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.

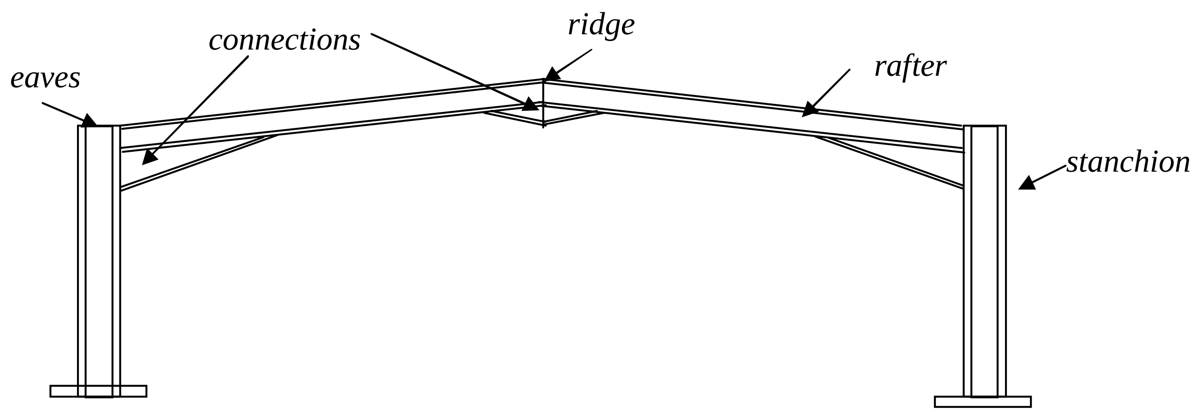
The slopes of rafters in the gable portal frames (Fig.1) vary in the range of *1* in *10* to *1* in *3* depending upon the type of sheeting and its seam impermeability. With the advent of new cladding systems, it is possible to achieve roof slopes as low as 1^0. But in such cases, frame deflections must 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.

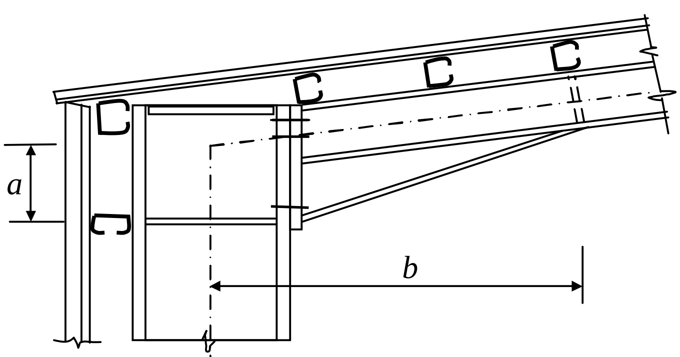
In the analysis, the problem is to find the ultimate load of a given structure with known plastic moment values of its members. But in design, the problem is reversed. Given a certain set of loads, the problem is to select suitable members.

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 according to new IS: 800.

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).

4.0 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 Combined Axial Force and Bending Moment

Under combined axial force and bending moment section strength as governed by material failure and member strength as governed by buckling failure have to be checked as given below.

4.1.1 Section Strength

4.1.1.1 Plastic and Compact Sections ─ In the design of members subjected to combined axial force (tension or compression) and bending moment, the following should be satisfied

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leq 1.0$$

Conservatively, the following equation may be used under combined axial force and bending moment

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

where

M_y, M_z = factored applied moments about the minor and major axis of the cross section, respectively

M_{ndy}, M_{ndz} = design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone, (**4.1.1.2**)

N = factored applied axial force (Tension T or Compression F)

N_d = design strength in tension (T_d) or in compression

$N_d = A_g f_y / \gamma_{m0}$

M_{dy}, M_{dz} = design strength under corresponding moment acting alone

A_g = gross area of the cross section

α_1, α_2 = constants as given in Table 4.1

$n = N / N_d$

TABLE. 4.1 CONSTANTS α_1 AND α_2

(Section 4.1.1.1)

Section	α_1	α_2
I and Channel	$5n \geq 1$	2
Circular tubes	2	2
Rectangular tubes	$1.66/(1-1.13n^2) \leq 6$	$1.66/(1-1.13n^2) \leq 6$
Solid rectangles	$1.73+1.8n^3$	$1.73+1.8\,n^3$

4.1.1.2 For plastic and compact sections without bolts holes, the following approximations may be used.

a) *Plates*

$$M_{nd} = M_d\,(1-n^2)$$

b) *Welded I or H sections*

$$M_{ndz} = M_{dz}\,(1-n) / (1-0.5a) \leq M_{dz}$$

$$M_{ndy} = M_{dy}\left[1-\left(\frac{n-a}{1-a}\right)^2\right] \leq M_{dy}$$

where $n = N / N_d$ and $a = (A - 2\,b\,t_f) / A \leq 0.5$

c) *For standard I or H sections*

$$M_{ndz} = 1.11\,M_{dz}\,(1-n) \leq M_{dz}$$

for $n \leq 0.2$ $\quad M_{ndy} = M_{dy}$

for $n>0.2$ $M_{ndy} = 1.56\, M_{dy}\, (1-n)\, (n+0.6)$

d) *For Rectangular Hollow sections and Welded Box sections* – When the section is symmetric about both axes and without bolt holes:

$$M_{ndz} = M_{dz}\,(1-n) / (1-0.5a_w) \leq M_{dz}$$

$$M_{ndy} = M_{dy}\,(1-n) / (1-0.5a_f) \leq M_{dy}$$

where $a_w = (A - 2\, b\, t_f) / A \quad \leq 0.5$

$$a_f = (A-2\, h\, t_w) / A \quad \leq 0.5$$

e) *Circular Hollow Tubes without Bolt Holes*

$$M_{nd} = 1.04\, M_d\, (1-n^{1.7}) \leq M_d$$

4.1.1.3 Semi-compact section – In the absence of high shear force (**4.2.1**) semi-compact section design is satisfactory under combined axial force and bending, if the maximum longitudinal stress under combined axial force and bending, f_x, satisfies the following criteria.

$$f_x \leq f_y / \gamma_{m0}$$

For cross section without holes, the above criteria reduces to

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

where

N_d, M_{dy}, M_{dz} are defined in **4.1.1.1**

4.1.2 Overall Member Strength

Members subjected to combined axial force and bending moment shall be checked for overall buckling failure as given in this section.

4.1.2.1 Bending and Axial Tension – The reduced effective moment, M_{eff}, under tension and bending calculated as given below, should not exceed the bending strength due to lateral torsional buckling, M_d,

$$M_{eff} = \lfloor M - \psi\; T\; Z_{ec} / A \rfloor \quad \leq M_d$$

where

M, T = factored applied moment and tension, respectively

A = area of cross section

Z_{ec} = elastic section modulus of the section with respect to extreme compression fibre

ψ = 0.8 if T and M can vary independently

= 1.0 otherwise.

4.1.2.2 Bending and Axial Compression – Members subjected to combined axial compression and biaxial bending shall satisfy the following interaction relationship.

$$\frac{P}{P_d} + \frac{K_y M_y}{M_{dy}} + \frac{K_z M_z}{M_{dz}} \leq 1.0$$

where

K_y, K_z = moment amplification factor about minor and major axis respectively

P = factored applied axial compression

M_y, M_z = maximum factored applied bending moments about y and z-axis of the member, respectively.

P_d, M_{dy}, M_{dz} = design strength under axial compression, bending about y and z-axis respectively, as governed by overall buckling as given below:

a) The design compression strength is the smallest of the minor axis (P_{dy}) and major axis (P_{dz}) buckling strength.

b) Design bending Strength M_{dz} about major axis as given below

$$M_{dz} = M_d$$

where

M_d = design flexural strength about z axis when lateral torsional buckling is prevented and where lateral torsional buckling governs

i) For flexural buckling failure

$$K_z = 1 - \frac{\mu_z P}{P_{dz}} \leq 1.5$$

where

μ_z is larger of μ_{LT} and μ_{fz} as given below:

$$\mu_{LT} = 0.15\lambda_y \beta_{MLT} - 0.15 \qquad \leq 0.90$$

$$\mu_{fz} = \lambda_z (2\beta_{Mz} - 4) + \frac{Z_{pz} - Z_{ez}}{Z_{ez}} \qquad \leq 0.90$$

$$K_y = 1 - \frac{\mu_y P}{P_{dy}}$$

$$\mu_y = \lambda_y (2\beta_{My} - 4) + \left[\frac{Z_{py} - Z_{ey}}{Z_{ey}}\right] \leq 0.9$$

β_{My}, β_{Mz}, β_{MLT} = equivalent uniform moment factor obtained from Table 4.2, according to the shape of the bending moment diagram between lateral bracing points in the appropriate plane of bending

P_{dy}, P_{dz} = design compressive strength as governed by flexural buckling about the respective axis

λ_y, λ_z = non-dimensional slenderness ratio about the respective axis

c) Design Bending Strength about Minor axis

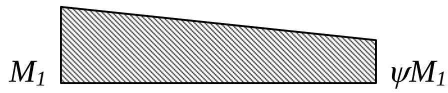
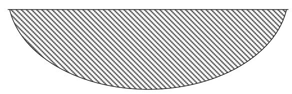
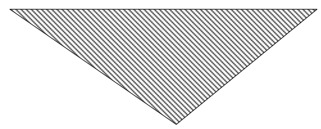
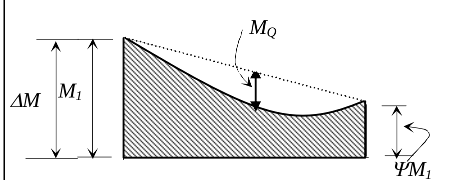
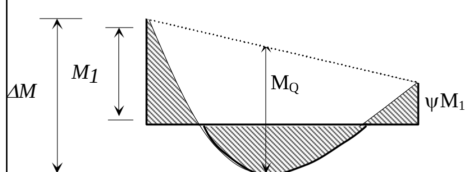
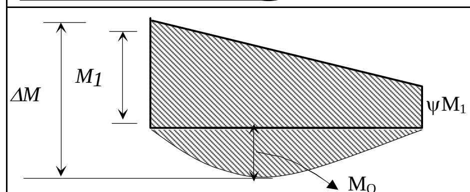
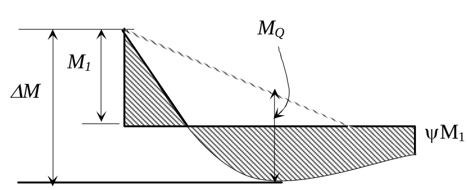
$$M_{dy} = M_d$$

where

M_d = design flexural strength about y-axis calculated using plastic section modulus for plastic and compact sections and elastic section modulus for semi-compact sections.

TABLE 4.2 EQUIVALENT UNIFORM MOMENT FACTOR

(Section 4.1.2.2)

<table>
<tr><th>Particulars</th><th>BMD</th><th>β_M</th></tr>
<tr><td>Due to end moments</td><td></td><td>1.8-0.7ψ</td></tr>
<tr><td rowspan="2">Moment due to lateral loads</td><td></td><td>1.3</td></tr>
<tr><td></td><td>1.4</td></tr>
<tr><td rowspan="4">Moment due to lateral loads and end moments</td><td></td><td rowspan="4">$$1.8-0.7\psi+\frac{M_Q}{\Delta_M}(0.7\psi-0.5)$$
M_Q=$\left|M_{\max}\right|$ due to lateral load alone
ΔM=$\left|M_{\max}\right|$(same curvature)
ΔM=$\left|M_{\max}\right|+\left|M_{\min}\right|$ (reverse curvature)</td></tr>
<tr><td></td></tr>
<tr><td></td></tr>
<tr><td></td></tr>
</table>

4.2 Combined Shear and Bending

4.2.1 No reduction in moment capacity of the section is necessary as long as the cross section is not subjected to high shear force (factored value of applied shear force is less than or equal to 60 percent of the shear strength of the section), the moment capacity may be taken as, $M_{d,}$ (**Cl. 8.2 of IS:800**) without any reduction.

4.2.2 When the factored value of the applied shear force is high (exceeds the limit in **4.2.1**), the factored moment of the section should be less than the moment capacity of the section under higher shear force, M_{dv}, calculated as given below:

a) Plastic or Compact Section

$$M_{dv} = M_d - \beta\left(M_d - M_{fd}\right) \;\; \leq 1.2\, z_e \;\; f_y \big/ \gamma_{m0}$$

where

$\beta = \left(2\,V/V_d - 1\right)^2$

M_d = plastic design moment of the whole section disregarding high shear force effect considering web buckling effects

V = factored applied shear force as governed by web yielding or web buckling.

V_d = design shear strength as governed by web yielding or web buckling

M_{fd} = plastic design strength of the area of the cross section excluding the shear area, considering partial safety factor γ_{m0}

b) Semi-compact Section

$$M_{dv} = Z_e\, f_y / \gamma_{m0}$$

where Z_e = elastic section modulus of the whole section

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 new IS: 800 the projection of flange or other compression element beyond

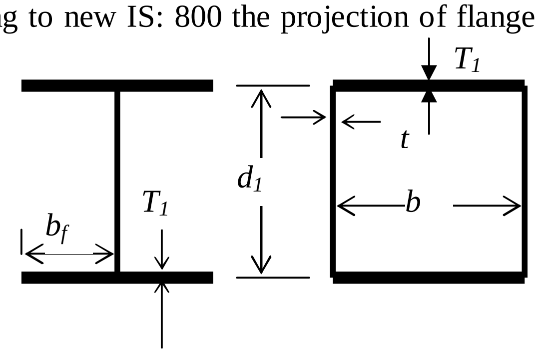


Fig. 2 Plate Elements in Steel Sections

its outermost point of attachment shall not exceed the value given below:

TABLE 4.3 LIMITING WIDTH TO THICKNESS RATIOS

Compression element			**Ratio**	**Class of Section**		
				Class 1 Plastic	**Class 2 Compact**	**Class 3 Semi-Compact**
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I-H-or box section [c]	Neutral axis at mid-depth		d/t_w	83.9ε	104.8ε	125.9ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\geq 42\varepsilon$	$\frac{104.8\varepsilon}{1+r_1}$	$\frac{125.9\varepsilon}{1+2\,r_2}$ but $\geq 42\varepsilon$
		If r_1 is positive:	d/t_w		$\frac{104.8\varepsilon}{1+1.5r_1}$ but $\geq 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε $9.4\ \varepsilon$	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component						
Circular tube subjected to moment or axial compression	CHS or built by welding		D/t	$44\varepsilon^2$	$55\varepsilon^2$	$88\varepsilon^2$
Stem of a T-section, rolled or cut from a rolled I-or H-section			D/t_f	8.4ε	9.4ε	18.9ε

Note 1: Section having elements which exceeds semi-compact limits are to be taken as slender cross sections

Note2: $\varepsilon=(250/f_y)^{1/2}$

Note 3: Check webs for shear buckling in accordance with **cl. 8.4.2 IS:800** *when d/t > 67 ε. Where, b is the width of the element may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate, t is the thickness of element, d is the depth of the web, D mean diameter of the element,*

Note 4: Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favourable classification.

Note 5: The stress ratio r_1 and r_2 *are defined as*

$$r_1=\frac{\text{actual average axial compressive stress}}{\text{design compressive stress of web alone}}\ ,\quad r_2=\frac{\text{actual average axial compressive stress}}{\text{design compressive stress of overall section}}$$

4.4 Laterally Unsupported Beams

Beam experiencing bending about major axis and not restrained against lateral buckling of the compression flange may fail by lateral torsional buckling before material failure. Effect of lateral torsional buckling on flexural strength also needs to be considered unless $\lambda_{LT} \leq 0.4$

where, λ_{LT} = non-dimensional slenderness ratio for lateral torsional buckling as given below

The design bending strength of laterally unsupported beam as governed by lateral torsional buckling is given by: $M_d = \beta_b Z_p f_{bd}$

where, β_b = 1.0 for plastic and compact sections

= Z_e/Z_p for semi-compact sections

Z_p, Z_e = plastic section modulus and elastic section modulus with respect to extreme compression fibre.

f_{bd} = design bending compressive stress, obtained as given below

$$f_{bd} = \chi_{LT}\, f_y / \gamma_{m0}$$

where, χ_{LT} = reduction factor to account for lateral torisonal buckling given by:

$$\chi_{LT} = \frac{1}{\left\{\phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2\right]^{0.5}\right\}} \leq 1.0$$

$$\text{in which } \phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\lambda_{LT} - 0.2\right) + \lambda_{LT}{}^2\right]$$

The values of imperfection factor, $\alpha_{Lt,}$ for lateral torisonal buckling of beams is given by:

where

α_{LT} = 0.21 for rolled section

α_{LT} = 0.49 for welded section

The non-dimensional slenderness ratio, λ_{LT}, is given by

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}}$$

$$= \sqrt{f_y / f_{cr,b}}$$

where, M_{cr}= elastic critical moment calculated as per **4.1**

$f_{cr,b}$ = extreme fibre compressive elastic lateral buckling moment

4.4.1 *Elastic Lateral Torsional Buckling Moment* – The elastic lateral buckling moment, $M_{cr,}$ can be determined as:

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{(KL)^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{(KL)^2}\right]\right\}}$$

The following simplified conservative equation may be used in the case of prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections, for calculating the elastic lateral buckling moment, M_{cr}.

$$M_{cr} = \frac{\beta_{LT}\pi^2 EI_y h}{2(KL)^2}\left[1+\frac{1}{20}\left[\frac{KL/r_y}{h/t_f}\right]^2\right]^{0.5} = \beta_b Z_p f_{cr,b}$$

where

I_t = torsional constant
I_w = warping constant
I_y = moment of inertia about the weak axis
r_y = radius of gyration of the section about the weak axis
KL = effective laterally unsupported length of the member (**4.2**)
h = overall depth of the section
t_f = thickness of the flange
β_{LT} = 1.20 for plastic and compact sections with $t_f / t_w \leq 2.0$
= 1.00 for semi-compact sections or sections with $t_f / t_w > 2.0$

4.4.2 Effective Length of Compression Flanges

4.4.2.1 For simply supported beams and girders of span length, L, where no lateral restraint to the compression flanges is provided, but where each end of the beam is restrained against torsion, the effective length KL of the compression flanges shall be taken as follows:

a) With ends of compression flanges unrestrained against lateral bending (that is, free to rotate in plan) at the bearings — $KL = L$

b) With ends of compression flanges partially restrained — $KL = 0.85 \times L$ against lateral bending (that is, not fully free to rotate in plan at the bearings)

c) With ends of compression flanges fully restrained against lateral bending (that is, rotation in plan at the bearings completely restrained) — $KL = 0.7 \times L$

Restraint against torsion can be provided by:

i) web or flange cleats, or
ii) bearing stiffeners acting in conjunction with the bearing of the beam, or
iii) lateral end frames or to the external supports to the ends of the compression flanges, or
iv) their being built into walls

Where the ends of the beam are not restrained against torsion or where the load is applied to the compression flange and both the load and flange are free to move laterally the above values of the effective length shall be increased by 20 percent.

The end restraint element shall be capable of safely resisting in addition to wind and other applied external forces, a horizontal force acting at the bearing in the plane in a direction normal to the axis of compression flange of the beam at the level of the centroid of the flange and having a value equal to not less than *2.5* percent of the maximum compressive force occurring in the flange.

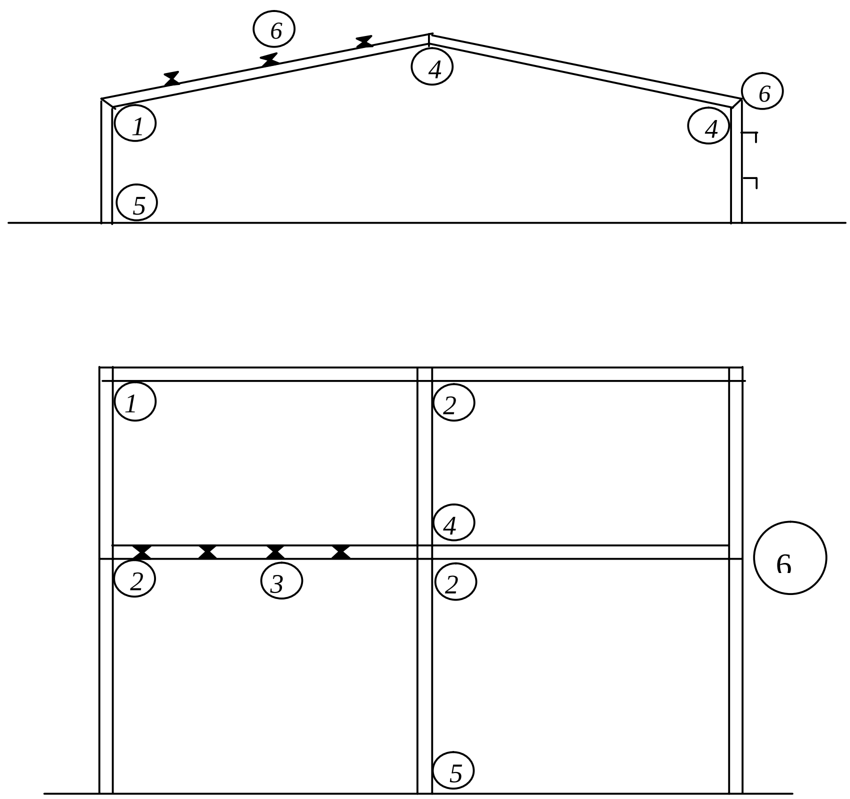
4.5 Connections

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.*3*.

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. 4 (a). For haunched connections, the critical sections are assumed at R_1 and R_2, [Fig. 4 (b)].



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

Fig. 3 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. 4(a).

$\Delta L = r_1 + r_2$

This requirement reduces to the following

$$\theta_h \leq \frac{M_p}{EI}.\Delta L$$

where θ_h is the joint rotation.

The above equation 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.

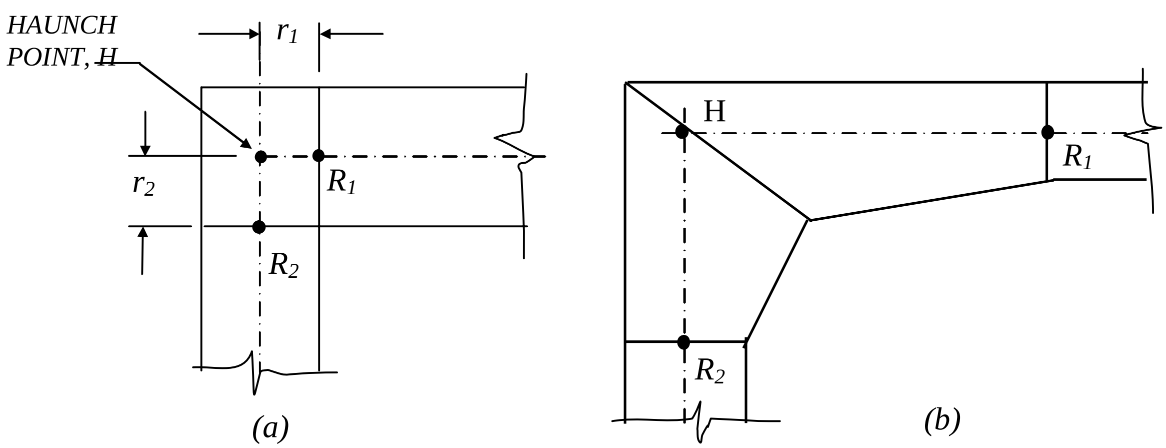


Fig.4 Designation of Critical Sections in Straight and Haunched Sections

4.6.1 Straight Corner Connections:

In the case of unstiffened corner connections, (Fig. 5) 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.(a), 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. 6), we can get the value of $M_h\ (\tau)$ as

$$M_{h(\tau)} = \frac{t d_1 d_2}{\sqrt{3}} f_y \qquad \text{(a)}$$

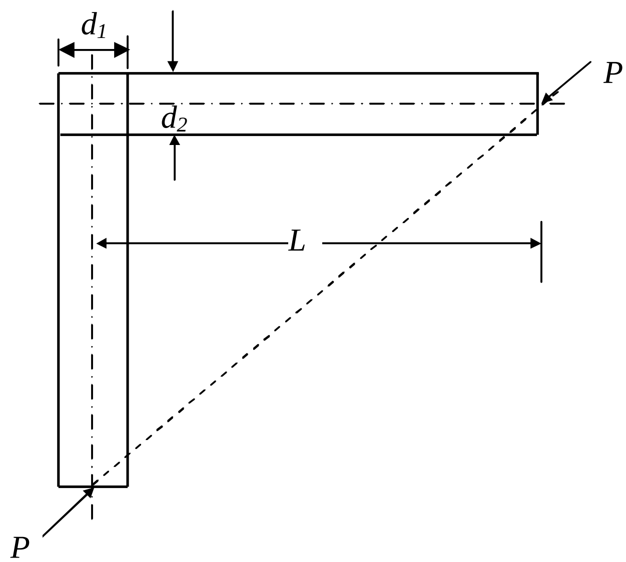


Fig. 5 Idealised Loading on Straight Corner Connection

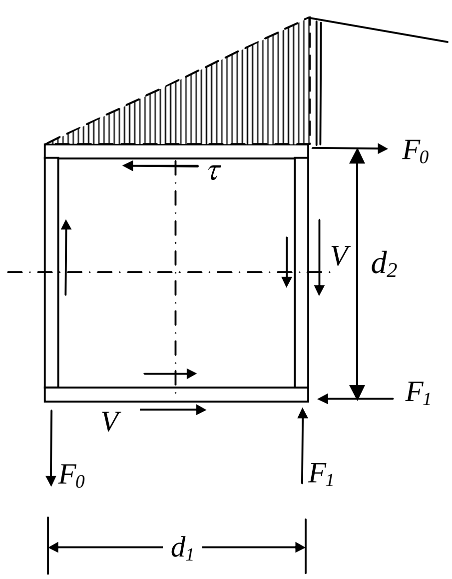


Fig.6 Forces and Stresses Assumed to act on Unstiffened Straight Corner Connection

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

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

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. 7). 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., $F_o = F_{web} + F_{stiffener}$.

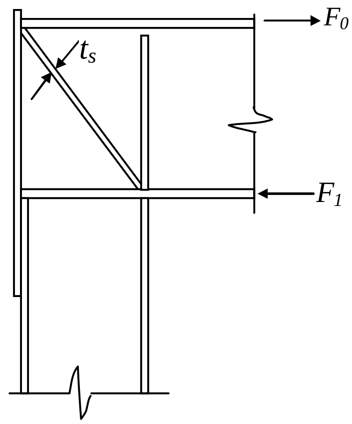


Fig. 7 Stiffened Corner Joint

When both web and diagonal stiffener have reached the yield condition

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

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:

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

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

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

4.6.2 Haunched Connections

Some of the typical haunched connections are shown in Fig. *8*. 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. *8*), 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.

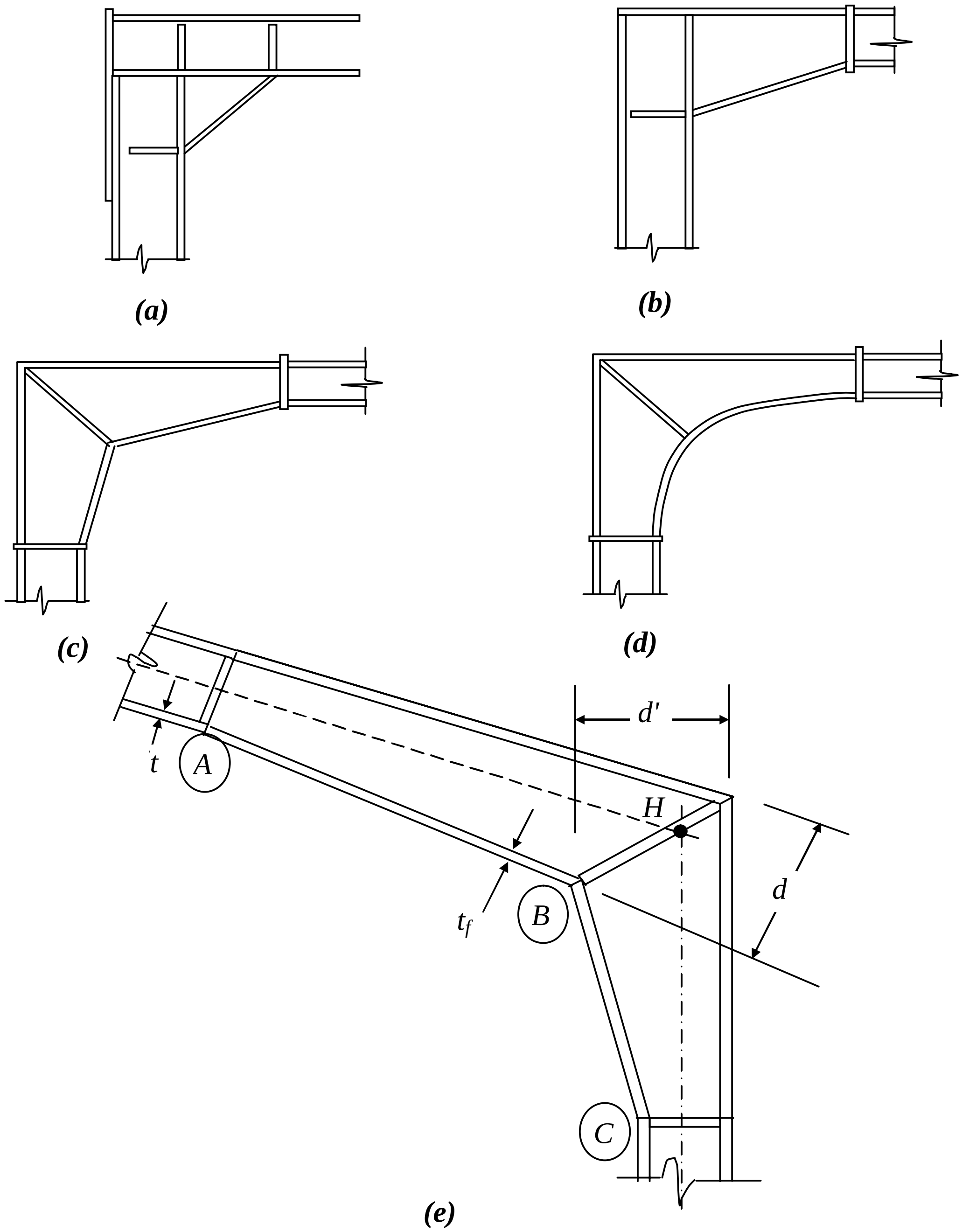


Fig. 8 Typical Haunched Corner connections

4.6.3 Interior Beam to Column Connections

Typical interior beam-column connections are shown in Fig. 9.

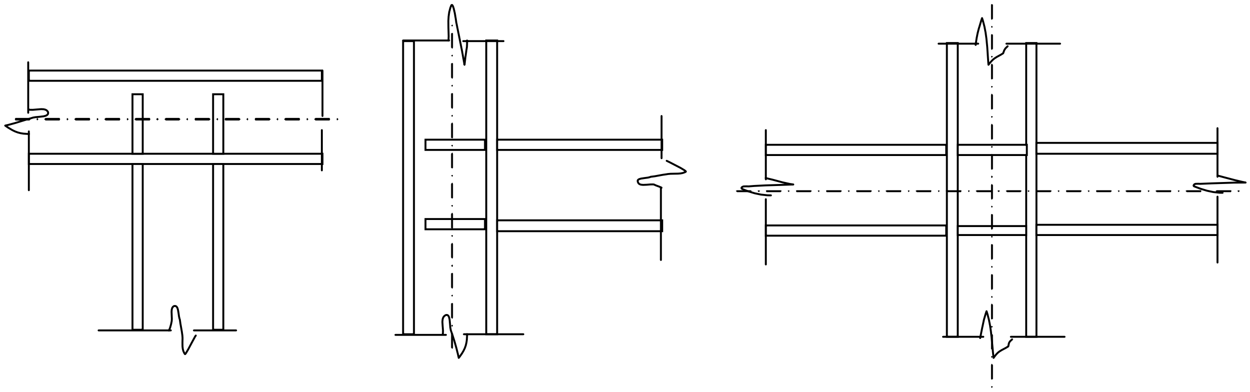


Fig.9 Beam to column connections of (a) Top, (b) Side, and (c) Interior Type

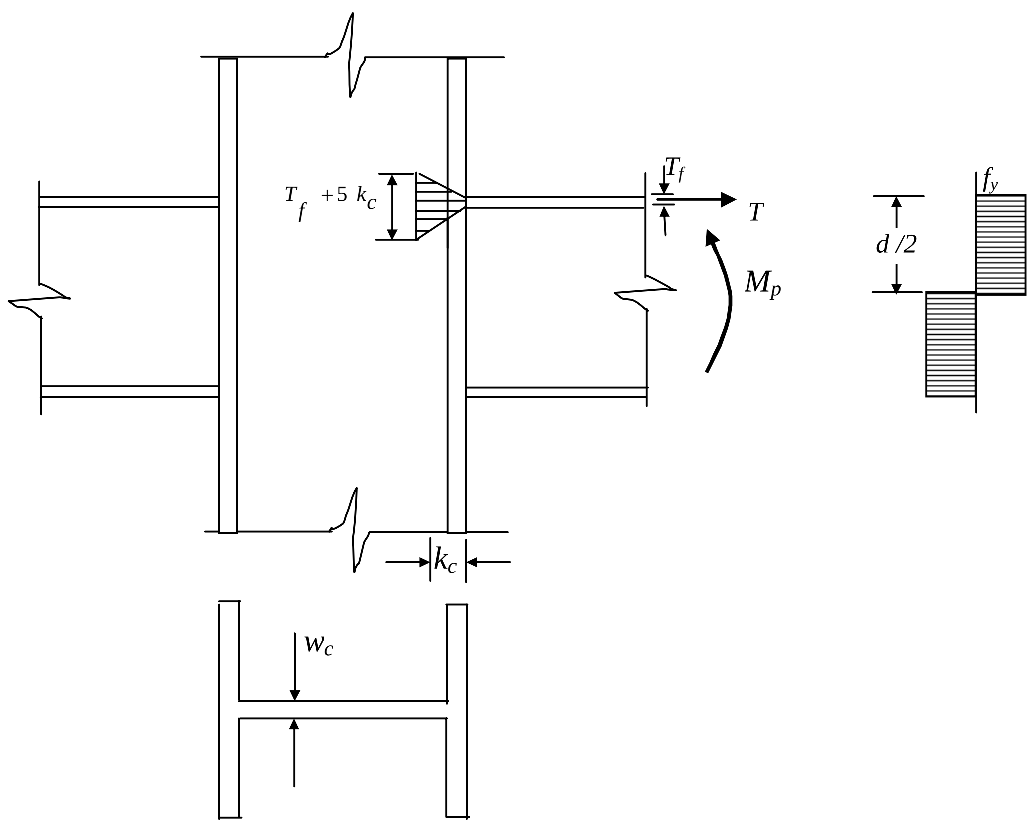


Fig. 10 Assumed stress Distribution in beam column connection with no stiffeners

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 the following equation (Fig.12)

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

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

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,

$$t_{wc} = \frac{A_{fb}}{T_{fb} + 5k_c}$$

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.11). 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

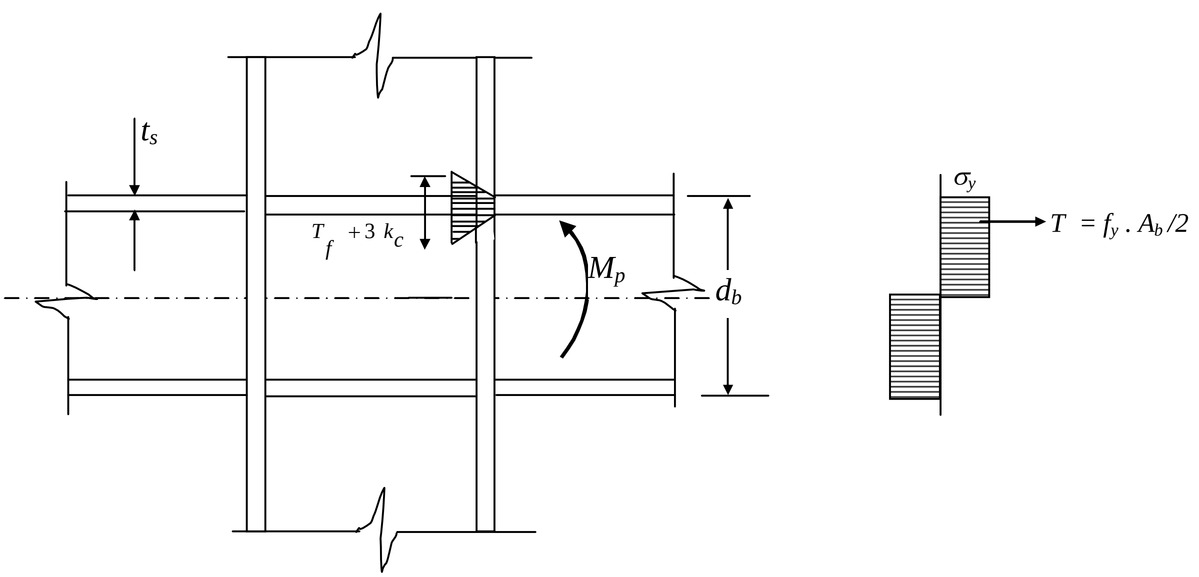


Fig. 11 Assumed Stress Distribution in Beam to Column Connection with flange type Stiffener

$T = T_w + T_s$

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

$T = f_y \cdot A_{fb}$

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

$$t_s = \left[A_{fb} - t_{wc}\left(T_f + 5k_c\right)\right]/2b$$

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: 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".