner similar to that of an unhaunched frame, the only difference being that the hinges, which were assumed to be at nodes, are forced away from the actual column-rafter junction to the ends of the haunches. Provided the haunch regions remain elastic, hinges can develop at their ends. The haunch must be capable of resisting the bending moment, axial thrust and shear force transferred by the joining members. The common practice is to make the haunch at the connection interface approximately twice the depth of the basic rafter section, so that the haunch may be fabricated from the same basic section.

3.0 GENERAL DESIGN PROCEDURE

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

a) Determine possible loading conditions.
b) Compute the factored design load combination(s).
c) Estimate the plastic moment ratios of frame members.
d) Analyse the frame for each loading condition and calculate the maximum required plastic moment capacity, M_p
e) Select the section, and
f) Check the design for other secondary modes of failure (IS: 800-1984).

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

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

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

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

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

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

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

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

4.0 SECONDARY DESIGN CONSIDERATIONS

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

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

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

4.1 Influence of axial force on plastic moment

This has been already explained in an earlier chapter on Beam Columns. Even though the presence of axial force tends to reduce the magnitude of the plastic moment, the design procedure may be modified to account for its influence, retaining the 'plastic hinge' characteristic.

IS: 800 recommend the following provisions to account for axial compression on M_p.

- Neglect the effect of axial force on the plastic moment unless $P > 0.15 P_y$, where P is the actual axial force and P_y is the axial force causing yielding of the full cross section.

- If P is greater than 15 percent of P_y,, the modified plastic moment capacity, M_{pc}, is given by

$$\frac{M_{pc}}{M_p} + \frac{1.18P}{P_y} \leq 1.18 \tag{1}$$

where

M_p is the plastic moment capacity of the section when the axial force is absent.
P is the actual axial force.
P_y is the axial force corresponding to yielding.

The required design value of plastic section modulus under combined compression and bending, Z_{req},, for a member is given by:

$$Z_{req} = Z\left(\frac{M_p}{M_{pc}}\right) = \frac{0.85Z}{1 - P/P_y} \geq Z \tag{2}$$

where

$Z = M_p / f_y$,

f_y = yield strength of the material.

4.2 The influence of shear force

The effect of shear force is also to reduce the plastic moment. Due to the presence of shear, two types of 'premature failure' can occur.

(a) General shear yield of the web may occur in the presence of high shear-to-moment ratios.
(b) After the beam has become partially plastic at a critical section due to flexural yielding, the intensity of shear stress at the centre line may reach the yield condition.

The calculated maximum shear capacity V_{ym}, of a beam under combined shear and moment shall be

$$V_{ym} = 0.55\, A_w . f_y . \tag{3}$$

where

A_w = effective cross sectional area resisting shear after deducting the area that has yielded under flexure.

Usually it is found that the reduction in moment capacity due to shear is more than compensated by the strain hardening of extreme fibre under flexure and consequently effect of shear on plastic moment capacity may be neglected in most cases.

4.3 Local Buckling of Flanges and Webs

If the plates of which the cross section is made are not stocky enough, they may be subject to local buckling either before or soon after the first plastic moment is reached. Due to this, the moment capacity of the section would drop off and the rotation capacity would be inadequate to ensure formation of complete failure mechanism. Therefore, in order to ensure adequate rotation at M_p values and to avoid premature plastic buckling, the compression elements should have restriction on the width-thickness ratios. The variables representing dimensions of typical sections are indicated in Fig. 2.

According to IS: 800 the projection of flange or other compression element beyond its

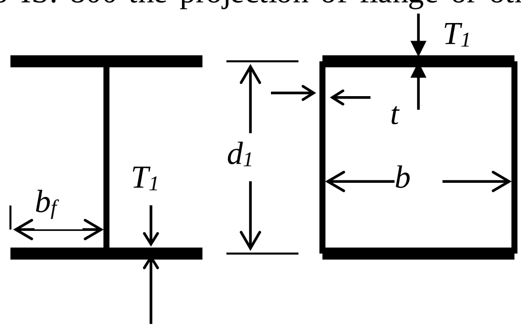


Fig. 2 Plate Elements in Steel Sections

outermost point of attachment, b_f (*in mm*) required to participate in a plastic hinge shall not exceed the value given below

$$b_f \leq 136 . T_1 / \sqrt{f_y} \tag{4}$$

where

T_1 is the thickness of flange of a section or plate in compression or the aggregate thickness of the plates forming the overhang in *mm*.

f_y is yield strength in *Mpa*.

Note: This equation relates b_f and T_1, i.e. a non - dimensional quantity $\frac{b_f}{T_1}$ is related through a dimensioned quantity $\sqrt{f_y}$. It is only valid if f_y is expressed in *Mpa*

The distance, b (in *mm*), between adjacent parallel lines of attachment of a compression flange or another compression element to other parts of member, when such flanges or elements are required to participate in a plastic hinge action, shall not exceed the limit as given below:

$$b \le 512.T_1 / \sqrt{f_y} \qquad (5)$$

where

b and T_1 are in *mm* and f_y is in *Mpa.*

The maximum permissible value of the web depth, d_1, corresponding to web thickness, *t*, in any plastic hinge zone shall be $1120\,t / \sqrt{f_y}$, where the quantities are in SI units (*mm* and *Mpa*). When the web is subjected to combined bending and compression, the following conditions apply (Fig.3).

a) where $\dfrac{P}{P_y}$ exceeds *0.27*, then the depth d_1 shall not exceed $\dfrac{688.t}{\sqrt{f_y}}$; and

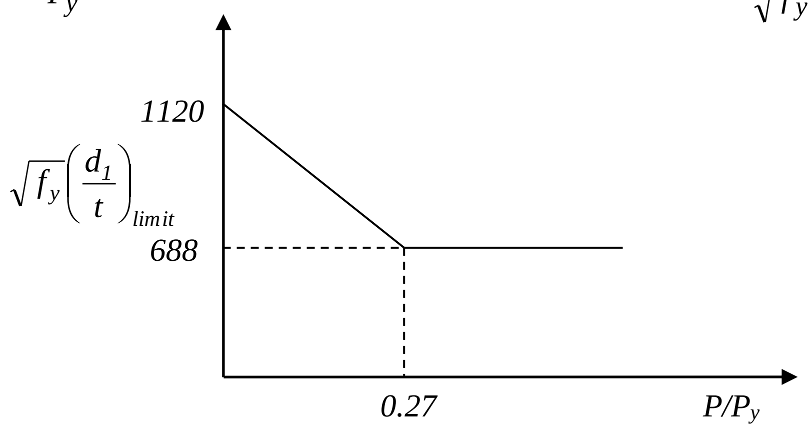


Fig. 3 Limitation on web slenderness

b) when $\dfrac{P}{P_y}$ is less than or equal to *0.27*, then the depth d_1 shall not exceed

$$d_1 \le \left[\frac{1120}{\sqrt{f_y}} - \frac{1600}{\sqrt{f_y}}\left(\frac{P}{P_y}\right)\right] t \qquad (6)$$

4.4 Lateral Buckling of flexural members

To avoid lateral buckling and torsional displacements, bracings should be provided to compression flanges at points as given below. (Fig. 4).

(a) Lateral support to the compression flange should be provided at the location of plastic hinges.

(b) The ratio of laterally unsupported length of the compression flange to the radius of gyration of the member about weak axis, ℓ/r_y , shall not exceed $640\upsilon/\sqrt{f_y}$ (with f_y in *Mpa)*, where υ is defined below in Eq.7.

(c) The slenderness ratio of compression flange, ℓ/r_y of the length, adjacent to the unsupported length where the moment exceeds $0.85\ M_p$, shall not be greater than $\dfrac{960}{\sqrt{f_y}}$ (with f_y in *Mpa*) (Fig. 4).

(d) The slenderness ratio, ℓ/r_y, of the rest of the elastic portion of the member shall be such that the lateral buckling strength of that portion is greater than actual maximum elastic moment in the region.

where

f_y = yield strength of the material in *Mpa* and υ may be taken conservatively as 1.0 or may be calculated using the following equation.

$$\upsilon = \frac{1.5}{\sqrt{1+(\theta/8)}} \tag{7}$$

where θ is the ratio of the plastic rotation at the hinge point just as the mechanism is formed to the relative elastic rotation of the far ends of the beam segment containing the plastic hinge.

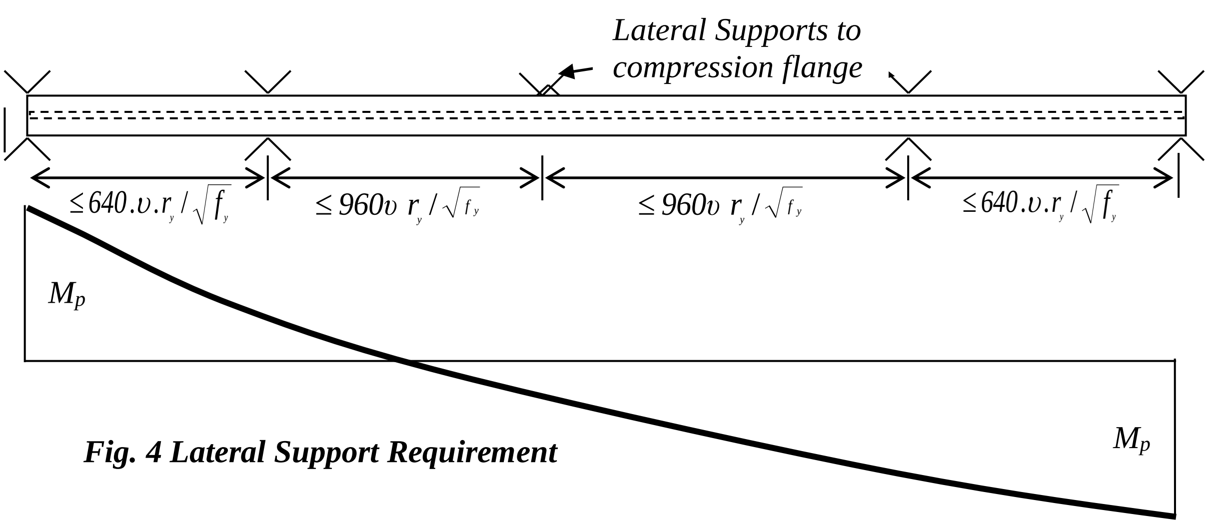


Fig. 4 Lateral Support Requirement

4.5 Column Buckling

In the plane of bending of columns which would develop a plastic hinge at ultimate loading, the slenderness ratio ℓ / r_x shall not exceed *120*, where ℓ is the centre-to-centre distance of bracing members connecting and providing restraint against weak axis buckling of the column or the distance from such a member to the base of the column. Further, columns in moment resisting frames, where sidesway is not prevented, shall be so proportioned such that

$$\frac{2P}{P_y} + \frac{\ell}{70 r_x} \le 1.0 \tag{8}$$

The slenderness ratio, ℓ / r_y, of the frame in the plane normal to the plane of frame action under consideration shall be such that the following condition is satisfied.

$$\frac{P}{P_y} \le 8700 \Big/ \left(\ell / r_y \right)^2 \qquad when \left(\ell / r_y \right) > 120 \tag{9}$$

M_{pc} / M_p, the ratio of end moment to the plastic moment capacity of columns and other axially loaded members, shall not exceed unity nor the value given by the following formula.

Case I - For columns bent in double curvature by moments producing plastic hinges at both ends of the columns:

$$\frac{M_{pc}}{M_p} \le 1.18 \left(1 - \frac{P}{P_y} \right) \tag{10}$$

Case II - For slender struts, where P / P_y in addition to exceeding *0.15* also exceeds $\dfrac{1 + \beta - \lambda_0}{1 + \beta + \lambda_0}$ Shall not contain plastic hinges. However, it is permissible to design the member as an elastic part of a plastically designed structure. Such a member shall be designed according to the maximum permissible stress requirements satisfying:

$$\frac{P}{P_{ac}} + \frac{M_{pc} C_m}{M_0 \left(1 - \dfrac{P}{P_e} \right)} \le 1.0 \tag{11}$$

where

P = axial force, compressive or tensile in a member;
M_{pc} = maximum moment (plastic) capacity in the beam - column;
M_p = plastic moment capacity of the section when no axial force is acting.
M_0 = lateral buckling strength in the absence of axial load

= M_p if the beam column is adequately braced against lateral buckling

P_{ac} = buckling strength in the plane of bending if only axially loaded (without any bending moment) and if the beam - column is laterally braced.
If the column is not adequately laterally braced, P_{ac} is the weak axis buckling strength under only axial compression.

P_e = Euler load = $\dfrac{\pi^2 \, E A_s}{(\ell/r)^2}$ in the plane of bending;

P_y = yield strength of axially loaded section = $A_s \cdot f_y$;

A_s = effective cross-section area of the member;

C_m = a coefficient whose value shall be taken as follows:

a) For member in frames where side sway is not prevented:

$$C_m = 0.85$$

b) For members in frames where side sway is prevented and not subject to transverse loading between their supports in the plane of bending:

$$C_m = 0.6 - 0.4\,\beta \geq 0.4$$

c) For members in frames where side sway is prevented in the plane of loading and subjected to transverse loading between their supports; the value of C_m is given by,

For members whose ends are restrianed against rotation

$$C_m = 0.85$$

For members whose ends are unrestrained against rotation

$$C_m = 1.0$$

r = radius of gyration about the same axis as the applied moment;

λ_0 = non -dimensional slenderness ratio = $\sqrt{\dfrac{P_y}{P_e}} = \dfrac{\ell}{\pi r} \cdot \sqrt{\dfrac{f_y}{E}}$

β = the ratio of end moment;

ℓ = actual strut length.

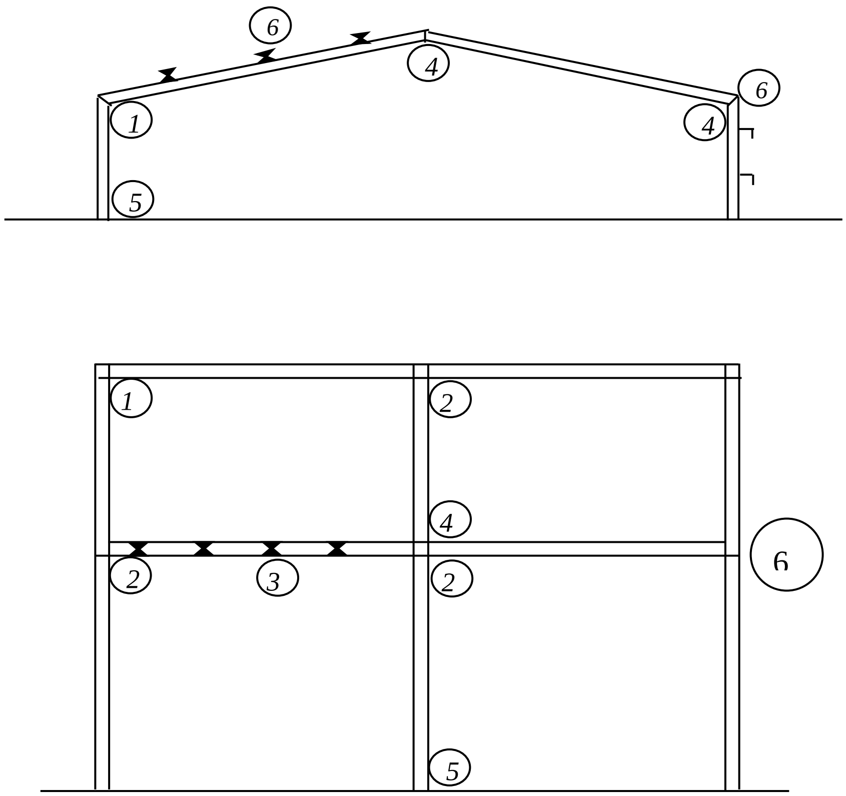
4.6 Connections

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

The various types of connections that might be encountered in steel frame structures are shown in Fig.*5*

There are four principal requirements, in design of a connection

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



1. Corner
2. Beam - column
3. Beam- Girder
4. Column Splice
5. Column Base
6. Miscellaneous

Fig. 5 Types of Connections in Buildings Frames According to Their Function

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

$\Delta L = r_1 + r_2$

This requirement reduces to the following

$$\theta_h \leq \frac{M_p}{EI}.\Delta L \qquad (12)$$

where θ_h is the joint rotation.

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

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

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

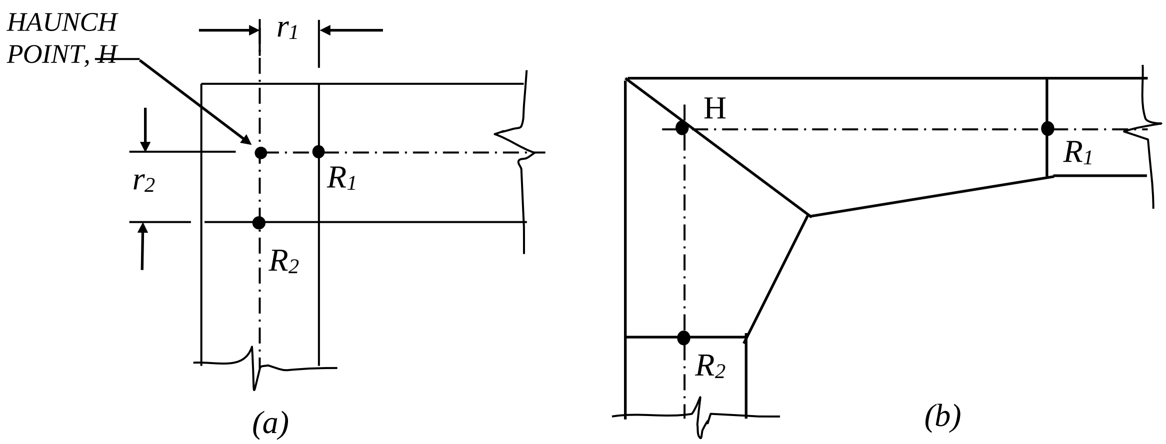


Fig.6 Designation of Critical Sections in Straight and Haunched Sections

4.6.1 Straight Corner Connections:

In the case of unstiffened corner connections, (Fig. 7) the design objective is to prevent yielding of the web due to shear force. For this the moment at which the yielding commences due to shear force, $M_h(\tau)$, given by Eq.13, should not be less than the plastic moment, M_p.

Using the maximum shear stress yield condition $\tau_y = f_y/\sqrt{3}$ and assuming that the shear stress is uniformly distributed in the knee web, and that the flange carries all of the flexural stress (Fig. 8), we can get the value of $M_h(\tau)$ as

$$M_{h(\tau)} = \frac{td_1d_2}{\sqrt{3}} f_y \qquad (13)$$

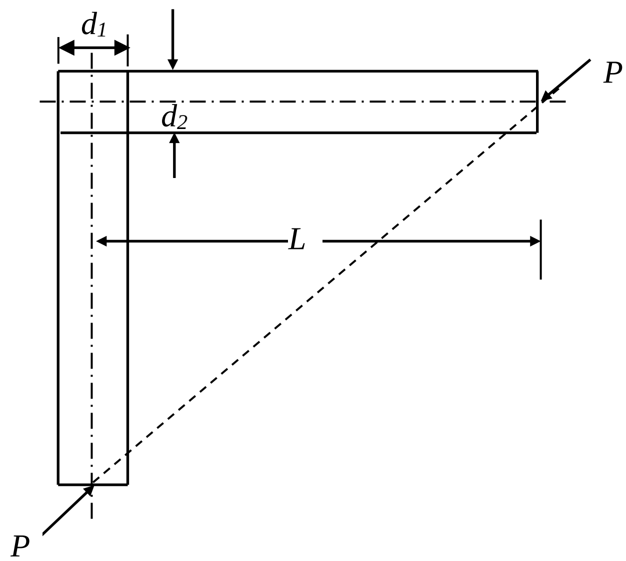


Fig. 7 Idealised Loading on Straight Corner Connection

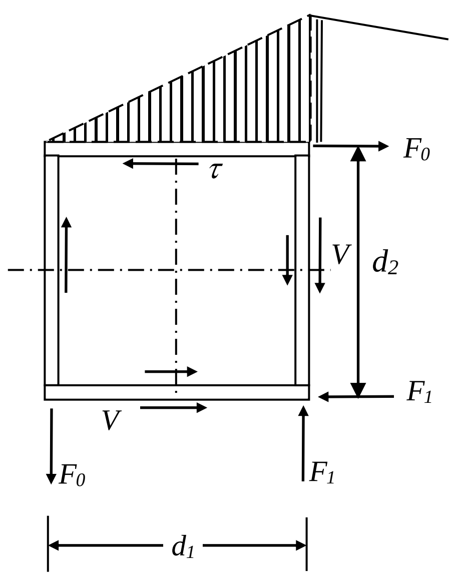


Fig.8 Forces and Stresses Assumed to act on Unstiffened Straight Corner Connection

Relating this to $M_p = f_y Z$ to obtain the required web thickness given by:

$$t_w \geq \sqrt{3}\frac{Z}{d_1 d_2} \tag{14}$$

where Z is the smaller of the plastic section modulus of the members meeting at the joint.

If the knee web is deficient in resisting the shear force, a diagonal stiffener may be used. (Fig. 9). Then the force F_0 is made up of two parts, a force carried by the web in shear and a force transmitted at the end by the diagonal stiffener. i.e., $F_o = F_{web} + F_{stiffener}$.

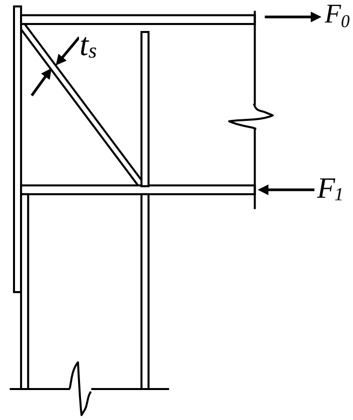


Fig. 9 Stiffened Corner Joint

When both web and diagonal stiffener have reached the yield condition

$$F_o = \frac{f_y\, t\, d_1}{\sqrt{3}} + f_y . b_s . t_s \frac{d_1}{\sqrt{d_1^2 + d_2^2}} \tag{15}$$

where b_s and t_s are the sum of the width and the thickness of the diagonal stiffeners provided on both the sides of the web.

The available moment capacity of this connection type is thus given by:

$$M_h = f_y d_2 \left[t \frac{d_1}{\sqrt{3}} + b_s t_s \frac{d_1}{\sqrt{{d_1}^2 + {d_2}^2}} \right] \tag{16}$$

The required thickness of diagonal stiffeners in corner connections, t_s that would ensure the moment as governed by shear resistance of the corner (M_h) which is greater than the plastic moment capacity $M_p = Z\ f_y$ is obtained from

$$t_s \geq \left[\frac{Z}{d_1 d_2} - \frac{t_w}{\sqrt{3}} \right] \frac{\sqrt{d_1^2 + d_2^2}}{b_s} \tag{17}$$

4.6.2 Haunched Connections

Some of the typical haunched connections are shown in Fig. *10*. Haunched connections are to be proportioned to develop plastic moment at the junction between the rolled steel section and the haunch. In order to force formation of hinge at the end of a tapered haunch (Fig. *10*), make the flange thickness in the haunch, to be *50* percent greater than that of section joined. Check the shear resistance of the web to ensure M_p governs the strength.

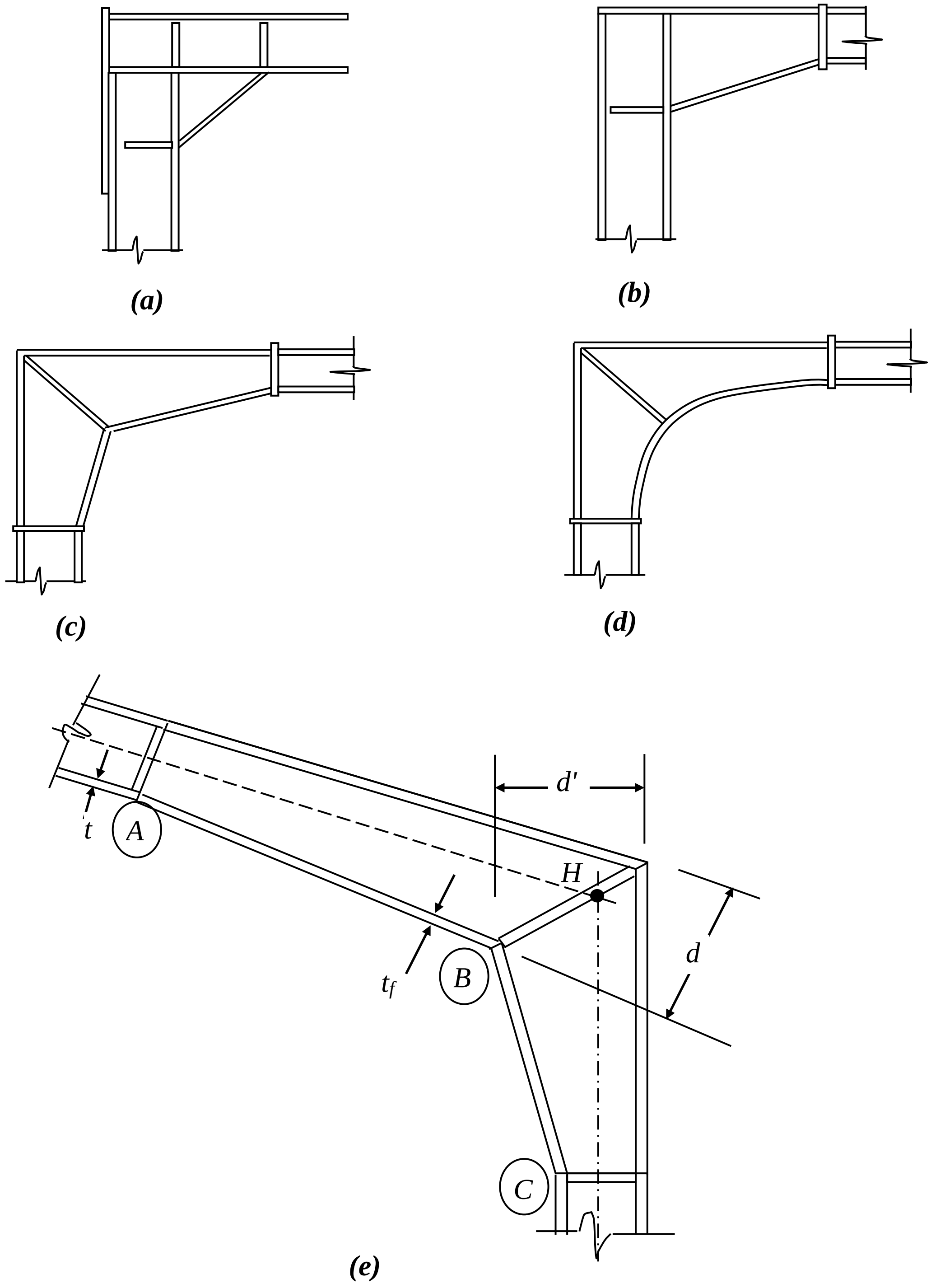
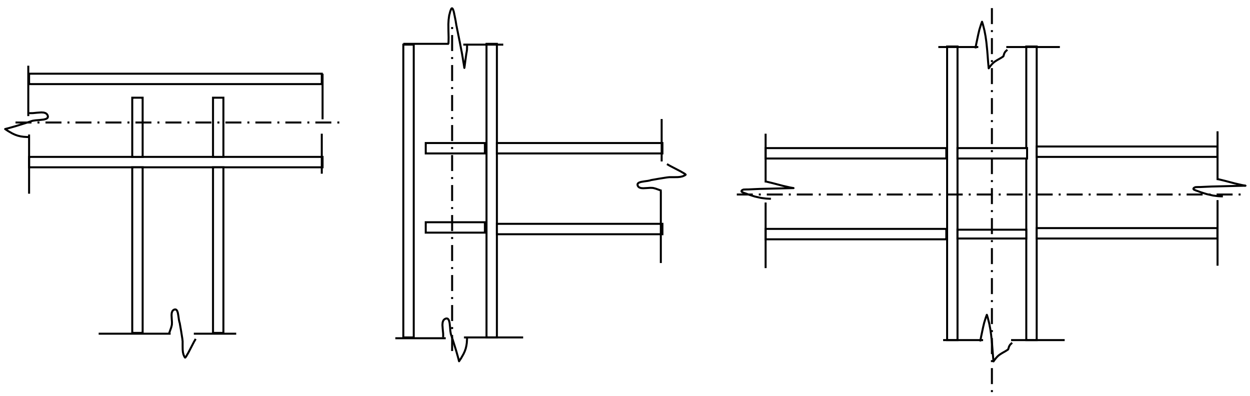


Fig. 10 Typical Haunched Corner connections

4.6.3 Interior Beam to Column Connections

Typical interior beam-column connections are shown in Fig. 11.



(a) Top *(b) Side* *(c) Interior*

Fig.11 Beam to column connections of (a) Top, (b) Side, and (c) Interior Type

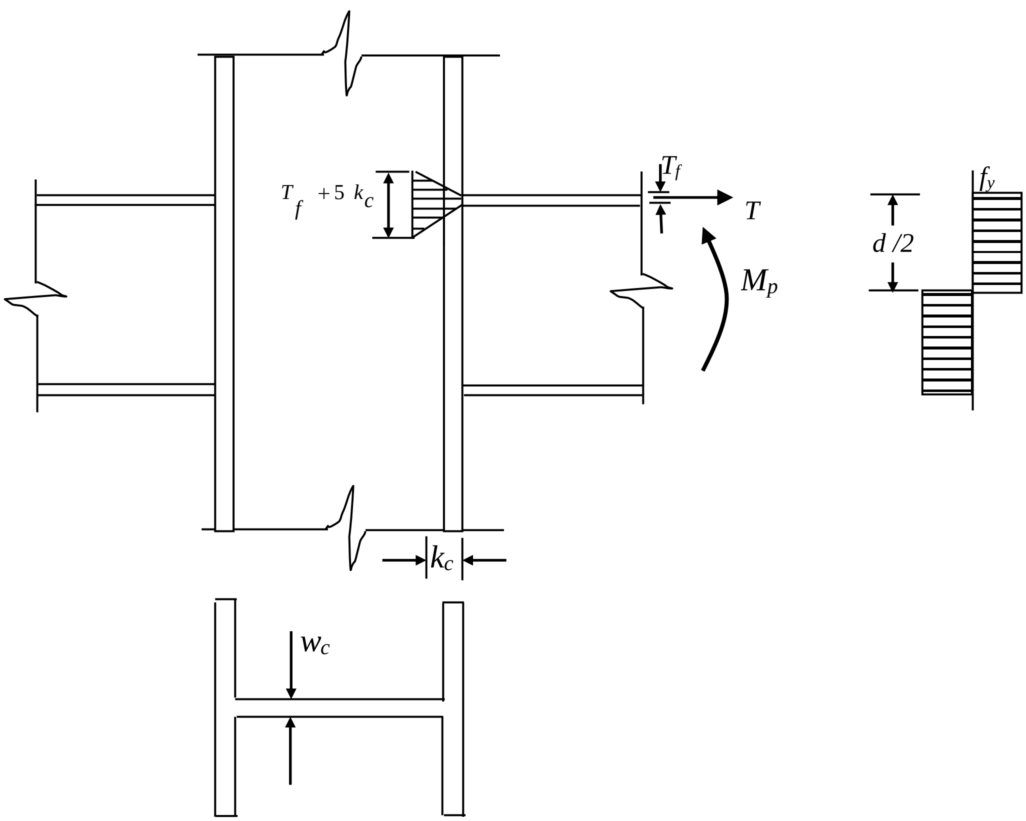


Fig. 12 Assumed stress Distribution in beam column connection with no stiffeners

The function of the 'Top' and the 'Interior' connections is to transmit moment from the left to the right beam, the column carrying any unbalanced moment. The 'side' connection transmits beam moment to upper and lower columns. The beam - column connection should have sufficient stiffening material so that it can transmit the desired moment (usually the plastic moment M_p) without the shear strength of the corner governing the design.

In an unstiffened beam-to-column connection, the concentrated force, T_f, from the beam flange, which the column web can sustain, is given by Eq.18 (*a*). (Fig.12)

The reaction width is equal to the column web thickness, w_c.

$$f_y . A_{fb} = \lfloor t_{wc}\left(T_{fb} + 5k_c\right)\rfloor\left(f_y\right) \qquad (18a)$$

where A_{fb}, T_{fb} are area and thickness of the beam flange, t_{wc} is the thickness of the column web and k_c is the distance to the roof of the column web. Thus,

$$t_{wc} = \frac{A_{fb}}{T_{fb} + 5k_c} \qquad (18b)$$

which gives the required minimum column web thickness to develop the plastic moment in the beam, without stiffening the corner.

If the column web thickness does not meet the requirement for preventing the column web from buckling, stiffeners may be provided (Fig.13). In that case, the plastic moment (M_p) will be acting at the end of the beam, and the thrust T from the beam flange should be balanced by the strength of the web (T_w) and of the stiffener plate (T_s) or

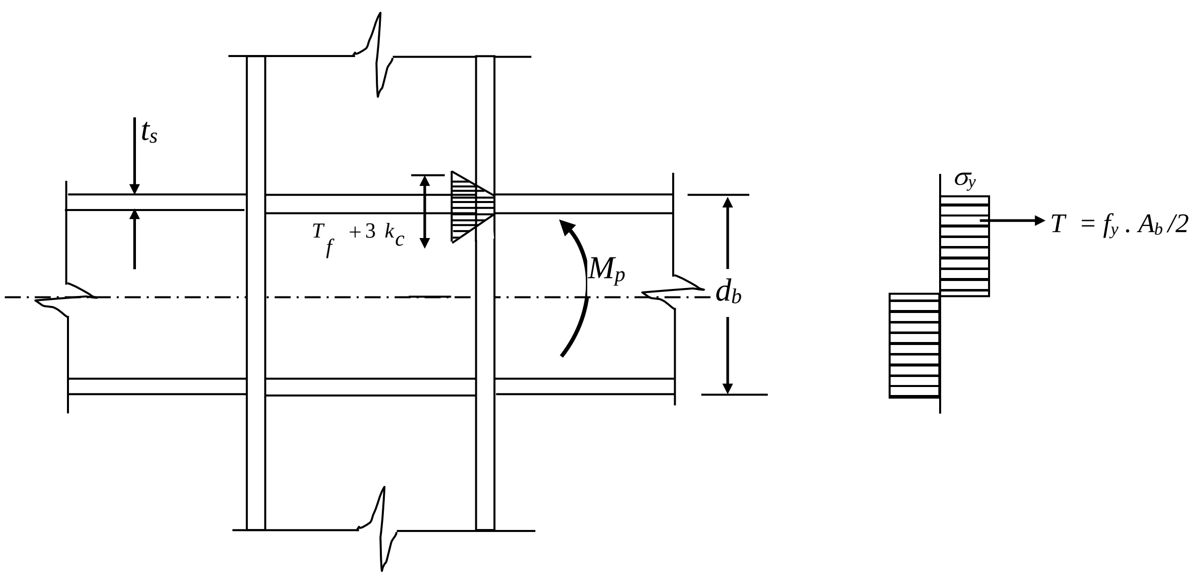


Fig. 13 Assumed Stress Distribution in Beam to Column Connection with flange type Stiffener

$$T = T_w + T_s \tag{19}$$

where T_w = force resisted by the column web $= f_y\, t_{wc}(T_f + 5k_c)$

T_s = force resisted by stiffener plate $= f_y.t_s.b_s$ and

$$T = f_y \,.\, A_{fb} \tag{20}$$

If 'flange' stiffeners are used for reinforcement, their required thickness of the stiffener is given by:

$$t_s = \lfloor A_{fb} - t_{wc}\left(T_f + 5k_c\right)\rfloor / 2b \tag{21}$$

where b is the width of the stiffener on each side of the web.

5.0 STRUCTURAL DUCTILITY

Ordinary structural grade steel for bridges and buildings may be used with modifications, when needed, to ensure weldability and toughness at lowest service temperature.

Fabrication processes should be such as to promote ductility. Sheared edges and punched holes in tension flanges are not permitted. Punched and reamed holes for connecting devices would be permitted if the reaming removes the cold-worked material.

In design, triaxial states of tensile stress set up by geometrical restraints should be avoided.

6.0 SUMMARY

The analysis, design of members and connections in steel portal frames encountered in single storey industrial buildings was discussed. Example problem is illustrated in the appendix to this chapter.

7.0 REFERENCES

1. IS800:1984 Code of practice for use of structural steel in general building construction.
2. SP:6 (6) – 1972, "Handbook for Structural Engineers – Application of Plastic Theory in Design of Steel Structures"